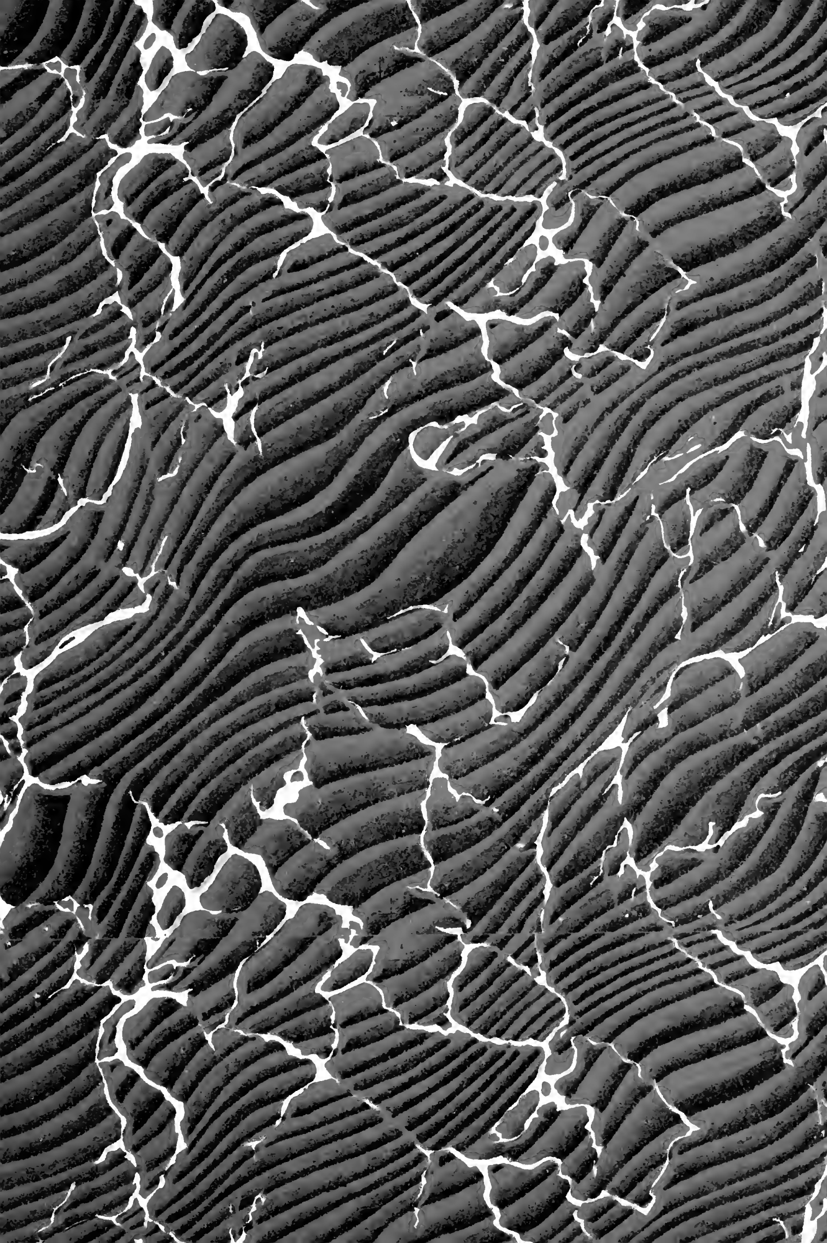

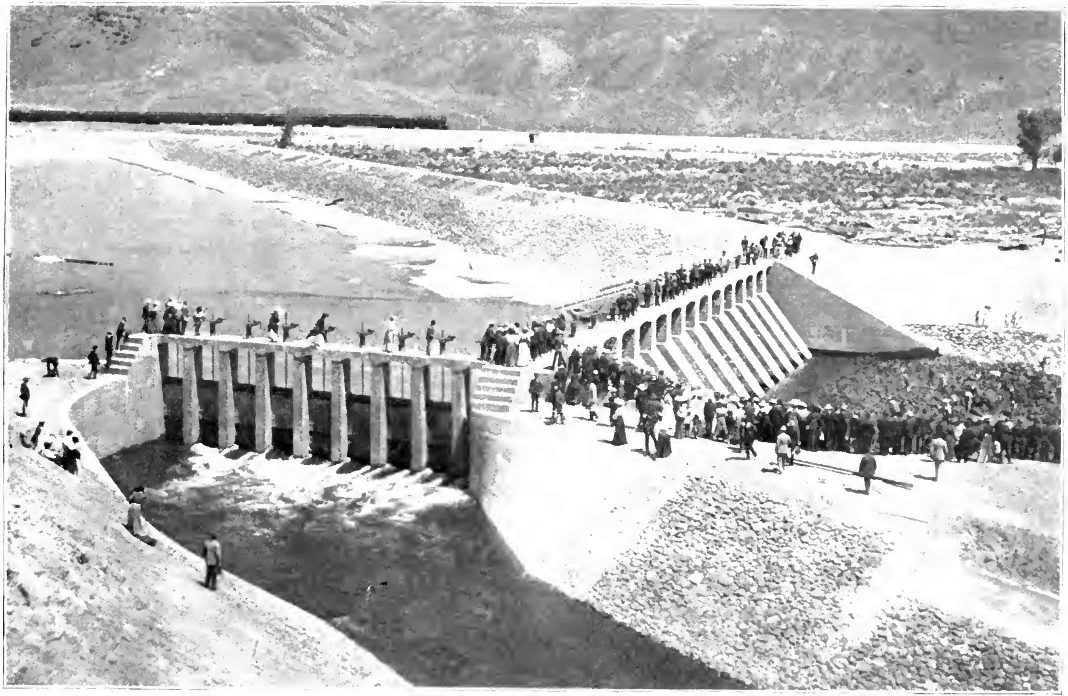

TRUCKEE-CARSON PROJECT, NEVADA

View showing gates of new diversion dam completely closed, and entire flow of Truckee River diverted into main Truckee canal, to be used in irrigating 350,000 acres of land in the Carson Sink Desert.

Photo by Reclamation Service.

Cyclopedia
of
Civil Engineering

A General Reference Work

ON SURVEYING, RAILROAD ENGINEERING, STRUCTURAL ENGINEERING, ROOFS AND BRIDGES, MASONRY AND REINFORCED CONCRETE, HIGHWAY CONSTRUCTION, HYDRAULIC ENGINEERING, IRRIGATION. RIVER AND HARBOR IMPROVEMENT, MUNICIPAL ENGINEERING, COST ANALYSIS, ETC.

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Assisted by a Corps of
CIVIL AND CONSULTING ENGINEERS AND TECHNICAL EXPERTS OF THE HIGHEST PROFESSIONAL STANDING

Illustrated with over Three Thousand Engravings

EIGHT VOLUMES

CHICAGO
AMERICAN TECHNICAL SOCIETY
1908

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Grateful acknowledgment is here made also for the invaluable co-operation of the foremost Civil, Structural, Railroad, Hydraulic, and Sanitary Engineers in making these volumes thoroughly representative of the very best and latest practice in every branch of the broad field of Civil Engineering; also for the valuable drawings and data, illustrations, suggestions, criticisms, and other courtesies.

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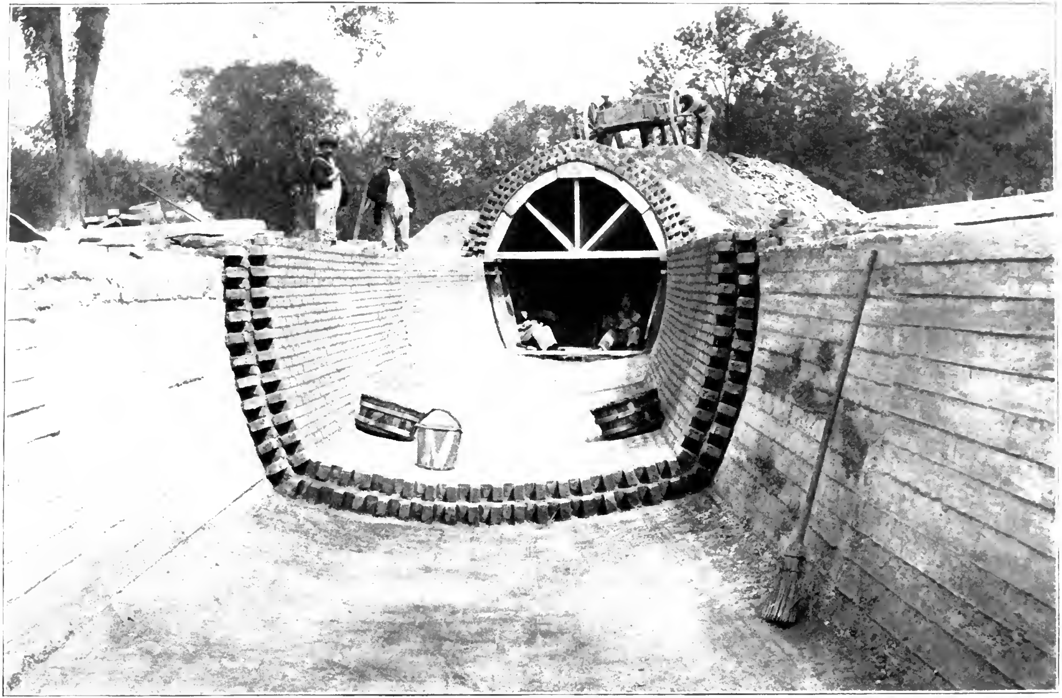


BOTTOM OF TRENCH OF CONCRETE AND BRICK-LINED SEWER

The entire work is covered with an earth embankment after completion. Metropolitan Water and Sewerage System of Boston, Mass.

Foreword

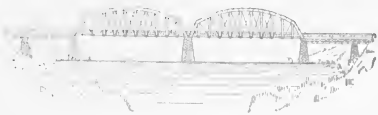
THE marvelous developments of the present day in the field of Civil Engineering, as seen in the extension of railroad lines, the improvement of highways and waterways, the increasing application of steel and reinforced concrete to construction work, the development of water power and irrigation projects, etc., have created a distinct necessity for an authoritative work of general reference embodying the results and methods of the latest engineering achievement. The Cyclopedia of Civil Engineering is designed to fill this acknowledged need.

The aim of the publishers has been to create a work which, while adequate to meet all demands of the technically trained expert, will appeal equally to the self-taught practical man, who, as a result of the unavoidable conditions of his environment, may be denied the advantages of training at a resident technical school. The Cyclopedia covers not only the fundamentals that underlie all civil engineering, but their application to all types of engineering problems; and, by placing the reader in direct contact with the experience of teachers fresh from practical work, furnishes him that adjustment to advanced modern needs and conditions which is a necessity even to the technical graduate.

¶ The Cyclopedia of Civil Engineering is a compilation of representative Instruction Books of the American School of Correspondence, and is based upon the method which this school has developed and effectively used for many years in teaching the principles and practice of engineering in its different branches. The success attained by this institution as a factor in the machinery of modern technical education is in itself the best possible guarantee for the present work.

¶ Therefore, while these volumes are a marked innovation in technical literature — representing, as they do, the best ideas and methods of a large number of *different* authors, each an acknowledged authority in his work — they are by no means an experiment, but are in fact based on what long experience has demonstrated to be the best method yet devised for the education of the busy workingman. They have been prepared only after the most careful study of modern needs as developed under conditions of actual practice at engineering headquarters and in the field.

¶ Grateful acknowledgment is due the corps of authors and collaborators — engineers of wide practical experience, and teachers of well-recognized ability — without whose co-operation this work would have been impossible.



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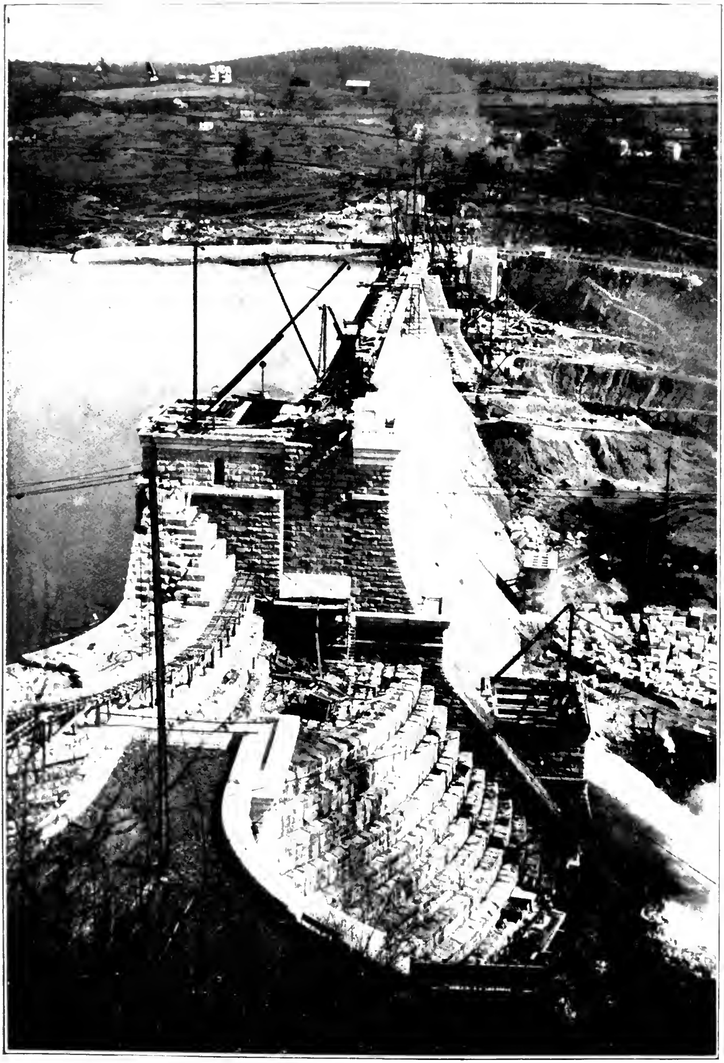


NEW CROTON DAM UNDER CONSTRUCTION

Largest masonry dam in the world, part of the waterworks system of New York City. Took 14 years to build, requiring about 850,000 cu. yds. of masonry. Completed in 1906. Cost, $7,700,000. Length, 2,400 feet; height, 301 feet; thickness, 216 feet at base, tapering to ten feet at top of spillway (at left) and 21 feet at top of main dam. Capacity, 30,000,000,000 gals., and with auxiliary dams 100,000,000,000 gals. Water at dam, 160 feet deep, the impounded river forming a lake 20 miles long and 2 miles in extreme width, burying under 30 feet of water the old dam 3 miles upstream.

WATER SUPPLY.

PART I.

INTRODUCTION.

1. Historical. The earliest method of artificially obtaining a water supply was by the digging of wells. These were at first mere shallow cavities scooped out of the ground in low places; but it is interesting to know that the sinking of deep wells through rock dates from a very early period, the Chinese having been familiar with such work from very early times. Besides wells, other works for water-supply purposes were constructed by the Ancients, such as reservoirs, cisterns, aqueducts, etc.

The greatest development of waterworks construction in ancient times took place during the prosperous period of the Roman Empire, some of the finest works having been built at this time. To supply the chief cities of the empire great aqueducts were constructed, many miles in length, and there were in some cases several such aqueducts supplying a single city. Rome was at one time supplied from fourteen different aqueducts some of which had a length of 40 miles. The first of these was built about 312 B.C. and the last about 305 A.D. Some of the other cities which were well supplied with water at this time were Paris and Lyons in France, Metz in Germany, and Segovia and Seville in Spain.

The distribution of water in this age was by no means general. From the aqueducts the water first passed into large cisterns, and from these it was distributed through lead pipes to the fountains, baths and various public buildings, and to a few private consumers. The masses of the people were obliged to get their supply from public fountains. While the actual amount of water used by private consumers was not great the liberality of the supply for public purposes was so great that the total consumption was in many cases very high, some estimates making the consumption of water in Rome as high as 300 gallons per capita daily.

After the fall of Rome the entire subject of water supply was neglected for many centuries, and as one result, Europe was ravaged many times by terrible pestilences, due to polluted water. In some cases even the purpose for which the ancient aqueducts had been built was forgotten by the inhabitants.

The development of modern waterworks began in Paris and London as early as the beginning of the 17th century, but little progress was made until the application of steam to pumping engines, first made in London in 1761. Since 1800 the development has been very rapid, both in Europe and America.

The first works in America for the supply of water to towns were those of Boston, built in 1652. Machinery was first used for pumping water at Bethlehem, Pennsylvania, where the works were put into operation in 1754. The first use of the steam engine was at Philadelphia in 1800, and in New York steam was applied in 1804. The principal development in this country has taken place since 1850, about ninety-eight per cent of all existing works having been constructed since that time. Nearly all towns of 2,000 inhabitants or more now have a public water supply, and the construction of works is progressing rapidly in many smaller towns and villages. While there is more work yet to be done in this direction, the chief work of the future will be in providing increased supplies for the rapidly growing cities and towns of this country, in developing new and better sources of supply and in the improvement of the quality of the existing supplies. There is also much opportunity for the engineer in the management of waterworks, in the direction of reducing cost of operation, prevention of waste and in the improvement of service in many other ways.

2. Value and Importance of a Public Water Supply. The most important use of a public water supply is that of furnishing a suitable water for domestic purposes. For such use the prime requisite is that the water should be pure. 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.

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.

A good water supply is also of great value to the manufacturing interests of a town. Many establishments, such as sugar refineries, starch factories, cleaning and dyeing houses, chemical works, etc., require an abundant water supply, and in some cases water of a high degree of purity. The question of water supply indeed often determines the location of factories. Large quantities are also used for operating elevators, for boiler purposes, and for many other uses that may be classed as commercial.

The most important public use of water supply is 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 waterworks, which at the same time provides a suitable water for other purposes. To permit of the establishment of a certain class 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.

CONSUMPTION OF WATER.

3. General Considerations. When a new or enlarged water supply is under consideration one of the first questions to be answered is that relating to the quantity of water which will be required in the near future. The knowledge which is required includes not only the average daily quantity which will be needed, but also the monthly, daily, and hourly variation in the rate of consumption. In designing certain parts of the works the average consumption for the year is sufficient, but in certain other parts, such as pumps and distributing pipes, we need to know the greatest rate of consumption for a very short period of time.

There are many influences which affect the rate of consumption per capita of any given town or city. One of these is the actual population of the town. 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 water 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.

4. The Average Daily Consumption Per Capita. In Table No. 1 are given the rates of consumption per capita for several American cities and towns in 1895.

It will be noted from Table No. 1 that a great variation exists in the rate of consumption in different cities and that the consumption in some of the cities is very high. For example, it is 271 gallons in Buffalo, New York, and 247 gallons in Allegheny, Pennsylvania. It will also be noted from a comparison of Tables No. 1 and 2 that the consumption is, on the average, much less in European than in

TABLE I.

Consumption of Water in American Cities and Towns.

City.	Population. 1900.	Daily consumption per inhabitant, 1895.
New York	3,437,202	100
Chicago	1,698,575	139
Philadelphia	1,293,697	162
Brooklyn	....	89
St. Louis	575,238	98
Boston	560,892	100
Cincinnati	325,902	135
San Francisco	342,782	63
Cleveland	381,768	142
Buffalo	352,387	271
New Orleans	287,104	35
Washington	278,718	200
Montreal	....	83
Detroit	285,704	152
Milwaukee	285,315	101
Toronto	....	100
Minneapolis	202,718	88
Louisville	204,731	97
Rochester	162,608	71
St. Paul	163,065	60
Providence	175,597	57
Indianapolis	169,164	74
Allegheny	129,896	247
Columbus	125,560	127
Worcester	118,421	66
Toledo	131,822	70
Lowell	94,969	82
Nashville	80,865	139
Fall River	104,863	35
Atlanta	89,872	42
Memphis	102,195	100

In Table No. 2 are given the rates of consumption for several European cities.

TABLE 2.

Consumption of Water in European Cities.

City.	Estimated population.	Daily consumption per capita, gallons.
London	5,700,000	42
Manchester	849,093	40
Liverpool	790,000	34
Birmingham	680,140	28
Bradford	436,260	31
Leeds	420,000	43
Sheffield	415,000	21
Berlin	1,427,200	18
Breslau	330,000	20
Cologne	281,700	34
Dresden	276,500	21
Paris	2,500,000	53
Marseilles	406,919	202
Lyons	401,930	31
Naples	481,500	53
Rome	437,419	264
Florence	192,000	21
Venice	130,000	11
Zurich	80,000	60

American cities. Both of these variations are due largely to the variation in practice in the use of meters to measure the water used and to charge accordingly. In some American cities meters are quite generally used, and without exception the consumption of water in those places is comparatively low. Meters are also generally used in European cities with the results as indicated in the table. It is true, however, that there is a greater general use of water for proper purposes in this country than in foreign countries.

5. Consumption of Water for Different Purposes. In studying the subject of the consumption of water it is desirable to consider the different uses of water under the following heads: (1) Domestic use; (2) Commercial use; (3) Public use; (4) Loss and waste.

(1) *Domestic Use.* Statistics collected from many sources where the supply has been actually measured by meter show that the amount of water used for domestic purposes will vary from about 15 to 40 gallons per capita; usually from 20 to 30 gallons. Where the supply is not metered, but is paid for according to the number and kind of fixtures in use, or the number of rooms in the house, the consumption may be several times the above figures. It has been known in some cases to go as high as 175 and 200 gallons per capita. Under these conditions it is difficult to predict what the consumption will be.

(2) *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 large cities the use for commercial purposes is likely to be more than in small cities. Various statistics show a consumption for these purposes of 10 to 40 gallons per capita. The nature of the industries will determine very largely this item.

(3) *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. The water used for street-sprinkling purposes is likely to be quite a large proportion of the total, as much as 10 gal-

lons per capita being used in some places. The average is, however, not more than one or two gallons per capita. For fire purposes the total consumption is relatively small, but during fires the rate of consumption is very high for a short time. The total consumption for public purposes may be estimated from 3 to 10 gallons per capita.

(4) *Loss of Water.* The chief cause of waste is bad plumbing and carelessness on the part of the private consumer, but this source of waste has already been mentioned under the first item. There is in addition considerable waste due to leakage of mains and reservoirs and minor uses of water, not included under the foregoing. It is estimated that at least 15 gallons per capita should be allowed for this item.

From the foregoing analysis it may be concluded that a reasonable estimate of the consumption of water where meters are largely used will be about 40 gallons as a minimum and 120 gallons as a maximum; 75 or 80 gallons may be taken as a fair allowance under average conditions. Where meters are not used extensively the statistics in Table No. 1 show that 200 gallons per capita would not be an excessive figure, but it is impossible under such circumstances to make a very close estimate.

6. Variations in Consumption. The foregoing sections 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.

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. The monthly variations in consumption for several places are illustrated by the data given in Table No. 3.

From the 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. 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.

Daily Variations.—The maximum daily rate is usually estimated at about 150 per cent of the average. In Table No. 3 very considerable differences are to be noted in the ratios for different places, these being caused by a variety of conditions, some accidental and some constant.

TABLE 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

The maximum daily rate will usually occur in the month of maximum consumption, and a rate considerably above the average for the month will occur 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 from 130 per cent to 140 per cent of the average, but for longer periods of time the rate will approach the monthly maximum.

Ordinary Hourly Variations. If there were no waste or leakage, the consumption during several hours of the night would be almost nothing and the consumption during several hours of the day would be two or three times the average for the twenty-four hours. It is a fact, however, that the rate of consumption at night is usually as much as 60 per cent of the average, and during the hours of maximum consumption it is not often more than one and a half times the average. Where waste is carefully prevented, and the consumption therefore low, the variation during the twenty-four hours will be relatively greater than where the waste is great and the total consumption great. Fig. 1 shows typical curves representing the hourly variation in consumption throughout the day. The curve for New York illustrates what occurs in a city where waste is fairly large, while that for Des Moines represents a case where consumption is small and the waste largely prevented. The average daily consumption per capita for New York was 100 gallons and for Des Moines 43 gallons.

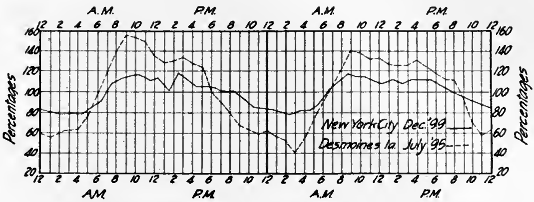


Fig. 1. Typical Curves Showing Hourly Variations of Water Consumption.

Consumption for Large Fires. The consumption for large fires must 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 Mr. Kuichling's estimate of the required number of fire streams.

If, for example, the average consumption is 100 gallons per capita, then the fire rate in per cent of the average will be as follows for different size 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 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.

The Combined Maximum Hourly Rate. In obtaining the total maximum rate of consumption at times of fires it is not necessary to assume that a great fire will occur coincident with the maximum use for other purposes. In fact, at times of great fires the use of water for many purposes would be interrupted. For average conditions the following may be taken as a reasonable allowance:

If the average daily rate is 100 per cent, then the maximum daily rate equals 150 per cent, and adding 20 per cent for increased consumption during day gives a total of 180 per cent. To this the fire consumption should be added by the use of the formula of the preceding paragraph.

Example. If the average daily consumption of a city of 20,000 inhabitants equals 80 gallons per capita, what will be the approximate maximum rate of consumption (a) for the day of greatest consumption, and (b), at the occurrence of a large fire?

From the foregoing discussion the maximum daily consumption may be estimated at 80 × 150 per cent = 120 gallons per capita.

From the above estimate the rate for ordinary use may be taken at 180 per cent of 80 gallons or 80 × 180 = 144 gallons. The fire rate $= \frac{1000}{\sqrt{20}} = 224$ gallons per capita. The total rate is therefore $144 + 224 = 368$ gallons per capita per 24 hours.

7. 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 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.

Probably the best way to estimate the population of a city for several years in the future is to take as a basis its growth in past years. If conditions do not change the per cent added each year or decade is apt to remain about the same. No one can predict closely the growth of a city, and in water-supply problems a close estimate is unnecessary.

Example. If the population of a city be 5,250 in 1880, 7,670 in 1890 and 11,400 in 1900, estimate the population in 1910.

The growth from 1880 to 1890 was 2,420, equal to 46 per cent, and from 1890 to 1900 it was 3,730, equal to about 49 per cent of the population in 1890. These figures show a steady growth, and it may be assumed that for the next decade the growth will be about the same, say 48 per cent. Then, 48 per cent of 11,400 = 5,472, and the estimated population in 1910 = 11,400 + 5,472 = 16,872.

The population for 1920 may be estimated in the same way, but the result will be much more uncertain than for 1910.

SOURCES OF SUPPLY.

8. **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 is at present furnishing many cities in the United States with a more or less satisfactory water.

The following table gives the number of waterworks in 1896 obtaining their supply from the various sources indicated.

Region.	Surface waters.	Ground waters.	Total.
Northeastern States	615	511	1,238
Southeastern States	143	180	340
North Central States	193	469	715
Western States	329	662	1,063
Total	1,280	1,822	3,356

SURFACE WATER SUPPLIES.

9. **Rainfall** is the source of all water supply, whether it be caught as it flows over the surface or is first allowed to percolate into the ground to furnish water for wells and springs. The amount

of rainfall is expressed in inches of depth upon a horizontal surface, snowfall being reduced to its equivalent amount of rainfall. 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 purposes continuous. Various forms have been devised, some weighing the water, others recording by volume.

Rainfall statistics for a large number of stations can now be readily obtained from the monthly reports of the Weather Bureau. 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.

10. **Mean Annual Rainfall.** The mean annual rainfall for a number of stations in the United States is shown in Table No. 4. The table also gives the ratio of the rainfall in the driest year, covered by the statistics, to the average.

The maximum rainfall is 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. The table also shows that in the driest years the rainfall is in most places only 50 to 60 per cent of the average. In the central and Western States the variation is greater than in the Eastern States.

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.

TABLE 4.

General Rainfall Statistics for the United States.

Station.	Mean yearly rainfall, Inches.	Per cent rainfall, driest year to mean rainfall.
Boston	45.4	60
New York	44.7	62
Philadelphia	42.3	70
Charleston	49.1	48
Jacksonville	54.1	74
Shreveport	48.2	67
Mobile	62.6	68
New Orleans	60.3	64
Vicksburg	52.7	70
Louisville	47.2	74
Cairo	42.6	62
Cincinnati	42.1	60
Cleveland	36.6	71
Marquette	32.3	69
Chicago	34.0	66
Milwaukee	31.0	66
St. Louis	40.8	55
St. Paul	28.2	53
Duluth	30.7	65
Omaha	31.4	57
North Platte	18.1	56
Denver	14.3	59
Salt Lake City	18.8	55
Spokane	18.6	73
Santa Fe	14.6	53
Yuma	2.8	25
San Diego	9.7	30
Los Angeles	17.2	33
San Francisco	23.4	51
Portland	46.2	67

11. Maximum Rates of Rainfall. In estimating the maximum flood discharges 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. Great rainstorms occur but rarely, but in hydraulic works where a failure would mean not only the destruction of property but often a great loss of life, it is necessary to provide against the greatest flood ever likely to occur. Accurate data of such floods must be based on many years of observation, but extraordinary rainfalls are likely to occur almost anywhere, and it may be assumed that what has happened in one locality may happen at any place in the same region. Examination of the data contained in the United States Weather Bureau Reports shows that in the Northern and Central States a rainfall at the rate of 4 inches for one hour and 8 inches for 24 hours represents the greatest rain likely to occur; in the South Atlantic and Gulf States these figures should be about 4 inches for one hour and 10 inches for 24 hours.

That excessive rainfalls are of sufficient extent to cover areas of such size as are ordinarily considered in water-supply problems is shown by the statistics of 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	"	21	"
8.90	"	30	"
8.44	"	42	"

FLOW OF STREAMS.

12. 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, but, 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.

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 5.

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.

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 watershed and the size of impounding reservoirs.

EXAMPLES FOR PRACTICE.

1. If a rain is falling at the rate of $\frac{1}{2}$ inch per hour, how many cu. ft. per sec. will this amount to over an area of 10 sq. mi.? 3,226 cu. ft. per sec. Ans.

2. If 1 inch of water is collected from an area of 20 sq. mi., how many days will this supply a town of 15,000 inhabitants using 100 gallons per capita daily?

The total amount of water collected = 2,323,200 $\times$ 20 = 46,-464,000 cu. ft. = 347,500,000 gal. This will last 231 days. Ans.

13. The Dry-Weather Flow. 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.

In Table No. 6 are given the minimum flows of several streams in different localities. It will be seen that the minimum varies greatly with the size of the stream and locality, and that streams of several hundred square miles of drainage area may have a minimum of zero.

TABLE 6.

Minimum and Maximum Flow of Streams.

Stream.	Place.	Drainage area. square miles.	Minimum flow, cubic feet per sec. per sq. mile.	Maxim'm flow, cubic feet per sec. per sq. mile.
	New England.			
Merrimack.	Lawrence	4,599	0.31	20.87
Connecticut.	Hartford	10,234	0.51	20.27
Nashua	Massachusetts. . .	109		104.5
Sudbury	Massachusetts. . .	78	0.036	44.2
	New York.			
Chemung	Elmira	2,055		67.1
Croton West Br. . . .	. . . .	20.37	0.016	54.43
	New Jersey.			
Delaware.	Stockton	6,790	0.17	37.5
Pequannock	. . . .	48		115
	Pennsylvania.			34.9
Perkiomen	Frederick.	152	0.39	
South Fork	Dam in Creole . Township	48.6		215
	Maryland.			131
Potomac	Cumberland	1,364	0.018	
	Illinois.			
Rock	Rockford	6,500	0.0158	
Des Plaines	Riverside	630	0	21.4

WATERWORKS PUMPING STATION AT ANDERSON, INDIANA

Here, in February, 1905, were carried out tests which finally demonstrated the possibility of the absolute removal from water of all disease germs of animal or vegetable origin, by means of a process using lime and sulphate of iron, with a small proportion of sulphate of copper. Over 40 cities in the United States have adopted this process for the purification of their water supply.

14. Flood Flow. 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.

Of great 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.

In Table No. 6 great variation in the maximum flow is observable, due partly to the varying rates of rainfall, but largely to the different characteristics of the streams. 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.

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

$$Q = \frac{200}{\sqrt[6]{M}} \qquad (1)$$

in which Q = discharge in cubic feet per second per square mile and M = area in square miles. It gives results probably somewhat too low for small areas.

Example. What will be the flood flow according to formula 1 for a drainage area of 10 square miles?

The flow will equal $\frac{200}{\sqrt[6]{10}} = 136$ cu. ft. per sec. per sq. mi.

TABLE 7.

Statistics of the Yearly Flow of Streams.

Stream.	Area drained, square miles.	Average yearly flow.		Dry year flow.	
		Rain, inches.	Flow, per cent of rainfall.	Rain, inches.	Flow, per cent of rainfall.
Cochituate	18.87	47.08	43.2	31.20	31.3
Croton	338.0	48.38	50.8	38.52	37.8
Genesee	1,060	39.82	32.5	31.00	21.5
Perkiomen	152	47.98	49.2	38.67	40.4
Potomac	11,043	45.47	52.7	37.03	39.2
Savannah	7,294	45.41	48.9	43.10	37.7
Upper Mississippi	3,265	26.57	18.4	22.86	7.1

15. Annual Discharge. Table 7 gives some statistics of the annual flow of streams as compared to rainfall. It will be seen that in the dry years the percentage running off is much less than in the average year. From these data and other statistics it is estimated that for a stream of average conditions east of the Missouri and Mississippi Rivers the percentage of rainfall flowing off for different annual rainfalls is about as follows:

Rainfall, inches.	Per cent running off.
20	25 to 35
30	30 to 40
40	35 to 45
50	40 to 50

In the nature of the problem there is a wide variation in percentage due to variations in the conditions of the watershed, climate, etc. Whatever tends to promote evaporation from the watershed decreases the run-off. 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. 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. All these things must be considered in estimating the flow of a stream from rainfall data and from statistics of the flow of other streams.

Examples. Estimate the flow of a stream during dry years where the average annual rainfall is known to be 40 inches.

The rainfall for a very dry year may be taken from Table No. 4 at say 60 per cent of the average or $40 \times .60 = 24$ in. For a rainfall of this amount the per cent running off will probably be between 25 and 35. If this is an average watershed we may put it at about 32 per cent. The run-off will then be estimated at 24 × .32 = 7.7 inches.

Note. The wide variation in percentage indicates that such estimates as this are very uncertain. Actual stream measurements are the only safe guide.

2. How much water can probably be collected in a dry year from an average watershed where the rainfall in very dry years is 30 inches?

By the estimates of section 15 it is probable that at least 33 per cent will run off or can be caught in a reservoir. This amounts to 30 × .33 = 9.9 inches.

By Table No. 5 this amounts in gallons per sq. mi. to 9.9 × 2,323,200 × 7.48 = 172,000,000 gallons.

16. Monthly Variation in Stream Flow. During dry years very little water can be collected from summer rains. Dependence must be had on winter snows and spring rains for filling storage reservoirs and nearly all the yearly supply will be caught in the months from December to May inclusive. During average seasons a large proportion of the stream flow occurs in the summer months. Generally about three-fourths of the yearly flow occurs in the months from December to May and only one-fourth from June to November, whereas in very dry years the summer flow may be considered as practically nothing.

17. Quality of Surface Waters. Surface water supplies are drawn from two general sources—rivers and lakes. River supplies may be divided into those obtained directly from large rivers and those obtained from impounding the flow of small streams in reservoirs. The quality of surface waters may be considered with reference to: (1) appearance, (2) mineral content, (3) the presence of disease-producing organisms.

(1) The appearance of a water is affected by the presence of clay and sand in suspension, rendering the water turbid, and by certain vegetable material giving the water a distinct color. Turbidity varies according to the nature of a watershed. While a turbid water is very objectionable for household use it cannot be said to be actually dangerous. Turbidity is removed by allowing the water to rest in

reservoirs, thus permitting the clay to settle, or by passing the water through filters. 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.

(2) While flowing surface waters do not dissolve so much mineral matter as ground waters, yet they take up an appreciable amount, depending considerably on the character of the soil over which they pass. A large part of the mineral content is usually carbonate of lime. In general surface waters are preferable to ground waters as regards their mineral content, a hard water (one containing lime) being less desirable for culinary and manufacturing purposes.

(3) The most important question relating to the quality of a water is whether it is dangerous to the health. It has been well demonstrated that certain diseases, particularly cholera and typhoid, are caused by certain minute organisms called bacteria. These inhabit the intestinal tract of persons sick with the disease and are present in enormous numbers in the sewage wherever these diseases exist. Whenever such sewage or drainage gets into the water supply of any town an outbreak of the same epidemic is sure to appear. Many cases are on record of whole villages being affected through the contamination of the water supply by a single diseased person. From such facts it is seen that the quality of a water supply from this point of view is exceedingly important.

A surface water supply can be absolutely safe only when it is drawn from an uninhabited area. A few scattered farm houses, if not located too near a water course, are not likely to cause serious pollution. But where the watershed is quite populous, and especially where villages are located in the valleys, the danger of the transmission of disease through the water supply is very great.

The danger in the use of water from a large stream depends on the amount and nearness of the pollution. All large streams receive more or less drainage from towns and cities, but if such pollution is relatively small and remote the danger is small. As a rule a surface water supply is not free from danger unless the water is artificially purified by some adequate means, but many large cities in the United States continue to use water supplies which are badly contaminated.

The result of such use is shown in the relatively high death rate from typhoid fever in such places.

The quality of lake supplies is likely to be better than that of rivers. Such water is usually quite free from turbidity, as the sediment brought into it by the tributary streams soon settles; and unless polluted by sewage in the immediate neighborhood, it is likely to be relatively safe from a sanitary point of view. Experiments show that in the settling of the clay and sand particles, the bacteria settle to a great extent, and a marked purification takes place in a polluted water. For the same reason that lake water is better than river water, it is true that a supply from a small stream is usually improved by storage in a large storage reservoir. Sometimes, however, vegetable growths occur in reservoirs which give to the water a disagreeable odor.

GROUND WATER SUPPLIES.

18. Occurrence of Ground Water. The rain which falls upon the ground is disposed of in three ways: A part flows off immediately in the streams, a part is evaporated from the ground and vegetation, and a part percolates into the soil.

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 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.

19. 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 downward and lateral flow which will on the average be equal to the percolation. In surface streams a very light inclination is sufficient to cause a rapid movement of water, but in the ground the resistance to movement is so great that a 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. 2. At the margin of streams as at A and B the level

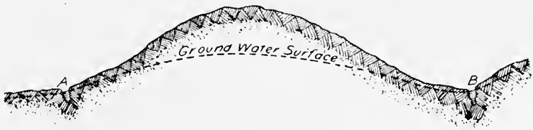


Fig. 2. Relation of Ground Water to Surface Water.

of ground and 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 slope.

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.

20. Porosity of Soils. All soils and rocks near the surface of the earth are capable of absorbing more or less water. In sand of a fairly uniform size the porous space is commonly from 35 to 40 per cent of the entire volume. 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 most favorable formations 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.

21. The Flow of Ground Water. It has been explained in the previous section that the water in the ground has in general a slow rate of flow through the ground. Where a supply of ground water of considerable amount is to be obtained this rate of flow is of much importance. To get water from a ground-water "stream" is exactly similar to the taking of water from a surface stream; in both cases the flow of water in the stream must be at least equal to the proposed draught or the supply will be inadequate. The notion is quite common that in many places the water in the ground is inexhaustible. This is an entirely mistaken idea as is well illustrated by the gradual failure of many ground-water supplies.

Ground water in large quantities is usually obtained either from large gravel deposits of comparatively small depth, forming broad underground streams, or from extensive deposits of porous rock like sandstone, the latter source being tapped by deep wells many of which are the well known "artesian" wells. In the case of a gravel deposit near the surface it is often possible to estimate the quantity of water actually flowing through a given section of the deposit.

The best method of estimating capacity of a ground-water source 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 which will be shown when the level of the water in the trial well ceases to lower. 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.

To do this we must estimate the velocity of flow, the cross-section of the porous stratum containing the water, and the percentage of porous space.

The rate of flow of ground water streams is very small compared to that of surface streams. It depends on the slope or inclination of the ground and upon the size of the grains of sand or gravel through which it passes. The following table shows about what the velocities are likely to be for various slopes and conditions of soil.

TABLE 8.

Velocities of Flow of Ground Water in Feet Per Day.

Material.	Slope of ground, feet per mile.					
	10	20	30	40	50	100
Fine Sand	0.2	0.4	0.6	0.8	1.0	2.0
Medium Sand	1.5	3.0	4.5	6.0	7.5	15.
Coarse Sand	4.0	8.0	12.0	16.0	20.	40.
Fine Gravel, free from sand ...	20–40	40–80	60–120	80–160	100–200	200–400

The velocity of flow having been determined it remains to estimate the actual quantity of water flowing through a given territory. Of the total volume of a body of sand or gravel, the water will occupy only about 25 to 30 per cent. The actual volume of water, therefore, which will pass through a given section will be only 25 or 30 per cent of the amount were it solid water. If v = velocity of flow in feet per day, A = area of cross-section of the porous bed at right angles to the direction of flow, then assuming a porosity of 25 per cent or $\frac{1}{4}$th, the actual volume of flow per day will be in cubic feet

$$Q = \frac{1}{4}\, vA \qquad (2)$$

Thus suppose we have a porous bed of coarse sand in which the water is 10 feet deep, the bed is 500 feet wide and slopes 20 feet per mile. The velocity of flow by Table 8 may be taken at about 8 feet per day. The cross-section $A = 10 \times 500 = 5{,}000$ square feet. Hence the volume of flow will be approximately $\frac{1}{4} \times 8 \times 5{,}000 = 10{,}000$ cubic feet per day, or about 75,000 gallons. This being the total rate of flow through the sand it is evidently the greatest amount of water that could be extracted from this sand deposit by means of any system of wells or other devices. To give the above results the bed must be of considerable length and the water in it must be about the same depth throughout and have the same slope as the surface.

SPRINGS.

22. 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.

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. 3 represents such conditions, the ground water escaping at the outcrop of the impervious material thus forming a spring. The porous stratum may be sand or gravel, or a porous rock; while the impervious layer is usually clay, or rock of an argillaceous character.

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 underlaid by slate. The largest spring, the Kaiserbrunnen, has an average flow of about 150 gallons per second, varying from 60 to about 250.

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

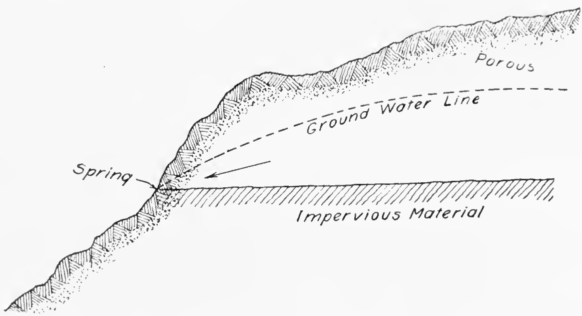


Fig. 3. Formation of Springs.

where the overlying impervious material is broken, or through a fault, or it breaks through at places where it is not sufficiently strong or compact to resist the upward pressure.

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. Conditions of this sort 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.

Third Class. The third class of springs 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. Such conditions also give rise to marshy places at the foot of hills and even on side hills.

23. The 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.

Springs of the first class will vary in yield with the variations in ground-water level and, therefore, will vary with the rainfall, but will not wholly cease to flow if the water is intercepted by suitable constructions.

Springs of the second class are apt to be much less affected by variations in rainfall than either the first or the third class.

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.

ARTESIAN WATER.

24. General Conditions. Whenever a water-bearing stratum dips below a relatively impervious one the former becomes in a sense a closed conduit or pipe, 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 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.

Fig. 4 shows an ideal condition for artesian or flowing wells. If A B is a porous stratum outcropping at A and B and covered by an impervious stratum of clay or impervious rock, water entering at A could escape at the lower end 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 A B. Water would rise to this line in a tube sunk to the porous stratum, and a flowing well would be possible wherever the surface of the ground lies below this line.

Actual conditions may be much modified from those represented in Fig. 4, as where the water is prevented from flowing out at B by reason of an increased density of the stratum or by the stratum becoming thinner. The effect in causing the water to exert an

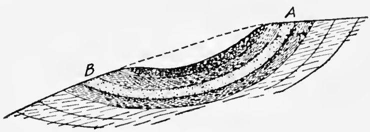


Fig. 4. Dip in Water-Bearing Stratum.

upward pressure is, however, the same. 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; and many instances are known of large springs which occur at points where the overlying stratum is broken as noted in the preceding section. 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. But wells sunk to tap an artesian stratum must not be placed too close together else they will interfere with one another and the yield per well will be reduced.

25. 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.

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.

QUALITY OF GROUND WATER SUPPLIES.

26. The quality of ground waters is, in general, quite different from that of surface waters. By percolating through the ground, practically all suspended matter is filtered out, and ground waters are usually clear and sparkling. At the same time this very filtration process causes the water to dissolve more of mineral substances, and the result is that ground waters usually contain much more mineral matter than surface waters. In a limestone country the ground water will be hard, as it will contain lime, and where the soil contains alkali the water will be changed with it. Water that contains little beside lime is not especially objectionable for drinking purposes, but for most other purposes it is more or less expensive and troublesome. An alkali water may be quite unusable.

As regards disease organisms a ground water is likely to be quite free on account of the filtering action of the soil. In the case of private wells, often located near outhouses, pollution is much more likely to occur than in public supplies where any source of pollution must be quite remote.

The temperature, odor and taste of ground waters are generally much more satisfactory than of surface waters. Ground waters constitute a most valuable source of supply for small cities and towns, and where such a supply can be had it should almost always be chosen in preference to a surface water.

CONSTRUCTION OF WORKS.

Before passing on to the details of waterworks construction it will be of assistance to obtain a general view of the subject, and to that end we will here briefly outline the various general features which go to make up a waterworks system.

27. Classification. The various constructive features of a water supply system may be divided into three groups—works for the collection of water; works for the conveyance and distribution of water; works for the purification of water.

28. 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 or 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 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 readily be 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.

29. 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, or 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.

30. 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. 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.

RIVER AND LAKE INTAKES.

In drawing a water supply from a large river or lake a pipe or tunnel must extend from the pumping works out some distance from shore and the construction of such pipe line or tunnel often involves some very difficult work.

31. River Intakes. The location of the point of intake must be selected with reference to (1) the quality of water, and (2) the cost of construction and maintenance of the works connected therewith. The point of intake should 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. The location of the intake must also be determined with special reference to the lowest water stage.

The form of construction to be used depends upon the character of the stream in question, especially whether the difference between low and high water level is small or great.

If the water level vary only a small amount, as in the case of streams near dams or near a lake or ocean, 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.

The intake pipes, usually of cast iron, may lead directly to the pumps, thus acting as suction pipes, or to a gate chamber and pump well. In the latter case the suction pipes of the pumps lead from this pump 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 are inserted in the gate house or placed over the ends of the suction pipes.

If there is a large fluctuation of water level considerably more work is involved. It is usually 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. Furthermore, pumps cannot lift by suction more than about 20 feet in practice, hence in order to enable the pumps to reach the water at the lowest stage, 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.

Another form of construction at the end of the intake is a masonry tower extending above high water and containing ports and sluice-

IMPROVEMENT OF THE MISSISSIPPI RIVER

Upper picture, a view of the Des Moines rapids and canal at Keokuk, Iowa; Burlington tracks alongside the canal. Lower picture, a view of the middle lock of the Des Moines rapids canal; government dry dock at right. All of these works, which cost the U. S. Government $5,000,000, will be submerged about 40 feet as a result of the construction of a 6,000-foot dam to be thrown across the Mississippi at this point.

gate similar in form to those used in reservoirs. 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 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. Such intake towers are used at St. Louis and at Cincinnati and tunnels connect with the tower through which the water is conveyed to the pumps.

The tower has the advantage over the crib construction in permanence and reliability. 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 or tunnel usually runs to a screening chamber or pump well and from this chamber suction pipes lead direct to the pumps.

Fig. 5 illustrates a good design for small works. Here the water flows by gravity to the wet well made of boiler steel and con-

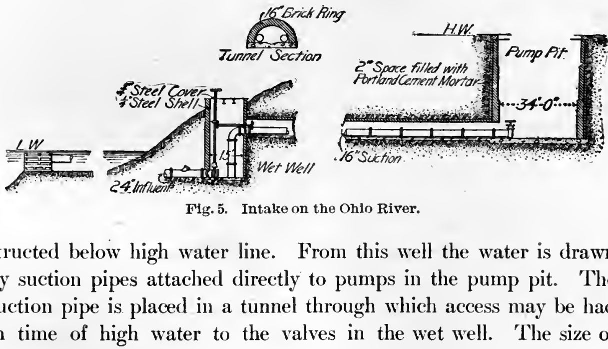


Fig. 5. Intake on the Ohio River.

structed below high water line. From this well the water is drawn by suction pipes attached directly to pumps in the pump pit. The suction pipe is placed in a tunnel through which access may be had in time of high water to the valves in the wet well. The size of intake pipe and suction pipe should be such that the velocity of the water in them will not exceed $1\frac{1}{2}$ to 2 feet per second.

32. Lake Intakes. 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 lake bottom and in conveying sediment from point to point.

The intake should if practicable be located at a sufficient depth to be free from any considerable wave action, both to secure a greater stability and to avoid the effect of the disturbance of the sediment by the waves. 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 the water is not too stagnant, 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.

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.

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.

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. Various methods of laying submerged pipe are described later.

Most lake intakes are protected at their ends by submerged crib work of timber partly filled with stone, the end of the pipe being raised 6 or 8 feet above the lake bottom to prevent the entrance of sand. At some of the larger ones, as at Chicago and Cleveland, large inlet cribs or towers built above the water surface are used, similar to river inlet towers.

The greatest difficulty met with in operating lake intakes is due to the clogging of the ports by anchor ice. This consists of needles and thin scales of ice which form in moving water and which are of such small size that they are readily carried below the surface by comparatively weak currents. They cease to form after the body of water has become frozen over. On coming in contact with submerged objects these particles of ice adhere and soon form large masses difficult to dislodge. Anchor ice has given much trouble at lake intakes both at the exposed cribs and at the shallower submerged ones. It is removed in various ways. Compressed air discharged near the port has been effective in some submerged intakes. Steam, water from hose, chains drawn back and forth through the ports, and pike poles, are some of the other means used. As tending to obviate difficulty with anchor ice all crib openings or port holes should have a large area so that the velocity of flow through them will not be more than 3 or 4 inches per second.

WORKS FOR THE COLLECTION OF GROUND WATER.

33. Collection of Water from Springs. 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.

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.

Mineral and other springs occurring in public places usually have open basins, and opportunities are offered in the walls and parapets for ornamentation.

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. If 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 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 may be made similar to that shown in Fig. 12, 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.

The gallery may be simply a line of drain tile or vitrified pipe laid with open joints at the upper part of the pipe. If large quantities are collected the gallery may be made of brick or stone and large enough to permit of the passage of a man.

Where springs originate in a deep porous stratum such stratum may usually be tapped by wells without much reference to the spring. Use of such wells will generally reduce or entirely stop the flow of the spring.

THE CONSTRUCTION OF WELLS.

34. **Principles Governing the Yield of Wells.** If a well, either large or small, be sunk into a body of water-bearing material the water will run into such well, and if no pumping is done the water will, after a time, reach a level in the well the same as the level of the water in the surrounding soil. Fig. 6 represents a section through such a well. The dotted line A B represents the level of the water in the ground and in the well. Now if water is pumped from this well the level of the water therein will be lowered and as a consequence water will tend to flow into it from the surrounding ground and the surface of the ground water will assume some such shape as shown by the full line C D E F. The amount which the water 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 pumping is continued the level will be more and more lowered until it is so low that water will run into the well as rapidly as it is pumped out, after which no further change will take place. If the pumping ceases the well will gradually fill up to the original level.

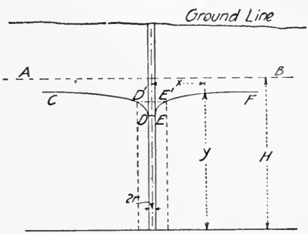


Fig. 6. Well Sunk in Ground Water.

Where the water flows under pressure, as 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 water, but the curve of *pressures* is of a form similar to the water surface in the case already treated.

The principles underlying the yield of wells have been investigated both theoretically and practically, but the subject is too difficult to be discussed in detail here. There are certain general principles, however, that are very important and which aid greatly to a clear understanding of the behavior of a set of wells under varying conditions. These may be stated as follows:

Having given a sand or gravel stratum of at least several feet in thickness in which water is flowing at some appreciable slope, such as 5 or 10 feet per mile, and a well is sunk into this stratum to a considerable depth, the yield of such a well when pumped from continually will follow approximately the following laws:

(1) The yield will be proportional to the distance the water level is lowered in the well below its normal level.

(2) The yield will be proportional to the thickness of the water-bearing stratum.

(3) For the same amount of lowering of the water the yield will be a little greater the larger the well, but the difference is not great except in case of very deep wells of small diameter in which the upward velocity of flow through the well is greater than 2 or 3 feet per second. A 10-foot well will yield only about 50 per cent more than a 6-inch well.

(4) For the same amount of lowering of the water the yield will be much greater in coarse material than in fine.

The following table will serve to give a rough idea of what may be expected from a single well sunk at least half way through a water-bearing stratum of various grades of material.

TABLE 9.

Approximate Yield of a 6-Inch Well When Sunk Into Water-Bearing Material 10 Feet Thick and When the Water Level Is Lowered One Foot by Continuous Pumping.

Material.	Yield in gallons per day.
Fine Sand	4,000
Medium Sand	30,000
Coarse Sand	80,000
Fine Gravel, free of Sand	500,000 or more.

For other thicknesses of material and other amounts of lowering the yield can be obtained by the law of proportion as stated above. The great increase in yield due to increasing coarseness of material is very marked and shows that for this very reason it is very difficult to make close predictions as to yield. Larger wells will give slightly better results.

Example. A well is sunk into a water-bearing stratum consisting of medium size sand to a depth of 30 feet below water level. What will be the yield if the water therein is pumped down 5 feet below its normal level?

By Table 9 the yield would be about 30,000 gallons per day for a 10-foot stratum and one foot of lowering. Hence for a 30-foot stratum and 5 feet of lowering the yield will be about $3 \times 5 = 15$ times as much, or $15 \times 30{,}000 = 450{,}000$ gallons per day.

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.

The amount of this interference depends mainly upon the distance the wells are apart. It also depends upon the amount the water is lowered by pumping, and upon the general capacity of the stratum. If the water is lowered a considerable amount, such as 10 feet, the wells should be placed 200 to 400 feet apart in order that the interference be not too great. A small spacing like 25 to 50 feet will give an interference of a large amount,—often as great as 50 per cent in the case of 3 or more wells. That is to say, if 4 wells are placed 50 feet apart the total yield is not likely to be more than 50 per cent of the yield if these 4 wells were placed 300 or 400 feet apart.

Where it can be done, the best way to determine the capacity of wells 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.

In the case of deep wells sunk into rock strata it is impossible to make an analysis of the conditions so as to be able to predict the yield. A pumping test is a necessity, but in this case also a very useful principle to remember is that of proportionality of flow to the lowering of the water level in the well. Thus if by pumping the level down 10 feet we get 200,000 gallons per day we may say with great certainty that the yield will be about 400,000 gallons if the water is pumped down 20 feet. In all cases this lowering of the water is to be measured from the level to which it rises when no water is pumped. In a flowing artesian well to get this level it is necessary to extend the casing above the ground as far as the water will rise, or to cap the well and determine this level by a pressure gauge.

35. Large Open Wells. As already explained, the yield of a well that is constantly pumped from is not much affected by its size. For other reasons, however, large wells are often advantageous.

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.

Large wells for waterworks 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 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.

In the construction of a large well large quantities of water will be met with, and 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

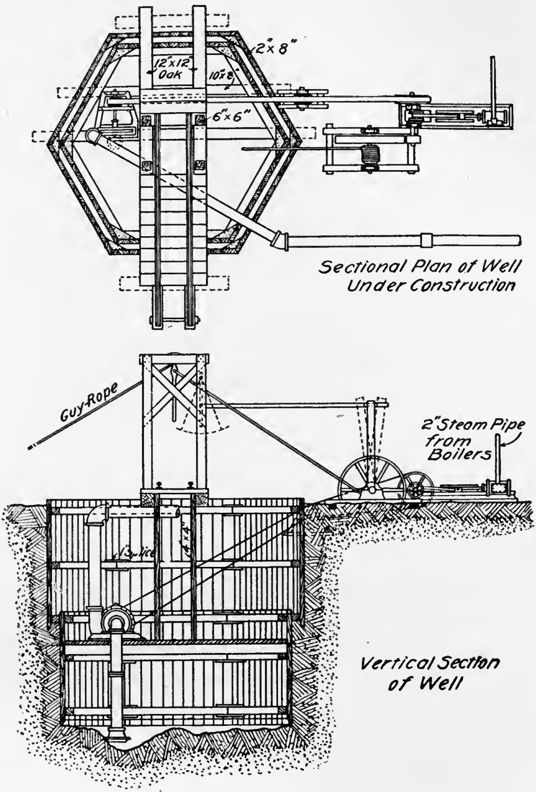


Fig. 7. Large Well under Construction.

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. 7.

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. 8 are illustrated two forms of shoes used in sinking wells. These are both constructed mainly of wood. To strengthen such curbs iron rods should 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.

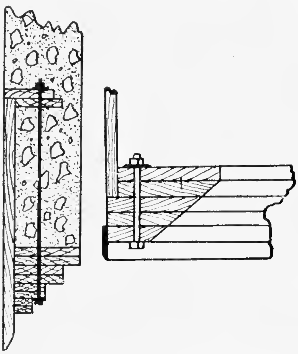


Fig. 8. Shoes for Sinking Well Curbs.

The lining or curb usually consists of a circular wall of brick or concrete masonry of a thickness varying with diameter and depth of the well, but ranging ordinarily from 2 to 5 feet. Dry rubble may be used for the lower portion, but the upper portion should be of concrete.

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.

36. Shallow Tubular or Driven Wells. 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. 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. We will here consider only the methods of sinking wells in earth or soft strata.

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.

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 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.

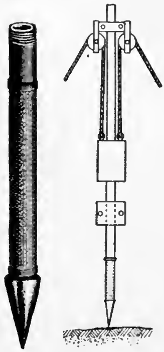


Fig. 9. Well Point and Driving Rig.

Fig. 9 shows a common form of well point and the method of driving wells by means of a weight operated by two men. The tube may also be driven by a wooden block operated by a pile driver or other convenient means. Such a well 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.

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.

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.

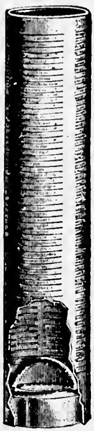


Fig. 10. Cook Well Strainer.

Fig. 10 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.

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.

In order to enable the pumps to draw as much water as possible from the wells the pumps and suction main should be placed as

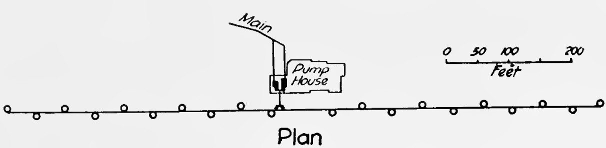


Fig. 11. Typical Arrangement of Wells.

deeply in the ground as practicable. A typical arrangement of wells is shown in Fig 11. In this plan the wells are 6-inch wells and are spaced 50 feet apart and are 35 to 50 feet deep.

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. 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.

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. 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. 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 steel placed on the suction pipe near the pumps through which the water passes at a slow velocity. A vacuum pump is attached to this drum.

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 drum or box inserted in the suction pipe and provided with suitable handholes for cleaning.

37. Deep and Artesian Wells. Where the depth exceeds 75 to 100 feet the small driven well is no longer practicable. 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. 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.

In soft material it is 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. 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, 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. 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 1,000 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 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 lowerd 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.

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.

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.

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 2,000 feet. For smaller sizes the cost will be somewhat less, especially for the shallow depths.

38. Connections for Deep Wells. The economical spacing for deep wells will be much greater than for shallow wells. It will likewise pay to spend more money in lowering the flow line by making deep connections, thus decreasing the number of wells and increasing the spacing. Generally speaking a spacing of from 400 to 800 feet will be found desirable.

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 considerable 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. This method is applicable to any depth, but involves the use of uneconomical types of machinery. The air lift is another form suited to this work.

39. Yield of Artesian Wells. 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. 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. Often a well or set of wells will show a gradual falling off in capacity. The chief cause of a decrease in the yield of a well is the influence of other wells sunk in the vicinity. Where large numbers of wells are sunk in the same region this effect may be very serious, as in some cases where it has reduced the pressure of flowing wells from 75 or 100 feet down to nothing.

40. Galleries and Horizontal Wells. 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 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

AN ARTESIAN WELL SPOUTING

View taken in Australia, showing a typical example of the spouting bores which are transforming vast areas of that continent from parched desert into the richest and most fertile of pastoral and agricultural regions. Australia has long been known as the driest of all the continents; but it has recently been discovered that the rock strata underlying the greater part of the country are storage reservoirs of inexhaustible water supply. Strangely enough, Victoria which is itself watered by fine rivers, is devoid of these sources of underground supply

water. Fig. 12 illustrates a form of gallery of concrete built in a water-bearing gravel.

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; or it may occur where a steeply inclined artesian stratum can be more readily reached in this way than by vertical wells. Tunnels or galleries are also sometimes run from the bottom of large wells for the purpose of increasing the yield. This method of increasing the flow is advantageous where it is necessary to lower the pumps and to concentrate the flow in a single well.

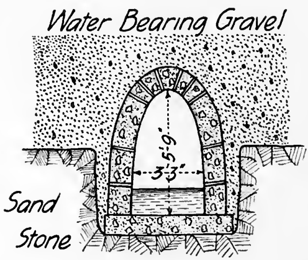


Fig. 12. Concrete Gallery.

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 lake or river.

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 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.

Wells and galleries are often constructed near streams for the purpose of getting all or a portion of the supply therefrom. 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.

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, 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.

RESERVOIRS AND DAMS.

Impounding Reservoirs.

41. Capacity. When the minimum flow of a stream is less than the daily demand of water it is necessary to store up the excess flow during the rainy season in large reservoirs called *impounding reservoirs.* The deficiency in the supply can then be made up by drawing from the reservoir. In this way the entire flow of a stream for a year or more may be stored and drawn off as wanted and streams that run dry at certain times may be made to supply quite a large population. Impounding reservoirs are made by constructing a dam across the valley in question, but natural lakes or ponds can often be utilized as reservoirs by building suitable works at this outlet.

In calculating the proper size of a reservoir we must consider (1) the yield of the source for successive intervals of time; and (2) the demand for all purposes for like intervals of time. The yield of the source of supply has been previously discussed. The demand to be considered includes not only the consumption for the city in question, but also the loss of water by evaporation 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 amount of leakage through the dam will usually be very small, but with certain forms of construction may be large. 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.

The capacity of the reservoir must be based on the supplying capacity of the stream during the dryest year. The probable yield of the stream for each month of such a year should be estimated and recorded. Then likewise the monthly demand for the city in question and whatever allowance, if any, should be made for the use of riparian owners below.

Then for all months in which the demand is greater than the flow subtract the latter from the former; this will give the deficiency for each month. Add all deficiencies together and the result will be the total deficiency which must be made up from the reservoir and therefore is the required capacity of the reservoir, provided, however, that the total surplus for the remaining months is at least equal to the deficiency. If not, then the total yearly flow of the stream is not equal to the total demand and additional water must be obtained from some other source.

42. Location. In determining upon the location of a reservoir several elements must be kept in mind. In the first place it is very desirable that it shall be at such an elevation that at least a part of the consumers may 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.

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. 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.

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. 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.

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. The volume up to any given contour having been determined, the necessary height of dam to hold any given quantity of water becomes known.

All vegetation and perishable matter should be removed from the reservoir site, as the decay of such material injures the quality of the water. It is also 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.

As a further protection to the quality of the 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. Shallow places should either be excavated to give a depth of 6 or 8 feet, or partly excavated and partly filled, the slopes being formed at about 3 to 1 and covered with sand or gravel.

43. 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. During seasons of low water, opportunity is offered for removing the vegetation from around the borders of the reservoir.

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 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.

EARTHEN DAMS OR EMBANKMENTS.

44. 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.

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. 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, due to faulty construction. For dams higher than 100 feet or thereabouts few engineers would recommend an earthen structure. If the foundations are suitable, a masonry dam is in such cases greatly to be preferred. It is more reliable, and with the great pressures occurring it is desirable to have all outlet arrangements built in masonry.

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. 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.

Earthen dams are of a trapezoidal form with top width, side slopes, etc., proportioned according to the material used. 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 materials towards the lower face. Where good material is scarce, imperviousness is usually obtained by means of a wall or "core" of impervious earth or masonry placed near the centre of the dam: If impervious foundation is reached only at a considerable depth, this portion only of the embankment is carried to the extreme depth.

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 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.

45. Core Walls. For a puddle wall of clay 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.

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.

Masonry core walls are made of various widths. Sometimes in case of embankments made of good material, they are made only a foot 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.

Figs. 13, 14 and 15 show cross sections of several forms of embankments. Fig. 13 is without core wall except in the trench, Fig. 14 has a core wall of puddle and Fig. 15 one of concrete.

46. Dimensions of Embankments. 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 a slope of 2 to 1 is 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 1½ to 1 would be suitable. On high embankments, bermes placed 30 to 40 feet apart vertically are a desirable feature.

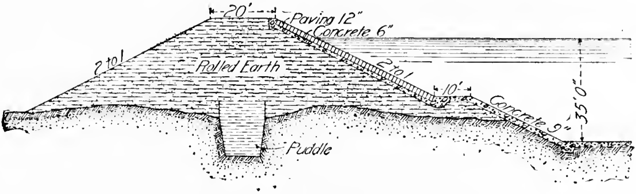


Fig. 13. Reservoir Embankment, Syracuse, N. Y.

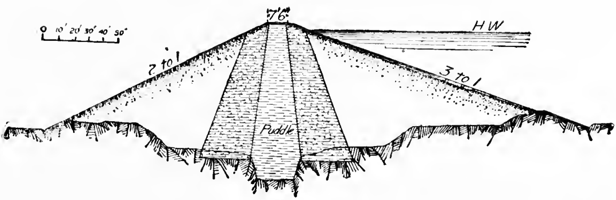


Fig. 14. Reservoir Embankment, Glasgow Waterworks.

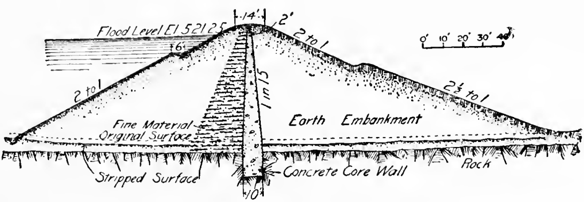


Fig. 15. Reservoir Embankment, Boston Waterworks.

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.

The width of top is frequently fixed by the 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.

47. Construction. 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 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. 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.

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. 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.

Much importance is attached to the work of compacting, and only by the best of supervision can 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.

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. 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.

48. 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.

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.

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 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. 16.

For some reasons an open culvert is much to be preferred to a simple 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 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. 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.

Figs. 16 and 17 show the two general methods of laying pipes through embankments.

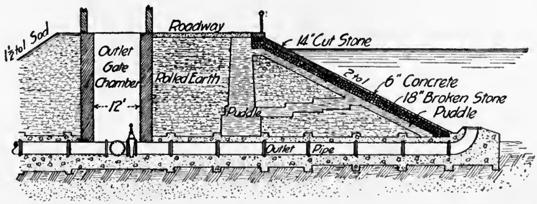


Fig. 16. Section Through Minneapolis Reservoir.

49. 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. 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. Fig. 16 shows the gate chamber placed near the middle of the embankment, while Fig. 17 shows it placed at the upper end of the outlet pipe. One advantage of the latter arrangement is that water may be drawn from different levels so as always to get water of the best quality. Fig. 18 shows a gate chamber for a small works located as in Fig. 17.

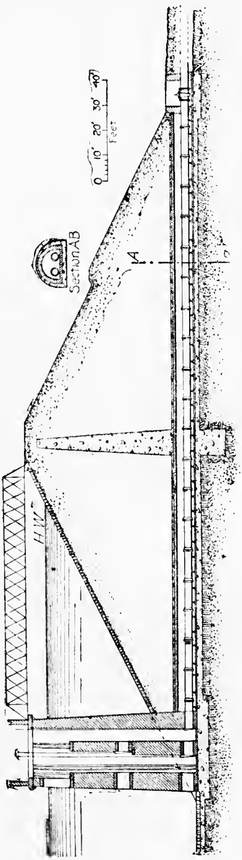


Fig. 17. Section Through Embankment and Gate Chamber.

The masonry of the chamber is usually of heavy rubble, faced with ashlar and lined with hard brick or cut stone. It should be laid in rich Portland-cement mortar to secure imperviousness. The walls will vary in thickness with their unsupported length, or the size of interior chamber, but 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. The foundation should be prepared with great care, as unequal settlement is liable to occur, causing cracks in the masonry of the culvert and displacing the outlet pipes.

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 sections 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.

50. 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, where, on account of insufficient wasteway, an earthen embankment was destroyed, resulting in the loss of over 2,000 lives and the destruction of property valued at 3 to 4 million dollars.

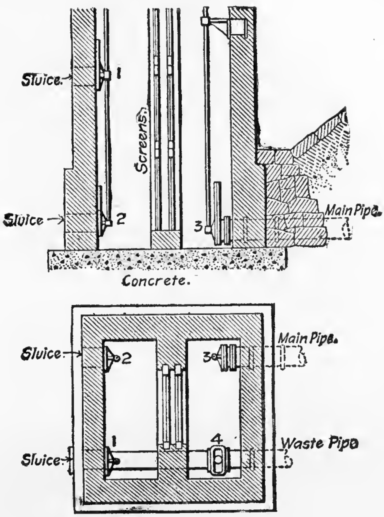


Fig. 18. Gate Chamber, Ipswich, Mass.

In section 14 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 wasteway.

(2) The wasteway may sometimes be formed at some low point in the dividing ridge, and the water led to another valley.

(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 section 54. 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 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 the weir formulas as given in "Hydraulics." 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 main part of the dam, while the weir crest determines the storage capacity.

MASONRY DAMS.

51. 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.

Masonry dams are designed in accordance with theoretical considerations so as to fulfill the following conditions: (1) The dam must not overturn or slide on its foundation, and (2) the pressures in the dam or foundation must be within safe limits.

The first consideration will govern the design of all dams up to a height of 100 feet or more and are therefore the only considerations which will be taken into account here.

Dams up to 30 or 40 feet in height are usually made trapezoidal in form, the saving obtained by making the faces curved or on broken lines not being enough to justify the extra trouble.

Let A B D C, Fig. 19, 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' = weight of a unit volume of masonry. Let g = specific gravity of the masonry $= \frac{w'}{w}$.

The water pressure is represented by P and the weight of the dam by G, and it is assumed that the water level is at the top of the dam We will consider a length of dam of one foot. The height is known and the top width a and front batter m are assumed. Usually the front face A C is made vertical, or at a slight batter of one inch to the foot or so. In the former case $m = 0$ and in the latter $m = \frac{1}{12} h$.

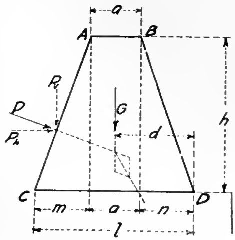


Fig. 19. Trapezoidal Dam.

From principles of mechanics we find the following value of the width of base l,

$$l = \sqrt{A + B^2} - B \qquad (3)$$

in which

$$A = a^2 + 2\,a\,m + \frac{h^2}{g} + \frac{m^2}{g}$$

and

$$B = \frac{m}{g} - \frac{m}{2} + \frac{a}{2}$$

In solving problems the numerical values of A and B should first be obtained and these values then substituted in the formula (3) for l.

If $m = 0$, as for a vertical face, then

$$l = \sqrt{\frac{5}{4}\,a^2 + \frac{h^2}{g}} - \frac{a}{2} \qquad (4)$$

Example. What width of base will be required if $h = 20$ ft.; $a = 5$ ft.; and the weight of masonry be considered 2.3 times the weight of water, or $g = 2.3$. Let value of m be 2 feet, giving a batter of 1 in 10.

Getting first the values of A and B we have $A = 5 \times 5 + 2 \times 5 \times 2 + \frac{20 \times 20}{2.3} \times \frac{2 \times 2}{2.3} = 221.$ $B = \frac{2}{2.3} - \frac{2}{2} + \frac{5}{2} = 2.37.$

Then $l = \sqrt{221 + 2.37 \times 2.37} - 2.37 = 12.7$ ft. Ans.

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. The simplest way of calculating the section of such a dam (up to a height of 100 feet at least) is to treat it at first as similar to the form previously considered, but with a vertical upper face and top width of 0. Then the formula for bottom width becomes

$$l = \frac{h}{\sqrt{g}} \qquad (5)$$

This gives the triangular section A B C in Fig. 20. This form can then be modified by adding a suitable width a at the top and joining the point F with the sloping face A C by a smooth curve F D.

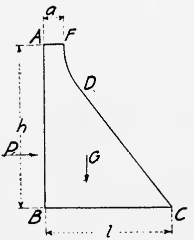
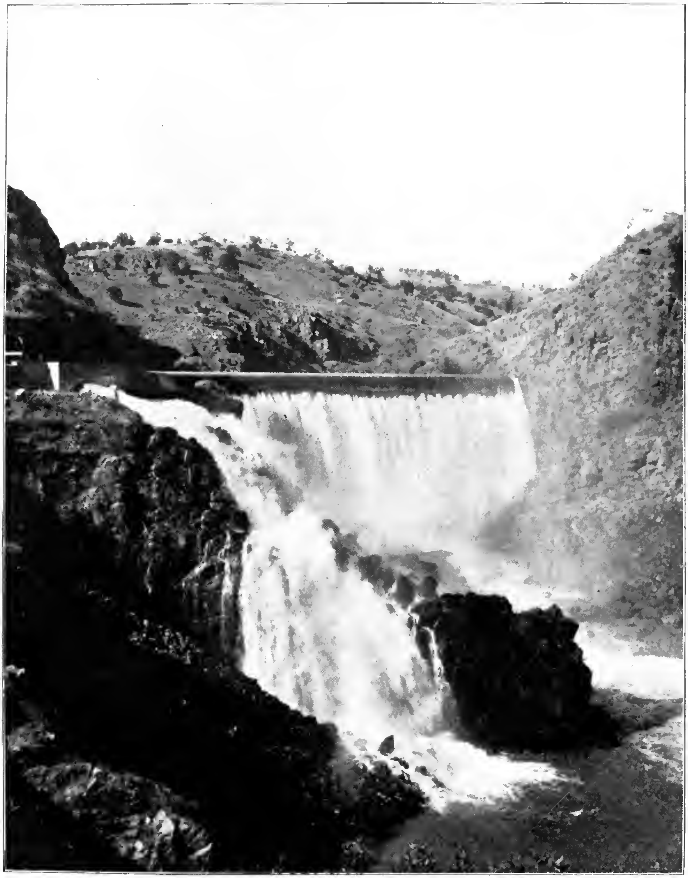


Fig. 20. Curved-Face Dam.

52. 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. 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.

53. Construction. 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.

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.



LA GRANGE IRRIGATION DAM, MODESTO, SOUTHERN CALIFORNIA

A typical example of work that is being done for the reclamation of arid areas in the West and Southwest. Under the direction of the U. S. Government Reclamation Service, irrigation projects are being carried out in thirteen States and three Territories. In every case the cost of the work is charged against the land, and thus ultimately paid back to the Government by the settlers. Among the most notable projects are the Salt river dam in Arizona, to irrigate 200,000 acres near Phœnix; the Yuma project on the lower Colorado river in California; the Klamath project, to reclaim 300,000 acres in California and Oregon; the Twin Falls project, on the Snake river in Idaho, to reclaim 250,000 acres; the Shoshone river project in Wyoming; the Gunnison river project in Colorado, to irrigate the Uncompahgre valley; and other projects in New Mexico, Oregon, Washington, Montana, and North Dakota.

Concrete, well placed, is in this respect an ideal material. Rubble masonry, in which all joints are thoroughly filled with mortar, and larger spaces with concrete, has been used for most of the high dams.

In constructing the masonry the principal points to be emphasized are clean surfaces, irregular surfaces, joints absolutely filled with compact mortar, 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.

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. If 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 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 earthen embankments. No danger is here to be apprehended from constructing the pipes in the body of the dam.

54. 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. 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.

Figs. 21, 22 and 23 show three forms of waste weirs. Fig. 21 permits the water to fall vertically and is suitable for small heights; Fig. 22 is preferable for larger quantities of water and greater heights; while Fig. 23 represents a form of construction suitable for the largest dams.

55. Timber Dams. Where a dam 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.

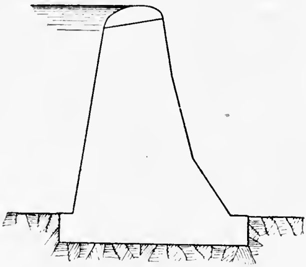


Fig. 21. Waste Weir.

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.

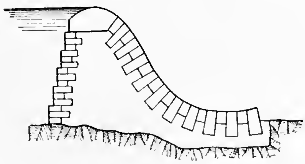


Fig. 22. Waste Weir.

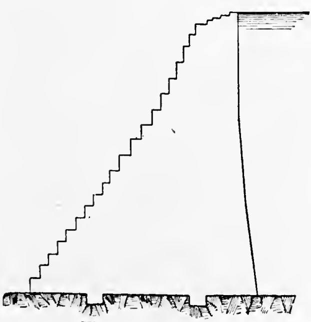


Fig. 23. Waste Weir.

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

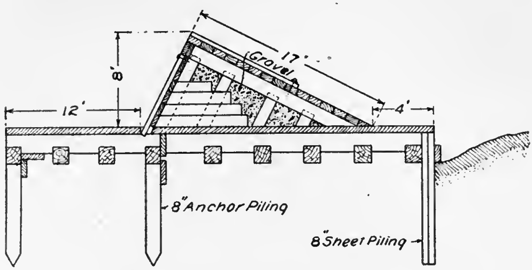


Fig. 24. Timber Weir.

stepped aprons below, or a single free wall to a water cushion. Rock and gravel, or puddle, is used for filling. Fig. 24 shows a form of dam on pile foundations and Fig. 25 a dam anchored to solid rock.

56. Loose Rock Dams. Dams composed largely of loose rock have been used to a considerable extent, and in some respects present

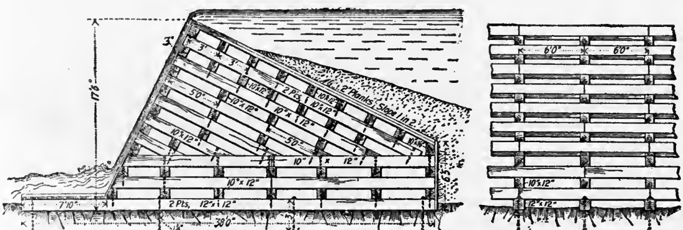


Fig. 25. Dam Anchored to Rock.

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 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.

A SPLENDID EXAMPLE OF A SIDE-HILL FLUME

Total length of flume, 10 miles, diverting water of the Puyallup River, Washington, to supply the power plant of the Puget Sound Power Company at Electron

WATER SUPPLY.

PART II.

CONDUITS AND PIPE LINES.

PIPES.

57. Materials and Stresses. 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.

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 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.

The tensile stress in a water pipe under pressure is given by the formula in section 10, of Hydraulics,

$$S = \frac{pr}{t} \qquad (6)$$

where S = tensile stress per sq. inch
p = pressure in lb. per sq. in.
r = radius of the pipe in inches,
t = thickness of pipe in inches.

If f represents the safe tensile strength of a pipe material, then the required thickness to resist the pressure will be given by the formula

$$t = \frac{pd}{f} \tag{7}$$

Example. What will be the required thickness of a steel pipe 3 ft. in diam. for a water pressure of 100 lb. per sq. in. assuming the safe stress = 10,000 lb. per sq. in.

From formula 7, $t = \frac{100 \times 18}{10{,}000} = .18$ inch. Ans.

Besides the stress due to steady water pressure, the pipe must be strong enough to resist the shocks due to the sudden stoppage of flowing water, called *water hammer,* also the pressure of the surrounding earth and the action of other outside forces, changes of temperature, and blows and shocks received in transportation and construction. The stresses due to these additional forces cannot be accurately calculated; but they are allowed for in practice by various empirical rules.

58. Cast-Iron Pipe. 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."

A formula for the thickness of cast-iron pipe applicable to diameters up to 3 feet is as follows:

$$t = \frac{(p + 140 - 4\,r)}{3300}\,r + 0.25 \tag{8}$$

where t = thickness in inches;

p = static pressure in pounds per square inch;

r = radius of pipe in inches;

0.25 = allowance for eccentricity, deterioration, and safety in handling.

In Table No. 10 are given the thicknesses of pipe for various pressures, also the average weight per foot, and the total weight of 12-foot lengths, as employed by the Metropolitan Waterworks, of Boston.

TABLE 10.

Thickness and Weight of Water Pipe.

Diameter in Inches.	Class A. 115-foot Head.			Class B. 150-foot Head.			Class C. 200-ft. Head.			Class D. 250-ft. Head.			Class E. 300-ft. Head.		
	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.
4	....	......	......	....	......	......	....	......	......	0.40	19.2	230	0.45	21.2	255
6	....	......	......	....	......	......	....	......	......	0.46	31.7	380	0.50	34.6	415
8	....	......	......	....	......	......	....	......	......	0.52	47.1	565	0.55	50.0	600
10	....	......	......	....	......	......	....	......	......	0.60	66.7	800	0.63	70.0	840
12	....	......	......	0.57	75.8	910	0.61	80.8	970	0.65	85.8	1030	0.69	91.2	1095
14	....	......	......	0.61	94.2	1130	0.65	100.0	1200	0.70	107.5	1290	0.75	115.0	1380
16	....	......	......	0.65	115.0	1380	0.70	123.7	1485	0.75	132.5	1590	0.81	142.9	1715
20	....	......	......	0.73	160.8	1930	0.79	173.3	2080	0.85	186.2	2235	0.92	201.2	2415
24	....	......	......	0.80	210 4	2525	0.87	228.3	2740	0.95	248.7	2985	1.03	269.2	3230
30	....	......	......	0.92	302.1	3625	1.00	327.5	3930	1.10	361.2	4335	1.20	393.3	4270
36	0.93	366.7	4400	1.03	404.2	4850	1.13	441.7	5300	1.25	491.7	5900	1.36	533.3	6400
42	1.01	467.5	5610	1.14	524.2	6290	1.27	581.2	6975	1.40	645.8	7750	....	......	......
48	1.15	605.8	7270	1.25	655.8	7870	1.40	730.0	8760	1.55	818.3	9820	....	......	......
54	1.23	730.8	8770	1.35	797.5	9570	1.53	919.2	11030	....	......	......	....	......	......
60	1.35	885.8	10630	1.50	979.2	11750	1.70	1132.5	13590	....	......	......	....	......	......

59. Joints. 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.

In Table No. 11 are given various dimensions of standard bell and spigot of the Metropolitan Waterworks (Fig. 26), 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.

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

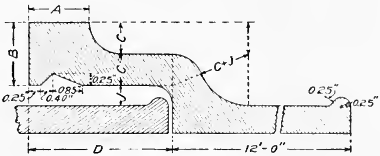


Fig. 26. Standard Bell and Spigot.

radius, a 16-inch pipe to a 250-foot radius, and a 36-inch pipe to a 500-foot radius.

TABLE 11.

Dimensions of Standard Bell and Spigot, Metropolitan Waterworks, Boston (Fig. 26).

Size of pipe.	Class.	Dimensions in inches.					Average wt. of lead per joint.		Weight of jute gasket per joint.
		A	B	C	D	J	With gasket.	Solid lead.	
4	All classes	1.50	1.30	0.65	3.00	0.40	7	9¼	.10
6	"	1.50	1.40	0.70	3.00	0.40	9¾	12¾	.15
8	"	1.50	1.50	0.75	3.50	0.40	12½	18¾	.25
10	"	1.50	1.50	0.75	3.50	0.40	15¼	23¼	.30
12	"	1.50	1.60	0.80	3.50	0.40	18	27	.35
14	"	1.50	1.70	0.85	3.50	0.40	20½	31	.40
16	"	1.75	1.80	0.90	4.00	0.50	31¼	50½	.65
20	"	1.75	2.00	1.00	4.00	0.50	38½	62	.80
24	"	2.00	2.10	1.05	4.00	0.50	45½	74	.95
30	B and C	2.00	2.30	1.15	4.50	0.50	56	100½	1.55
30	D and E	2.00	2.50	1.25	4.50	0.50	57	102	1.55
36	A, B, and C	2.00	2.50	1.25	4.50	0.50	67	120½	1.85
36	D and E	2.00	2.80	1.40	4.50	0.50	68½	122½	1.85
42	A, B, and C	2.00	2.80	1.40	5.00	0.50	77½	154	2.60
42	D	2.00	3.20	1.60	5.00	0.50	78½	156	2.60
48	A, B, and C	2.00	3.00	1.50	5.00	0.50	88½	176	3.00
48	D	2.25	3.50	1.75	5.00	0.50	89½	178	3.00
54	A and B	2.25	3.10	1.55	5.50	0.50	99½	215	3.95
54	C	2.25	3.90	1.95	5.50	0.50	100	215½	3.95
60	A and B	2.25	3.20	1.60	5.50	0.50	110½	239	4.40
60	C	2.25	4.20	2.10	5.50	0.50	111	241	4.40

60. Special Castings. The ordinary special castings required are the ¼, ⅛, 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. 27. 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 manu-

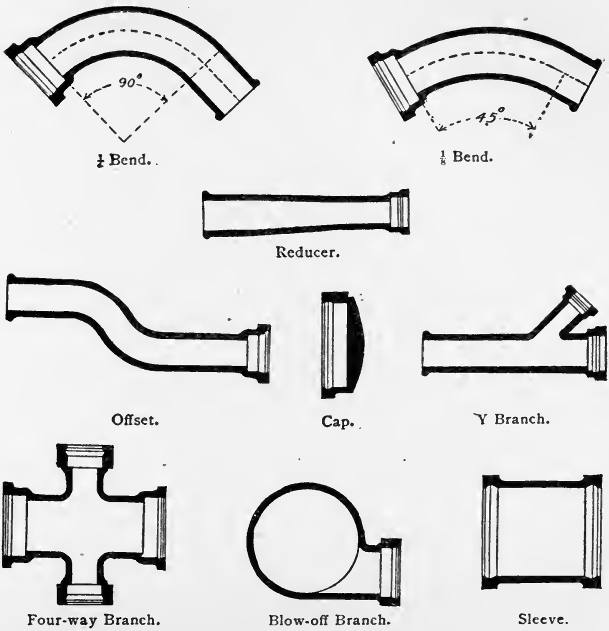


Fig. 27. Special Cast-iron Pipe Fittings.

facturers' 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 preferred for branches, as it enables connections to be readily made by means of pieces of pipe.

61. 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. 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 on account of the increased friction.

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. The design of the riveting is too large a subject to be taken up here.

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.

62. Wooden 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. About 1,500 miles of this pipe is reported to be now in use. It is very durable and is said to cost somewhat less than cast iron where the transportation charges are not excessive. Wooden stave pipe is another form that has been extensively used in the West.

The durability of wooden pipe is chiefly a question of the life of the bands. Wood, itself, when kept saturated with water, has an almost indefinite life, old water mains in Philadelphia, New York, and Boston having been found perfectly sound after sixty or seventy years of use.

63. 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.

64. Materials for Service Pipes. Service pipes, or pipes for conducting water to individual consumers, are made of a considerable variety of materials. 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.

Tin-lined pipe is now being used to a considerable extent. It is quite expensive, but the experience with it so far indicates that it may be very durable.

CONSTRUCTION OF CONDUITS.

65. 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.

Long conduits usually include both masonry 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.

66. 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. 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.

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 as explained in Hydraulics. 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. If vegetation is allowed to accumulate in the canal, a large allowance must be made for increased resistance caused thereby. The cross-section of a canal is usually trapezoidal in form. Fig. 28

shows a section built principally by embankment. Clay puddle is placed in the center of each embankment.

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. If the soil is very porous, a lining of concrete or puddle may be necessary.

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.

Waste weirs and sluice gates should be provided at intervals along the canal to prevent flooding and to permit of rapid emptying, but 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.

Canals are carried across valleys on trestles or bridges, or, in the case of short crossings, on embankments with a culvert or arched

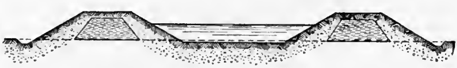


Fig. 28. Canal Section in Embankment.

bridge beneath. Under crossings are made by means of inverted siphons of pipe.

67. Masonry Conduits. For conveying relatively large quantities of water over territory where the conduit can readily follow the hydraulic grade line, the masonry conduit 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. Masonry conduits should 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. The velocity should preferably be such as to prevent deposit of sediment, which requires 2½ 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. 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.

Kutter's formula is usually employed in these calculations. (See Hydraulics.) The value of n to be used will vary with the character of the masonry about as given in Hydraulics.

Brick is the most suitable material for linings, and is commonly used also for the entire arch crown. For the side walls and foundation, rubble masonry or concrete is employed. For places of great wear paving brick in cement is a good substitute for granite. Concrete is better suited than either stone or brick for irregular forms and especially for light sections. For small aqueducts a rectangular form has often been used, as in Fig. 29, the cover being of stones, slabs, or arches. For large sizes the horseshoe shape is better adapted, as shown in Fig. 30, which represents the form adopted for a large conduit for Boston.

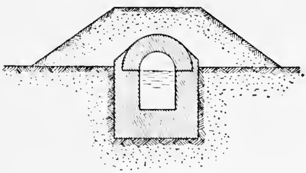


Fig. 29. Gallery, Vienna Waterworks.

The thickness of the arch is made about one-tenth to one-sixth the width of the opening, and of two, three, or four rings of brick, or a corresponding of concrete, depending on span and weight of covering. The arch is generally segmental in form. The invert, in compact ground, is made only thick enough to secure a firm and impervious bottom, two or three rings of brick, or a thin layer of concrete with brick lining, being usually employed. A timber foundation and sometimes piling may be required on soft soils. Settlement must be reduced to very low limits, or cracks and leakage will result. It is unnecessary to state that in work of this kind the masonry must be constructed with the most careful supervision. Concrete and stone masonry should be

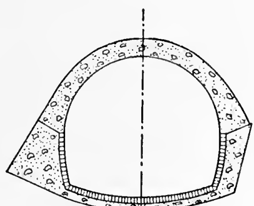
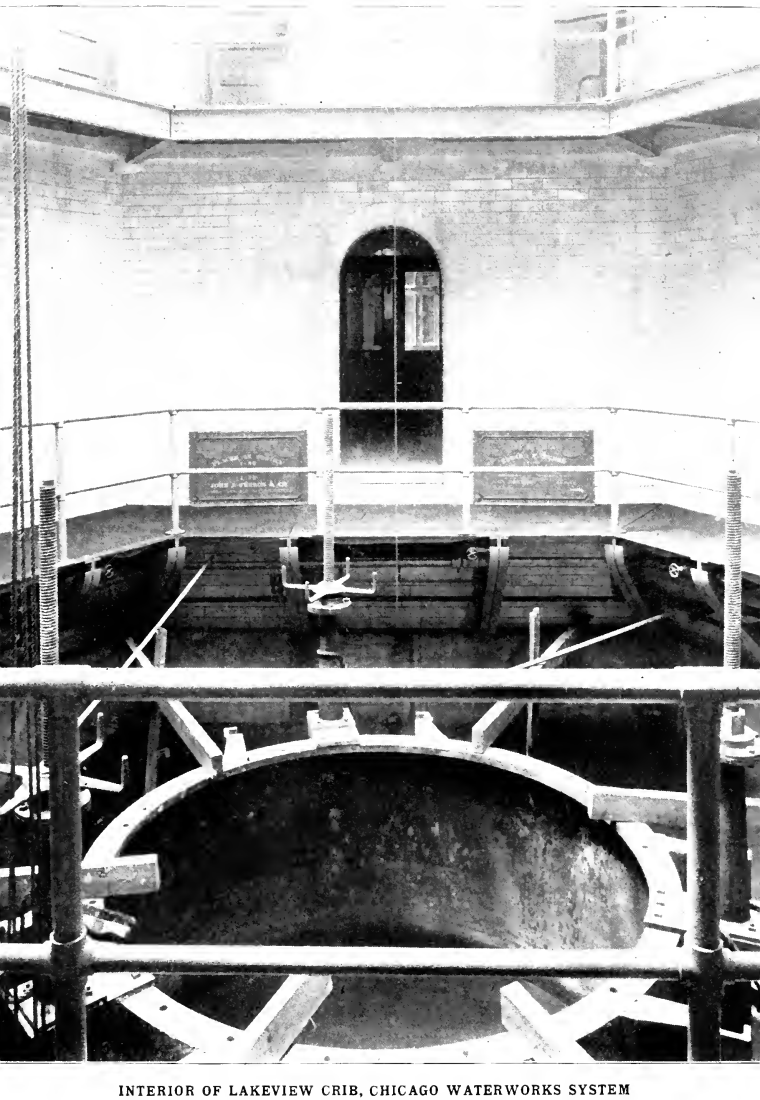


Fig. 30. Forms for Large Conduit.



INTERIOR OF LAKEVIEW CRIB, CHICAGO WATERWORKS SYSTEM
Showing intake shaft.

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.

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.

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 often be the more economical design.

68. Pipe Lines. As to location, 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.

Where the total available head is fixed, the size required for any given capacity is readily determined by the table of the flow of water in pipes in Hydraulics. 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.

If pumps are used to force the water through the pipe then the proper size depends on the relative cost of pipe, and of pumping against an increased head. A large pipe gives low friction head of the water and therefore saves in pumping expenses, but a large pipe is more expensive than a small one. In general it may be assumed that a proper size of pipe is one which calls for a velocity of flow of from $1\frac{1}{2}$ to $2\frac{1}{2}$ feet per second, the former value for pipes of 6 to 12 inches in diameter, and the latter for pipes of 3 or 4 feet in diameter.

69. Laying of Pipes. 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 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. 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. 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.

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. After riveting, all field joints should be calked, and these and all other abraded places painted. Some re-calking may be needed after the pipe is tested.

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 a general way it may be stated that for a latitude of 40° the depth of cover should be 4 to 6 feet and for 45° should be 6 to 8 feet, the smaller depths being used east of the lakes and the greater depths for the country between the lakes and the Rocky Mountains. In sandy soil the depth should be greater than in clay.

70. Special Details. 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. Valves of all kinds and designs are furnished by various special manufacturing concerns.

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. 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. Check valves should be 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. 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.

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. 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 crossing under other structures, such as railways, buildings, sewers, etc., special precautions should be taken to avoid all danger of future breakage.

Streams are crossed either on bridges, or by laying the pipe beneath the stream bed, or by the use of a subway.

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. At the angles at ends of bridge and submerged crossings special care is necessary to keep the pipe from separating at the joints. 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.

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.

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, 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.

Various methods are employed in laying pipes beneath watercourses. In the case of small streams the usual method is to employ a cofferdam 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.

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.

(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 $2\frac{1}{2}$ 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 $1\frac{1}{4}$-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.

(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.

(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.

71. Cost of 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 readily determined. Cast-iron pipes laid under average conditions will cost approximately as follows, assuming the pipe itself to cost $1\frac{1}{2}$ cents per lb

Size of pipe.	Cost per foot.
4 inch	$.50
6 "	.70
8 "	1.00
10 "	1.30
12 "	1.70
16 "	2.50
20 "	3.50
24 "	4.50

THE DISTRIBUTING SYSTEM.

Distribution Reservoirs.

72. Use. The rate at which water is actually used is not at all uniform, as fully pointed out in section 6. It varies from day to day according to the season, from hour to hour according to the time of day, and at times of large fires the rate will be greatly increased. If all parts of a system were to be designed of a capacity equal to the greatest possible rate of demand the cost would frequently be prohibitive, and in most cases it would not be the most economical plan. It will usually be more economical to store up a quantity of water in a small reservoir or elevated tank which may be drawn upon when the demand is excessive and thus relieve to some extent a part of the system.

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.

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 standpipes 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. 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 standpipe or a tank of wood or steel is usually employed.

73. Capacity. 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, owing to the character of the topography, it becomes necessary to artificially elevate a reservoir in the form of a standpipe 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 standpipes which are often used in such cases serving merely to equalize the action of the pumps.

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 to the day only.

In general the best size of elevated tank will range from about 75,000 gallons for very small works up to 200,000 or 300,000 gallons for towns of the size mentioned above.

74. Location and Arrangement. 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.

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 proper elevation of a reservoir depends on the required pressure in the mains, a subject fully discussed in section 89.

EARTHEN AND MASONRY RESERVOIRS.

75. Form and Proportion. 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. 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. 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.

76. Construction. The construction of the embankment is based on the same principles as discussed in section 47, but the conditions are somewhat different from those obtaining with impounding reservoirs. In this case 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. Fig. 31 shows a puddle being placed near the face and Fig. 32 shows a puddle core connected to the puddle lining of the bottom.

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, more commonly, by a coat of Portland-cement

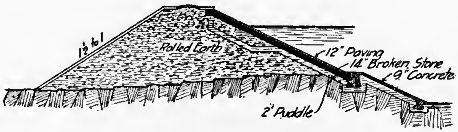


Fig. 31. Section of Reservoir Embankment, Pittsburg.

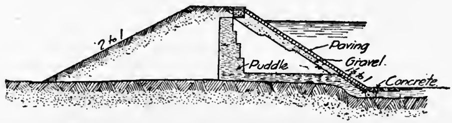


Fig. 32. Section of Reservoir Embankment, Brooklyn.

mortar, neat or 1 to 1, which it is well to finish by a brush coat of neat cement. The latter method is more likely to be satisfactory with covered reservoirs, where the temperature changes are small, than with open reservoirs.

While it is comparatively easy to secure imperviousness at the start 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. The foundation for the walls should be broad enough to reduce settlement to very small limits, and as further precaution against cracks the floor lining should be constructed after the walls are complete, but should be thoroughly bonded thereto. Junctions between floors and walls are preferably made curved.

The most common form of lining consists of about $1\frac{1}{2}$ 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. A layer of paving brick laid in cement makes a good finish for a concrete lining which is to be frequently exposed. Concrete can be made impervious by plastering with cement mortar, neat or 1 to 1, but where it extends over large areas, cracks will form, due to temperature changes and to settlement of embankments. To minimize this difficulty, concrete may be laid in blocks, with asphalt joints between.

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 to effectually prevent the washing out of any of the material. Seepage water is also sometimes taken care of by means of drains.

Asphalt has for some time been extensively used for reservoir linings on the Pacific coast, and recently its use has become quite general. Compared to concrete it has the advantages of elasticity and greater imperviousness, both of which are of great importance in this connection. 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 to give good results.

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.

Reservoirs with masonry walls 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. 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.

77. Inlet Pipes and Valves. 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 section 48. A by-pass should be provided to enable the reservoir to be cut out at any time. 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.

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 oustide.

78. Covered Reservoirs. 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. 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.

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.

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. 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.

Masonry covers consist usually of segmental or elliptical masonry arches supported by small brick piers; or, for very small reservoirs, a dome may be used. 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. The piers are spaced from 10 to 15 feet apart, and are made from 1 to 2 feet square in cross-section, depending upon the span and weight of filling.

Piers are usually made about 18 inches square of brick and spaced about 12 to 15 feet apart. Concrete is now generally used for the roof, being made in the form of groined arches of about 3 feet rise and 6 inches thick at the crown. Fig. 33 shows the interior of such a reservoir used as a filter.

79. Cost. The cost of reservoirs varies, of course, greatly according to local conditions, kind of reservoir and capacity. According to the capacity the cost per unit will be less the larger the reservoir. The actual cost of a large open reservoir varies from $3 to $5 per 1,000 gallons capacity. Covered masonry reservoirs will cost usually from $10 to $15 per 1,000 gallons capacity.

STANDPIPES AND ELEVATED TANKS.

80. Where a reservoir requires to be artificially elevated it is usually built as a standpipe—a tall slim tank resting on the ground—or as an elevated tank of steel or wood, supported by a tower of steel, wood, or masonry. 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

Fig. 33. Covered Filter.

in section 72, 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. The capacities of standpipes and tanks range ordinarily from 50,000 gallons up to a maximum of about 1,500,000 gallons for small villages and cities up to 30,000 population or more. The useful capacity of a standpipe is only that part of the volume which is at a sufficient elevation to give 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.

81. Location. For storage purposes only, the location would be the same as that for any reservoir. 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.

82. Design of the Standpipe. The chief elements in the design of a standpipe are the thickness of plates, the riveting, the foundation and anchorage and the pipe details. The forces to be considered in the design of a standpipe 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 of the pipe from the top, d = diameter of pipe in feet, r = radius in feet, and t = thickness of shell in inches at the given point.

From equations in Hydraulics we find that the water pressure causes a bursting stress per vertical lineal *inch* of pipe equal to

$$S = \frac{62.5hd}{2 \times 12} = 2.6hd \qquad (9)$$

The stress per square inch of metal is

$$s = \frac{2.6hd}{t} \qquad (10)$$

This is the only stress that need be considered in determining the plate thickness, as the effects of wind and weight are much smaller than this and cause a stress in a *vertical* direction.

The safe tensile stress on net section of metal, 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 equation 10 the required thickness to resist the water pressure will be

$$t = \frac{2.6hd}{s} = \frac{2.6hd}{ae} \qquad (11)$$

or, if $a = 12{,}000$ and $e = \frac{2}{3}$, then, approximately,

$$t = \frac{2.6hd}{8{,}000} = .000325hd \qquad (12)$$

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 plates forming a standpipe 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 standpipe. Lap joints are most commonly used, but for thickness exceeding $\frac{1}{2}$ inch, double-butt strap joints are much preferable and are stronger.

Table No. 12 gives suitable proportions for riveted joints, together with their approximate efficiencies or ratio of strength of joint to strength of plate.

TABLE 12.

Proportions for Riveted Joints for Standpipes.

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.	Efficiency, per cwt.
	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 " "	$\frac{5}{16}$	$\frac{5}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{8}$	60
" " "	$\frac{3}{8}$	$\frac{3}{4}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{4}$	
" " "	$\frac{7}{16}$	$\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{3}{8}$	
" " "	$\frac{1}{2}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{1}{2}$	
" " butt	$\frac{9}{16}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	70
" " "	$\frac{5}{8}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	
" " "	$\frac{1}{16}$	1	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{3}{4}$	
" " "	$\frac{3}{4}$	1	3	$2\frac{1}{2}$	$1\frac{3}{4}$	
" " "	$\frac{13}{16}$	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

Horizontal joints are made single-riveted lap joints, with rivet spacing of about three diameters. 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. 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 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.

Standpipes must be anchored to the foundation to prevent being overturned by the wind. 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 value the overturning moment in foot pounds at a distance h below the top is

$$M = 50 \times \frac{dh}{2} \times \frac{h}{2} = 12.5dh^2 \qquad (13)$$

This movement causes an uplift on the leeward side for each inch along the circumference of the pipe of

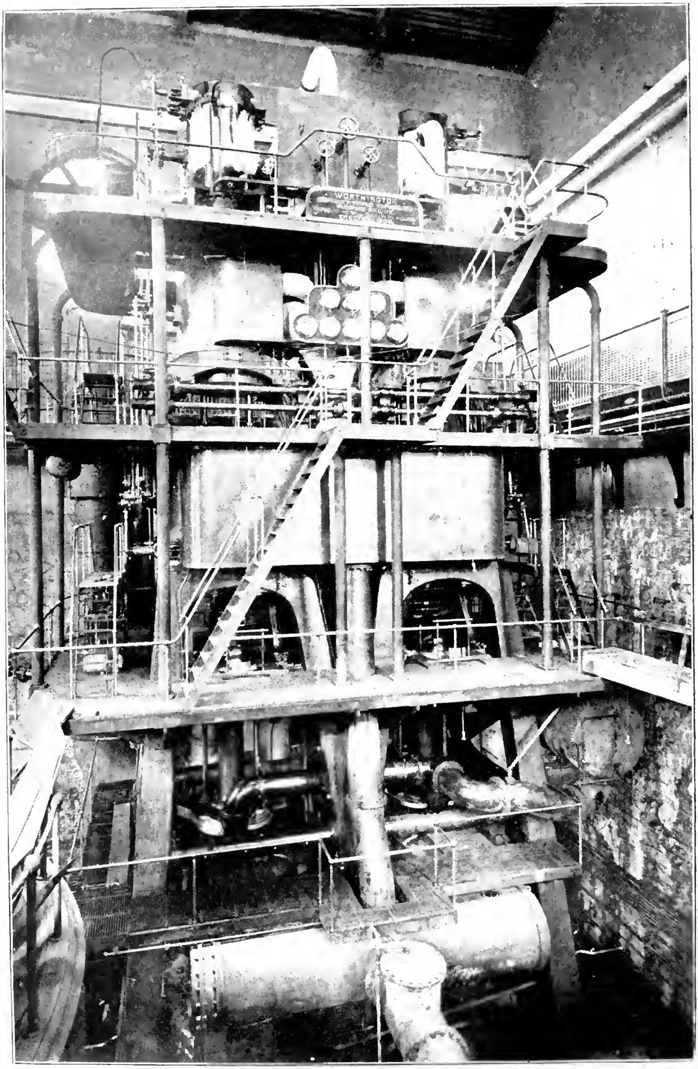
$$S^* = 1.33 \frac{h^2}{d} \text{ pounds.} \qquad (14)$$

Then if anchor bolts are placed p inches apart around the bottom of the tank the stress in each bolt will be

$$S \times p = 2.33 \frac{h^2}{d} \times p \qquad (15)$$

If numerous bolts are used, their size need not be great, and they may be put through the exterior bottom angle iron 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

*The derivation of this equation comes from the formula $S = \frac{Mc}{I}$ of Mechanics in which M is the moment of inertia of the standpipe shell. The process of derivation cannot well be entered upon here.



WATERWORKS PUMPING ENGINE AT CENTRAL PARK AVENUE PUMPING STATION, CHICAGO, ILL.

Worthington pump; capacity, 40,000,000 gallons daily.

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 standpipe by long vertical pieces of angles, and the bolts placed close to the pipe as shown in Fig. 34. 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.

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 standpipe.

The effect of ice action is a very serious matter in unprotected standpipes, but is very difficult to calculate or provide for. 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 importance of this matter is attested by the many accidents traceable to the action of ice.

The material used for standpipes 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.

83. 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 standpipe 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. 34. A drain-pipe through which the tank may be drained or flushed should also be provided.

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 standpipe can be arranged to give an alarm at any desired pressure.

84. **Other Details.** 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.

It is not customary to roof standpipes, 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 a framework.

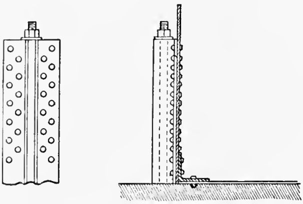


Fig. 34. Arrangement of Rivets.

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.

Standpipes 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.

A standpipe is often surrounded with a masonry shell in order to furnish protection from cold, or to improve the appearance of the structure. This 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. The walls are usually made from $2\frac{1}{2}$ to 4 feet thick at the bottom and $1\frac{1}{2}$ to 2 feet at the top.

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.

85. Elevated Tanks. If the lower portion of the water in a standpipe 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. Besides being cheaper, a tank is much less objectionable in appearance than a standpipe, and experience indicates that trouble from ice is less likely to occur. 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.

The bottom of a tank supported on an iron trestle is usually made hemispherical, as this requires no support except at the outside edge where the legs of the tower are located. The thickness of side plates is the same as for standpipes, and the details are similar. If the bottom is hemispherical the stresses therein will be one-half those in the lowest side course of plates.

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. The stresses in the various parts of the tower and the design of the details belong to the domain of structural engineering and cannot be elaborated here. Suggestions for the upper column connections, the anchorage and the roof are given in Fig. 35.

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 foundation should be rigid, and large and heavy enough to serve as anchorage and to give only safe pressures on the ground. There should be practically no settlement, as any unequal settlement will greatly change the stresses in the tower.

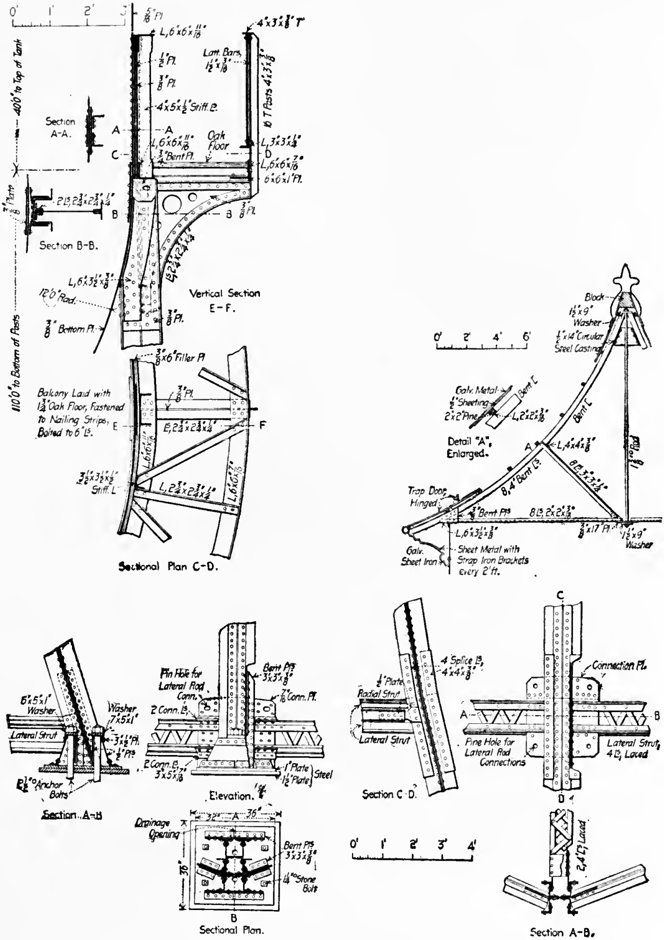


Fig. 35. Details of Elevated Tank, Ames, Ia.

The inlet pipe is usually made to enter the tank at the center of the bottom, and should be provided with an expansion joint. 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 a packing of some non-conductive material. If the tank is encased, it will be necessary to provide an overflow pipe.

86. 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.

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.

87. Storage Under Compressed Air. In small works, air chambers or their equivalent may be used to provide a considerable storage of water and thus avoid the use of standpipes or elevated tanks. In the design of such storage tanks the larger the proportion of air space the less will be the variation in 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 $p = P\left(1 - \frac{v}{V}\right)$. 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 is maintained in the tank by occasionally admitting a little air into the pump.

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 in the water tank. By reducing valves in the connecting pipes, the pressure on the water may be maintained constant, or may be increased in case of fire up to the pressure in the air tank. Air compressors must be used here to keep up the air supply. A number of plants of this kind have been installed. 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.

THE DISTRIBUTING PIPE SYSTEM.

88. 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 waterworks system, the uses of water naturally fall into two very distinct classes: (1) the ordinary, everyday 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.

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.

89. The Pressure Required. 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 20 to 30 pounds in residence districts, and usually from 30 to 35 pounds in business districts, according to the character of the buildings.

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, 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 standpipe if there be one, and to furnish the necessary fire pressure direct from the pumps.

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 hydrant fire pressure is to be supplied it should not be less than 60 pounds for residence districts and 70 pounds for business districts. Pressures 20 pounds higher than these are to be desired. If dependence is to be placed on fire engines, as is usual in large cities, the domestic pressure of 25 to 30 pounds is sufficient.

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. In the case of gravity supplies much higher pressures may be possible, but on account of the increased cost of plumbing and piping to withstand high pressures it will not be desirable often to exceed 130 to 140 pounds.

90. 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. For average conditions the number may be calculated from the formula

$$y = 2.8\sqrt{x}, \qquad (16)$$

where y = number of streams, and x = population in thousands. About two-thirds of this number should be capable of being concentrated upon a single block or group of buildings.

In small cities and towns the requirements for fire protection may differ widely. For example, in a country town of 4,000 to 5,000 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.

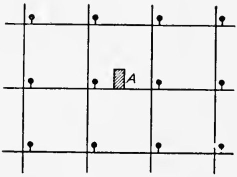
The number of fire streams is based upon a size of stream of about 250 gallons per minute, which is generally considered to be about right as an average value for good fire streams in business districts. For a residence district 175 to 200-gallon streams will usually meet the requirement. Fire hydrants must be sufficiently numerous and so located as to meet the requirements regarding number and size of fire streams set forth in the preceding paragraphs. 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 Hydraulics), 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. 36 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 about 450 feet, giving sixteen 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. 37, 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.

91. General Arrangement of the Pipe System. From the data on page 9 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. 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.

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,



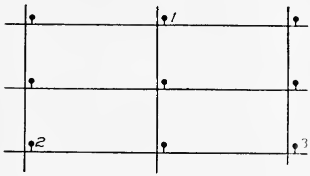


Fig. 36. Location of Hydrants. Fig. 37.

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. 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 never be used to supply hydrants except where the pipe is comparatively short and is fed from larger pipes at each end.

92. Calculation of the Pipe System. 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 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 maximum rate of consumption for the entire city has already been discussed in section 6. 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, 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 1,000 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 data so collected will enable the main distributing pipes to be calculated. The size of the cross mains and smaller pipes will be determined almost entirely by the local requirement as to fire streams. For all practical purposes an arrangement of 6-inch pipe in one direction, and 4-inch pipe crossing these, is ample for cities up to about 10,000 inhabitants, and six-inch pipes in both directions will suffice for populations up to about 50,000. For villages up to 1,000 or 2,000 population and the residence districts of small cities all but the general supply main may be 4-inch provided there are no dead ends and that there is a cross line at least every other block.

The size of the main supply pipe and the main branches feeding isolated districts can be calculated by the aid of Table No. 12 of Hydraulics giving the friction loss in pipes, an estimate of the maximum rate of demand having been made. For most cases the desirable velocities in the main pipes will be from 3 to 6 feet per second for the maximum rate of flow. The lower velocity is that suitable for a plant where the available head is limited and not much friction loss can be permitted, as for example in a gravity system where the elevation of the source is barely sufficient to furnish the desired pressure. The higher velocity is suitable where a considerable loss of head may be allowed, as for example in a gravity system with a high source, or in a pumping system where the fire pressure is furnished by pumps and only during the fire.

The number of fire streams of 250 gallons per minute each which can be supplied reasonably through pipes of different size are given in Table No. 13, the smaller number corresponding approximately to the lower velocity mentioned above and the larger the higher velocity. Where a pipe is fed from both directions double the number of streams can be supplied.

TABLE 13.

Number of Fire Streams Obtainable From Pipes of Various Sizes.

Size of pipe.	No. of 250-gal. streams.
4	1
6	1– 2
8	2– 4
10	3– 6
12	4– 8
16	8–16
20	12–24
24	18–36

Example. A town of 3,000 inhabitants is to be supplied through a force main 4,000 feet long. Assuming the average daily consumption to be 75 gallons per capita and that the town is of average character as regards fire demands, what would be a suitable size of main?

Referring to section 6, we find that the maximum rate for ordinary use may be taken at 180 per cent of the average, which would be $1.80 \times 75 = 135$ gallons. The rate per minute will be $\frac{135 \times 3,000}{24 \times 60} = 280$ gallons. The number of fire streams required is by formula 16 equal to $2.8 \sqrt{3} = 4.8$ or, say, 5. Each being assumed as 250 gallons the total rate will equal $280 + 5 \times 250 = 1,530$ gallons per minute, or practically equal to 6 fire streams. From the table No. 13 we see that a 10-inch pipe may be used if a considerable loss of head is permissible or a 12-inch pipe if but little loss is desired. From Table 12 of Hydraulics the actual loss of head in the 10-inch pipe for a flow of 1,530 gallons per minute is 16 feet per 1000, or 64 feet for the entire length of main. For a 12-inch pipe the loss is only about 6.5 feet per 1,000, or 26 feet total. Where the available head is not more than 150 feet the former loss would be too great.

93. Separate Services for Different Elevations. 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.

94. 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.

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.

95. Hydrants. The general location of hydrants has already been considered in section 90. 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.

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. In Fig. 38 are shown two forms of hydrants which illustrate the two general types of valves used—the gate valve and the compression valve. In ordering hydrants care should be taken to have the nozzles of the same standard as those used in adjoining large 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.

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.

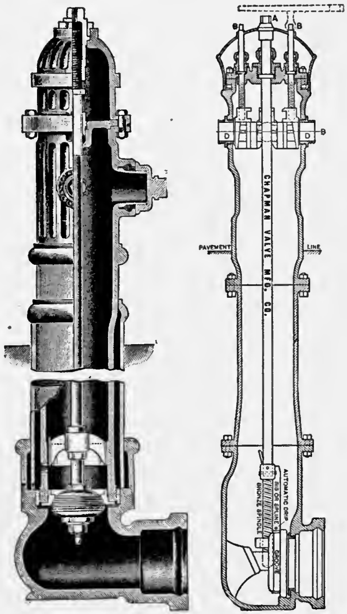


Fig. 38. Fire Hydrants.

96. Service Connections. Service pipes are usually from $\frac{3}{4}$ inch to 1 inch in diameter, and are made of lead, galvanized iron or tin-lined iron pipe. In making the connection between service pipe and main, the latter is tapped and a brass "corporation" cock screwed in. At the curb is usually placed another stop cock, with a suitable valve box, at which point the supply to the consumer is controlled.

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.

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 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.

OPERATION AND MAINTENANCE.

97. 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. If the water carries sediment and has a low velocity, the pipe line should be occasionally flushed by opening the blow-off valves.

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.

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 certain waters, particularly those

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relatively soft, the interior of cast-iron pipes corrode quite rapidly as explained elsewhere. This tuberculation, as it is called, 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.

Large pipes can be cleaned by sending workmen through them, but ordinary pipes can only be cleaned by flushing or sending through them some form of mechanical scraper which nearly fills the pipe and which is propelled by the water pressure. Very badly corroded pipes have been successfully cleaned in this way.

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.

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.

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. The remedies for electrolysis should apparently rest entirely with the railway companies. 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.

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. 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.

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.

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.

In the management of the pumping station the best results can only be obtained by employing thoroughly competent 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. 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.

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.

The maintenance of earthen reservoirs calls for little more than has already been mentioned in section 76. 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. Standpipes 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.

98. Detection and Prevention of Waste. From the data given in section 6 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 waterworks system.

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. 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 re-inspected shortly to make sure that the order has been complied with. Persistent refusal is followed by the shutting off of the supply.

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. 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.

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.

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.

99. Water Rates. The several services performed by a waterworks 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 standpipes, cost of hydrants, and increased cost of maintenance. Estimates of careful observers place the proportion of cost chargeable to fire protection at one-third or one-half the entire cost.

The sources of revenue are the water rates and the fund 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 section 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 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.

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, and rates per unit of volumes 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. A good example of such a schedule is as follows:

For the first	5,000	cu. ft.	per 6 months,	20	cts.	per	100 cu. ft.
" " next	15,000	"	" "	10	"	"	100 "
" " "	10,000	"	" "	5	"	"	100 "
" " "	30,000	"	" "	3	"	"	100 "
" " "	30,000	"	" "	2	"	"	100 "
" over	90,000	"	" "	5	"	"	100 "

A minimum charge of $2.00 per 6 months is made.

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 exists as to the actual amount of water used by different fixtures, and the rates are largely arbitrary.

PURIFICATION OF WATER.

100. Object and Methods. In the purification of public water supplies the primary object is usually to remove from the water any traces of pollution that may give rise to disease, or, in general, to remove any disease germs that may possibly infect the supply. It is often important also to remove the suspended matter where the water is turbid. Sometimes also the water contains so much dissolved mineral matter that it is desirable to remove a part of this to render the water more suitable for manufacturing as well as for domestic purposes. Thus, a very hard water is undesirable to use for boiler purposes as well as for culinary and laundry uses.

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.

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 so-called mechanical filters. 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. 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.

SEDIMENTATION.

101. 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. The amount and character of the sediment varies greatly from time to time; 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. In the Ohio River water at Louisville it varies from 1 to 5,000 parts per million, ranging ordinarily from 100 to 1,000; the bacteria varies from a few hundred per cubic centimeter to as high as 50,000. The size of the suspended particles also 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.

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.

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.

Sedimentation may be employed as a preliminary treatment of a water that is to be filtered or it may be used as the final and only treatment. In the former case a fully clarified water is not essential, but in the latter case it is greatly to be desired, although not always possible.

There are two methods to be considered: (1) Plain sedimentation; (2) Sedimentation with the addition of a coagulant. For plain sedimentation a period of 24 hours' subsidence is about the minimum limit adopted, but this will seldom give a clear water. A considerably longer time is often necessary to give acceptable results. For some waters it requires weeks and even months to remove all the turbidity, 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. In the case of the Mississippi at St. Louis, it is practically impossible to clarify the water in the spring by simple sedimentation, owing to the attenuated condition of the clay particles.

There is a marked degree of bacterial purification in sedimentation, yet it should be kept in mind that such a method is extremely hazardous, especially where the water supply is subject to any sewage pollution. Experience shows that sedimentation alone is insufficient to protect a city against a polluted water supply.

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 much more readily removed by sedimentation or filtration. Color may also frequently be removed to a large extent by this treatment. If a water can be satisfactorily purified the greater part of the year by plain sedimentation, the use of a coagulant at other times as an aid in the process is well worth consideration.

Several substances can be used as coagulants. That most commonly employed is sulphate of alumina. When this substance is introduced into water containing carbonates and bicarbonates of lime and magnesia, it is decomposed, the sulphuric acid forming sulphates 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. 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. The regulation of this matter must be put into the hands of an expert, as an excess of alum in the water is very undesirable if not actually dangerous.

The amount of chemical required 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 per gallon. The proper amount can only be determined by experiment.

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. With a fair amount of coagulant, the sediment remaining after 24 hours' subsidence will settle very slowly; and this period may be taken as about the maximum economical figure. Much less time than this can be used in many cases. Where the water contains large amounts of sediment, it will often be more economical to allow the coarser particles to settle before applying the coagulant. This will reduce the cost of chemicals and give a more satisfactory result.

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, 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 a clear-water reservoir, or to a pump well, or to filters, as the case may be.

102. Methods of Operation. There are two general methods of operating settling basins: (1) the constant-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. It is probable that certain waters can be treated best by one system and others by the other system, but this is a matter which can only be determined by experiment. The method of fill-and-draw, used at St. Louis, is thought by the engineers in charge to be more suitable for conditions at that place.

In several plants using Missouri River water the constant-flow system is used. 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. If the basins are operated on the constant 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. In case a coagulant is used after partial sedimentation, two basins would be necessary and three would be desirable. With the fill-and-draw method, the number becomes a question of economical construction and operation. This will usually be from 4 to 6.

For a single rectangular basin of given area the square is the most economical form. For a number of basins they should be made rectangular in shape with a width about three-fifths of the length and arranged side by side in one row.

In general, settling basins are built similar to ordinary reservoirs, partly in excavation and partly by embankment, so as to secure the greatest economy. Earthen 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 in spite of this. 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.

Fig. 39 illustrates the arrangement of the large St. Louis basins. 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. The filling is done through a 60-inch pipe leading from a filling conduit of masonry. The drawing is done at the east end through two sluiceways, 4 by 5 feet in size, which take the water from about 5 feet above the bottom. The water passes thence through a 60-inch pipe into the main delivery conduit.

In the continuous flow system the object to be obtained is the distribution of the water on entering as evenly as may be across one side or one end of the basin 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. The ordinary inlet is usually a single large pipe laid through the embankment, or a single sluice gate in a gate chamber built in the walls. A better distribution of the water could be obtained by means of several inlets, or several branches from a single inlet pipe. 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.

In the fill-and-draw system 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. To enable the sediment to be removed, the bottom of the basin should be made slightly sloping (1 to 2 per

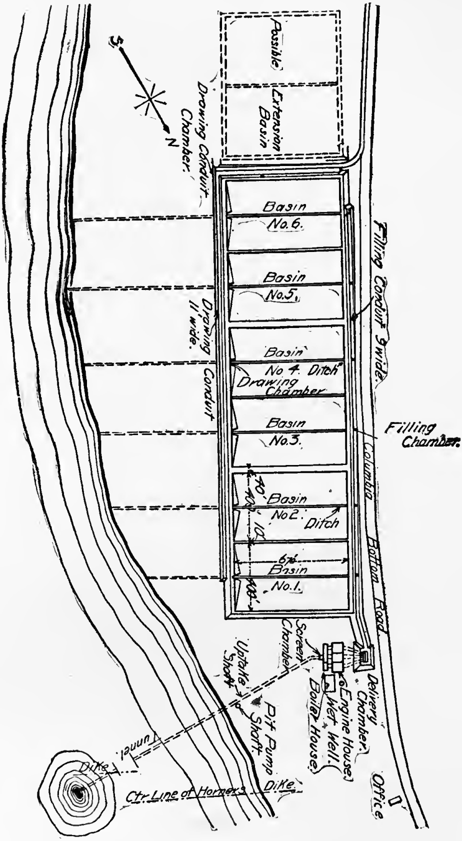


Fig. 39. St. Louis Settling Basin.

cent grade) towards a central drain leading to an outlet gate or to a drain pipe. The mud is removed by flushing into it 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.

Where the basins are operated on the continuous-flow system and the water passes from them directly to the pumps, it is necessary to construct a small clear-water or pump well to avoid the necessity of too frequent adjustment of the rate of supply to the basins.

SLOW SAND FILTRATION.

103. The first filter of which we have any record was established 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. 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 made compulsory. In the United States it is only very recently that this subject has received the attention that it merits, but within the last few years many small cities and several large ones have installed efficient filter plants.

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 a bed of from 2 to 5 feet of sand which forms the true filter. The water flows by gravity, or is pumped, upon the filter, passes through the under drains to a collecting well, and thence to the consumer. Water containing much sediment is usually first passed through settling basins, where a large part of the sediment is removed.

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.

Besides the method of construction, the chief characteristic of this system of filtration is the slow rate of operation, usually not exceeding 2 or 3 million gallons per acre per day.

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.

104. 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. In general high rates of filtration will give less efficiency than low rates, but until the rate exceeds a certain amount the difference in efficiency is small.

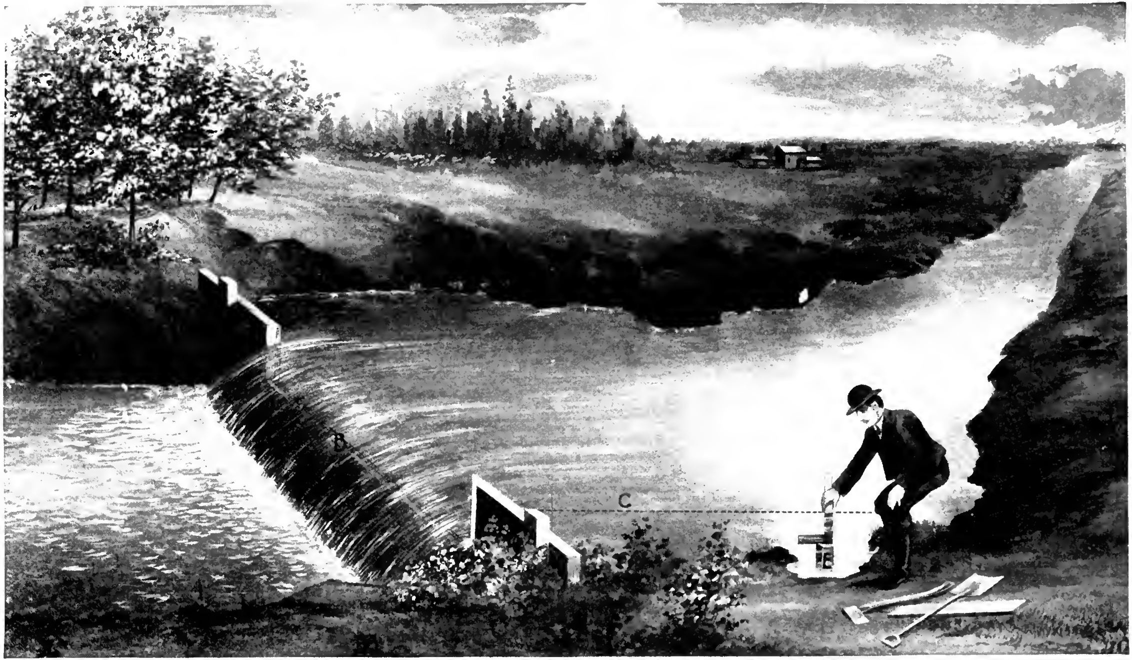
Rates of filtration are in this country usually stated in terms of gallons per acre per day or per hour. The experience of European works has resulted in the adoption of a rate, for most places, of between 2 and 3 million gallons per acre per day, but in this country somewhat higher rates have been favored. 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. It should not be overlooked that there may be cases, where, with a moderately polluted water, all the practical benefits of filtration can be secured at rates much higher than are usually employed. Each case demands independent consideration in order that the best and most economical solution may be arrived at. Sudden changes of rate are apt to produce disturbances in the filter and to give a reduced efficiency. In practice, absolute uniformity of operation is unnecessary, but sudden changes in rate should be avoided, and especially any large increase above the normal.

105. 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. In section 6 it was shown that the maximum daily rate of consumption is likely to be about 150 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.

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. 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 under ordinary conditions the number should be about as follows:

For a	total	area of	1 acre	3	to	4	beds.
"	"	"	3 "	5	"	6	"
"	"	"	6 "	7	"	10	"

Filter beds are usually made rectangular in form and arranged side by side in one or two rows according to the number. In general construction a filter basin is built in a way similar to small distributing reservoirs. (See section 76.) Earth embankments for the sides are cheaper than masonry walls, but require more ground. If the filters are covered, masonry walls are usually employed. Partic-



ILLUSTRATING WEIR METHOD OF MEASURING FLOW OF WATER IN SMALL STREAMS ular care must be taken to render the basin water-tight both on 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.

The depth of open filters is made only sufficient to contain the necessary depth of filtering materials and water, as explained subsequently, and still have a margin of 2 or 3 feet from the water surface to top of the embankment. This will give a total depth of 9 or 10 feet. In closed filters the distance from top of sand to cover must be sufficient to give head room for workmen when cleaning the filter, a distance of about 6 feet.

Covers for filters are constructed of the same general form and arrangement as described in section 78. Masonry or 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. Walls and piers should be built with small offsets near the bottom in order to insure good filtration at that point. A covered filter is illustrated in Fig. 33.

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, requiring the removal of the ice or the use of special methods giving inferior results. If the filters are drained for cleaning, trouble also arises from the freezing of the sand. The cleaning of beds is thus not likely to be done as promptly as desirable, and the result of winter operation will be a decreased effective area and a lowered efficiency. 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.

106. Filter 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 there is 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.

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. 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.

The depth of sand must be sufficient to form an effective filter and, besides, to allow of several scrapings without renewing the sand. 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 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. A depth of 3 feet is about right, with one foot allowed for scraping.

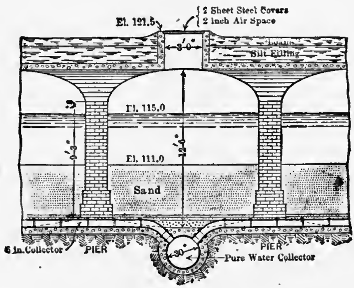
The depth of water on the filter should be sufficient to enable the desired maximum head to be used without reducing the pressure in the filter below atmospheric; and as the resistance is nearly all at the surface of the sand, the depth must be about equal to the maximum head to be used in forcing the water through the filter. 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.

107. 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 either of large vitrified pipe, as in Fig. 40, or of masonry. The branch drains are usually of 4 to 8-inch round or special tile, laid with open joints.

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. 40. 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.

The gravel used should be carefully screened and, if dirty, washed. It is readily sized by revolving or fixed screens, using for this purpose



Section through Main Drain.

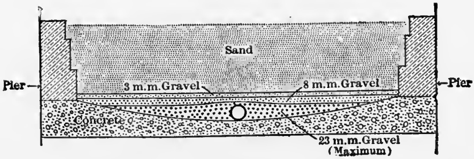


Fig. 40. Details of Drains, Albany Filter Beds.

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. 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 greater depth of filter, and have a detrimental effect in compacting the sand. A maximum loss of head of 4 or 5 feet may be taken as good practice.

108. Arrangement of Inlet and Outlet. 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 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. 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.

The regulation of head requires, first, some form of measuring device, such as a weir or orifice, by which the rate of filtration can be ascertained at any time by floats and indicators; and, second, the controlling of the head on this weir or orifice 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.

Automatic regulators for delivering water at a constant rate are in use in a number of places. They usually 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. 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. An overflow pipe is also necessary to provide against any failure on the part of the inlet regulator. This connects with the drain pipe.

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.

109. 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 by means of broad thin shovels a layer of clogged sand from ½ to 1 inch in thickness. The surface is then smoothed with a rake. The sand is removed from the filter by means of wheelbarrows or small cranes, and deposited at a convenient point where it is cleaned after a considerable quantity has accumulated—or is wasted in case it is cheaper to use new sand than to clean the old. 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 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. After cleaning and filling, the filter should be started slowly and gradually. The sand that has been removed is allowed to accumulate until it is desired to replace it in the bed. Before replacing it, however, it is washed to free it of the accumulated sediment. Various effective devices are employed for this washing operation.

110. 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 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. The careful control of the operations is a matter of great importance. 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.

111. Results of Filtration. The most apparent result of filtration is in the removal of all the suspended matter in the water; but more important than this, a sand filter will remove very nearly all the bacteria originally present in the water. This is specially exemplified in the reduction of the deathrate from typhoid fever following upon the installation of a filter plant. But to secure continually good results it is essential that such works as filter plants be under efficient control.

RAPID FILTERS.

112. The *Rapid,* or as it is often called, the *Mechanical 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 like the sand filter in that the filtering material consists of a bed of 2 to 4 feet of sand or crushed quartz, but in other respects the construction and operation are widely different. The chief points 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. Methods of operation and mechanical details are, to a large extent, covered by patents, and the filters are manufactured and sold by various filter companies.

Briefly, the filter consists of a wooden, steel or concrete tank in which the filtering material is placed and supported on a system of screens of various designs. In some forms the tanks are open and operated by gravity, and in others are closed and operated by pressure from the pipe system. When the filters become clogged through the accumulation of sediment on the surface, they are washed by forcing water in a reverse direction through the sand. During this process, the sand is, in most cases, agitated by means of mechanical agitators reaching deeply into the sand layer or by means of compressed air.

Two well-known types of filters are illustrated in Figs. 41 and 42. In the first, the water enters the settling chamber at the bottom, passes up through the central tube to the top of the filter and thence downwards through the sand to the collecting pipes located between filter and settling tank. Where a coagulant is used it is introduced before admission to the filter bed. The figure shows the agitators which are operated when the filter is washed. When not in use they are raised out of the sand. The wash water passes off on all sides over the top of the inner tank. In the other type the settling tank is not connected with the filter. No agitators are used; but to loosen the sand air is passed through the bed from below. In both forms shown the filtration is by gravity. The Warren filter is another

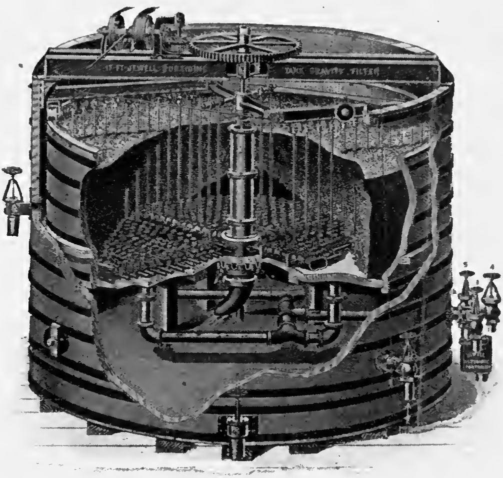


Fig. 41. Jewell Gravity Filter.

type commonly employed. It uses agitators like the Jewell, but the settling tank is disconnected from the filter.

113. Principles of Operation. The action of mechanical filters is somewhat unlike that of 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 section 101. 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 the coagulant which enables such high velocities to be employed. To avoid too frequent washing, it is common to employ heads as high as 10 to 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 about 15 to 20 minutes. The amount of wash water used is commonly from 2 to 5 per cent of the applied water, which fact must be considered in determining the gross capacity of the plant. Crushed quartz is often used for the filtering material, but ordinary sand would probably do as well if of very uniform grain so as not to be

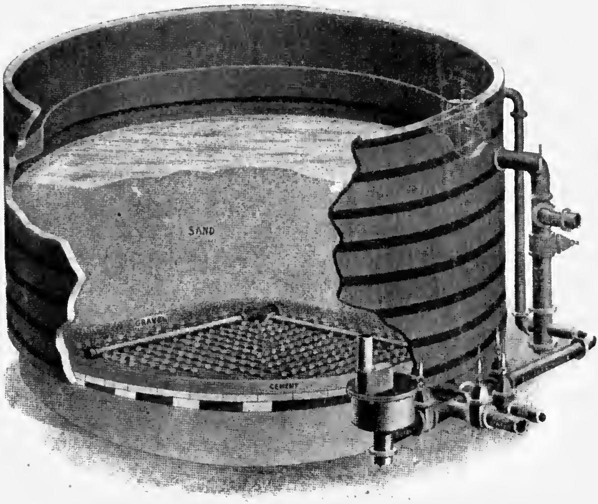


Fig. 42. Continental Gravity Filter.

carried away in the washing process. The coagulant employed is usually sulphate of alumina, but common alum is sometimes used. The relative merits of these and some other coagulants have been discussed in section 101. Ferrous sulphate is also employed as a coagulant with good results. Rapid filter plants were formerly installed by the companies who manufactured the mechanical devices, but some very complete plants have lately been designed and constructed under the supervision of consulting engineers. In this case some of the patented forms of strainers have been adopted and the other details been designed by the engineer in charge.

In most of the early plants rapid filters have been adopted with particular reference to the removal of turbidity or color, but in some of the more recent plants the elimination of bacteria has been the chief object.

Extensive experiments with rapid filters indicate that when they are properly operated, turbidity can be practically all removed, a large percentage of color, and a considerable portion of dissolved organic matter. With the quantities of coagulant ordinarily used, they are probably slightly less efficient than sand filters in the removal of bacteria. In some places satisfactory clarification is obtained without a coagulant, but for good bacterial results a coagulant is necessary.

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 proper operation of a mechanical plant involves greater care on the part of the attendants than that of a sand filter. It is fully as important in this case also that the whole plant should be under the control of bacteriological tests, regularly and frequently made. Many points of operation, such as period between washings, wasting of water, thickness of sand layer, and best kind of sand, can be learned only after experience in the light of such analyses.

Considering the economic advantages of mechanical 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 with a sand filter plant, and in small plants where the unit for sand 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. On the other hand, if the total annual expense of sand filters and mechanical filters is equal, the evidence points to the desirability of adopting sand filters, especially for sewage-laden waters, but the difference in efficiency is not great enough to warrant any considerable additional expense. With very turbid waters both systems are likely to prove unsatisfactory unless supplemented by adequate settling basins.

OTHER METHODS OF PURIFICATION.

114. Aeration. Aeration of a water, or the bringing of the water into close contact with the air, is useful in certain cases. It will have little or no effect upon organic matter present, but it does have a very important action in the case of waters coming from stagnant ponds and reservoirs in which putrefactive changes have taken place. Such waters will have offensive odors and tastes, due to the dissolved gases contained, and it is in the removal of these gases that the process of aeration can be successfully applied. It has been so used in a large number of cases.

Aeration may be accomplished by forcing air into the mains, or by passing the water over cascades or weirs, or by spraying it from a fountain into a reservoir as is done in many places, or by still other methods. The benefit of aeration explains why a well water raised by buckets is more commonly free from bad tastes and odors than where a pump is used, although such odors and tastes are not in themselves dangerous to the health.

115. Softening Water. Water is rendered hard by the presence of lime and magnesia, chiefly in the form of carbonates and sulphates, but occasionally as chlorids and nitrates. The carbonates cause so-called temporary hardness (removable by boiling), while the sulphates and other compounds cause permanent hardness. 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. Sulphate of lime precipitates at high temperatures and forms a very hard, objectionable scale, particularly if the water contains other suspended matter. The softening of water is therefore of great economical importance.

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.

The $CaCO_3$ (lime carbonate) and the CO_2 (carbonic acid) are present in the water; the $Ca(OH)_2$, ordinary lime, is the chemical added.

To remove the sulphate, sodium carbonate (Na_2CO_3) is used. The reaction is

$$CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4.$$

The carbonate of lime precipitates out as before while the sodium sulphate (Na_2SO_4) is not especially objectionable. Various methods of carrying out the details of the process, relating principally to the application of the chemical 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, a solution of slaked lime in water.

In general the water to be treated is run into large tanks, the chemical added and then the precipitate allowed to settle as far as practicable. The water is then drawn off and the remaining precipitate removed by rapid filtration. In purifying water for boiler use the precipitate can be removed to a sufficient extent by the use of settling tanks alone. The chemicals used are lime and usually soda ash, or crude sodium carbonate. The cost for such small plants is reported to be from 4 to 15 cents per 1,000 gallons.

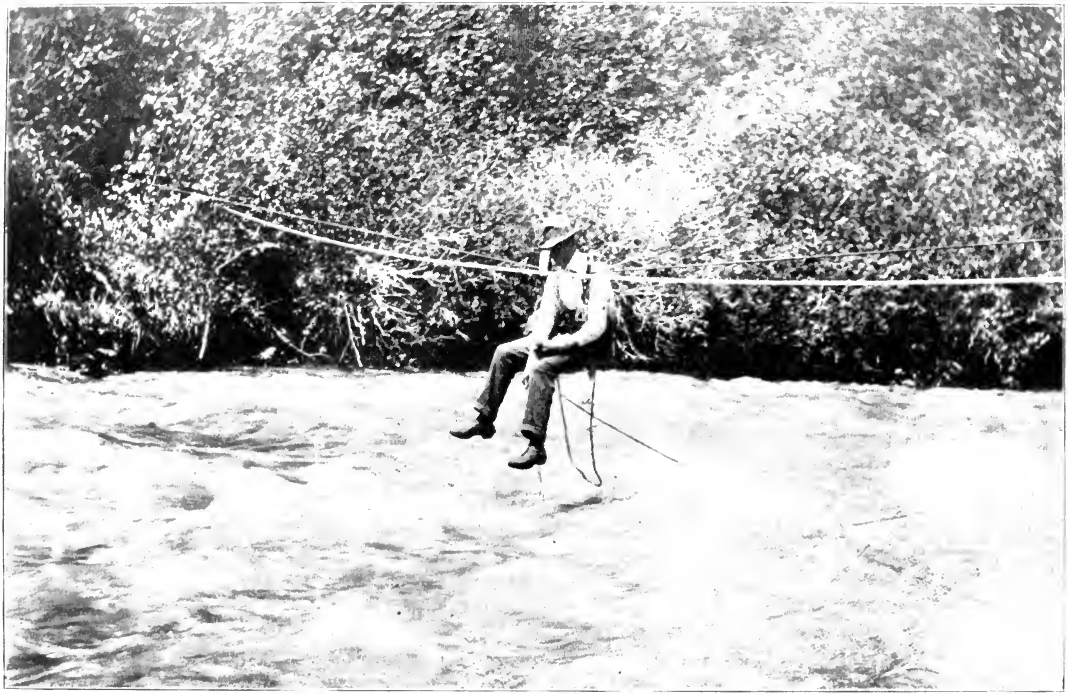
Many scale preventives have been proposed for use in boilers, but probably the best in general use is sodium carbonate. This breaks up the sulphates as previously shown, and thus prevents the formation of a hard deposit; but the precipitation of the carbonates is increased by the process. The sodium sulphate remains in solution, but should not be allowed to concentrate too greatly or it will cause foaming.

116. 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). 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. 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.

Filters of this class are not often used for municipal purification, but are admirably adapted for schools, garrisons, prisons, or hotels as well as for private use.

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.

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 the influence of boiling water for a period exceeding 10 to 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. 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 seacoast for similar purposes.



HYDROGRAPHER GAUGING A WILD MONTANA STREAM

IRRIGATION ENGINEERING

The control and distribution of water for irrigation presents to the engineer about the same problems as the control and distribution of water for domestic and manufacturing purposes in large cities and towns. The water must be diverted from a flowing stream at a sufficient elevation to command the territory to be irrigated; or it must be impounded in reservoirs at a season of floods or unusual flow due either to the more or less regular recurrence of rainy seasons, or to the melting of snow and ice. Again, it may be derived from subterranean sources, either deep or shallow wells, lifted to the proper elevation by pumps, and stored in reservoirs for future use.

The principal difference between securing a water supply for domestic and irrigation purposes, is that in the former case the water must be as pure as possible, while in the latter the impurities that gather in ponds and streams have a distinct commercial value as fertilizers, and hence are especially desirable for irrigation purposes. The sewage of the city of Paris, for example, is used successfully for purposes of irrigation.

Except where water is scarce and difficult to procure at a reasonable cost, it is not necessary to make such elaborate provision for the distribution of water for irrigation purposes as is the case for domestic supply. In the majority of cases, water for irrigation flows in open channels. However, to guard against loss by evaporation and seepage, it may be necessary to distribute the water through a system of underground or enclosed pipes or conduits.

Irrigation works in the West range from rude and simple ditches taking their supplies from mountain brooks where the water has been diverted by means of small brush dams, to great masonry walls blocking the outlet of deep canyons, holding back the water, which is transported through canals, pipes, or flumes to lands situated miles away.

QUANTITY OF WATER REQUIRED

It will first be necessary to define the term *duty of water* as applied to irrigation, in order to determine, at least approximately, the amount of water necessary to supply a given area for a specific purpose; and it may be defined as *the ratio between a given quantity of water and the area of the land which it will irrigate.* On the duty of water depends the financial success of every irrigation enterprise, as it involves the dimensions and cost of construction of canals and reservoirs, and the feasibility of furnishing a sufficient supply of water at a reasonable cost.

Units of Measure for Water Duty and Flow. In hydraulic problems the standard unit is the *cubic foot.* In irrigation problems, however, where large volumes of water are to be considered, the cubic foot is too small a unit and the *acre-foot* is the standard adopted by irrigation engineers. An acre-foot is the amount of water which will cover an acre of land one foot in depth—or, in other words, 43,560 cubic feet. In considering mining streams, as rivers or canals, the *volume* of flow must be coupled with a factor representing the *rate* of flow. As in other hydraulic problems, the time unit usually employed by irrigation engineers is the *second*; and the unit of measure of flowing water is the *cubic foot per second* (or the *second-foot,* as it is termed for brevity). Thus the number of second-feet flowing in a canal is the number of cubic feet passing a given section in a second of time. Another unit still generally employed in the West is the *miner's inch.* This differs widely in different localities, and is generally defined by State statute. In California one second-foot of water is equal to about 50 miner's inches, while in Colorado it is equivalent to about 38.4 miner's inches. The period of time during one season, determined between the first watering and the completion of the last watering, is the *irrigating period.* This is usually divided into several *service periods*—that is, the times during which water is allowed to flow on the land for any given watering. The irrigation period in most of the Western States, extends from about April 15 to August 15. The service period, or the duration of one watering, is generally from 12 to 24 hours, according to soil and crop; and the number of waterings making up the irrigation period varies between 2 and 5, depending upon the soil, climate, and crop.

SHOSHONE FALLS OF THE SNAKE RIVER, IDAHO

Height 210 feet. The bed of the Snake river canyon lies at a general level of 700 feet below the valley which its waters irrigate through the Twin Falls project. The Shoshone Falls, Twin Falls (187 feet), and Augur Falls (140 feet)—three of the six large falls in the canyon—all lie within five miles of the town of Twin Falls, center of an area of 250,000 acres of rich soil to be reclaimed. The waters are impounded by means of a dam at Milner, at the head of the canyon, 35 miles upstream from Twin Falls.

TABLE I

Units of Measure

1 Second-foot	= 450 gallons per minute
1 Cubic-foot	= 7.5 gallons.
1 Cubic-foot	= 62.5 pounds at average temperature.
1 Second-foot	= 2 acre-feet in 24 hours (approximately)
1,000,000 Cubic feet	= 23 acre-feet (approximately).
100 California inches	= 4 acre-feet in 24 hours.
100 Colorado inches	= 5⅙ acre-feet in 24 hours.
50 California inches	= 1 second-foot.
38.4 Colorado inches	= 1 second-foot.
1 Colorado inch	= 17,000 gallons in 24 hours (approximately).
1 Second-foot	= 59.5 acre-feet in 30 days.
2 Acre-feet	= 1 second-foot per day.
100 California inches	= 3.97 acre-feet per 24 hours.
1 Acre-foot	= 25.2 California inches in 24 hours

The duty of water may be expressed by the number of acres of land which a second-foot of water will irrigate; by the number of acre-feet of water required to irrigate an acre of land; or in terms of the total volume of water used during the season. It is also sometimes expressed in terms of the expenditure of water per linear mile of the canal, when the location of the canal has been previously determined. On account of the losses of water by evaporation, seepage, etc., while flowing through the canal, care should be taken to state whether the duty is reckoned upon the basis of the water entering the canal or upon the amount of water applied to the land. In a long line of canal, the losses may amount to 33⅓ per cent of the total amount entering the canal, and the relative duties would vary accordingly.

The duty of water in various portions of the West is variable. Experiments show that it is rapidly rising; for, as land is irrigated through a series of years, it becomes more saturated, and as the level of the ground-water rises, the amount of water necessary to the production of crops diminishes. The cultivation of the soil causes it to require less water; and the adoption of more careful methods in designing and constructing distributaries, and care and experience in handling water, increase its duty. In the same State, and even in the same neighborhood, the duty will vary with the crop, soil, and altitude.

Experiments have shown that a good, heavy rain amounting to 5½ inches soaks into the earth to a depth of from 16 to 18 inches.

If this amount of water were applied three times in the season, it would be equivalent to a total depth of 16½ inches to the crop. An average depth of 3 inches of water on the surface is sufficient to water an average soil thoroughly. In sandy soil, 4 inches is required, which is equivalent to about ¼ of an acre-foot per acre. The average crop requires about four waterings in the season. At the above rate, this would be equivalent to nearly an acre-foot per acre. It will be seen, therefore, that from 1¼ to 2 acre-feet in depth applied to the land is sufficient to irrigate it. In estimating the duty of water stored in a reservoir, allowance must be made for losses due to evaporation and absorption in conducting the water to the fields. This will seldom average below 25 per cent, and the amount of water stored in the reservoir must be increased accordingly.

In every irrigated area, only a small percentage of the total area commanded is irrigated in any one season. Some of the land is occupied by woods, farm-houses, or villages. Some is occupied by pasture lands that receive sufficient moisture by seepage from adjoining irrigated fields, and some lands are allowed to lie idle during a season. From estimates made of the area under cultivation in wild portions of the West, it is found that if water is provided for 500 out of every 640 acres, it will be sufficient to supply all of the demands of the cultivators. It will be seen, therefore, that the actual duty of water when estimated on large areas is at least 20 per cent greater than the theoretic duty per acre.

In examining into the feasibility of a proposed irrigation project, the first consideration is the land to be irrigated. The area of this must be considered; its proximity to markets; the nature of the soil; the climate; and the value and character of the crops. The value and ownership of the land must also be considered, for, unless the proposed irrigation results in increased quantity and improved quality of the crops, the value of the land will not be enhanced and the project will result in failure.

To determine the area and configuration of the land under consideration, a topographical survey will be necessary, and a plot should be made to as large a scale as possible, upon which the contours should be drawn at intervals of from 5 to 10 feet.

Having determined all the matters relative to the area, quantity, and value of the land, and the necessity of supplying water for irrigation, the next step is to determine the source of supply and its location relative to the lands. This supply may be taken from an adjacent perennial stream; or it may be necessary to transport it from a neighboring watershed; or, again, it may be necessary to conserve in reservoirs the intermittent flow of minor streams. Or the water may be derived from subterranean sources, from which it may flow under pressure, or be lifted by pumps to storage reservoirs for future distribution. The relation of the water supply to the land, the extent and value of the latter, and the volume and permanency of the former, are the vital points to be determined in the preliminary investigations of an irrigation project.

If the source of supply is a perennial stream, it must be examined as to its velocity and quantity of flow during high and low stages. A topographical survey must be made of the watershed, to determine its area, the minimum and maximum depth of rainfall, and the probable run-off. Minor streams of more or less intermittent flow must be examined as to the feasibility of bringing them together and impounding their supplies for use over dry seasons. Subterranean supplies must be examined by driving test wells to determine their source, quantity of flow, and permanency.

Classification of Irrigation Works. Having determined the source of water supply and its relation to the irrigable lands, the next step is the design of the irrigation works. These may be divided into two great classes:

(1) *Gravity Works.* This name covers all those forms of irrigation by which the water is conducted to the land with the aid of gravity or natural flow. It includes:

Perennial canals;	Inundation canals;
Periodic and intermittent canals;	Storage works;
Artesian water supplies;	Subsurface or ground-water supplies.

(2) *Pumping or Lift Irrigation.* Under this name are included those forms of irrigation in which the water does not reach the land by natural flow, but is transported to it by pumping, by means of:

Animal power;	Water wheels;
Windmills;	Elevators;

Steam Power.

The sources of supply for gravity works are perennial streams; intermittent streams; artesian wells; or the storage of perennial, intermittent, or flood waters. The sources of supply for lift irrigation may be wells, canals, storage works, or flowing streams.

The conditions necessary to the development of an irrigation canal are: *First,* that it shall be carried at as high a level as possible in order to command as great an area as possible on either side; *second,* it should be fed by some source of supply that will maintain the water at a constant level; third, it should have such a slope and velocity as to prevent as far as possible the deposition of sediment and the growth of weeds, and at the same time such a velocity that the cross-section may be a minimum for a given discharge, provided that the scouring action is not so marked as to endanger the canal itself.

Climate, geology, and topography are the determining factors as to the particular class of works adopted to a particular region. Perennial canals deriving their supplies from perennial streams or storage reservoirs, may be divided into two classes according to the location of the headworks—namely:

(1) Highline Canals;
(2) Low-Service or Deltaic Canals.

Highline canals are usually designed to irrigate lands of limited area, and are given the best possible slope in order that their grades may be as high as possible, to command the maximum area of land. In such canals the headworks are usually located high up on the streams, frequently in rocky canyons where the first portions of the line may encounter heavy and expensive rock work. Low-service canals are constructed where the majority of the lands are situated in low-lying and extensive valleys, and when the location of the head of the canal depends, not so much on its being at a relatively high altitude and commanding a great area, as upon the suitability of the site for purposes of diversion.

Deltaic canals have been constructed in India and Egypt at the deltas of some of the great rivers. They are low-lying canals of relatively large cross-section and low velocity of flow.

Intermittent and periodic canals are usually of small dimensions, commanding relatively small areas of land, and are generally employed by individual farmers to eke out a supply for which the annual precipitation is nearly sufficient.

Storage works may be of almost any capacity, depending upon the nature of the project, the source of supply, and the area of the land to be irrigated. They may be built in connection with perennial canals, and are especially necessary in connection with intermittent canals, artesian wells, and subsurface or ground-water supplies.

Fig. 1 *A*.

Fig. 1 *B*.
Two Views at Deming, New Mexico, Showing Windmills for Pumping Water for Irrigation.

Inundation canals are used almost exclusively in India and Egypt, and derive their supply from streams the beds of which are at a relatively high altitude compared with the surrounding country. It is only necessary, when the water in the river is high, to make a cut through its banks and permit it to flow out into the canals, which distribute it over the surrounding country. They rarely require any permanent headworks to control the entrance of the water to the canal.

Pumping or Lift Irrigation. Frequently large volumes of water are found situated at such low levels that the water cannot be distributed by gravity, and must be raised by pumps or other lifting devices to be distributed directly over the lands, or stored in reservoirs

Fig. 2. Private Reservoirs and Garden, in the Region of the Rio Grande Project, New Mexico.

Water pumped from well by gasoline engine, and stored in reservoir to obtain sufficient head for irrigation.

for distribution by gravity. Lift irrigation may be employed to raise water from the canals to higher levels.

When the gravity sources have been entirely utilized, large areas of land may still be brought under cultivation by the employment of pumps. As irrigation is practiced, the subsurface soil becomes saturated, the ground-water level rises, and much of the water delivered by gravity may be pumped up and re-employed for irrigation, increasing thereby the duty of the ultimate sources of water supply. In this country the value of irrigation by pumping is coming to be fully appreciated, and thousands of acres of land are amenable to cultivation by this method, that cannot be irrigated by gravity supplies.

Windmills have been extensively used in portions of the West for raising water for irrigation, by the individual farmer. Fig. 1 shows the windmills in and around Deming, New Mexico. In this particular instance, the water is reached at a depth of about 60 feet from the surface, and is pumped to a wooden tank, from which it is distributed over the land as required. Windmills, however, are rather

Fig. 3. Irrigation Canal in Experimental Gardens, Los Cruces Agricultural College, Rio Grande Project, New Mexico.

The Rio Grande project calls for the building of a $6,000,000 dam near Engle, New Mexico, to supply water for lands in New Mexico, Texas, and Old Mexico. By decision of the U. S. Supreme Court, 60,000 acre-feet are awarded to the Mexicans on account of priority of water rights obtained during the period of Mexican possession of that territory.

expensive to maintain, requiring constant repairs; and the storage tanks need constant attention.

The use of steam pumps for irrigation purposes is increasing, and there are many places where water can be pumped at comparatively small cost, providing irrigation for lands that otherwise would remain useless. The chief differences between pumping for irrigation and pumping for mines and waterworks, is in the height to which the water has to be raised. For irrigation purposes, the head will rarely exceed 25 feet, or possibly 35 feet, it being necessary to raise the water from a river or well to a sufficient height to enable it to flow over the field by the action of gravity.

The varieties of pumps most commonly used in the West are:

Centrifugal pumps, which, for this operation, require small steam or oil engines;

Vacuum pumps, pulsometers, etc.;

Pumping engines.

Parts of a Canal System. Taking up the design of gravity systems, a great perennial canal consists of the following parts:

(1) Main canal;
(2) Head and regulating works;
(3) Control and drainage works;
(4) Distributaries and laterals.

The principal units of this system are the main canals and distributaries. Between different canal systems, the greatest points of difference are found in the headworks and in the first few miles of diversion line, where numerous difficulties are frequently encountered, calling for variations in the form and construction of drainage works and canal banks.

The headworks usually consist of the diversion weir, with its scouring sluices; of the head regulating gates at the canal entrance; and of the head or first escape gates. The control works consist of egulating gates at the head of the branch canals, and of escapes on the line of the main and branch canals. The drainage works consist of inlet or drainage dams, flumes, or aqueducts, inverted siphons, and drainage cuts. In addition to these works, there are usually constructed falls and rapids for neutralizing the slope of the country; and tunnels, cuttings, and embankments. Modules or some form of measuring box or weir, are necessary for the measurement of the discharge.

The headworks of a canal are usually located high up on the supplying stream, in order to command a sufficient area and to tap the stream where the water is clear and contains the least amount of silt. By so locating the headworks, it is usually possible to reach the watersheds with the shortest possible diversion line. The disadvantages of this class of location are serious, since the canal line is seen to be intersected by hillside drainage, entailing serious difficulties in construction; and as the adjacent slopes of the country are heavy, much expensive hillside cutting is required.

Diversion lines are those portions of a canal line that are required to bring the water to the neighborhood of the irrigable lands. Since they do not of themselves command any irrigable land, the endeavor should always be to reduce the length of diversion lines to a minimum, so that the canal shall command irrigable land and derive revenue at the earliest possible point in its course.

The alignment of the canal should be such that the canal will reach the highest part of the irrigable lands with the least length of line and at a minimum expense of construction. The line of the canal should follow the highest line of the irrigable lands, skirting the surrounding foot-hills, and passing down the summit of the water-shed dividing the various streams. The best alignment can be determined upon only after careful preliminary and location surveys have been made of the country involved. These should include a complete topographic survey, and a plot of the same to as large a scale as possible, the contour lines being spaced at vertical intervals of from 5 to 10 feet. On such a plot it is possible to lay down, with a close degree of approximation, the final position of the canal line. Such a plot frequently renders possible an improved location, saving many miles of canal by the discovery of some low divide or some place in which a short but deep cut or tunnel will save a long roundabout location. The final location may now be made in the field, with the aid, perhaps, of a few short trial lines. A direct or straight course is the most economical, as it gives the greatest freedom of flow with the least erosion of the banks. It also greatly diminishes the cost of construction, as well as the losses by absorption and evaporation consequent on the increased length of a less direct location. It is an error in alignment to adhere too closely to grade lines following the general contour of the country. By the insertion of an occasional fall, it may be possible to obtain a more desirable location and to diminish the cost of construction by avoiding a natural obstacle.

The careless location of curves is a serious error in alignment, as the insertion of sharp bends results in the destruction of the banks, or requires that they be paved to protect them from erosion. Curvature diminishes the delivering capacity of a canal. As the cross-section becomes smaller or the velocity increases, the radius of curvature should be correspondingly increased. To maintain the discharge of a canal constant throughout its length, either its cross-section or its grade should be increased in proportion to the sharpness of the curve. Such obstacles as streams, gullies, and unfavorable or low-lying soil or rocky barriers are frequently encountered in canal alignment, and the best method of passing these must be carefully studied. It may be cheaper to carry the canal around these obstructions; or it may be better to cross them at once by aqueducts, flumes, or inverted siphons, or to cut or tunnel through the ridges. Careful study should be made of each case, and estimates made of the cost, not only of first construction, but of ultimate maintenance. In crossing swamps or

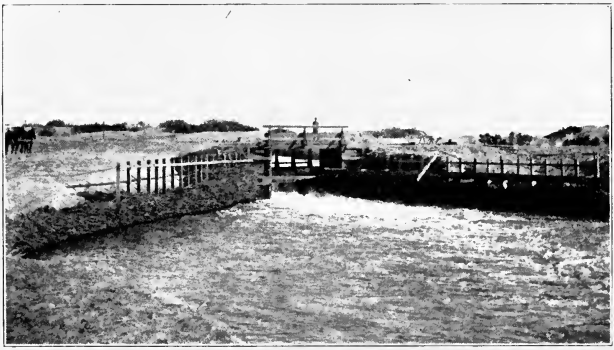


Fig. 4. Drop on Main Imperial Canal at Sharps, in the Imperial Valley, California. Showing brush wing walls, type of drop, etc.

sandy bottom lands, it may be cheaper, because of the losses due to evaporation and absorption, to carry the canal in an artificial channel. If water be abundant, it may be less expensive, on hillside work, to simply build the canal with an embankment on its lower side, permitting the water to flood back on the upper side according to the slope of the country. The relative cost of building a hillside canal wholly in excavation or partly in embankment, should be considered. If the hillside is steep and rocky, the advisability of tunneling, of building a masonry retaining wall on the lower side of the canal, or of carrying the water in an aqueduct or flume, will have to be considered.

In finally locating an expensive work, borings and trial pits should be made—the former by means of a light steel rod, and the latter by simple excavation—in order to discover the nature of the material to be encountered. In making the final survey of a canal, it is well to place at convenient intervals permanent benchmarks of stone or other suitable material. The establishment of these along the side of the canal in some safe place, will give convenient datum points to which levels may be referred wherever it may be necessary to make repairs or new branch lines.

Slope and cross-section are dependent the one upon the other. Having determined the discharge required, the carrying capacity for this discharge may be obtained by increasing the slope and the consequent velocity and diminishing the cross-sectional area, or by increasing the cross-sectional area and diminishing the velocity. The proper relation between cross-section and slope requires the exercise of careful judgment. In order to reduce the deposition of silt and the growth of weeds to a minimum, it is desirable to give the water as high a velocity as the material will stand. This may result, however, in bringing the water to too low an elevation to command the area of land desired. Too great a cross-sectional area may result in excessive cost, if the material is in rock or for any other reason is difficult to remove. Other things being equal, the correct relation of slope to cross-section is that in which the velocity will neither be too great nor too small, and yet the amount of material to be removed will be reduced to a minimum. When the fall will permit, the slope of the bed of the main canal should be less than that of the branches; and the latter should be less than that of the bed of the distributaries and laterals, the object being to secure a nearly uniform velocity throughout the system, so that sedimentary matter in suspension will not be deposited until the irrigable lands are reached.

In order that the proper slope may be chosen—one that will prevent deposit on the one hand and at the same time not erode the banks—it is necessary to know the limiting velocities for different materials. In a light, sandy soil, surface velocities of from 2.3 to 2.4 feet per second, or mean velocities of 1.85 to 1.93 feet per second, give the most satisfactory results. Velocities of from 2 to 3 feet per second are ordinarily sufficient to prevent the growth of weeds and the deposition of matter in suspension; and other things being equal, this velocity should be maintained whenever possible. Ordinary soil and firm, sandy loam permit velocities of from 3 to 3½ feet per second; while in firm gravel, rock, or hardpan, the velocity may be as high as 5 or 7 feet per second. Brickwork or heavy dry-laid paving or rubble will not stand velocities higher than 15 feet per second; and only the most substantial form of masonry construction is capable of resisting still higher velocities.

The grade required to produce these velocities is dependent chiefly upon the cross-sectional area of the channel. Much higher grades are required in canals of small cross-sectional area than in large, in order to produce the same velocity. The velocity required being known, the grade may be ascertained from Kutter's or some similar formula. Or, on the other hand, if the grade is limited, the resulting velocity may be determined. In large canals of 60 feet bed width and upwards, and in sandy or light soil, grades as low as 6 inches to the mile produce as high velocities as the material will stand. In firmer soil this grade may be increased to 12 to 18 inches to the mile, whereas smaller channels will permit of slopes of from 2 to 5 feet per mile, according to the material and dimensions of the channel.

The most economical form of cross-section of channel is one with vertical sides and a depth equal to one-half of the bottom width; but of course this form is applicable only to the firmest rock. The best trapezoidal form is one in which the width of the water surface is double the bottom width and equal to the sum of the side slopes. The side slopes above water level should be as steep as the nature of the material will permit. The particular form of cross-section will depend upon the nature of the material and the topography. The greater the depth, other things being equal, the greater will be the velocity and the consequent discharge for the same form of cross-section.

Very large canals, such as some of those in India, have been given a proportion of depth to width similar to that of great rivers. This proportion has been found to be most nearly attained when the bed width is made from 15 to 16 times the depth. In sidehill excavation, the greater the proportion of depth to width, the less will be the cost of construction; and in rock and heavy material it is desirable to make the bottom width not greater than from 2 to 3 times the depth.

The cross-section of a canal may be so designed that the water may be wholly in excavation, wholly in embankment, or partly in excavation and partly in embankment. The conditions that govern the choice of one of these three forms, are dependent upon the alignment and grade of the canal, and incidentally upon the character of the soil. It may be desirable at times to keep the canal wholly in cutting, provided the topography and consequent location will permit of it. For if the material of which the banks are constructed is light and porous, the water may filter through and stand in stagnant pools on the surface, causing unnecessary waste as well as unsanitary conditions. If the material is impervious and will form good, firm banks,

Fig. 5. Completed Portion of Upper Main Canal at Pogue Flats, Okanogan Project, Washington.

The Okanogan project, by storage of the waters of the northern Salmon River, provides for the irrigation of 8,000 acres of fruit lands near Okanogan.

it may be well to keep the canal in embankment where possible, although this may necessitate the expense of borrowing material. To reduce the cost of construction, it is desirable, where the location will permit, to keep a canal half in cut and half in embankment, thus reducing to a minimum the amount of material to be handled.

Most main canals follow the slope of the country in grade contours running around sidehill or mountain slopes. In such cases it is necessary to build an embankment on one side only, when the cutting will be entirely on the upper side. If there is a gentle slope on the upper side, and consequently an embankment on that side, it is desirable to run drainage channels at intervals from this embankment to prevent the water making its way through it to the canal. These drainage channels may be taken through the embankment into the canal, or may be led away to some natural watercourse. In large canals it is always desirable to have a roadbed on at least one bank, and the width of this will determine the top width of the bank. The inner surfaces of the canal are usually made smooth and even; while the top is also made smooth, with a slight inclination outward to throw drainage water away from the canal. The inner slopes of the banks vary in soil from 1 to 1 to 4 to 1, according to the character of the material. In firm, clayey gravel or hardpan, slopes of 1 to 1 may be constructed. In ordinary firm soil mixed with gravel, or in coarse, loamy gravel, slopes of $1\frac{1}{2}$ to 1 are necessary. In other soils, a slope of 2 to 1 will be required; and light, sandy soils will require a slope of 4 to 1.

The top width of the canal bank is generally from 4 to 10 feet, according to the material and depth, and whether or not the water is in embankment. If there is to be no roadway on the top of the embankment, and the surface of the water does not rise more than a foot or so above the foot of the embankment, a top width of 4 feet is sufficient. Where the depth of water on the embankment is greater, the top width should be 6 or 5 feet, and in light soil should be 10 feet. It may be necessary to build a puddle wall in the embankment, or to make a puddle facing on the inner slope in case the material is particularly pervious. The same effect is obtained by sodding or by causing grass to grow on the bank. During the construction of the work, the material may be compacted by putting it in place in layers and thoroughly rolling. The carrying capacity of a canal should be so calculated that the surface of the water where in cut shall not reach within less than one foot of the ground surface. In fill, the surface of the water should not come within less than $1\frac{1}{2}$ feet of the top of the bank, while in extreme cases it may be unsafe to allow the water to approach closer than 2 feet of the top of the bank.

The term *weir*, as distinguished from *dam*, refers to any structure for the impounding or diversion of water, over which flood waters may flow without endangering the structure. Thus weirs are usually built across streams, near the heads of canals, for the diversion of the water of the stream into the canal, while the surplus water flows over the weir and passes down the stream. In some cases, however, dams are built, over which it would be unsafe to allow water to pass, and the surplus is then discharged through a spillway constructed around one end of the dam.

The headworks of a canal are usually located at the highest possible point where the stream emerges from the hills. At such a point, the topography of the country and the elevation of the stream make it possible to conduct the water to the irrigable lands with the shortest diversion line and the least loss of head. At such a location, also, the width of the channel of the stream is usually contracted, permitting of a reduction in the length of the weir and in the consequent cost of construction. At times it may be necessary to locate the headworks at a point where the distance between banks is greater, in order that the depth of water on the weir during flood discharge may not be excessive, thus endangering its stability. Obviously the points to be kept in mind in the location and design of headworks are: Permanency; economic first cost; cost of maintenance; and the command of the maximum area of irrigable lands at the least cost of diversion canals. These requirements may at times be conflicting; and complete surveys, supplemented by careful study of the problem in all of its details, are necessary to a satisfactory final conclusion.

The headworks of a canal consist of:

(1) The diversion weir or dam;
(2) The scouring sluices;
(3) The regulator at the head of the canal, for its control;
(4) An escape for the relief of the canal below.

Diversion weirs or dams, as the term indicates, are structures built for the purpose of backing up or impounding a body of water in order that it may be diverted into the proper channels to reach the irrigable lands. They may be classified as follows, according to the construction of the superstructure:

(1) Temporary brush and bowlder barriers;
(2) Rectangular walls of sheet and anchor piles, filled with rock or sand;
(3) Open weirs and closed weirs;
(4) Wooden crib and rock weirs;
(5) Masonry weirs.

Brush and bowlder weirs are of the simplest construction,

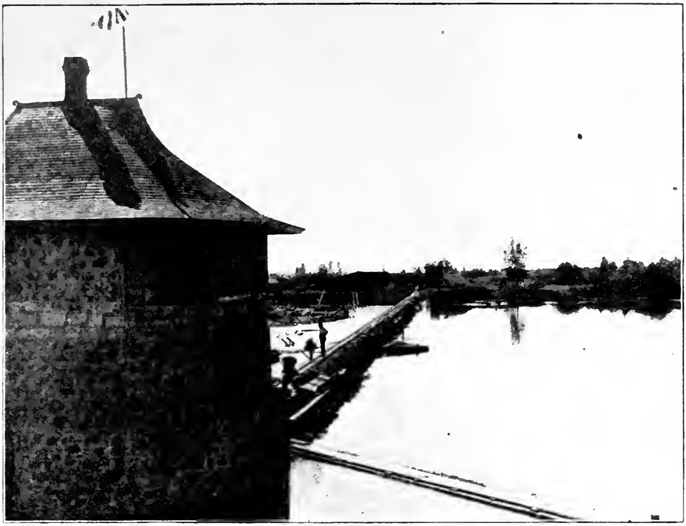


Fig. 6. Gate House and Dam, Sunnyside Canal Heading.

Fig. 7. Gate House and Intake Gates, Upstream Face, Sunnyside Canal.

Two views of the Yakima Project, Washington. This project calls for the reclamation of 160,000 acres of land in the Yakima valley. Flood water is saved by storage in Lakes Keechelus, Karhees, and Clealum, in the Cascade Range. The private headworks of the Sunnyside Canal Company were purchased by the Government, and have been reconstructed of reinforced concrete, the canal being sufficiently enlarged to care for the necessary irrigation waters.

ORANGE GROVE AND CEMENT IRRIGATION CANAL NEAR RIVERSIDE, CALIFORNIA temporary in character; and are formed by driving stakes across the channel, and attaching thereto fascines or bundles of willows; these are laid with the brush end upstream, and are weighted with bowlders and gravel. More willow or cottonwood branches are laid on top of these, which are in turn weighted down; and this operation is continued until the barrier reaches a height of three or four feet. Such structures are built without any engineering knowledge, and are constructed by the individual farmer for diverting the water to his land.

Rectangular pile-weirs consist of a double row of piling driven into the river bed. The rows are spaced about 6 feet apart, and the individual piles about 3 feet apart, center to center. Between these piles is driven sheet piling to prevent the seepage of water; and the upper portion of the structure is planked, forming a sort of crib, which is filled with sand, gravel, etc. Weirs of this kind seldom exceed 8 feet in height, are cheaply built, and form a substantial barrier that may last for many years. They are not adapted to locations in which the flood flow is very great, owing to the danger of undermining.

A *closed weir* is one in which the barrier is solid nearly the entire width of the channel, the flood waters passing over the crest. Such weirs have usually a short open space, called the *scouring sluice*, in front of the regulator, the object of which is to produce a swift current past the regulator opening, to prevent the deposit of silt at that point. An open weir is one in which scouring sluices are provided throughout its entire length.

An *open weir* consists of a series of piers of wood, iron, or masonry, set at regular intervals across the bed of the stream, and resting on a masonry or wooden floor constructed flush with the river bed, and protected from scouring by curtain walls constructed up and down stream. The piers are fitted with flashboards or movable weirs so constructed that the afflux height of the river may be controlled. The distance between piers varies from 3 to 10 feet. In a river subjected to sudden floods, the gates are constructed to drop automatically when the flow overtops them.

A *closed weir* consists of an apron built upon a substantial foundation and carried across the entire width of the weir, flush with the level of its bed, and protected from erosion by curtain walls up and down stream. The superstructure may consist of a solid wall, or may consist in part of upright piers, the interstices being closed by some temporary construction. This latter portion of the weir is called the *scouring sluice.*

Wooden crib and rock weirs are generally built when the bed and banks of the river are of heavy gravel and bowlders or of solid rock; and they may be employed for diversions of greater height than is possible with open weirs. Crib weirs consist of a framework of heavy logs, drift-bolted together, and fitted with broken stones and rocks to keep them in place. The structures may be founded by sinking a number of cribs, one on top of the other, to a considerable depth in the ground, or they may be bolted to the solid rock. They may consist of separate cribs built side by side across the stream and fastened firmly together, or they may be built as one continuous weir.

In case the weir is intended to be permanent, only steel and masonry should be used in the construction. Masonry weirs may be built of plain concrete throughout; of concrete reinforced with steel; of uncoursed rubble masonry laid in cement mortar; of ashlar masonry; of brick masonry; or of combinations of all of these. Or they may be built of loose rubble maintained in place by masonry walls.

As regards the superstructure, masonry weirs may be classified according to the method of discharge, as follows:

(1) Simple weirs of moderate height and overflow, with a clear fall to the bed of the stream;

(2) Simple weirs with clear fall to an artificial apron;

(3) Weirs with roller-way on lower face;

(4) Weirs of heavy cross-section curved on the face to break the fall of the water;

(5) Weirs with clear fall to a water cushion.

Where possible, masonry weirs should be founded on solid rock; but on account of the depth of the rock, comparatively small, low dams may be constructed upon heavy timber foundations resting upon piles or a solid hardpan or gravel foundation. In such cases, however, the greatest care must be exercised to prevent undermining of the foundation; and a large factor of safety must be used in designing the superstructure, to render such a dam secure. Close sheet-piling should be carried entirely across the upper end of the foundation, placed in a trench dug down to and well into hardpan or clay, and puddle should be well rammed into the trench on either side of the sheeting. The sheeting should be carried up two or three feet higher than the foundation, and the space between it and the masonry filled with concrete or puddle (see Fig. 8). The foundation should be carried downstream for a distance at least equal to the height of the dam, to act as an apron to prevent undermining from the falling water. At the end of the apron should be placed sheet piling similar to that already described.

All dams or weirs over 10 or 12 feet in height should be founded on rock. Masonry is practically inelastic; and so an inflexible foundation is necessary if there are to be no unknown and unprovided-for stresses in the structure caused by a yielding and insecure

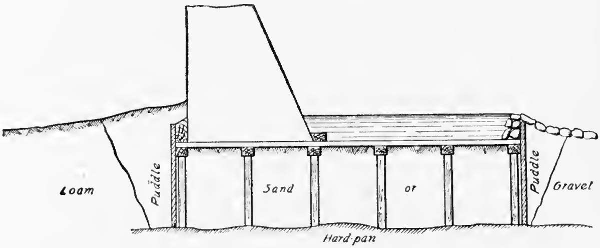


Fig. 8. Use of Sheet Piling in Weir Construction.

foundation. All loose or decomposed rock on the foundation should be removed; and if the rock has a smooth top surface, a shallow trench should be excavated into it the entire length of the dam, or the bed should be cut in steps to prevent leakage and the sliding of the dam on its bed. Every square foot of the foundation should be thoroughly cleaned and washed, and covered with a thick bed of mortar just before the masonry is laid upon it; and the spaces between the dam and the sides of the excavation should be cleaned out and filled with concrete. Seams in the rock should be filled with cement, to prevent the escape of the water behind the dam. Springs should be diverted or sealed up, to avoid risking the stability of the structure.

The ends of the dam should be carried to bed-rock if possible, and treated in the same manner as the bottom. If rock does not extend up to the level of the crest of the dam at its ends, these should be carried some distance into the banks, to prevent leakage around them; or tight masonry river-walls should be carried for some distance upstream from the dam. The banks for a short distance downstream, if of earth, must be protected from wash by similar walls.

In building a stone dam, care should be taken to break courses at every joint, the masonry being uncoursed rubble, except the faces, which should be broken ashlar. Every stone and its bed must be perfectly clean and have a damp surface when it is laid; and all spaces must be absolutely filled with mortar or fine concrete. If a tight dam is to be built, these precautions must be rigidly observed.

Fig. 9. Assuan Dam, River Nile, Egypt.

About 1¼ miles long. Contains over 1,000,000 tons of masonry. Reservoir extends 140 miles up valley, with greater capacity than any other artificial basin in the world. First insubmergible dam without overflow waste weirs. Silt-laden flood waters pass freely through the sluices placed at different levels, which are gradually closed after subsidence of flood, to provide water to supplement the natural supply during period of deficient discharge.

In constructing the dam, the work, so far as possible, should be carried up so that the finished masonry is approximately horizontal. All stone should be set by derrick; and no dressing of stones should be done on the wall, since that would be likely to disturb or jar the masonry already set. Careful attention to details will result in a dam perfectly water-tight under all practicable heads.

Concrete has been used in the construction of a number of dams, and the use of the material for this class of work is increasing daily.

Concrete reinforced with steel has been used in the erection of several notable structures, resulting in marked economy of cost.

The designing of masonry dams and weirs will not be taken up here, as the matter is fully treated in "Masonry and Reinforced Concrete" and "Water-Power Development."

Scouring Sluices. Scouring or undersluices are built in the bottom of every well-designed dam or weir, and at the end immediately adjacent to the regulator head. Their function is to remove by erosive action of the water any sediment that may be deposited in front of the regulator. In case the flow in the stream is sufficient to warrant it, the sluices are kept constantly open, preventing the deposit of silt by the constant motion of the water in front of the regulating head. If the flow of water is not sufficient to permit of leaving the sluices constantly open, they should be opened during flood and high waters, creating a swift current which is effective in removing the silt deposited during slack water.

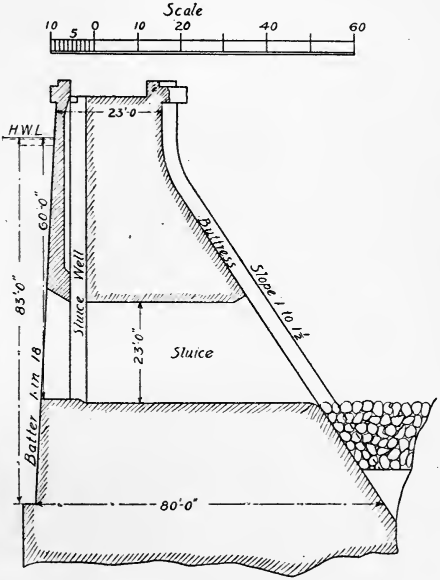


Fig. 10. Section of Assuan Dam on River Nile.
For photographic view, see page 22.

The scouring effect of sluices constructed in the body of the weir is produced by two classes of structures: (*a*) *open scouring sluices,* and (*b*) *undersluices.* The open scouring sluice is practically identical with the open weir, as the latter consists of scouring sluices built across the entire width of the channel. When the weir forms a solid barrier across the channel and is open for only a short portion of its length adjacent to the canal head, the latter is called a *scouring sluice.* The waterway of a scouring sluice is open the entire height of the weir from crest to bed of stream, and consists of foundation, floorway, and superstructure. The floor must be deep and well constructed, and carried for a short distance up and down stream upon either side of the axis of the weir. On this floor are built piers, grooved for the insertion of planks or gates that may be opened or closed at will.

Undersluices are more generally constructed where the weir is of considerable height and the amount of silt deposited is small. The opening does not extend as high as the crest of the weir, nor does the sill of the sluiceway necessarily reach to the level of the stream bed. It is necessary, however, that the sill be as low as the sill of the regulator head.

Regulators. A diversion weir retards the flow of the stream, and raises its level to such a height as to enable it to pass into the head of the canal. The regulator controls the flow of water, admitting it to the canal when necessary, and at other times causing it to pass on down the stream over or through the weir. The regulator should be so located that the water held back by the weir will pass rapidly, and with the least loss of head, through it and into the canal. To accomplish this successfully, the canal head should be placed immediately adjacent to the weir and continuous with it. The weir should not be so located and aligned as to cross the stream diagonally, either toward or from the regulator head. In the former case, it tends to force the water against the regulator, causing unnecessary scour at that point and undue pressure upon the head. In the latter case, the reverse action would take place, and it would not be effective in directing the water into the canal. The best alignment for the weir is to have it cross the stream at right angles to the line of the regulator. This gives a clear, even scour past the regulating gates and keeps them clear of silt, at the same time furnishing the required amount of water.

The regulator should not be placed at a distance from the end of the weir, as thereby dead water, in which deposits of silt occur, is created between the weir and the regulator, blocking the entrance to the canal and diminishing the available supply.

The type of regulator employed depends upon the character of the foundation and the permanency desired. Regulators may be classified according to the design of the gate and its method of operation. With any type of foundation, varying degrees of permanency may be given to the superstructure, and various methods employed for operating the gates.

Regulators may be classified as follows:

(*a*) Wooden gates in timber framing;
(*b*) Wooden gates in masonry and iron framing;
(*c*) Iron gates in masonry and iron framing.

Fig. 11. Gates at Sharp's Heading.
At this point, seven miles east of Calexico, Mexico, the Alamo River drops from the Imperial Canal.

They are further classified according to the method of operating the gates, as:

(*a*) Flashboard gates;
(*b*) Gates raised by hand lever;
(*c*) Gates raised by chain and windlass;
(*d*) Gates raised by screw gearing.

Simple flashboard gates can be used only when the pressure upon them is low. When the gates are under great pressure, the opening is generally closed by a simple sliding gate that may be raised by hand lever or windlass. Under high pressures a double series of gates may be employed, one above the other, each separately raised by hand lever or windlass.

The regulator should be so designed that the amount of water admitted to the canal may be perfectly controlled at any stage of the stream. To this end the gates should have such dimensions that they can be quickly opened or closed as desired. When the canal is large and its width considerable, the regulator should be divided into several openings, each closed by a separate and independent gate. The channel of the regulator way should consist of a flooring of timber or masonry, to protect the bottom from the erosive action of the water; and the side walls or wings should be of the same construction, to protect the banks. The various openings will be separated by piers of wood, iron, or masonry; and the amount of obstruction offered to the channel should be a minimum, so that the width of the regulator head will be as small as possible for the desired amount of opening. For convenience in operating, the regulator should be surmounted by arches of masonry or a wooden flooring, so as to provide an overhead bridge from which the gates may be raised or lowered. The height of the regulating gates, and the height of the gates surmounting them, must exceed the height of the weir crest by the amount of the greatest afflux height which the flood water may attain, so that the flood flow may not top the regulator and destroy the canal. The regulator must be firmly and substantially built to withstand the pressure of floods, and a drift fender should be built immediately in front of, or at a short distance in front of, the gates, to protect them from floating logs, debris, etc.

The regulator head should be placed as close to the end of the weir as possible, to prevent the deposit of silt at that point. It is sometimes necessary, however, to set the regulator back in the canal a short distance, owing to the nature of the banks and the consequent difficulties of construction. Under such circumstances the escape should be placed in front of and adjacent to the regulator, to relieve it of undue pressure.

Escapes. In order to maintain complete control over the water in a canal, provision should be made for disposal of any excess flow that may arise from sudden and excessive rains or floods, or of any water not required for irrigation. This is effected by means of *escapes* (or, as they are more commonly called, *wasteways*). These are usually short-cuts from the canal to some natural drainage channel or stream into which the excess water may be wasted without fear of damage. Wasteways may also be utilized for flushing the canal, preventing the deposit of silt, or scouring it out.

Opening the distributaries relieves the main canal, and opening the escapes in turn relieves the distributaries. Escapes should be provided at intervals along the entire canal line, the lengths of the

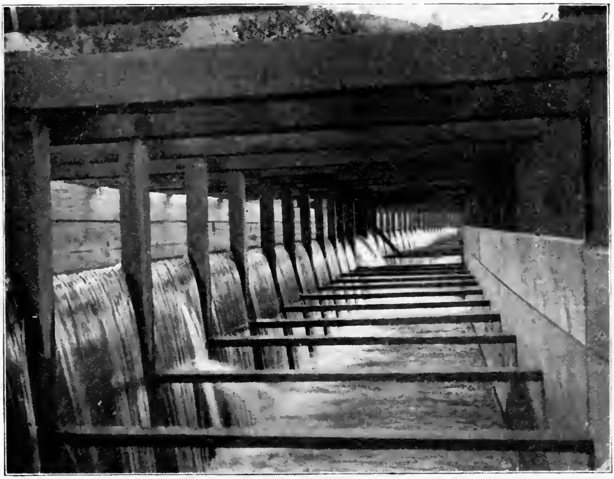


Fig. 12. Interior of Automatic Spillway for Control of Water Supply in Canal on Yakima Project, Washington.

intervals depending upon the topography of the country, the danger from floods or inlet drainage, and the dimensions of the canal. The first or main escape should be located at a distance from the regulator not exceeding half a mile, so that in case of accident to the canal the water may be drawn off. This main escape may also be used as a flushing gate for the prevention and removal of silt deposits. The slope of the canal between its head and the escape, should be decreased in order that the matter carried in suspension may be deposited at the head of the escape.

Escapes should be located above weak points, as embankments, flumes, etc., in order that the canal may be quickly emptied in case of accident. Their channels should be of the shortest possible length; and they must have sufficient capacity to carry off the whole body of water from both directions so that the canal below the escape may be emptied for repairs while the canal above the escape is still in operation.

The greatest danger to a canal occurs during local rains when water is not being used for irrigation, leaving the canal supply full, while the discharge is augmented by the flood waters. It is essential where a drainage inlet enters the canal, that an escape be placed opposite it for the discharge of surplus water. In order that the escape may act most effectively, the slope of its bed should be increased by at least a foot immediately below its head; besides this, the slope of the remainder of the bed should be somewhat greater than that of the canal, and it should tail into the drainage channel with a drop of a few feet.

Escape heads, and the regulators necessary to their operation, are similar in design to the regulating gates at the head of the main canal. They should be as large as can be conveniently operated, and there should be a sufficient number of them to discharge the canal without unduly retarding the velocity of flow. The gates may be of timber or iron, and may be framed between piers of timber, masonry, or iron. They are operated the same as the gates at the head of the main canal; but, as the pressure upon them is not so great, the lifting apparatus need not be so elaborate.

Sand gates are practically escape gates, but they may be so designed as to be of service only for scouring and the removal of sediment. The main gates on a canal system should be designed as much for scouring purposes as for the control of the water in the canal.

DRAINAGE WORKS

When the diversion line of a canal is carried around a hill or is located on sloping ground, provision must be made for side drainage; and this is sometimes one of the most difficult problems encountered in the design and construction of irrigation works. The higher the canal heads up on a stream, the more liable it is to encounter side drainage. On low ground and flat slopes, drainage may be provided for by diverting the water-courses through cuts into natural drainage areas. When this cannot be done, the drainage may be passed by one of the following methods:

(*a*) Simple Inlet Dam;
(*b*) Level Crossing;
(*c*) Flume or Aqueduct;
(*d*) Superpassage;
(*e*) Culvert or Inverted Siphon.

Inlet Dams. In case the drainage is intermittent and its volume small compared with that of the canal, the water may be admitted directly into the canal, and the excess of flow may be discharged at the first escape along the line of the canal.

Inlet dams may be built of wood, masonry, or loose stone. If the depth of the canal is small, and the height of the fall from the crest of the dam to the bed of the canal is also small, a wooden flooring may be laid in the bed of the canal, and a barrier or dam of piles and sheet-piling built across the upper side. After a time, the sediment carried by the stream will fill in behind the dam to the level of its crest, and the water will then simply fall over the crest of the dam onto the wooden apron. The inlet dam may be built as a loose rock retaining wall, in which case the bed and banks of the canal below, as well as the opposite side, should be riprapped to protect them from erosion. If the drainage flow is considerable, more substantial works may be required, and a masonry inlet dam will be needed, a portion of the canal channel being built of masonry, and the opposite bank protected with loose stones.

Level Crossings. When the discharge of the drainage channel is large and is at the same level as the canal, it may be passed over, or under, or through the latter. In the latter case the water is admitted by an inlet dam on one side, and discharged through an escape in the opposite bank. The discharge capacity of the escape must be sufficient to pass the greatest volume of flood flow likely to enter; and a set of regulating gates should be placed in the canal below the escape, in order properly to control the amount of water passing down the canal.

Flumes and Aqueducts. The term *flume* is more generally applied to a wooden structure intended to carry the waters of a canal around steep, rocky hillsides or across drainage lines. The term *aqueduct* is more generally applied to works of greater magnitude, which are built of more permanent material, as iron or masonry. In case the drainage encountered is at a lower level than the bed of the canal, it may pass under the latter, the canal passing over the drainage in the form of a flume. The drainage stream must be carefully studied as to its discharge, in order that the capacity of the waterway under the flume may be ample to pass the maximum flood dis-

Fig. 13. Huntley Main Canal, Huntley Project, Montana, on Line of Northern Pacific Railroad.

At the point where the canal disappears under the crossing, it is carried by means of pressure pipe underneath Pryor Creek, which in turn is carried over the canal by reinforced-concrete structure, this arrangement being necessary on account of the flood conditions of Pryor Creek. View taken from above tunnel No. 3.

charge. Particular attention must be paid to the stability and permanency of the foundations of the flume; and they should not unnecessarily obstruct the waterway, as this would cause a high velocity of flow in the drainage channel, which might result in scouring the bed and possibly destroying the work. In connecting the ends of the flume with the canal banks, care must be exercised that leakage does not occur at such points.

The length of a flume or aqueduct may be shortened by making the approaches at either end, of earth embankments, causing the canal at the end of the flume to flow on top of an embankment, which must be of the most careful construction and of sufficient width so

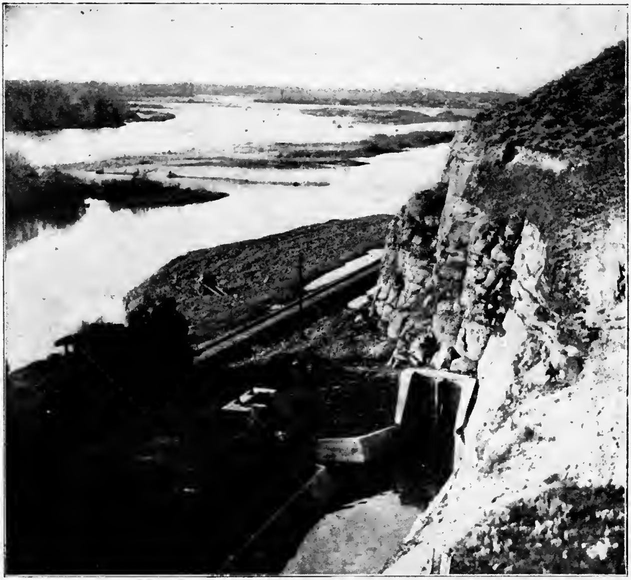


Fig. 14. General View of Wasteway and Portal of Tunnel No. 3 on Huntley Main Canal, and Passing Northern Pacific Train.

The Huntley project, Montana, calls for the irrigation of 26,000 acres of Crow Indian Reservation lands by means of water diverted from the lower Yellowstone River.

that it will not settle greatly or be washed away. The embankments should be faced with masonry abutments and wing-walls at their junction with the flume, to protect them from erosion. The cross-section of the flume may be diminished, and in consequence its slope may be made somewhat greater than the canal at either end, so as to carry the required volume of water. This will result in a considerable saving in the cost of construction.

Bench flumes are built on steep hillsides to save the cost of canal construction. Structures of this kind should never be built on embankments, but in excavation or upon trestles to avoid the danger of subsidence and consequent destruction. The excavated bench should be several feet wider than the flume, and the flume itself should be supported on a permanent foundation of mudsills or posts.

Superpassages. When the canal is at a lower level than the drainage channel, the latter is carried over the former by what is termed a *superpassage*. This is practically an aqueduct, though differing from an aqueduct in some important details. The volumes of streams that are to be carried in superpassages are variable: at times the streams may be dry, while again their flood discharges may be enormous. Since the discharge of the canal below the superpassage is fixed, no provision is necessary for passing flood waters. The waterway of the superpassage, however, must be ample to carry the greatest flood flow that may occur in the stream; and especial care must be exercised in joining the superpassage to the stream bed above and below, to prevent injury by the violent action of flood waters. Constructing superpassages of wood is not to be recommended, as the alternate wetting and drying will soon result in decay. They have occasionally been built of iron; but proper precautions must then be taken to provide against expansion and contraction, since at times the amount of water flowing in the channel will not be sufficient to maintain the temperature constant. The most common practice in the United States is to carry the canal under the stream by means of an *inverted siphon*, though at times it may be more convenient to carry the stream through the siphon and then build the canal over the siphon in the form of an aqueduct. The proper dimensions of the siphon may be computed by means of any one of the numerous formulæ for the flow of water in pipes.

Siphons may be of wood—built up—or they may be of wrought iron, cement, reinforced concrete, or wooden pipe.

DISTRIBUTARIES

Distributaries bear the same relation to the main canal system as the service pipes in a city waterworks system. From them the irrigator takes his water supply for irrigating his crops. These minor ditches, or *laterals*, should never be diverted from the main canal or from its upper branches, as these should have as few openings as possible, in order to reduce the liability to accident to a minimum. The water should be drawn at proper intervals from the main canal into moderate-sized branches so located as to command the greatest area of land and to supply the laterals and small ditches of the irrigator in the simplest and most direct manner possible. Wherever water is plentiful and not particularly valuable, it is conducted to the lands in open ditches and laterals excavated in the earth. Where, however, water is scarce and its cost relatively high, the losses from evaporation and percolation should be reduced to a minimum. In such a case the laterals should consist of wooden flumes; of paved or masonry-lined earth channels; or, in extreme cases, the supply may be conducted underground in pipes and applied to the crops from these, instead of being flowed over the surface. This results in the highest possible duty and the most effective use of the water. The most economical location of a main canal is along the surface of a ridge, whence the water may be supplied to its branches and to private channels on either side. Such a location, however, can seldom be realized. The distributaries taken from the main lines should conform to the dividing lines between water courses; and the capacity of each will then be proportioned to the duty required of it, the bounding streams limiting the area it will have to serve.

In preparing for the design of a distributary system, a careful topographical survey should be made of the entire area to be traversed by the system; and the location should be made only after a thorough study of the topographical features. Cuts and fills should be balanced as nearly as possible, and the loss of water by percolation reduced to a minimum. Provision should be made for the least mileage of channels consistent with the command of the greatest area of irrigable land on either side. Attention should be paid to drainage, that it may be secured with the least possible interference with the proposed channels and distributaries.

For the complete and efficient distribution of water, the engineer should consider the design and location of the distributaries of as much importance as the design and location of the main branches. Besides the alignment, attention should be paid to the character of the soil; to the safe and permanent crossing of drainage lines; and the capacity of the channels should be proportioned to the duty required, the cross-sectional area being reduced as the quantity of water is decreased by its diversion to private channels.

The bed of the distributary should be above the level of the bed of the canal, in order to get the clearest water from the canal, and in order that it may be kept at as high a level as possible and provide surface irrigation throughout its length. Care should be taken, in level country, that the drainage lines into which the distributaries tail should have ample capacity to accommodate any flood flow that it may be necessary to discharge into them, as otherwise the streams might become clogged and flood the surrounding country. The slope of the distributary should be as nearly as possible parallel to the surface of the land it traverses, thus commanding the maximum area and at the same time avoiding expensive and heavy construction. To realize this condition, falls may be frequently introduced, and storm-water escapes into natural channels should be provided at intervals in the course of the distributary.

In determining the dimensions of distributaries, it must be borne in mind that the greater the amount of discharge, the smaller will be the cost of maintenance. A single channel the width of which is equal to that of two separate channels, will have a carrying capacity more than double that of each of the smaller channels; and the cost of patrolling and maintaining the single channel will be one-half that of the two smaller channels. Experience has shown that irrigation can be most profitably carried on from channels having a width of 18 feet on the bottom and a depth of about 4 feet of water. The surface of the water should be maintained at from 1 to 3 feet above the surrounding area, to provide gravity irrigation and to reduce the loss by absorption.

Distributary heads should be arranged much as are the heads of main canals and escapes. They consist essentially of two parts: (*a*) a *regulator* or *check* below the head on the main canal, to divert the water into the distributary; and (*b*) a *regulating gate* in the distributary, to admit the proper amount of water. These heads usually consist of a wooden fluming, built in the form of an apron to the bed of the distributary, with planking to protect the banks. In this flume are built the gates, consisting either of flashboards or of simple wooden lifting gates.

Distributary channels are often open channels; but when the

EXCAVATED CHANNEL OF MAIN CANAL, KLAMATH PROJECT, OREGON AND CALIFORNIA

View a short distance below the town of Klamath Falls, Oregon. This project calls for the drainage and irrigation of 200,000 acres of land in southern Oregon and northern California.

water is conveyed in pipes, these may be of steel, of wood, or of reinforced concrete. Steel pipe should always be coated with hot asphaltum to protect it from corrosion. This class of pipe will bear internal pressures of from 100 to 200 pounds per square inch. Air-valves should be introduced at all high places; blow-off valves at the proper intervals; and a standpipe air-chamber may be placed at the highest point in the line.

Several types of wooden pipes are in use, but for the larger conduits, wooden stavepipes are to be preferred. The pipe is built continuously in the trench, and the staves are formed with radial edges and bound together by means of round or oval bands of steel or iron, spaced and sized according to the pressure. Two types of stave-pipe are employed. In one of these (the Allen patent), the outside and inside surfaces of the staves are 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, 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 in length.

Fig. 15. Erecting a 12-Inch Pipe of California Redwood.

Stavepipe is suited to pressures up to as high as 100 pounds per square inch. Above this limit it is less economical than steel, owing to the size and number of the bands required. It has been built as large as 9 feet in diameter. The staves should be made of clear stuff, somewhat seasoned. California redwood and Oregon fir are the materials most generally used for irrigation pipes in the West.

For the smaller sizes of conduits, bored pipe is better adapted. This pipe is made from solid logs, but depends for its strength upon spiral bands of flat iron wrapped tightly about it from end to end. The exterior of the pipe is coated with pitch to protect the bands and preserve the pipe. 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 tightly together by means of a wooden ram. The pipe is made in sections 8 feet long and varying in diameter from 2 to 17 inches. The bands are spaced according to the pressure; and branch connections are made by means of cast-iron specials having long sockets into which the wooden pipe is driven. This pipe is very durable, and is cheaper than cast-iron pipe, especially when transportation rates are high.

STORAGE RESERVOIRS

All varieties of natural or artificial impounding reservoirs designed for the storing up of superfluous or flood waters, are termed *storage works*. Such works are intended to equalize and insure a constant supply of water during each and every season, regardless of the amount of rainfall. They may be classified (1) according to the character and location of the storage basin, or (2) according to the design and construction of the retaining wall or dam enclosing the basin.

Under the former classification are:

(*a*) Natural lake basins;

(*b*) Reservoir sites on natural drainage lines, as a valley or canyon through which a stream flows;

(*c*) Reservoir sites in depressions on bench lands;

(*d*) Reservoir sites that are in part or wholly constructed by artificial methods.

The second classification gives rise to the following structures:

(*a*) Earth dams or embankments;

(*b*) Combined earth and loose-rock dams;

(*c*) Hydraulic mining type of dam, or dams constructed of loose rock or loose rock and timber;

(*d*) Combined loose-rock and masonry dams;

(*e*) Masonry dams.

Knowing the position and extent of the irrigable lands, an extended and careful preliminary survey of the surrounding region should be made, in order to discern all possible reservoir sites. The catchment areas of these should be mapped, gaugings made of the streams, and examinations conducted for the purpose of ascertaining the minimum discharge as well as their flood heights, from which may be estimated the amount of evaporation and percolation.

The location of the reservoir site having been decided upon in a general way, a detailed survey of it should be made. This can best be done with the transit and stadia. If the valley above the dam site be long and narrow, the reservoir also will be of this shape, and in many cases two or more valleys of contributing streams may unite in one reservoir site. It is desirable that the enclosing hills be steep, and that the valley be narrow at the dam site, to avoid shallow water and expensive construction. A basin or valley with little longitudinal slope will provide a given amount of storage with less height of dam than one with a steep longitudinal slope, and for that reason is to be preferred. The larger the drainage area above the dam site, the greater the quantity of yield; and so the position of the dam with reference to the head of the area is important. The reservoir must be sufficiently high up the valley for the water to command the irrigable lands.

In conducting the surveys of the reservoir site, a tentative location of the dam should be made, and an approximate idea of its height obtained. The extreme height of the dam should be taken as the basis of the surveys, and a closed contour run out at this level around the entire site. A main traverse should be run through the lowest line of the site, from the dam to the extreme end, where it will connect with the top contour. Cross-section lines may now be run with the stadia from this line, and the topography of the site sketched by means of 5-foot contours and plotted to as large a scale as possible. Such a map will enable the engineer to determine the capacity of the reservoir for different depths of water.

The dam site should be surveyed with greater care and more in detail, several possible sites being cross-sectioned and plotted to 1-foot contour intervals, to a scale of 100 feet to the inch if possible. In deciding upon the exact location of the dam, the cost will generally be the controlling consideration. The elevation of the crest having

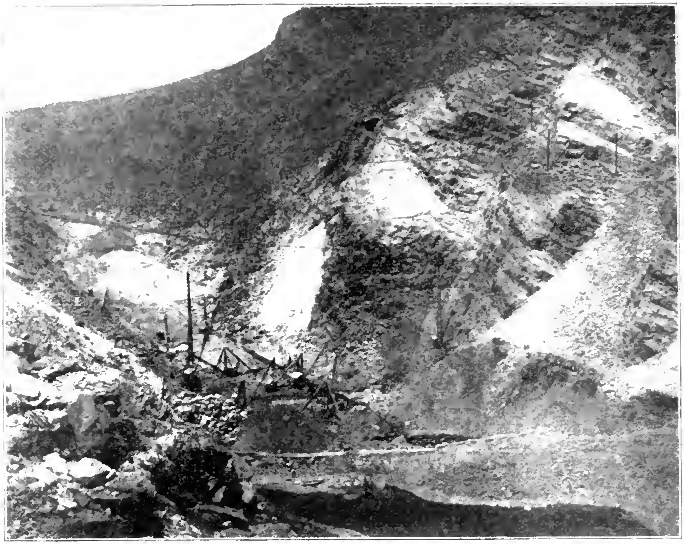


Fig. 16. Site of Roosevelt Dam, Salt River, Arizona.

Fig. 17. Foundation Work on Roosevelt Dam, Salt River, Arizona.

The Salt River project calls for the construction of the Roosevelt Dam, an arched masonry gravity structure, 280 feet high and 1,020 feet long, forming the Tonto Reservoir holding 1,200,000 acre-feet of water, to be used in reclamation of 250,000 acres of arid land near Mesa and Phoenix.

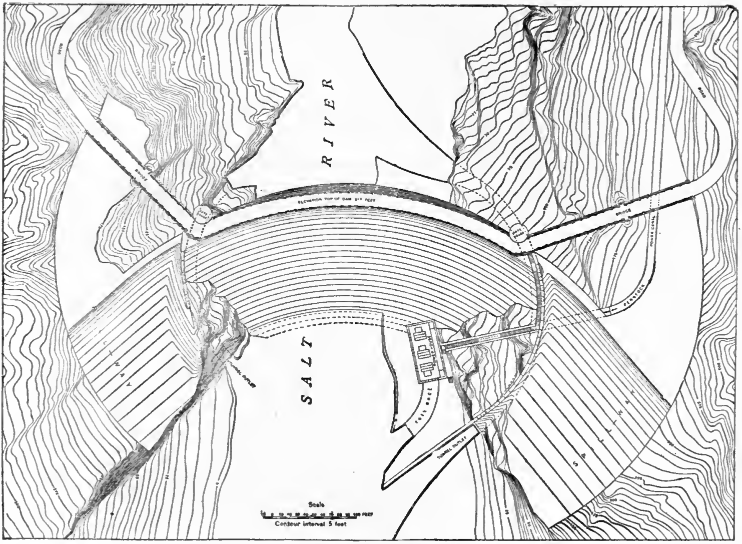


Fig. 18. Plan of Salt River Dam.

Showing the Location of the Power House, Power Canal, Tail-Race, and Contour of the River Banks.

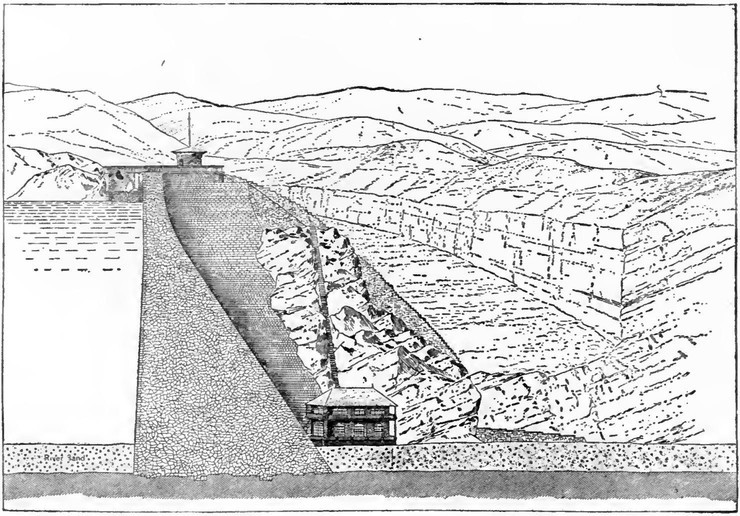


Fig. 19. Section of Salt River Dam and Elevation of Spillway and Power House.

been fixed, that location is best which requires the least expense for excavation and construction; and this is generally where the least quantity of material is necessary in construction. With such a knowledge of the topography of a catchment basin and of the reservoir and dam sites as the resulting map will give, the engineer may readily compute the cost of construction of dams for various heights, as well as the contents of the reservoir for these heights, determining thereby what height and location of dam will be most economical of construction. In many cases a straight dam at the narrowest point offers the best location; but conditions of topography are frequently met with which make more economical a dam whose center line is curved upstream.

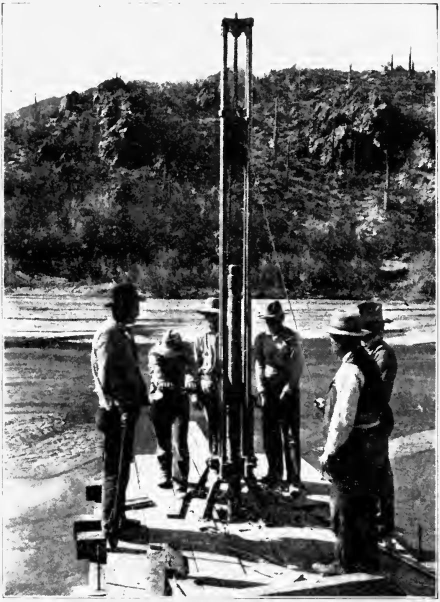


Fig. 20. Drilling in a River Bed to Find Depth of Solid Rock for Foundation of a Proposed Dam.

Having determined the desirability of a reservoir site, both topographically and hydrographically, test boring and trial pits should be sunk at various points in the reservoir basin, and particularly at the dam site, to ascertain the character of the soil and the character and dip of the strata underlying the proposed reservoir. The geological conformation may be such as to contribute to the efficiency of the reservoir, or it may be so unfavorable as to be irremediable by engineering skill. A reservoir site situated in what is called a *synclinal valley* is the most favorable, as in such a case the strata slope from the hills toward the lower lines of the valley, so that any water that falls upon the hills will find its way by percolation through the strata into the reservoir, thus adding to the storage volume. On the other hand, an *anticlinal valley* is the least favorable for a reservoir site, as the strata slope away from the reservoir site, and water falling on the hills is diverted by percolation away from the reservoir site and to other drainage areas. A third class of geological formation intermediate between the other two, is that in which the valley has been eroded in the side of the strata that dip in one direction. In such a case the upper strata lead water from the adjoining hills into the reservoir, while the strata on the lower side tend to carry the water away from the reservoir by percolation. If the surface of the proposed reservoir site is made up of a deep bed of coarse gravel, sand, or even limestone, crevices in the latter or between the interstices of the former will tend to diminish the capacity of the reservoir by seepage from it. On the other hand, the geological formation may be most unfavorable, yet, if the surface of the reservoir site be covered with a deep deposit of alluvial sediment or of clay or dirty gravel, or of other equally impervious material, little

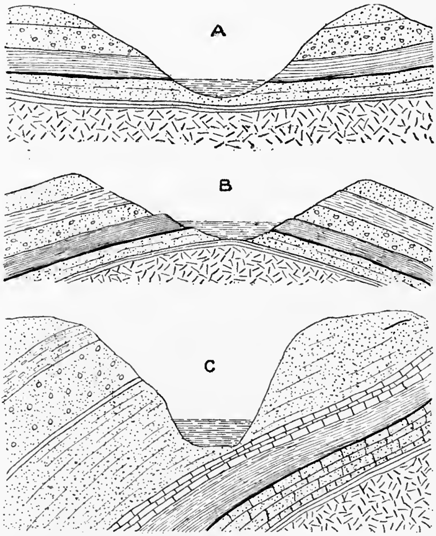


Fig. 21. Types of Geological Strata for Reservoir Sites. *A*—Synclinal Valley: *B*—Anticlinal Valley; *C*—Intermediate Type.

danger is to be apprehended from seepage. Fig. 21 illustrates the three types of geological strata described above.

In considering the relation of the reservoir site to the irrigable lands, the former should be situated at a sufficient altitude above the latter to allow of the delivery of water to them by gravity. The area of these lands should be sufficient to require the entire amount of water stored, that the maximum return may be derived from water rates; and the reservoir should be located as near as possible to the irrigable lands, in order that the loss in transportation shall be a minimum. It may often happen, however, that the reservoir is necessarily located at a considerable distance from the irrigable lands, requiring either a long supply canal, or that the water be turned back into the natural channels, down which it flows until diverted in the neighborhood of the irrigable lands. This is very wasteful of water, since the losses by absorption, percolation, and evaporation are large.

Fig. 22. Government Dam Site, Shoshone River, Wyoming. Reservoir to store 300,000 acre-feet. Height of dam to be 210 feet; walls of canyon 700 feet high. Diamond drills bored 80 feet below river bed before striking solid foundation rock.

If a reservoir is situated in a natural lake basin, a short drainage cut or a comparatively cheap dam, or both, may give a large available storage capacity. The most abundant reservoirs are usually those on natural drainage lines, though these may be the most expensive in construction owing to the precautions which are necessary, in building the dam, to provide for the discharge of flood water. Almost equally abundant are those reservoir sites found in alkaline basins or depressions on bench or plain lands. The utilization of such basins as reservoir sites is comparatively inexpensive, as they can be converted into reservoirs by the construction of a deep drainage cut or of a comparatively cheap earth embankment. Scarcely any provision is necessary for the passage of flood water, and the heaviest item of expense is the supply canal for filling them from some adjacent source.

Artificial reservoirs are occasionally constructed where water is valuable, by the erection of an earth embankment above the general level of the country, or by the excavation of a reservoir basin by artificial means. Reservoirs of this kind are usually of relatively insignificant dimensions, as the expense of building large reservoirs of the kind would be prohibitive. Shallow reservoirs should never be constructed. The loss from evaporation and percolation is proportionately great, and the growth of weeds is encouraged where the depth is less than seven feet, by the sunlight penetrating to the bottom.

EVAPORATION

If all of the precipitation upon a given area reached the adjacent stream as run-off, the calculation of the latter would amount to finding the product of the total area of the watershed into the precipitation. Much of the precipitation, however, returns to the air by evaporation. As all but a very small portion of the rainfall upon a given watershed which does not leave it as evaporation, leaves it ultimately as yield, it would seem as though the amount of the latter might be obtained by taking the difference between the total precipitation and the total evaporation. The great difficulty, however, is in the determination of the evaporation. The rapidity with which water, snow, and ice are converted into vapor, depends upon the relative temperature of these and the atmosphere, and the amount of motion in the latter. Evaporation is greatest when the atmosphere is driest, when the water is warm, and when a brisk wind is blowing. It is least when the humidity is highest, the air quiet, and the temperature of the water low. In summer-time the cool surfaces of deep waters condense moisture from the atmosphere, and really gain in moisture when they are supposed to be losing water by evaporation. When, however, the reverse conditions exist in the atmosphere, and the winds are blowing briskly across the water, the resultant wave motion increases the agitation of the water, and the vapors from it escape freely into the unsaturated air which is constantly coming in contact with it and absorbing its vapors. Evaporation is therefore going on at a rate due to the temperature of the surface, and condensation is likewise going on from the vapors existing in the atmosphere; and the difference between the two represents the amount of water lost by evaporation. Evaporation, therefore, should be greatest in amount

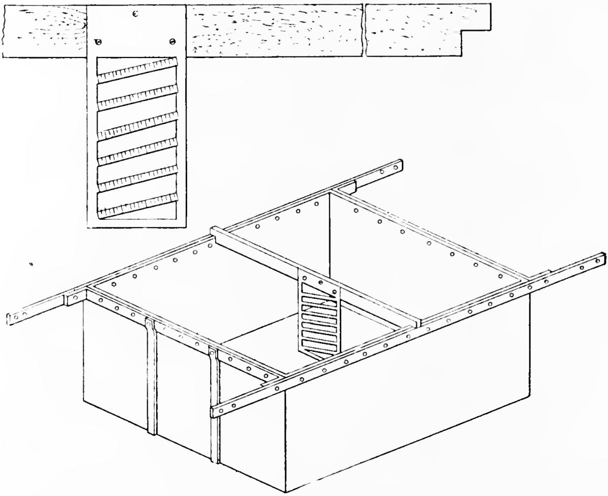


Fig. 23. Floating Pan Evaporometer.

in the desert regions of the Southwest, and least in the high mountains; and it is known that in the same latitude evaporation differs greatly with the altitude.

Different methods have been devised for measuring evaporation, none of which are wholly satisfactory. Most measurements of evaporation have been made from water surfaces or by an *evaporometer*, the Piche evaporometer being the one most used in this country. Water surfaces form but a small proportion of the total area of most catchment-basins; but the amount of evaporation from these and from the reservoirs themselves can be ascertained much more accurately than that from earth and vegetation.

The Piche evaporometer and the Richards evaporation-gauge are thought to give inaccurate results, since they are affected by the temperature of the air only, which is seldom the same for any length of time as that of a near body of water. However, in 1888, a series of observations with a Piche evaporometer were made by Mr. T. Russell, of the United States Signal Service, to ascertain the amount of evaporation in the West; and the results obtained showed such slight discrepancies from the results obtained by other methods that they must be considered reasonably reliable. Observations were made with this instrument in wind velocities varying from 10 to 30 miles per hour, from which it was discovered that with a velocity of 5 miles per hour the evaporation was 2.2 times that in quiet air; 10 miles per hour, 3.8 times; 15 miles per hour, 4.9 times; 20 miles per hour, 5.7 times; 25 miles per hour, 6.1 times; and 30 miles per hour, 6.3 times.

A more accurate method would seem to be to measure the actual loss from a pan filled with water and floating in a lake or other body of water. Such a pan and scale are shown in Fig. 23. The pan is so placed that the contained water has as nearly as possible the same temperature and exposure as that of the body of water the evaporation from which is to be measured. This pan is of galvanized iron, 3 feet square and 10 inches deep, and is immersed in water and kept from sinking by means of floats of wood or hollow metal. It should be placed in the body of water in such a position as to be exposed as nearly as possible to the average wind currents. The pan must be filled to within 3 or 4 inches of the top so that the rocking due to the waves produced by the wind shall not cause the water to slop over; and it should float with its rim several inches above the surface, so that the waves from this will not enter the pan. The device for measuring the evaporation consists of a small brass scale hung in the center of the pan. The graduations are on a series of inclined cross-bars so proportioned that the vertical heights are greatly exaggerated, thus permitting a small rise or fall of a tenth of an inch or so, to cause the water surface to advance or retreat on the scale 0.3 inch. In this way, multiplying the vertical scale by three, it is possible to read to 0.01 of an inch.

TABLE II
Evaporation from Water Surfaces, in Inches

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

From experiments conducted at the Boston Waterworks, the evaporation from snow was found to average 0.02 inch per day; that from ice to average 0.06 inch per day (or $2\frac{1}{2}$ inches and 7 inches per season, respectively). In the arid regions of the West, the evaporation from snow will exceed this, especially on barren mountain tops exposed to the action of the wind and bright sunshine.

Still more important than the evaporation from water, is that from soils of different characters. That from vegetation is also important, but in regard to both of these the available data are scarce and unreliable. Evaporation from the ground depends upon the contained moisture, the temperature, and the nature of the vegetation or other soil covering. The moisture in the soil depends upon the rainfall, and upon the ability of the ground to receive and retain the water falling upon it. In a general way, the greater the rainfall, the greater the evaporation. If, however, the soil is coarse and sandy, percolation will be rapid; the water will soon be beyond the reach of vegetation, resulting in small loss from evaporation If, on the other hand, the soil is fine, or hard and impervious, the evaporation will be small unless the surface happens to be level. The maximum evaporation will probably occur wherever the soil is sufficiently porous to receive and retain the water within reach of vegetation.

The amount of evaporation from earth in the Western States is a doubtful quantity, as the principal experiments conducted to determine the evaporation from earth have been conducted in England. Soil covering of any kind greatly affects the amount of evaporation; and calling the amount of evaporation from water 100, Prof. B. E. Fernow gives it for bare soil 0.60; sod, 1.92; cereals, 1.73; and forest, 1.51. Evaporation from ground covered with forest leaves is 10 to 15 per cent, and from sand 33 per cent, when from bare ground it is 100 per cent.

The value of water storage for irrigation in the West is realized chiefly during the dry season, May to August, inclusive. Little or no rain falls in the arid region during this interval; and it is during these months that the evaporation from storage supplies is principally felt. In Central California the average rainfall during these months is a trifle less than 1 inch. The evaporation during the same interval is about 21 inches, causing a deficiency due to evaporation, of 20 inches. When the reservoirs are located at high altitudes in the mountains, the losses from evaporation are less than upon the hot lowlands.

Canals and storage reservoirs are subject to a large loss of water due to percolation, the amount varying according to the soil. The losses due to percolation alone are difficult to estimate, but, combined with evaporation, may vary from 25 to 100 per cent. The combined losses, however, may be most conveniently considered under the head of *absorption*. In considering the losses in canals and reservoirs from absorption, the nature of the bed must be taken account of. If the bottom is of sandy soil, the losses from percolation and evaporation together will about double those from evaporation alone. If, however, the bottom is of clayey material, or if the canal or reservoir is old and the bottom protected by sediment, the loss from percolation will be limited and will not exceed that due to evaporation. In new canals and reservoirs the losses due to absorption are greatest, and in a long line of canal may amount to from 40 to 60 per cent of the volume entering the head. In shorter canals the loss will be proportionately less, though rarely falling below 30 per cent. As the canal increases in age, the silt carried in suspension is gradually deposited upon the banks and bottom, filling up the interstices and decreasing the amount of percolation. In favorable soil, old canals varying in length from 30 to 40 miles may not lose more than 12 per cent, though the loss of canals of average length will range from 20 to 25 per cent of the total volume entering the head.

To reduce the losses from percolation, it is recommended by Mr. J. S. Beresford, of India, that pulverized dry clay be thrown into canals near their headgates. This will be carried long distances and deposited on the sides and bottom of the canal, forming a silt berme. The losses by absorption may be greatly increased by giving the canal a bad cross-section, and in this feature of construction the attempt should be made to reduce the wetted perimeter and the surface of the water exposed to the atmosphere.

Where canals and reservoirs are built on steep hillsides, the amount of water lost may prove to be much less than was to be expected from absorption, etc. This is due to gains from seepage water from the surrounding country, which may enter the canal and replenish the losses due to absorption. In considering the matter of seepage, it will be of interest to inquire into the effect of irrigation upon the subsurface water level. Before irrigation becomes universal, the subsurface water level may be so low that it will frequently be impossible to derive water from wells. The practice of irrigation, however, for a considerable time, seems to fill the soil with water so that the level of the subsurface water is raised and shallow wells often yield uniform and persistent supplies. It is stated that in Fresno, California, where the subsurface level was originally at a depth of from 60 to 80 feet below the surface, the seepage from the canals has raised this level so that wells 10 to 15 feet in depth now receive constant supplies. In "Irrigation Institutions," Mr. Mead says:

"Up to a certain limit, irrigation on the headwaters of a river is a benefit to the users of water below. About one-third of the water diverted returns to the stream as waste and seepage. The waters diverted during the flood season, which return as seepage, come back slowly and help swell the stream when it is low and water is most needed. The exact time of the return varies, of course, with the location of the lands irrigated and with the character of the soil; but in a general way the effect of the diversion of floods in irrigation is to equalize the flow of rivers. They carry less water when high, and more water when low. Some rivers leaving the eastern slope of the Rocky Mountains, which formerly ran dry every year, now have a perennial flow; and on others, the period every year at which they become dry is traveling eastward rather than westward.

"Up to a certain limit, the storage of water also tends to equalize the flow of streams. Reservoirs are filled when there is an abundance, and the water is turned out when there is a scarcity. Hence the people who live along streams below where the stored water is used derive an indirect benefit from the increased seepage thereby created. There is, however, a limit beyond which irrigation on any stream does not improve the supply of those living below. If the irrigated valley is long enough, and the irrigated district broad enough, the ultimate absorption of the water-supply is inevitable."

As illustrating the possible effects of seepage, the State Engineer of Colorado, during the years 1890 to 1893, inclusive, conducted examinations on the South Platte and Cache la Poudre rivers, with the object of determining the amount of seepage water returned to them. In 1893, on the South Platte river, in a distance of 397 miles, there was a gain of 573 second-feet over that found at the upper measuring station. In 1891 there was a gain of 300 per cent over the flow at the upper measuring station.

From experiments conducted in 1889 on the Cache la Poudre, it was found that the discharge at a point considerably down the stream from the canyon was 214.7 second-feet, as against 127.6 second-feet at the canyon—and this, after supplying fifteen canals and without receiving additional supplies from drainage. These results were borne out by experiments conducted through succeeding years.

Experiments of a similar nature conducted elsewhere point to the same results. The amount of water returned by seepage will depend upon the soil, the nature of the underlying strata, the amount and direction of slope, and the extent of the drainage area above and tributary to the streams. It is probable that in many cases the amount of seepage water returned to the streams will be practically nothing; and in designing reservoirs it will be safer to assume that the gains by seepage water will be offset by the losses due to percolation, evaporation, etc.

RUN-OFF

By the *yield* or *run-off* of a catchment area, is meant the total amount of water flowing from a given drainage area, generally as streams fed by the rainfall upon such area. This is never the whole of such rainfall. In the temperate and frigid zones, rain (including snow) is considered the source of water. Whether for domestic or manufacturing purposes or for purposes of irrigation, rain, since it does not fall continuously, must be caught and stored up to tide over periods of longer or shorter duration. In a natural way this is accomplished to a certain extent through the agency of porous soil and rocks, underground caverns, lakes and ponds, glaciers, etc. Artificial storage, on the other hand, is accomplished by means of cisterns and reservoirs. The amount of rain falling directly into a reservoir is generally an insignificant part of its total capacity; but it is the water flowing from some drainage area, as a watershed of thousands of acres, which gives the greater part of the supply. If this drainage area is the surface of the ground, the run-off or yield is called *surface water.*

In falling, some rain is intercepted by the foliage and stems of trees and smaller plants, to be returned later to the air through evaporation. Of that which reaches the earth, a portion flows over the surface, and the remainder enters the soil. If the soil is very porous, almost all the rain falling upon it may be absorbed; if non-porous, very little may enter it. All soils, even the densest rocks, are more or less porous, and will absorb water to some extent.

After a rain has ceased, the small streams carry less and less water; but those of any size seldom become entirely dry though weeks and even months may elapse between rainfalls and the surface of the ground may become absolutely dry. During this time the immediate supply is not the rain, but is that portion of previous rainfalls absorbed by the earth and now being yielded slowly. In general, the more porous the soil, the more water it will receive for this purpose during a given rainfall; and the finer its grain, the more slowly will it yield its supply and become exhausted. The ground flow need not necessarily reach the same stream as the surface flow, but the dip of the strata may carry it into another valley. The ground flow frequently emerges as springs; but the larger portion of it generally reaches the stream through the banks, and in some cases through the bottom of the channel.

A study of the material and dip of the strata, and of the surface conditions, such as topographical features, vegetation, location of ponds, etc., as well as of the rainfall and other meteorological conditions, is necessary to forming any estimate of the probable amount and yield of a given watershed, where this cannot be measured directly. The total amount of rainfall reaching the ground is not yielded by the combined surface and ground flow; a large part of it is lost by evaporation from the surface of the ground, and from the surface of ponds or other bodies of water; much is taken up by vegetation, to be returned to the air by evaporation from the foliage. A portion is held in the soil by capillary attraction. Probably none of the precipitation settles into the lowest strata which have no outlet,

IRRIGATION WHEEL OF A PRIVATE ENTERPRISE ON THE SNAKE RIVER, IDAHO since these were filled ages ago. If, however, water is drawn from these deep strata by wells, the amount thus withdrawn must be replenished.

The dividing line between surface supplies and river supplies is indefinite; but when the supply is taken directly from a river or lake without impounding or storage, it should be called a *river supply* or *lake supply*. The conditions are in many respects similar to those affecting surface waters; but the supply is somewhat more constant and of greater volume, owing to the larger drainage area. Lakes act as regulators of flow, and take the place of artificial storage reservoirs. They are generally but enlargements of a river channel, although some lakes are formed direct from surface flow or from large springs and from the sources of rivers; while still others have ground water as both source and outlet. Lakes can in most cases be relied upon as more constant than rivers in regard to the quantity of water available.

The water flowing underground towards a surface stream may emerge as springs or be washed by a well dug or bored to and through the porous strata through which it flows. Such water seldom flows in the form of a stream; nor is it collected in caverns as *underground lakes*, but generally fills the porous stratum throughout, and moves slowly through the interstices towards the outlet. This movement is not always downward, but the outlet must be lower than the point where the water enters the soil. If, in traveling through such a stratum, the ground water encounters a fault, this may be followed to the surface, where the water will emerge in the form of springs. The run-off of a given catchment area may be expressed either as the number of second-feet of water flowing in the stream draining that area, or it may be expressed as the number of inches in depth of a sheet of water spread over the entire catchment. Or run-off may be expressed volumetrically, as so many cubic feet or acre-feet.

As run-off bears a direct relation to precipitation, it would seem that, knowing the amount of rainfall and the catchment area, the amount of run-off could be ascertained directly. This is not so, however, as the amount of run-off is affected by many varying climatic and topographic factors. Many formulæ, none of which gives satisfactory results, have been worked out, expressing the relation between precipitation and run-off. The climatic influences bearing most directly on run-off are the total amount of precipitation, its rate of fall, and the temperature of earth and air. When most of the precipitation occurs in a few violent showers, the percentage of run-off is higher than when the water is given abundant time to enter the soil. If the temperature is high and the wind blowing briskly, much greater loss will occur from evaporation than if the ground is frozen and the air is quiet.

Within a given drainage area, the rates of run-off vary on the different portions. In a large basin, the rate of run-off for the entire

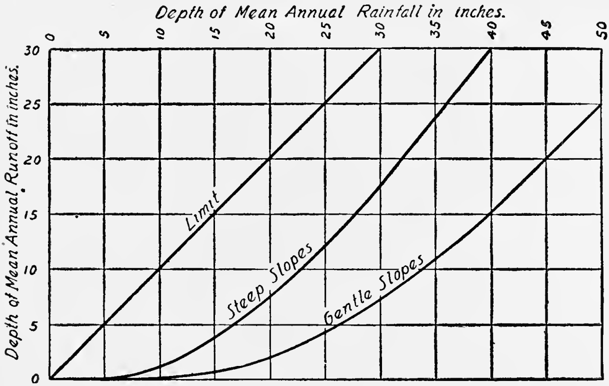


Fig. 24. Curves Showing Relation of Run-Off to Rainfall.

area may be low if the greater portion of the area is nearly level; but at the headwaters of streams, where the slopes are steep and perhaps rocky, the rate of run-off will be higher. Other things being equal, the coefficient of run-off will increase with the rainfall; and in humid regions, where the rainfall is greatest, the rate of run-off will naturally be highest.

The accompanying diagram (Fig. 24), prepared by Mr. F. H. Newell, illustrates the relation between mean annual run-off and mean annual rainfall. Along the vertical axis is plotted the mean annual run-off in inches for a given drainage area; while the annual rainfall in inches is plotted along the horizontal axis. The diagonal line represents the extreme limit of run-off that would occur upon a steep, smooth, impenetrable surface; the horizontal line represents the limit upon a level, porous surface from which there would be no run-off. The upper curve represents an average condition in mountainous regions, from which the run-off is large; the lower curved line represents the condition in a catchment area consisting of broad valleys and gentle slopes, from which the run-off is relatively small. For instance, in an area of the latter kind having an annual rainfall of 40 inches, the annual run-off, as indicated by the diagram, will be 15 inches. However, the relation between these two quantities will be largely influenced by topography.

The maximum discharge from a catchment area tributary to a reservoir is of the utmost importance in the design of a dam or of a spillway. Various formulæ, both empirical and theoretical, have been devised for expressing the volume of discharge; but it is evident that no formula is generally applicable to all of the conditions that occur in practice. They should be used with the utmost discretion, and only after a careful study of all the factors, such as topography, nature and depth of soil, vegetation, average temperature, humidity, etc. The following are a few of the formulæ proposed:

Fanning's formula $Q = 200M^{\frac{5}{6}}$

Dredge's formula $Q = 1{,}300\frac{M}{L^{\frac{2}{3}}}$

Col. Dicken's formula $Q = CM^{\frac{3}{4}}$

In these formulæ,

Q = Cubic feet per second yielded from the whole area;
M = Area of watershed, in square inches;
L = Length of watershed, in miles;
C = 200 in flat country, 250 in mixed country, 300 in hilly country, for a rainfall of 3.5 to 4 inches; or 300 to 350 for a 6-inch rainfall.

It is necessary to know the monthly and daily rates of run-off from a catchment area, as well as the mean annual rate of run-off, as these will affect the design of the spillway for a dam. The greatest floods will occur either on barren catchment areas with steep slopes, or wherever heavy snowfalls are followed by warm, melting rains. In some portions of the West, sudden flood discharges have been recorded of 30 second-feet per square mile of catchment area, where a few days previously the flow was at the rate of one-twelfth of a second-foot per square mile.

Table III, derived from a series of observations over a period of

TABLE III

Discharge and Run-Off from Catchment Areas of Important Streams in Arid Region of the United States

State	River Basin	Observing Station	Altitude (Feet)	Drainage Area (Sq. Miles)	Discharge				Run-Off			
					Maximum (Sec. Feet)	Minimum (Sec. Ft.)	Mean Annual (Sec. Feet)	Total Annual (Acre-Feet)	Depth Max. (In.)	Depth Min. (In.)	Depth Mean Annual (In.)	Per Sq. Mile per Annum (Sec. Feet)
Ariz.	Salt	Arizona Dam		12,260	143,290	320	3,170	2,297,000	....	....	3.50	0.26
"	Gila	Florence		17,834	102,566	0	500	400,000	....	....	0.5	0.06
Cal.	Sacramento	Collinsville	0	26,187	160,000	5,050	37,630	26,000,000	....	....	16.32	1.20
"	Cosumnes	Live Oak	150	580	......	......	1,234	914,000	....	....	29.52	2.18
"	Tuolumne	Lagrange	290	1,500	22,900	30	2,685	1,960,000	....	....	22.56	1.64
"	San Joaquin	Herndon	295	1,637	19,960	60	3,074	2,220,000	....	....	25.44	1.84
"	Kern	Bakersfield	412	2,345	5,342	80	1,110	650,000	....	....	5.22	0.38
"	Kings	Red Mountain	1,810	1,775	22,732	145	......	....	....	....	......	1.30
"	San Gabriel	Azusa	614	222	1,765	0	......	....	....	....	......	0.30
"	Santa Anna	Warm Springs	43	188	580	18	......	....	....	....	......	0.40
Colo.	Arkansas	Canyon City	5,330	3,060	4,750	124	814	570,000	1.20	0.08	3.80	0.27
"	Rio Grande	Del Norte	7,865	1,400	5,930	200	1,040	755,000	3.57	0.21	10.12	0.74
"	South Platte	Denver	5,183	3,840	2,425	42	......	....	....	....	......	0.10
"	Cache la Poudre	Fort Collins	4,984	1,060	5,060	32	......	....	....	....	......	0.32
Idaho	Snake	Idaho Falls	4,714	10,100	54,300	2,000	9,380	6,870,000	4.56	0.20	12.72	0.94
"	Weiser	Weiser	2,125	1,670	17,940	10	1,211	877,000	....	....	9.85	1.00
"	Teton	Wilford	......	967	5,270	425	......	....	....	....	......	1.00
"	Boise	Boise	2,880	2,450	40,130	550	......	....	....	....	......	1.20
Mont.	W. Gallatin	Salesville	5,380	860	10,750	100	1,930	710,000	5.48	0.39	15.70	1.15
"	Madison	Red Bluff	5,020	2,420	8,613	910	975	1,430,000	2.67	0.55	12.60	0.93
"	Missouri	Craig	3,028	17,615	28,650	1,740	5,100	3,800,000	1.34	0.14	4.00	0.27
"	Yellowstone	Horr	5,120	2,700	15,500	285	2,880	2,180,000	4.17	0.12	15.12	1.11
Nev.	East Carson	Rodenbah	......	415	5,540	290	760	550,000	7.38	1.00	25.00	1.84
"	Humboldt	Battle Mt.	4,515	7,800	3,131	5	......	....	....	....	......	0.08
N. Mex.	Rio Grande	Embudo	5,806	7,000	8,555	140	1,280	920,000	0.98	0.02	2.70	0.18
Ore.	Owyhee	Nyssa	......	9,875	18,000	32	2,114	1,536,000	1.53	0.02	2.90	0.21
"	Umatilla	Gibbon	......	353	4,217	74	......	....	....	....	......	1.50
Texas	Rio Grande	El Paso	3,697	30,000	17,000	0	1,490	1,090,000	0.45	....	0.70	0.05
Utah	Bear	Colliston	......	6,000	10,590	540	2,210	1,600,000	1.52	0.06	5.00	0.37
"	Weber	Uinta	4,514	1,600	7,980	100	980	690,000	3.26	0.13	8.07	0.59
"	Provo	Provo	4,456	640	2,600	146	519	376,000	3.47	0.26	11.01	0.81
"	Sevier	Leamington	4,674	5,595	2,330	35	485	350,000	0.35	0.01	1.15	0.09
"	Ogden	Ogden	4,310	360	2,433	30	......	....	....	....	......	0.50
"	Green	Blake	......	38,200	68,800	610	......	....	....	....	......	0.25
Wash.	Yakima	Kiona	499	5,230	30,000	612	......	....	....	....	......	0.98
"	Spokane	Spokane	1,886	4,005	32,875	1,972	......	....	....	....	......	2.20
Wyo.	Shoshone	Lovell	......	2,770	12,340	150	......	....	....	....	......	....
"	Laramie	Uva	7,153	3,179	2,570	0	......	....	....	.	......	0.06
"	North Platte	Orin	4,704	14,820	19,160	......	......	....	....	....	......	0.15

years up to 1900, shows the discharge and the run-off from catchment areas of the more important streams of the arid region.

The run-off may be expressed in percentages of the depth of precipitation. Thus, for a drainage area having an average annual precipitation of 45.47 inches, the run-off from which amounts to 24.03 inches in depth, the run-off may be expressed as 52.8 per cent of the precipitation. In case the number of storage basins is limited, it becomes necessary to store all of the water possible; and frequently it becomes necessary to impound enough water to "carry over" a period of two or three years of minimum rainfall. A measurement of the drainage area having been obtained, a decision must be made as to the probable average, minimum, and maximum run-off, both by year and by cycle of years. An estimate of the consumption of water must also be made, and from these figures a calculation of the storage to be provided may be made. If the minimum *annual* yield is equal to or greater than the consumption, storage will be required for only the dry season of one year of drought; if the minimum *daily* yield equals the maximum daily consumption, no storage will be required; if, however, the assumed consumption is nearly or quite equal to the mean yield, all of the surplus from the years of greatest rainfall must be stored and carried over until times of drought.

In making the calculation for storage, evaporation from the reservoir must be considered, and should be added to the consumption. This, of course, is proportional to the area of water surface of the reservoir; and therefore, in the calculations, an assumption must be made as to the probable area. This is usually a certain per cent of the drainage area, and will vary with the average depth of the reservoir and with the nature of the ground at the reservoir site—that is to say, whether the side slopes are steep or otherwise. A study of the run-off for a series of years at any location will give the approximate capacity required that the consumption may not exceed the average yield. This capacity, divided by the average depth of reservoir, will then give the area.

The loss from evaporation has already been discussed. There will be loss from seepage through the dam. The loss into the ground may usually be considered as additional storage; for although a part of this may be absorbed by vegetation, the proportion will probably be little if any greater than the loss from evaporation. The loss from seepage through a masonry dam should be small. The loss through an earthen embankment may be considerable, and the amount so lost will depend upon the character of the embankment, which should be so constructed that the daily seepage shall not exceed 10 gallons per square foot of vertical longitudinal section of embankment. With good materials and care in construction, the loss by seepage may be reduced to 5 gallons, or even as low as 3 gallons, per vertical square foot of embankment.

When irrigation is practiced, all of the water flowing in the streams is not available for storage, since much of it has already been appropriated by irrigators, and of course the quantity must be deducted from that available for storage. A large portion of the discharge occurs in winter when the streams are covered with ice, which renders it practically impossible to divert the water for storage, though it is available for such reservoirs as may be located on the main streams. As nearly all of the flow occurring in the irrigating season is appropriated, only the surplus and flood water are available for storage.

PRECIPITATION

In any region where the climate and soil are adapted to the production of crops, the necessity for irrigation will depend upon the amount of precipitation available, and the season of the year when this precipitation is available. The available amount of precipitation cannot be judged, however, from the total annual precipitation. Where the annual precipitation is less than 20 inches, irrigation is assumed to be necessary; and the arid region of this country is usually considered as including that area in which the annual precipitation is below 20 inches—or most of the territory west of the 97th meridian of longitude. As illustrating seasonal influence, irrigation is necessary in Italy, because, while the annual precipitation averages about 40 inches, most of this occurs during the winter months or at times other than the agricultural or cropping season. In certain parts of India, the rainfall is as high as 100 to 300 inches per annum; and yet nearly all of this occurs in one or two seasons of the year, and the actual rainfall during the winter months, when most of the cropping is done, may be as low as 5 or 10 inches. The cropping season in the arid West may be taken as occurring between April and August, inclusive, and this constitutes the driest season of the year.

In referring to the lands of the United States, those of the extreme West are usually designated as *arid*; those between the Mississippi Valley and the Rocky Mountains, where the rainfall is occasionally sufficient to mature the crops, are designated as *semi-humid*; and the lands to the east of the Mississippi Valley, over which the rainfall is always sufficient to mature the crops, are spoken of as *humid*. This distinction is based largely upon the amount of the mean annual precipitation; but the true basis of distinction between arid and humid regions is dependent upon the amount of precipitation during the crop-growing season. On this basis, the humid portion of the United States embraces those regions over which the precipitation during the cropping season is from 10 to 15 inches, depending upon the character of the soil and other modifying considerations.

Table IV shows in a general way the amount and extent of the precipitation over the arid region. It will be noted that the average rainfall over the northern portion of the Pacific Coast would be sufficient for the production of crops, provided it fell during the proper season of the year. Other areas may be noted over which the annual rainfall is apparently sufficient for the maturing of crops. That the amount of precipitation is greatly influenced by altitude, is shown by comparing the relative amounts of precipitation at places having the same latitude. Thus in the region between Reno, Nevada, and San Francisco, California, the average annual precipitation in the Sacramento Valley is about 15 inches; while to the eastward the precipitation increases in amount with the altitude, until, along the summits of the mountains, it averages from 50 to 60 inches. till farther east, the precipitation again diminishes with the decreasing altitude, until, in Nevada, it varies from 5 to 10 inches. All through the West, precipitation in the high mountains is much in excess of that in the adjacent low valley lands. As a result, while the precipitation is often insufficient to mature the crops in the lowlands, sufficient precipitation occurs in the mountains to furnish a constant supply for the perennial discharge of streams or for the filling of storage reservoirs.

Rainfall, being the ultimate origin of all water supplies, is the

TABLE IV

Precipitation by River Basins in the Arid Region of the United States

Station	Altitude (Feet)	Mean Annual Precipitation (Inches)
Rio Grande River—		
Summit, Colorado	11,300	29.00
Fort Lewis, Colorado	8,500	17.19
Fort Garland, "	7,937	12.74
Saguachi, "	7,740	12.60
Santa Fé, New Mexico	7,026	14.69
Fort Wingate, New Mexico	6,822	14.71
Las Vegas, " "	6,418	22.08
Albuquerque, " "	5,032	7.19
Socorro, " "	4,560	8.01
Deming, " "	4,315	8.95
Gila River—		
Fort Bayard, New Mexico	6,022	14.72
Prescott, Arizona	5,389	17.06
Fort Apache, Arizona	5,050	21.04
Fort Grant, "	4,914	16.65
Phœnix, "	1,068	7.38
Texas Hill, "	353	3.47
Yuma, "	141	2.81
Platte River—		
Pike's Peak, Colorado	14,134	28.65
Fort Saunders, Wyoming	7,180	12.92
Fort Fred Steele, "	6,850	11.03
Cheyenne, "	6,105	11.32
Colorado Springs, Colorado	6,010	14.79
Denver, "	5,241	14.32
Fort Morgan, "	4,500	8.08
Missouri River—		
Virginia, Montana	5,480	16.00
Fort Ellis, "	4,754	19.60
Helena, "	4,266	14.26
Fort Shaw, "	2,550	10.22
Poplar, "	1,955	10.50

basis of calculation of the amount of water available from whatever source; and consideration of the amount and intensity of rainfall is a necessary preliminary to the design and construction of storage works, whether for purposes of irrigation or for domestic supply. The amount of rain that will fall at any one place in any day, month, or year cannot be predicted with certainty by any method known to science. A record of past rainfalls, however, will afford a guide to our judgment in estimating the probable amount of future rainfalls, and in fact forms practically the only basis for such judgment. The total annual rainfall is seldom the same for any two years at the same place, or at any two places for the same year, and these variations seem to follow no definite law. During a dry season at one place, another only a few miles away may have conditions entirely the reverse.

The Weather Bureau has divided the United States into twenty-one districts for meteorological purposes, and the precipitation averages for these districts are given in Table V. In each meteorological district the general law and average amount of precipitation are practically uniform; in some the variation between the precipitation at different stations is as pronounced as the variation between the extreme district means. The factor most influential in determining the amount of rainfall in a given district, is its proximity or other relation to mountain ranges and to the sea or other large body of water. Thus the warm, moist winds of the North Pacific lose a large portion of their moisture upon the western slopes of the Sierra Nevada and the Cascade range, so that little is left for the plateau to the east of these ranges. The winds blowing over the Gulf Stream into the South Atlantic and Gulf States yield their moisture to them, so that, as they ascend the valley of the Mississippi and its tributaries, their moisture and consequent precipitation decrease. The departure of the Gulf Stream from the coast north of Cape Hatteras, causes its influence to be felt to a less extent in precipitation in the North Atlantic States.

One of the most important considerations in designing irrigation works, and especially storage reservoirs, is the maximum amount of rainfall which may occur. Great floods are the immediate result either of heavy, protracted rainstorms, or of the sudden melting of snow in the mountains. In nearly all river valleys, there are periods of maximum

TABLE V

Mean Annual Precipitation in the United States

(Mean, Maximum, and Minimum Averages of Stations in Each Meteorological District, in Inches)

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

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

rainfall, the recurrence and effect of which should be given careful study. The following are examples of unusual precipitation:

In the neighborhood of Yuma, Arizona, the average annual rainfall is about 3 inches; but in the last week of February, 1891, 2½ inches fell in 24 hours.

In the neighborhood of San Diego, California, the average annual rainfall is about 12 inches; but in the storms of 1891, 13 inches fell in 23 hours, and 23½ in 54 hours. The average annual discharge of the Salt River in Arizona is about 1,000 second-feet, and the average flood discharge is about 10,000 second-feet; yet, as the result of an unusually violent rainstorm in the spring of 1890, the flood discharge amounted to 140,000 second-feet; a year later, as the result of a still more violent storm, the discharge increased to the enormous amount of nearly 300,000 second-feet.

It is of course out of the question to design irrigation works so that they shall control and safely pass away the flood discharge of unusual storms such as described above. Such cloudbursts may occur once in a lifetime, but the increased expenditure necessary to provide for them is generally unwarranted.

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

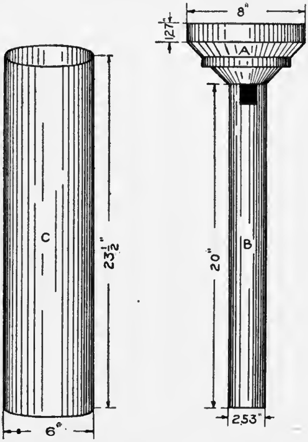


Fig. 25. Rain-Gauge. *A*—Collector; *B*—Receiver; *C*—Overflow Attachment.

Measurements of rainfall are usually made in this country by means of the *rain-gauge* or *pluviometer* (Fig. 25). The most common form consists of a circular cup of thick brass, its top brought to a chisel edge, the bottom being cone-shaped and connected with a deep tube of known diameter into which the rain flows from the cup. The area of the top of the cup and that of the tube bear a known relation to each other—usually 10 to 1—and the depth in the tube is measured by a stick so graduated that when it is lowered to the bottom of the tube, the scale will give the actual depth of rainfall, allowance being made in the scale for both the relative areas and the displacement caused by the stick. The depth is usually expressed in inches and decimals of an inch. The reading should be taken daily and at the beginning and end of each storm.

The size of the collector-cup seems to have some effect upon the catchment. Of four 3-inch cups and one 8-inch cup in use on Mt. Washington, the average total amount collected by the 3-inch cups in one year was 46.26 inches, while that collected by the 8-inch cup was 58.70 inches. The larger the collector-cup, probably the more accurate will be the result. The position of the gauge relative to the ground-surface will also have an important influence upon the amount of catchment, those placed near the surface generally giving the higher results. It has been found that a gauge 100 feet above the ground will give on the average only 65 per cent as much rainfall as one upon the surface. The intensity of the wind seems to be the controlling factor in these variations. It is maintained by many that gauges at the surface give less accurate results, since they receive not only the actual precipitation, but also a certain amount of moisture from the surrounding ground, which, after falling, again rises by splashing and evaporation, and is once more precipitated. A large number of the gauges of the signal service are placed upon the roofs of tall buildings, and in cities this is generally necessary. In open country, a height of from 3 to 6 feet from the surface will probably give the most accurate results. The gauge should be at least as far from any building or other obstacle as the top of this is above the gauge, and the rim of the collector-cup should be level. The United States Weather Bureau has for years been taking records of precipitation in various parts of the country, and many stations are now so operated, records being received from hundreds of voluntary observers as well.

For many purposes it is desirable to know the rate of fall for short intervals of five minutes or less; and for ascertaining this, self-recording gauges are necessary. Several styles of such gauges have been used, one of which, the *tipping tank*, tips and empties itself as soon as it has received 0.01 inch of rainfall, immediately returning to an upright position, the time of each discharge being recorded automatically. Another style of gauge consists of a tank suspended by a spring balance, a pencil attached to the tank continuously recording its vertical position upon a cylinder revolved by clockwork once in twenty-four hours. In using any recording gauge, the total water caught should be retained, and measured or weighed each day as a check upon the record. The records of the United States Weather Bureau are available to anyone, and should be freely consulted in the study of the precipitation of any locality.

FLOW OF WATER IN OPEN CHANNELS

If an open channel be given the smallest possible inclination in one direction, the water contained therein will at once be set in motion by the action of gravity upon the particles. The effect of the force of gravity in producing motion will depend upon the head over a certain point, and this is expressed as the *slope*, which is the ratio of the vertical height of fall to a given horizontal distance; thus a slope of $\frac{1}{100}$ means a fall of one foot in one hundred feet. The velocity of flow, therefore, will depend upon the length of channel (l) for a vertical fall of any height (h). The amount of friction offered by the sides and bed of the channel will depend upon l and upon the nature of the material composing these—that is, upon the nature of the lining or surface of the channel in contact with which the water flows. The coefficient of flow in a channel will also depend upon the hydraulic mean radius (r), which expresses the ratio of the area of cross-section in square feet (a) to the wetted perimeter, in linear feet (p). For a semicircular or circular cross-section of flow, r is one-fourth the diameter.

There are many formulæ for calculating the *mean* velocity of flow in open channels. All of these have constant coefficients, and are therefore incorrect outside of a small range of dimensions. The velocity of flow at the surface of a channel or along the wetted perimeter, is less than the mean velocity of flow of the cross-section. The mean velocity of rivers is generally about 98 per cent of the mid-depth velocity, and from 70 to 80 per cent of the maximum surface velocity.

The Chezy formula, $v = c\sqrt{rs}$, is generally used as the basis of velocity formulæ, in which $v =$ the mean velocity, in second-feet; c is a constant; r is the hydraulic radius; and s is the sine of the angle of slope, or rate of fall of the surface of the water.

Values of c adapted to different conditions have been ascertained by experiment; and Chezy's formula, as modified by Kutter, is the one now most approved for determining the velocities of flow in open channels; it is as follows:

$$v = c\sqrt{rs} = \left\{ \frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{s}}{1 + \frac{\left(41.6 + \frac{.00281}{s}\right) \times n}{\sqrt{r}}} \right\} \times \sqrt{rs},$$

in which n is a coefficient of roughness of the wetted surface of the conduit, and has the following values:

	$n =$
For channels of well planed timber	.009
" " " neat cement, glazed sewer pipe, or very smooth iron pipe and butt-joint wrought-iron pipe	.010
" " " 1 : 3 cement mortar or smooth iron pipe	.011
" " " unplaned timber and ordinary cast iron	.012
" " " smooth brickwork	.013
" " " ordinary brickwork or smooth masonry	.015
" " " lap-joint wrought-iron pipe	.012 to .016
" " " rubble masonry	.017
" " " firm gravel	.020
" " " rivers free from stones and weeds	.025
" canals and rivers with some stones and weeds	.030
" " " " in bad order	.035

Considerable care must be exercised in selecting the proper value of n; and an actual measurement of the flow is always to be preferred to the best formula used with the most expert judgment.

TABLE VI

Value of v in Feet per Second, and of c, for Earth Channels by Kutter's Formula

Slope = s	$n = .0225$; $\sqrt{r}$, in feet.									
1 in	0.7		1.0		1.8		2.5		4.0	
	v	c	v	c	v	c	v	c	v	c
1,000	1.17	51.5	2.01	62.5	4.88	80.3	7.08	89.2	12.73	99.9
1,250	1.04	51.3	1.79	62.3	4.17	80.3	6.38	89.3	11.43	100.2
1,667	.85	51.0	1.54	62.1	3.58	80.3	5.54	89.5	9.95	100.6
2,500	.72	50.4	1.25	61.7	2.95	80.3	4.54	89.8	8.19	101.4
3,333	.62	49.8	1.07	61.2	2.54	80.2	3.94	90.1	7.14	102.2
5,000	.50	48.9	.87	60.5	2.05	80.3	3.24	90.7	5.92	103.7
7,500	.42	47.5	.73	59.4	1.77	80.3	2.78	91.5	5.13	106.0
10,000	.34	46.4	.60	58.5	1.49	80.3	2.33	92.3	4.55	107.9
20,000	.22	43.0	.40	55.7	1.20	80.2	1.70	94.8	3.29	115.0

Slope = s	$n = .035$; $\sqrt{r}$, in feet									
1 in	0.7		1.0		1.8		2.5		4.0	
	v	c	v	c	v	c	v	c	v	c
1,000	0.67	29.9	1.19	37.6	2.85	51.6	4.69	59.3	8.76	69.2
1,250	.60	29.8	1.06	37.6	2.51	51.6	4.20	59.4	9.86	69.4
1,667	.57	29.6	.92	37.4	2.40	51.6	3.62	59.5	6.84	69.8
2,500	.42	29.2	.74	37.1	1.85	51.6	3.00	59.7	5.63	70.4
3,333	.36	28.9	.64	36.9	1.60	51.6	2.60	59.9	4.92	71.0
5,000	.29	28.3	.51	36.4	1.30	51.6	2.10	60.4	4.08	72.2
7,500	.24	27.7	.42	35.8	1.11	51.6	1.80	60.9	3.55	73.9
10,000	.19	27.1	.35	35.3	.93	51.6	1.50	60.5	3.02	75.4
20,000	.13	25.4	.24	33.8	.65	51.5	1.10	63.1	2.28	80.6

Pipes of metal or wood being of a uniform material and cross-section, the flow through them is capable of more exact determination than that through channels of earth or ordinary masonry.

Table VI from Flynn's "Flow of Water in Open Channels," gives values of c for a wide range of earth channels, and will cover nearly every case occurring in ordinary practice.

The quantity of discharge of a canal or river (Q) in second-feet, is obtained by multiplying its velocity v, in feet per second, by the cross-sectional area A of the channel, in square feet. Or,

$$Q = Av.$$

Substituting the value of v from Chezy's formula:

$$Q = A \times c\sqrt{rs}.$$

Measuring or Gauging Stream Velocities. The simplest method of gauging the velocity of a stream, when approximate results are sufficient, is by means of wooden floats or bottles, or some similar arrangement, thrown into the center of the stream and timed for a given distance. For convenience, a base of 100 feet may be measured off on the bank, parallel to the stream, and note taken of the time required by the float in passing over this distance. Several observations of the time of passage of the float over the same base may be made; or, better still, the time of passage of floats over different 100-foot lengths may be observed. The mean of these observations will give the central or maximum surface velocity. The mean surface velocity may be determined by the use of floats in different portions of the width of the streams, and timing the passage along a fixed length of base. The resulting velocity per second, multiplied by 0.8, will give approximately the mean velocity of the entire cross-section of the stream.

To determine the velocity of a stream with greater accuracy, the velocity should be determined, not of the surface alone, but of the entire body of the stream, by timing upright rods so weighted that their bottoms float at different depths beneath the surface of the water. The rods should be of wood or of tin jointed so as to be used in different depths of water. Finally, the stream should be cross-sectioned at various points to determine the average area; and the quantity of discharge may thus be determined.

Current Meters. The following description of the *current meter* and its use in gauging streams, is very complete, and is from "Irrigation Engineering," by H. M. Wilson:

Current meters are mechanical contrivances so arranged that by lowering them into a stream, the velocity of its current may be ascertained with accuracy by a direct reading of the number of revolutions of a wheel, and a comparison of this with a table of corresponding

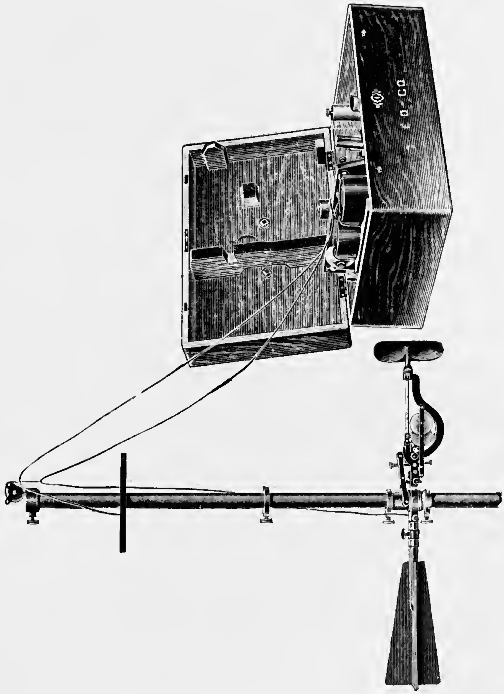


Fig. 26. An Efficient Type of Current Meter.
Registers mechanically in connection with electrical contact every 100 revolutions, two graduated wheels registering to 1,000 revolutions.
Courtesy of Eugene Dietzgen Company, Chicago.

velocities. Various forms of current meters have been designed and used, the three general classes being the *direct-recording meter,* in which the number of revolutions is indicated on a series of small gear-wheels driven directly by a cog and vane wheel; the *electric meter,* in which the counting is done by a simple make-and-break circuit, the registering contrivance being placed at any desired distance from

MINIDOKA DAM AND HEADWORKS, MINIDOKA PROJECT, IDAHO

A rock-fill dam 60 ft. high, 600 ft. long, across Snake River, diverts water onto 80,000 acres in the Snake River Plains. Soil is disintegrated lava of wonderful richness. At time this photograph was taken, 1.9 ft. of water was flowing over the spillway.

Photo by Reclamation Service.

the meter; and the *acoustic meter*, in which counting is done by hearing, through an ear-tube, the clicks made by the revolution of a wheel, and counting the same. There are several makes of current meters of nearly all of these varieties.

Of direct-acting meters, one which has been found effective in turbid waters, is the *Colorado meter*. In this, the stem is of iron pipe, several lengths of which may be joined together. This meter is difficult to handle for depths over 8 feet, or for less than 1 or 2 feet. Of the several varieties of electric meters, one which is chiefly used in the United States Coast and Geodetic Survey and in the United States Geological Survey, is a modification of the *Haskell meter*. This meter is not so good for very high velocities as that next described, as the rapidity of revolution is so great as to make counting difficult. The electric meter which has been found to work most satisfactorily under nearly all conditions of depth and velocity by the hydrographers of the United States Geological Survey and the United States Engineer Corps, is the small *Price electric current meter*. This meter undoubtedly gives the most satisfactory results in large streams of high velocity. It is very accurate for streams of nearly any velocity, and is now exclusively standard with both organizations.

The only acoustic meter now on the market is the *Price meter*, a modification of the electric meter, invented by Mr. W. G. Price, United States Assistant Engineer. This meter is especially desirable for its portability and the ease with which it can be handled, as it weighs but little over a pound. In very shallow streams, it gives the most accurate results of any meter. It is held at the proper depth by a metal rod in the hands of the observer, as is the Colorado current meter. This meter is designed especially to stand hard knocks which may be received in turbid irrigation waters, and it can be used in high velocities, as only each tenth revolution is counted. It consists of a strong wheel, composed of six conical-shaped cups, which revolve in a horizontal plane; its bearings run in two cups holding air and oil in such manner as entirely to exclude water or gritty matter. Above the upper bearing is a small air-chamber, into which the shaft of the wheel extends. The water cannot rise into this air-chamber, and in it is a small worm-gear on the shaft, turning a wheel with twenty teeth. This wheel carries a pin which at every tenth revolution of the shaft trips a small hammer against the diaphragm forming the top of the air-chamber; and the sound produced by the striking hammer is transmitted by the hollow plunger-rod through a connecting rubber tube to the ear of the observer by an ear-piece. The plunger-rod is in 2-foot lengths, and is graduated to feet and tenths of feet, thus rendering it serviceable as a sounding or gauging rod.

Gauging Stations. The first operation in making a careful gauging of velocity by means of a current meter, is the choosing of a good station. This consists in finding some point on the course of the stream where its bed and banks are nearly permanent, the current of moderate velocity, and the cross-sections uniform for over 200 feet above and below the gauging station. At this point a wire should be stretched across the stream and tagged with marks placed every 5, 10, or 20 feet apart, according to the width of the stream. An inclined gauge-rod is firmly set in the stream at some point where it can be easily reached for reading. It should be not less than 4 by 4 inches, and marked for feet and tenths of vertical depth. The gauge heights are recorded through a long period of time, in order that the variation in the velocity and discharge may be had for different flood heights.

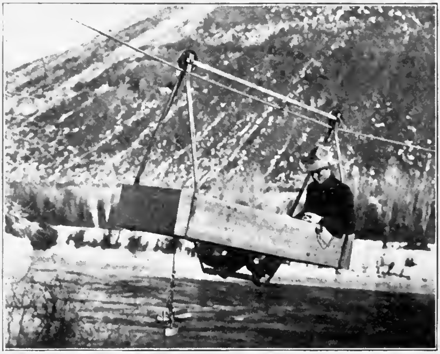


Fig. 27. Gauging the velocity of the stream by means of the Current Meter.

Fluctuations in the height of streams can be measured with even greater accuracy than is obtainable by the readings of a gauge-rod as above described, by using a *Nilometer,* which is a self-reading gauge. The chief objection to the use of the instrument is that its maintenance requires the attention of a person of considerable mechanical skill, in order that it may be kept in proper order. There are three general forms of Nilometer employed by the hydrographers of the United States Geological Survey. These have horizontal recording cylinders, vertical recording cylinders, and vertical record discs. All of these devices are driven by clockwork, and are designed to run a week before removal of the recording paper. The record of stream height by the Nilometer is on a scale less than the actual range of the water, and the recording pencil is connected by a suitable reducing device with a float which rises or falls with the stream. This float is usually placed in a small well near the stream bank, its bottom communicating with the stream bed by a pipe of such size as will not readily become clogged. The fluctuations of the water in this pipe correspond with those in the stream, and turn the recording wheel through the agency of a cord wound around the wheel and having its lower end attached to the float.

Use of the Current Meter. The current meter may be conveniently used, either from a boat attached to a wire cable strung a little above the tagged wire, or from a bridge which does not impede the channel so as to make currents or eddies in the water. In using the direct-acting meter, the gauger holds it in his hands by the rod; and, inserting it in the water at any desired depth, allows it to register for a certain number of seconds. In obtaining the mean velocity of the stream he plunges it slowly up and down from the bottom of the stream to its surface a few times for a given length of time at each section marked on the tagged wire, and in this way gets the mean velocity of each section. The area of this section is of course already ascertained by a cross-section made by measurement or sounding of the stream; and the mean velocity, multiplied into the area of each section, gives the discharge at that point. Care must be taken to hold the rod vertically, as any inclination of the meter materially affects its record. In using the electric meter, it is suspended and inserted in the same manner for moderately shallow streams; but in deep flood streams it is generally suspended by a wire, instead of being pushed down by a rod, and a very heavy weight is attached to its bottom to cause it to sink vertically.

Rating the Meter. Before the results can be obtained, each meter must be rated; that is, the relation between the number of revolutions of the wheel and the velocity of water must be ascertained. This is usually done by drawing the meter through quiet water over a course the length of which is known, and noting the time. From the observations thus made, the rating is determined either by formula or by graphic solution. The distance through which the meter is drawn, divided by the time, gives the rate of motion or velocity of the meter through the water. The number of revolutions of the wheel, divided by the time, gives the rate of motion of the wheel. The ratio of these two is the coefficient by which the registrations are transferred into velocity of the current. This is not a constant. Taking the number of registrations per second as abscissæ (represented by x), and the velocity in feet per second as ordinates (repre-

Fig. 28. U. S. Geological Survey Rating Station at Los Angeles, California. Here the meters used in the measurement of the flow of streams are calibrated.

sented by y), we get the equation $y = ax + b$, in which a and b are constants for the given instrument.

In determining the rating of the meter graphically, the values of x and y obtained directly from the instrument are plotted as coordinates, using the revolutions per second as abscissæ, and the speed per second as ordinates. In this way a series of points are obtained through which a connecting line is drawn, giving the average value of the observations. From the position of the line thus plotted, the coefficient of velocity can be read off corresponding to one, two, or any number of revolutions per second. When in actual use, it is evident that at each rate of speed of the meter there is a different coefficient of velocity. Three or four of these for average variations in velocities may be used in getting the true velocity from the meter record.

Rating the Station. After daily readings of the gauge height of the water have been taken at the station for some time, and the velocity measured by means of the meter at different heights of stream, the results should be plotted on cross-section paper, with the gauge heights as ordinates and the discharges (obtained by multiplying the velocities into the cross-section) as abscissæ. These points generally lie in such a direction that a line drawn through them gives nearly half a parabolic curve, and represents the discharge for different heights. Having once plotted this line, it becomes possible to determine the discharge of the stream at any time by knowing the height of the water from the gauge-rod.

Measuring Weirs. The simplest method of measuring the discharge from canals and streams of moderate size, is by means of weirs. It is extensively used in Colorado and other portions of

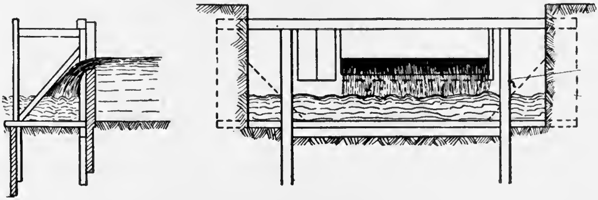


Fig. 29. Rectangular Measuring Weir.

the West, and especially commends itself to the use of irrigators, because of the simplicity and cheapness of construction of the weir, its accuracy of measurement, and its ease of operation. The results of weir measurement are easily interpreted by use of tables giving quantities of flow directly in second-feet.

By a *standard weir* is meant one in which the inner face is a vertical plane, and the edge of the weir, or its crest, is sharp. Weirs are generally made rectangular; and the crest of such a weir may extend to the side of the flume or canal conducting the water to it, or it may be a rectangular notch cut in the weir-plate, the vertical edges being bevelled similarly to the horizontal edge. In the former case, the contractions are said to be *suppressed*; in the latter, the weir is said to have *end contractions*. If the contractions are not suppressed, the weir-plate should extend as a plane on each side of the weir a distance at least three or four times the depth of water on the crest; and for an equal distance below the crest whether there be contraction or not. On account of the difficulty of measuring the depth of water over the crest, the height of water above the crest is measured on the surface of the water before it begins to curve downward toward the weir.

Francis Formula. The theoretical equation for the quantity of discharge over a weir is given by:

$$Q = Av \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The area A is evidently the product of the effective length of the weir into the depth of water flowing over it, while v is the mean velocity of flow through the section. Calling the length of weir l, and the depth h, we have: $A = lh$. From Hydraulics, the mean velocity is given by:

$$v = \tfrac{2}{3}\sqrt{2gh}.$$

Substituting these values in Equation 1, above, there results:

$$Q = lh\,\tfrac{2}{3}\sqrt{2gh} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

or,

$$Q = \tfrac{2}{3}lh^{\frac{3}{2}}\sqrt{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

As was pointed out before, the height h must be taken at some distance above the weir, to be free from the downward curve of the water. The reduction of volume of flow due to crest contraction may be allowed for by a coefficient m inserted in Equation 3:

$$Q = m\sqrt{2g}\,\tfrac{2}{3}\,lh^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Or, representing the product of $\tfrac{2}{3}m$ and $\sqrt{2g}$ by a constant c, there results:

$$Q = clh^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

In the experiments conducted by Mr. J. B. Francis, the value of this constant was found to be 3.33. Substituting in Equation 5, we have:

$$Q = 3.33\,lh^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

On account of the falling away of the surface at the crest of the weir, and the contraction at the ends, if l_1 is the effective length of the weir, one end contraction makes $l_1 = (l - 0.1h)$; and any number of end contractions makes $l_1 = (l - 0.1nh)$, so that:

$$Q = 3.33\,(l - 0.1nh)\,h^{\frac{3}{2}} \qquad \textbf{(7)}$$

Equation 7 is what is known as the *Francis Formula.*

In using the ordinary weir, it should be placed at right angles to the direction of flow of the stream, the upstream face being in a vertical plane. The crest and sides should be chamfered so as to slope downward on the lower side with an angle of not less than 30°, while the crest itself should be horizontal and the ends vertical. The dimensions of the notch should be sufficient to carry the entire stream and still leave the depth of water on the crest not less than five inches. The sectional area of the jet should not exceed one-fifth that of the stream; and in order that the proper proportion may be maintained between the area of the jet and that of the stream, several contractions may be introduced, dividing the weir crest into several orifices.

In the application of formulæ 6 and 7 to determining the quantity of flow over a weir, attention should be paid to the following details:

The water shall not be more than 24, nor less than 4, inches in depth; the depth of water on the crest shall not exceed one-third the length of the weir; there shall be complete contraction and free discharge, the water approaching without perceptible velocity and free from cross-currents. The distance from the side walls to the weir opening should be at least twice the depth of water on the weir; and the height of the crest above the bottom of the channel should be at least twice the depth of the water flowing over the crest. Air should have free access under the falling water, and the channel of approach should be straight and of uniform cross-section.

To determine the depth of water flowing over a weir, a post should be set in the stream a short distance above it, and to this a gauge-rod suitably marked should be attached. For very accurate measurements a hook gauge should be employed, consisting of a hook attached to a divided rod and fitted with a slow-motion screw and vernier. The hook is below the surface of the water; and by turning the slow-motion screw it may be raised until the point of the hook is just in contact with the surface, as indicated by a slight elevation of the surface at the point of the hook. The difference of elevation between the hook and the weir may be taken with a leveling instrument.

Measurement of Canal Water. In order that water flowing in open channels may be sold by quantity, it is necessary that the volume admitted to the canal shall be readily ascertained at any time, and that the method of admission shall be so regulated that it cannot be tampered with. No method has yet been devised for accomplishing this easily and cheaply; and as a result, water is almost universally disposed of by canal owners by means other than direct sale by quantity. In this country, rentals are charged per acre irrigated, rather than by the amount of water required for the irrigation.

Prof. L. G. Carpenter specifies the following conditions as most desirable for a *module*, or apparatus for measuring water for purposes of irrigation:

"Its discharge should be capable of conversion into the common measure, which is cubic feet per second. The ratio of discharge indicated from two outlets should be the actual ratio. The same module should give the same discharge wherever placed; it should be capable of being used on canals of all sizes, and of being set to discharge any fraction of its capacity for the process of distributing pro rata. Attempts to tamper with or alter its discharge should leave traces easy to recognize; and it should be simple enough to be operated by men of ordinary intelligence, so that calculations should not be required to regulate the discharge of different modules or to determine the amount thereof. It should occupy but small space, and the discharge should not be affected by variations of the water-level in the supplying canal. It should be inexpensive, and cause the least possible loss of head. Nearly all modules attempt to maintain a constant pressure of water above the opening, the orifice remaining unchanged."

The following upon the measurement of water is from "Irrigation Institutions," by Elwood Mead:

"The distribution of water for irrigation is attended by many perplexing conditions. Streams vary in volume from day to day. Wells which cannot be lowered in April, often fail in August. The water supply is subject to continued waste and loss. It sinks through the bottom of the canal by seepage, and is taken up by the air through evaporation.

"When the supply was abundant and the seepage limited, these vicissitudes were of small importance; but with the growing use of water, changes in methods and policies are necessary. This is especially true regarding the care taken in its measurement, and in the attention now being paid to the contracts under which it is supplied to irrigators. When streams carried more than was needed, water was seldom measured. Canal companies took what they wanted, and the irrigator was charged for the acres irrigated without any reference to how much he used. The result of this lavishness does not warrant its continuance. It led farmers to substitute water for cultivation, and to injure their land and exhaust streams by wasteful and careless methods. The need of a definite unit of measurement for the commodity bought and sold is now manifest. Without this there can be no satisfactory basis for transactions in water, or any intelligent or certain measure of value for irrigation purposes. In the establishment of such a unit, several things have to be taken into account. It should be in accordance with the requirements of agriculture, so that the quantities to be measured can be regulated by simple and not too costly devices, and be stated in a unit convenient of computation. Any unit, to be generally adopted and enforced, has to be both feasible in operation and in accord with the needs or prejudices of water users. Water cannot be delivered to irrigators by the pound or ton. Three units of measurement are now in general use; and some one of these three is recognized in the laws of nearly every arid State, and is nearly always stipulated in water contracts They are the *inch*, the *cubic foot per second*, and the *acre-foot.*

"In the measurement of water for irrigation, there are two distinct principles involved which it is desirable to have clearly defined and to keep separate in the mind. The first is the unit of volume to be employed, wholly apart from the method by which this unit may be measured in actual practice. Thus, in irrigation, if we say that the unit of measure is the cubic foot per second, the character and volume of the unit are not affected whether water is measured by the flow over a weir, or through a flume, or by the strokes of a pumping engine. The unit may sometimes be the quantity of water which issues from an opening of fixed dimensions, with or without pressure; or the unit may be the acres of land irrigated under certain conditions.

"In some cases, however, the unit of measurement is associated with a special device or instrument by which it is to be actually determined. The form of this apparatus should be in accord with the principles of hydraulics, and be determined by scientific considerations. The *inch* is such a unit of measurement: it has to be associated with some particular device or instrument of measurement. Its use is as old as irrigation. In this country it is older than modern irrigation, having been first used by the placer miner, and borrowed from him by the irrigator. In both mining and irrigation, it is the volume of water which will flow through an inch-square orifice under a uniform and designated pressure. The slope and size of the orifice and the pressure upon it are fixed by law in a number of States, and in others regulated by custom.

"The ruling custom in the United States is to have the orifice through which water is delivered, 6 inches in height, and wide enough to deliver the required number of inches. The pressure on this orifice varies from 4 inches above the center in some places, to 6 inches above the top in others. In Nevada, the inch has sometimes an opening 4 inches in height, with a pressure of 6 inches above the top. Irrigators who are not able to compute the quantity of water flowing over weirs or through flumes, prefer, as a rule, to have their water measured by the inch. They can tell by looking—or believe they can—whether or not the quantity contracted for is being delivered; and when the conditions presented by the statute are complied with, they can tell, with a close approximation to the truth, whether or not they get what they pay for.

"The most serious objection to this unit is the name. Men accustomed to *square inches* and *cubic inches,* confuse them with *miner's inches* and *statute inches.* Because of the confusion, they frequently determine the inches of water being furnished them, by ascertaining the number of square inches in the cross-section of their ditch or lateral, and calling this the number of inches of water received, although in doing so they disregard both the absence of an orifice, the pressure upon it, and the grade or velocity of the stream measured. It is the common practice on many streams in Utah, for the water masters to measure the inches of water in the ditches by taking the cross-section of their flow, and wholly disregarding pressure and velocity.

"A simple device for measuring miner's inches consists of a board 2 inches thick, 12 inches wide, and about 8 feet long. The opening is 1 inch wide and 50 inches long, and the distance from the top of the board to the *center* of the opening is exactly 4 inches on the upstream side. On the downstream side, the opening is beveled so that the whole presents sharp edges to the stream. A sliding board is hung upon the top of the first board, with a strip screwed along its upper edge, this sliding board being wide enough to cover the opening on the upstream side. In the slot, there is a closely fitting block made to slide on the beveled edges, and fastened by a screw to the sliding board. When the sliding board is moved backward or forward by means of its end, which is extended for a handle, the block moves in the slot and determines the length of the opening.

"When used to determine the flow of a stream, the board is placed so as to dam the flow completely, and the sliding board is moved backward or forward until the water is all passing through the slot, the water being kept to the top of the board, or 4 inches above the center of the opening. The length of the opening measures the number of miner's inches of water flowing through. If the flow is too great to pass through the opening 1 inch wide, the opening may be made wider, the water still to be kept 4 inches above the center of the opening.

"Many measuring boxes in European canals are constructed in the most substantial manner, of masonry. The orifice is cut through stone, with edges of metal, and with the utmost precision in its dimensions. Thus far, in this country, but little attention has been paid to accuracy, either in the form or size of openings, although much ingenuity has been shown in designing automatic regulators. The pervading practice in the West is to make the measuring boxes of wood, and to give slight regard either to the freedom of delivery or to securing uniform pressure. One of the reasons why no more consideration has been given to the accuracy of measuring devices, is the fact that the conditions of water contracts are so often not in accord with the way water has to be used. The field of usefulness of the inch is restricted to the measurement of comparatively small quantities of water. It is well adapted to the distribution of water to irrigators, from canals or from the main laterals of canals; but it is not suited to the measurement of rivers or to the distribution of water from a river. Where large volumes, or widely fluctuating volumes, are to be measured, the construction of a satisfactory device for measuring by inches is not practical. There are a number of canals in this country which carry from 50,000 to 125,000 statutory inches. It is manifest that while the width of an orifice can be extended indefinitely without materially affecting the accuracy of the measurements, every change in its depth must materially increase the velocity, and hence the quantity of water discharged by each square inch of its cross-section. Nearly all of the statutes prescribe a maximum depth for the orifice, and require that increase in volume delivered shall be secured by extending its length. To measure the water required to fill the Del Norte Canal, would require an opening 1,736 feet in length, which would be practically impossible.

"The limitations of mechanical devices render the inch unsuited to measuring the flow of rivers. In States where the inch is recognized as the legal unit in the distribution of water among irrigators, some other has to be employed in the measurement of the flow of streams.

"**Cubic Foot per Second.** The cubic foot per second has come into general use as the unit of volume for gauging and dividing weirs, and in measuring the flow of ditches and canals. Nearly all of the arid States and Territories have made it the legal unit in water contracts, and for defining the amounts of appropriation from streams. It has the double advantage of precision in statement, of being well adapted to the measurement of large as well as small volumes of flowing water, and of permitting the employment of varied methods of measurement. In many States it is used in connection with the inch. The flow of the stream and the amounts of appropriations are stated in cubic feet per second. The water, after it is turned into the ditches, is measured out to farmers in inches. This renders it desirable that there should be some basis of comparison, some legally defined ratio between the inch and the cubic foot per second.

"A number of States have passed laws fixing the number of inches which equal a cubic foot per second. Legislation fixing the ratio has been of decided service in the States where the inch is still employed.

"The following is the ratio assumed by law or custom in a number of States:

Colorado,	One	cubic	foot	per	second	= 38.4	statute	inches.
Montana,	"	"	"	"	"	= 40	"	"
Idaho,	"	"	"	"	"	= 50	miner's	inches.
Arizona,	"	"	"	"	"	= 40	"	"
Nevada,	"	"	"	"	"	= 50	"	"
Utah,	"	"	"	"	"	= 50	"	"

"In many places the inch is retained as a term where it has no existence in fact. The farmers who have been accustomed to estimating the flow of water in inches find it hard to think of this flow in cubic feet per second. Because of this, engineers who measure the flow of ditches or canals in cubic feet per second convert this into inches according to the statutory ratio. On many ditches where the inch is still retained as the unit of measurement, there is no measuring box for its delivery. Farmers pay for their water supply in inches, and estimate the flow in their laterals. Where the water is measured, the volume is determined in cubic feet per second, and converted into inches on some arbitrary ratio. The real unit is the cubic foot per second.

"**The Irrigating Stream.** *The irrigating stream* is a unit in common use in Utah. It is a stream which one man can control to advantage, but no rules for its measurement have ever been prescribed. The size of the stream is left to the water masters, who are charged with distributing water to the farmers. The following extract from a notice sent out by a Utah canal company, illustrates the use of this unit:

" 'The books of the company show that you are the owner of ...... shares of stock, and you will therefore be entitled to the use of an "irrigating stream" for .... hours.'

"The water in this canal is not measured, nor are the diversions. The water master estimates the number of 'streams' in his canal, and these streams are used in turn by the farmers. They are supposed to be equal; but measurements made upon a canal to determine the accuracy of the judgment of the water master, gave widely varying results.

"The amount of money paid for water each year by irrigators is so large that it seems surprising that they have not paid more attention to the accuracy with which it is measured. In transactions involving any other kind of property, care is taken to see that it is accurately measured; but, although water costs more than any other commodity used by the irrigator, it is bought and paid for without either buyer or seller knowing how much is delivered. The need of greater accuracy in water measurements has led to the passage of a law in Utah requiring the State Engineer to give information and advice about the placing of measuring devices. The State Engineers of Colorado and Wyoming are required to advise irrigators in respect to the measurement of water; and a recent Colorado statute also provides for the use of registers which will keep a continuous record of the quantity of water delivered. The introduction of registers and greater accuracy in the construction of measuring boxes, is one of the developments of the near future. Their installation will do much to reform water contracts, prevent the awarding of excessive amounts of water in decrees, promote economy and efficiency in use, and extend the reclaimed area."

SEWER IN FILLED EMBANKMENT, BOSTON, MASSACHUSETTS
Metropolitan Sewerage System.

SEWERS AND DRAINS

PART I

1. Introductory Definitions and Discussions. *Sanitary Engineering* is that branch of engineering which has to do with constructions affecting health. It thus might be claimed to include the manufacture and transportation of foods, the architecture of buildings, and many other things which affect the health of communities; but in ordinary use, a more restricted definition of the term is adopted.

In common practice, the term *Sanitary Engineering* is taken to include only *water supply engineering* and *sewerage engineering,* the former branch dealing with securing a satisfactory supply of water, and the latter with the satisfactory removal of surplus and waste liquids. Sewerage is the subject of this instruction paper, water supply being treated by itself.

Sometimes *sanitary engineering* is given a still more restricted meaning, and is taken to include sewerage only.

A *drain* is a canal, pipe, or other channel for the gradual removal of liquids. In sanitary engineering, the two principal kinds of drains are, first, those for the removal of comparatively pure ground waters and surface waters, as in land drainage; and, second, those for the removal of polluted liquids, as in sewerage systems.

A *sewer* is a drain for the removal of foul, waste liquids. Usually sewers are closed, underground conduits. An *open sewer* is an open channel which conveys foul, waste liquids.

Sewerage is a general term referring to the entire system of sewers, together with any accessories, such as pumping plants, purification works, etc. Thus we may speak of the "sewerage" of a city, or of the "system of sewerage," or of the "sewerage system."

Sewage is any foul, waste liquid.

Sanitary sewage is the foul wastes of human or animal origin from residences, stables, stores, public buildings, and other places of human or animal abode. By far the greater part (usually 99.8 per cent or more) of sanitary sewage, commonly, is ordinary water, which is added to the wastes themselves in this large volume simply to facilitate removal.

Manufacturing sewage is the foul wastes from factories. In different factories, it is of extremely different nature. It is often exceedingly strong, and very offensive and difficult to dispose of, as compared with sanitary sewage.

Storm sewage is the storm water flowing from city surfaces during and after rainstorms. Though polluted, especially at the beginning of a storm, from the droppings of animals and the other surface filth of cities, it is not so foul, nor so liable to swarm with disease germs, as is sanitary sewage.

The terms *sewage* and *sewerage* are often misused by persons not engineers, to mean the same thing. Thus such persons often speak of the "sewage system" instead of the "sewerage system;" of the "disposal of the sewerage" instead of the "disposal of the sewage," of a city. So common is the misuse that some sanction can be found in the dictionaries; but engineers should be careful to restrict the meaning of the word "sewage" to the liquid which flows in the sewers, while the word "sewerage" should never be so applied.

Sewer air, often miscalled *sewer gas,* is the air in the sewers above the liquid contents. It has no definite chemical composition, but contains varying proportions of pure air and of carbonic acid gas, marsh gas, sulphuretted hydrogen, and the various products of decaying organic matter. Sewer air is constantly changing in composition even in the same sewer. While considered injurious to health when breathed, it has not been proved to be in itself the direct means of communicating infectious diseases.

2. Historical Review. Sewers and drains are of very early origin. Among the ruins of all ancient civilizations, are found the remains of masonry and tile conduits constructed for drainage purposes.

In Fig. 1, for example, (from Fergusson's *History of Architecture*), are shown the remains of a large masonry sewer or drain built by the ancient Assyrians in the eighth or ninth century B. C., for one of their palaces at Nimrud. This is one of the earliest examples found of the use of the arch in masonry.

In Fig. 2 is shown the mouth of the *Cloaca Maxima,* or great sewer, of ancient Rome, built in the seventh century B. C., and still

SECTION OF A SAND-CATCHER

Metropolitan Sewerage System, Boston, Mass.

in use after the lapse of 2,500 years. Without this sewer, a large tract of ancient Rome could not have been inhabited; and in speaking

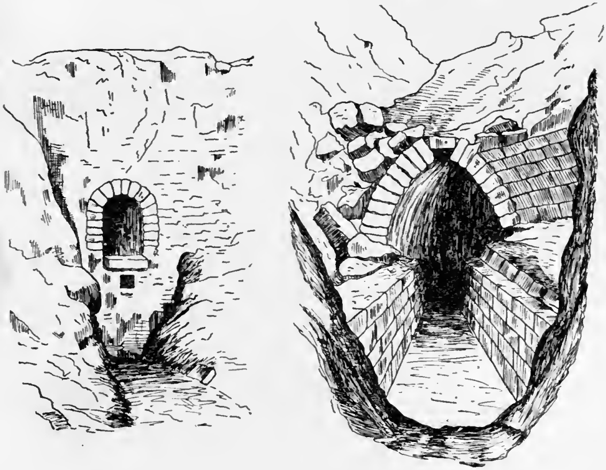


Fig. 1. Ancient Assyrian Sewers at Nimrud.

of it, one authority says: "To this gigantic work, admired even in the time of the magnificent Roman Empire, is undoubtedly owing the

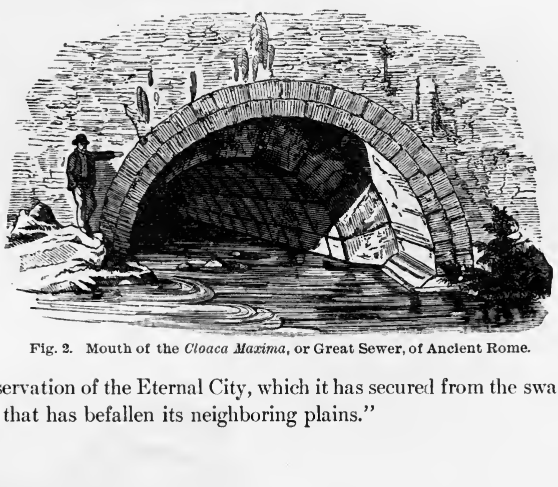


Fig. 2. Mouth of the *Cloaca Maxima*, or Great Sewer, of Ancient Rome.

preservation of the Eternal City, which it has secured from the swamping that has befallen its neighboring plains."

In many other ancient cities and structures, the remains of intelligently planned drainage systems have been discovered; and it is evident that the ancients paid great attention to this matter so vitally affecting health. The art reached its highest ancient development in the time of the Roman Empire. The Romans, in fact, were the greatest engineers of antiquity, and especially excelled in sanitary engineering (both water supply and drainage). They were proficient in land drainage, as well as in sewerage.

With the fall of the Roman Empire, sanitary engineering suffered the same retrogression which befell learning and science; and for a thousand years—throughout the Middle or Dark Ages—it was almost entirely neglected. The impure water supplies and the accumulated filth of mediæval cities produced fearful consequences in the terrible pestilences which desolated Europe.

With the revival of learning and science in the 14th and 15th centuries, attention again came to be paid to sanitary engineering; but for three or four hundred years more, little was done toward putting drainage and water supply on a scientific basis. Drains, rather than sewers, were built in the various towns as absolute necessity made imperative; but they were constructed piecemeal, and not so as to form comprehensive systems. They were not made water-tight or self-cleaning; but it was usually considered necessary to make them large enough for men to enter to remove the filth, whose accumulation and festering in them were believed unavoidable.

In England, modern sanitary engineering may almost be said to have had its origin; yet so late as 1815, laws were enforced forbidding the emptying of fæcal matter into the sewers. "Such matter was generally allowed to accumulate in cesspools, either under the habitations of the people or in close proximity thereto." * In fact, though no longer enforced, these laws were not repealed until 1847, when Parliament passed an exactly contrary act, making it compulsory to pass fæcal and other similar foul matter into the sewers.

Modern sanitary engineering, especially as regards sewerage and drainage, has had almost its entire development since 1850. It was not until 1873 that there was published a comprehensive treatise on sewerage, that of Baldwin Latham, already quoted. At about this time, also, much attention began to be paid in England to sewage

*Baldwin Latham.

purification. It was reserved, however, for America to put sewage purification on the road to a satisfactory scientific solution, by the thorough investigations of the Massachusetts State Board of Health, begun in 1887 and still under way.

In America, much was done in the third quarter of the 19th century to advance sewerage engineering, through the studies of able engineers in connection with the design of systems for Chicago, Brooklyn, and other large American cities, the results being published in papers and reports, or in book form.

About 1880 the *separate* system of sewerage came strongly into prominence in America, as advocated by the late Col. Geo. E. Waring; and the construction of the Memphis (Tenn.) sewers on this system at that time, together with their great success in putting a stop to the fearful epidemics which had so often desolated that city, did much to make sewerage possible for small cities. At present, sewers have become so common and so necessary in modern life, that villages of 2,000 population, or sometimes of even less, are very generally taking up their construction.

With the present wide adoption of sewers, even by small communities, sewage disposal has come to be of very great importance, and is now undergoing great development. Many discoveries remain to be made in this line, in which the guiding principles have not yet been so thoroughly worked out as in the construction and maintenance of sewers themselves.

3. Importance and Value of Sewerage and Drainage. The importance and value of the constructions of sanitary engineering can hardly be exaggerated. Upon them absolutely depends the health of every city. One needs but to read descriptions of the great modern epidemics of yellow fever at Memphis and New Orleans, or of cholera at Hamburg, or to have been engaged to visit as sanitary engineer an American town during one of the numerous recent outbreaks of typhoid, to understand the truth of the scripture, "All that a man hath will he give for his life." Yet not only could sanitary engineering absolutely prevent every such epidemic; but, in addition, it could annually save thousands upon thousands of other lives which now succumb to bad sanitation.

Already very much has been accomplished in this direction by improved sanitation, though ideal conditions are yet seldom attained.

A prominent sanitary engineer estimated from actual statistics, that as early as 1885 there was a saving from this cause of 100,000 lives and 2,000,000 cases of sickness, annually, in Great Britain, in a total population of only 30,000,000. Figuring on the basis of the money value alone of the lives saved, and of the sickness and loss of time avoided, the money value of the above result would be almost incalculable.

In many individual cities, statistics have shown in death rates an immediate lowering, due to the construction of sanitary improvements, more than sufficient in money value to the community to pay for the entire cost. Funeral and sickness expenses saved, alone, often make enormous sums.

In this connection, it should be said that pure water supply and good sewerage are both essential, and that it is impossible to separate the value of one from that of the other. A polluted water supply may spread disease, no matter how perfect the sewerage, and an abundant water supply is essential to the proper working of sewers. On the other hand, without sewers and drains, an abundant water supply serves as a vehicle to enable unmentionable filth to saturate more deeply and more completely the soil under a city. Cesspools are even more dangerous than privy vaults.

In addition to direct prevention of communication of disease by unsanitary conditions, modern sewerage facilities are so great a *convenience* that this advantage alone is usually more than worth the cost. This is shown by the increased selling and rental value of premises supplied with sewerage facilities. No sooner is a partial or complete sewer system constructed in a town, than prospective buyers or renters begin to discriminate severely against property not supplied with modern sanitary conveniences; and persons looking for new locations for business ventures or residence purposes, discriminate in like manner in favor of towns having good sewerage.

So great has become the demand for sanitary conveniences, that they are now being installed in farmhouses as well as in the city. It is now possible for any farmer, at an expense of only a few hundred dollars, to have hot and cold water piped under pressure in his house, a bathroom and other plumbing fixtures, and his own sewage-disposal plant. This has already been accomplished in many cases. Such improvements, if made in accordance with correct principles, greatly better the sanitary conditions of the home; and they also prevent much disease by doing away with the exposure to inclement weather, which is so dangerous an accompaniment of the old-fashioned, barbarous, outdoor privy.

The great importance of sewerage may be realized by giving some consideration to the enormous sums of money which have already been spent for sewer systems in this country alone. Villages of 3,000 population in rural communities, often spend $50,000 or more upon a system. The city of Chicago has in recent years spent $50,000,000 in securing merely a satisfactory outlet for its sewers, without counting a dollar of the vast sums expended on the sewers themselves. In the United States, hundreds upon hundreds of millions of dollars have been invested in sewers.

SYSTEMS OF SEWERAGE

4. A *privy vault* is a receptacle, usually a mere excavation in the ground, for the reception of fæcal matter and urine. To prevent dangerous pollution of the surrounding soil and ground water, privy vaults should be lined with water-tight masonry; but this is seldom attempted, and even if attempted, is still more seldom accomplished, for it is difficult in such work to secure absolute freedom from leakage. The privy vault, frequently, is simply abandoned and covered over with earth when full, it being cheaper to change the location than to clean out the old pit.

The privy vault, with its inevitable befouling, in the immediate vicinity of the home, of earth, air, and water, the three great requisites of health, and with its danger from pneumonia and other diseases which may be contracted from exposure, should be adopted only in case of absolute impossibility to secure something better, and even then only as a temporary resort. It is not so objectionable in the country as in the city, if located far away from the well; but here the trouble is that it is usually placed too close to the well which furnishes the drinking water. In the country the leachings from hog pens, cattle yards, and manure piles frequently add to the contamination of the drinking water. It is impossible to set any safe distance at which a well may be placed from a privy, owing to the variable nature of the soil. The contamination may be carried very far in gravel strata or rock crevices. Impervious clay confines filtration within narrower limits.

5. A *cesspool* is a receptacle for receiving and storing liquid sewage. It consists usually of an excavation dug in the ground, lined with masonry, and covered, into which the sewer from the house discharges. To prevent contamination of the surrounding soil and ground water, the cesspool should be made absolutely water-tight, and its contents should be removed whenever it becomes full.

A *leaching cesspool* is one not made water-tight. The liquid contents partly leach away into the surrounding soil, and often into sand or gravel strata, or crevices in the rock, which may carry the contamination to great distances. Owing to the offensive nature of the work of cleaning out cesspools, and to the expense thereof, cesspools as a usual thing are deliberately made not water-tight. The owner congratulates himself if he strikes a crevice in the rock or a gravel stratum which prevents his cesspool from filling up, though even a little thought will often show that he is thus directly contaminating the water vein which supplies his own or his neighbor's well. Even then he does not usually escape permanently the expense and annoyance of being forced to clean out the cesspool, for in time almost any crevice or porous stratum will clog so as to permit only partial escape of sewage.

Leaching cesspools should be absolutely prohibited by law. They are even more dangerous than the privy, for the liquid sewage in them can penetrate further into the surrounding soil than the fæcal matter of the privy vault.

The frequent effect of cesspools and privies is illustrated in Fig. 3, which does not at all exaggerate conditions very frequently found in cities and villages. Often the tearing down of old buildings, prior to the erection of new, exposes to view the rear of lots, and shows sometimes a half-dozen privies grouped within a few rods of several wells. The nose and the eye give convincing evidence of foulness in such cases; and chemical or bacterial analyses are not necessary to demonstrate the danger in using the wells; but the same dangerous conditions pass unnoticed in many other places in the same city, because not exposed to casual view. In time, the whole ground water under such a village or city becomes contaminated, and poisons wells and damp cellars and the exhalations from the ground.

6. A *dry closet* is a privy having a tight, removable receptacle in place of the vault, and provided with means for covering the contents with dry dust, ashes, or lime each time the closet is used. Usually a small shovel and a box are used to hold the dust or other absorbent material. Enough of the dry material should be used to absorb all liquids. The contents should be removed and hauled away in the tight box when it is full, to be emptied in a safe place or used for fertilizer. The dry earth closet is an improvement over the privy vault, but is not a safe or otherwise satisfactory arrangement.

7. The *pail system* is one in which the fæcal matter and urine are received in tight pails, which are removed daily, or at least every few days, by regular city employees. The pails are carried to some safe place, there emptied, and returned after disinfection. Although the pail system has been tried in America under exceptional conditions, it is entirely unsuited for use here, and is almost never employed, even in Europe, where the people will submit to the police interference necessary for satisfactory operation.

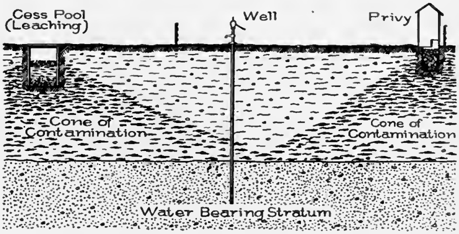


Fig. 3. Showing How Contamination of Well Water may Occur through Proximity of Cesspools and Other Sources of Filth.

8. *Pneumatic systems* of sewerage are those in which the sewage is forced through the street pipes by air, either by a partial vacuum, as in the *Liernur system* (tried in Holland), or by compressed air, as in the *Berlier system* (tried in France). Neither system is used at all in America, or to any important extent in Europe. The expense of construction and operation, and the liability of all such mechanical appliances frequently to get out of order, make them unworthy of consideration.

9. *Crematory systems* are devices for disposing of fæcal matter, urine, and garbage on the premises by drying and then burning. There are several patented methods. The matter to be disposed of is received in a furnace-like structure on the premises, built usually

of masonry, which is open to a chimney, as well as to the various closets in the building. The chimney is supposed to maintain a current of air out of the rooms in which the closets are located; this dries the material, which is then burned at intervals.

Where sewers have not been available, crematory systems have been installed in many schools and other public buildings in the United States; but, while sometimes fairly satisfactory for a while, they are usually soon found to be troublesome, expensive, and dangerous. The air-currents sometimes reverse into instead of out of the rooms containing the closets; danger ensues unless the burning is regularly attended to; and, without constant care in the attendance, the whole apparatus is likely to get out of order. Moreover, it is entirely unadapted to the disposal of liquid wastes such as those from sinks, washbowls, laundry basins, and bathtubs, which are as necessary to be taken care of as fæcal matter and urine.

In the foregoing paragraphs (Arts. 4 to 9), various makeshifts for caring for sewage have been described which are not worthy the name of "systems," although the privy vault and the cesspool are in very wide use. We next come to the only methods for removing sewage which are at present worthy of serious consideration when planning a sewerage system.

10. Water-Carriage Systems. Water-carriage systems of sewerage are those in which water is added to the fæcal matter and other foul wastes in such quantities as to permit of their rapid removal by gravity in sewers. As already stated, the water so added usually constitutes 99.8 per cent or more of the resulting sewage.

Water-carriage systems are now so universally used for sewerage purposes, that usually the two terms may be considered synonymous. That is, in the present day, a sewerage system is practically always a water-carriage system.

There are two kinds of water-carriage systems—namely, the *Combined System* and the *Separate System.*

11. Combined System. The combined system of sewerage is that in which the storm sewage flows in the same sewers with the sanitary and the manufacturing sewage. The combined system came into use prior to the separate.

12. Separate System. The separate system of sewerage is that in which separate sewers are provided for the storm sewage and for the sanitary and manufacturing sewage.

13. Comparative Merits of Combined and Separate Systems. The separate system came into prominence about 1880. At that time and for many years following, there was an active discussion over the relative merits of the two systems, some prominent engineers advocating one, and some the other. At the present time, the discussion has died down, and sanitary engineers use both, adopting whichever is best suited to local conditions, and often using a combination of the two.

In favor of the separate system, the following points have been cited:

1. The sanitary sewage which constitutes the dry-weather flow of combined sewers is so very small in comparison with the storm sewage, that in circular sewers, which are the most economical to build, it forms merely a trickling stream, with little velocity, over the bottom of the large sewers required; while in the separate system the sewers are proportioned for this small volume, and the sewage consequently has good depth and velocity. Moreover, sanitary sewers are free from the sand and other street detritus which are inevitably washed into combined sewers during storms, and which are especially troublesome in forming deposits. Hence, in the separate system, it is easier to make sewers self-cleansing from deposits.

2. Above the low-water line in combined sewers, the extensive interior surfaces of the large sewers required become smeared with filth in times of flood, which remains to decay and produce foul gases after the flood subsides.

3. On account of the comparatively small size of the sanitary sewers of the separate system, it is easier to flush them so as to keep them clean. Automatic flush-tanks can be used at small expense to do this very satisfactorily.

4. On account of the comparatively small size of the sanitary sewers of the separate system, the air in them is much more frequently and completely changed by the daily fluctuations in the depth of sewage and by the currents of air through ordinary ventilation openings. Hence, in the separate system, ventilation is easier and more perfect.

5. In case the sewage has to be purified, the separate system is more economical, because only the sanitary sewage need be treated, the storm sewage being discharged into nearby natural watercourses.

6. In small cities, and in large portions of large cities, the storm water can usually be carried some distance in the gutters, and then removed by comparatively short lengths of storm sewers, laid at shallow depths and discharging into the nearest suitable natural watercourses. In such cases, a separate system of sewers will usually cost only a fraction, frequently only one-third, as much as a combined system. For small towns, the great cost of a combined system would often prohibit the construction of sewers entirely, or postpone it almost indefinitely, were it not that a separate system can be built so cheaply. On this account alone, the introduction of the separate system of sewers has been of incalculable benefit in America.

7. On account of their relatively small size, sewers of the separate system can be made almost entirely of vitrified sewer-pipe, which has the important advantages over brick sewers, of greater smoothness, of being impervious, of having few joints, and of ease in making the joints practically water-tight. It is impossible to make even a pipe sewer absolutely water-tight, and with brick sewers the difficulty is very much greater.

In favor of the combined system, the following allegations, corresponding to the above points, have been made:

1. By making combined sewers egg-shaped with the small end down, or by making a small, semicircular channel in the bottom (see Figs. 19, 24, and 25), the depth and velocity of the dry-weather flow can be made sufficient to cause the sewer to be self-cleansing.

2. The coating on the interior surface of large sewers above the low-water line is not dangerous, and in fact is of very little importance.

3. While it is true that the smaller, separate sewers can be flushed more perfectly for the same expense, the larger, combined sewers are more convenient for removing obstructions, and are flushed out very completely (though at too long intervals in dry weather) by the floods of storm sewage during rains.

4. In regard to ventilation, the larger volume of air over the sewage in the larger, combined sewers dilutes to a much greater degree the gases from the sewage.

5. In case the sewage must be purified, it must be remembered that the early flow of storm sewage from the streets is foul, to some extent, from the droppings of animals and other surface filth; and it may in some cases be questionable whether this may not require purification in addition to the sanitary sewage.

6. Wherever, as in the case of the business districts of large cities, it is necessary to provide as great a length of storm sewers as of sanitary sewers, it will be cheaper to build one set of sewers, as in the combined system, rather than two, as would be required in such districts with the separate system.

The *general conclusions of sanitary engineers* at present regarding the relative merits of the separate and combined systems, are as follows:

a. Either system can be made satisfactory from a sanitary point of view.

b. The cost of a properly designed system, including means for safe disposal of sewage, should ordinarily decide which of the two systems should be built.

c. On the basis of cost, the separate system is usually the better for small cities, for suburban and sometimes residence districts of large cities, and for all cases, even those of large cities, where the sanitary sewage requires treatment while the storm sewage can be safely discharged into nearby watercourses. The separate system has just been recommended for the city of Baltimore on this last account.

d. Similarly, on the basis of cost, the combined system is usually the best for the business and other very thickly built-up districts of large cities, and, in general, where storm sewers must be coextensive with sanitary sewers; also for cases where both storm sewage and sanitary sewage require purification.

e. Often a combination of the two systems can be made to advantage, storm water being admitted to the sewers only in certain portions of the system, such as the business districts.

GENERAL FEATURES OF SEWERS

14. Kinds of Sewers. *Sanitary sewers* are those constructed to carry foul waste liquids of human or animal origin—that is, sanitary sewage. Since sewage of human or animal origin is most apt to contain the germs of human diseases, sanitary sewers require special precautions in design, construction, and maintenance, to render them safe. Manufacturing sewage is often, however, even stronger and more offensive than sanitary sewage, and hence requires equal precautions. In the separate system, the manufacturing sewage should go into the sanitary sewers or into special sewers of similar character.

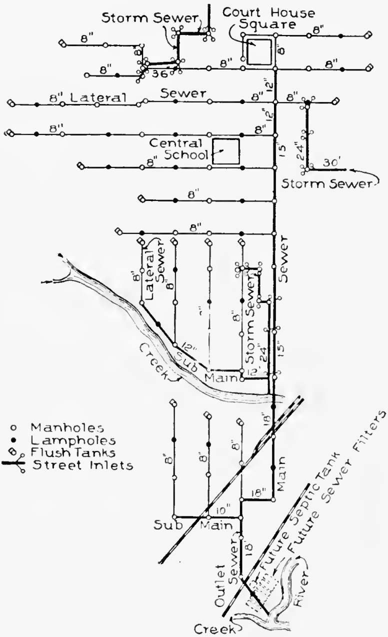


Fig. 4. Kinds of Sewers and Arrangement of Accessories.

Combined sewers are those constructed to carry both sanitary sewage and storm sewage. With the combined system, the manufacturing sewage also usually goes into the combined sewers.

Storm sewers are those constructed to carry storm sewage only.

An *outlet sewer* is one connecting a sewer system, or a part thereof, with the point of final discharge of the sewage.

A *main sewer*, or *sewer main*, is the principal sewer of a city, or of a large district thereof, into which branch sewers discharge.

A *sub-main sewer* is a branch of a main sewer, receiving in its turn the discharge of smaller branches.

A *lateral sewer* is one not receiving the discharge of other sewers, hence serving only property closely adjacent.

In Fig. 4, the various kinds of sewers above described are shown, from a portion of the actual sewerage map of a small city, sewered on the separate system.

15. *Intercepting sewers* are those built across lines of other sewers, to intercept the sewage flowing in them and carry it away to different outlets.

In Fig. 5 are shown the intercepting sewers of the city of Chicago, built along the lake front to intercept the sewage in the sewers which formerly discharged into and polluted Lake Michigan, from which the water supply of the city is taken. From the intercepting sewers, the sewage is pumped into the Chicago River, which now discharges through the great Drainage Canal into the Desplaines river, the Illinois River, the Mississippi River, and the Gulf of Mexico.

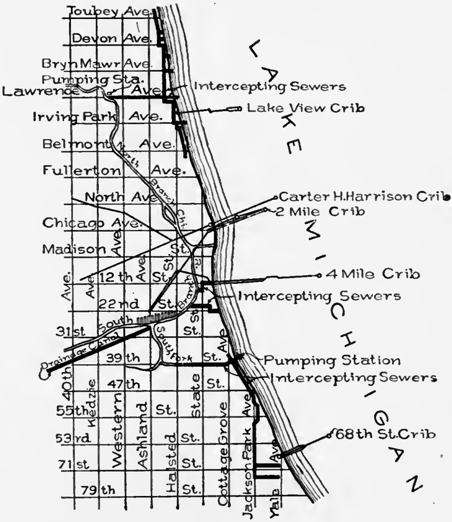


Fig. 5. Intercepting Sewers of the City of Chicago, Ill.

16. General Description of Sewers. Sewers, as usually built, are smooth pipe or masonry conduits, as nearly water-tight as practicable, buried in the ground as deeply as necessary to serve the adjacent houses and drain other territory tributary upstream. They are very carefully constructed to an exact grade line, determined by the engineer who made the sewer plans:

Unless special circumstances require other forms, sewers are usually made circular, this shape giving the greatest strength and area for a given amount of material. For other shapes, and the circumstances to which they are adapted, see Figs. 19 to 25.

The *invert* of a sewer is the lowest point on the interior surface (being so called because the interior curve is there inverted). When the grade of a sewer is mentioned, or the elevation of the sewer at a given place is spoken of, the invert is always meant. The invert is also sometimes called the *flow line.*

Almost all sewers up to 24 inches' diameter, and many from 24 to 36 inches' diameter, are made of vitrified or cement pipe. Above these sizes, concrete or brick masonry is ordinarily used. Stone masonry and iron pipe are also used, but only seldom. A comparison of these materials is given elsewhere in this paper.

At intervals along sewers, *manholes* (Art. 21) and *lampholes* (Art. 22) are placed to permit examination and repairs, and often *flush-tanks* (Art. 23) are provided to keep the sewers clean. In the case of storm sewers and combined sewers, either *street inlets* or *catch-*

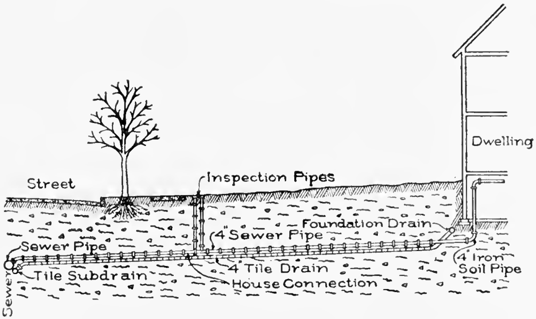


Fig. 6. Street Sewer, Subdrain, and House Connection.

basins (Art. 27) must be provided, for admitting the storm water to the sewers. These are usually placed at or near the curb corners at the street intersections.

A general idea of the relation of a sewer to a building served by it, may be gained from Fig. 6. The sewer there shown is a pipe sewer. Usually all lateral sewers are made of pipe; and in the separate system, the submains and mains also, unless the city is quite large.

17. Location of Sewers. Sanitary sewers are usually placed on the center lines of the streets, so as to give equal fall from the houses on both sides. On this account, water, gas, and heating mains, storm sewers, and other conduits should be constructed far enough from the center lines not to interfere with the sanitary sewers. Not infrequently the center of the street is found already occupied by other conduits which were located without proper foresight; and it is then necessary to place the sewer nearer to one side than the other.

In cases of streets on side hills, it is sometimes necessary to place the sewer close to the downhill side of the street, in order to serve houses on that side which are lower than the street grades.

In a few cases of excessively wide avenues, especially if paved, it is cheaper to build two lines of sanitary sewers, one on each side, than to construct the longer house connections required.

In any town having a fairly extensive system of alleys, careful consideration should be given by the sewerage engineer to the feasibility and desirability of locating part or all of the sanitary sewers in them instead of in the street. In Memphis, this plan was followed as far as practicable. It is not usually feasible to locate combined or storm sewers in alleys, because such sewers must receive storm water from the streets running in both directions, and hence must usually have the street inlets placed at the street corners.

Streets vs. Alleys for Sanitary Sewers. Location of the sanitary sewers in the alleys has a great advantage in avoiding the tearing up of the streets and pavements for sewer repairs and for new house connections, which not infrequently causes them serious injury. Pavements are often ruined by the trenches dug for water, sewer, gas, and other connections. Also, if the sewers are in the alleys, the trenches for house connections do not cross the lawns in front of the houses.

On the other hand, the system of alleys in the ordinary town is a public nuisance. They are usually filled mainly with manure piles, garbage, and debris of all descriptions; and they open through the middle of the blocks vistas which suggest most forcibly a neglected city dumping ground. Owing to their vile sanitary condition, the alleys are usually the first danger spots demanding attention when a town is threatened with an epidemic. Except in the business districts where they can be paved and policed, there is no necessity for alleys unless the lots are very narrow, for in almost every town there are sections which do without and never miss them. Teams can without inconvenience drive in from the front, along a cinder or gravel drive. Such sections are better off without the alleys, from both the sanitary and the æsthetic points of view.

For the above reasons, it is often unwise to perpetuate, or perhaps even extend, the alley system by locating sewers in them.

Again, the system of alleys, more often than not, is far from being as complete as the street system; and in such cases it will usually add considerably to the total length of sewers required to serve a given territory, if part of them are placed in the alleys. The alleys, also, are usually too narrow to permit the construction of sewers of considerable depth, without trouble as regards the excavated material, the handling of pipe, etc. Moreover, houses and the fixtures in them are usually so located that the house connection would be longer to the alley than to the street, requiring a deeper sewer for equal service. This, however, is not always the case.

The sanitary engineer should study each town by itself, and decide this question after giving due weight to all these various considerations.

18. Depth of Sewers: The depth of sanitary and combined sewers should be great enough to afford good drainage to the basements of all buildings. This will usually call for the tops of the sewers to be about $3\frac{1}{2}$ feet below the basement floors, as follows:

MINIMUM DEPTHS FOR SANITARY AND COMBINED SEWERS

Fall from sewer to house	2 ft. 0 in.
Fall from basement floor to house connection	1 ft. 6 in.
Total from *top* of sewer to basement floor	3 ft. 6 in.
For *sewer laterals*, add to the above for fall at sewer	1 ft. 0 in.
Total from *invert of lateral sewer* to basement floor	4 ft. 6 in.
For *residence districts*, add for ordinary-depth basements below street level	4 ft. 0 in.
Total minimum depth to invert of *lateral sewers in residence districts*	8 ft. 6 in.
For business districts, add for ordinary-depth basements	8 ft. 0 in.
Total minimum depth to invert of *lateral sewers in business districts*	12 ft. 6 in.

Hence, under average conditions, the depth of sanitary and combined pipe sewers of 12-inch diameter and less, should be not less than $8\frac{1}{2}$ feet in residence districts, and $12\frac{1}{2}$ feet in business districts. If, however, there is only a short stretch of low-lying ground on a residence street, it may be advisable to reduce the above depth, say to 6 feet as a minimun, when by so doing a very long stretch of sewer can bé lessened that much in depth throughout, and a large saving in cost made thereby.

SEWER TRENCH IN BROOKLINE, MASSACHUSETTS
Metropolitan Sewerage System of Boston.

In the case of sanitary and combined sewers more than 12 inches in height, the above depths should be increased by the excess over 12 inches, for the house connections should enter near the top of the sewer.

In the case of storm sewers and of outlet and intercepting sewers, the depth will no longer be determined by the depth of basements alongside. In these sewers three other considerations determine the depth: (1) the depth at the upper end necessary to afford a good outlet for the sewage; (2) the grade necessary to give good velocity; (3) the depth necessary to prevent injurious heaving of the sewer foundations by frost.

In regard to the third point, no danger need be apprehended of the sewer itself freezing up, even if it be laid practically at the surface, for a stream of warm, flowing sewage will not freeze. There will be little or no danger of trouble from heaving, if the sewer foundation be four feet under ground; and many stretches of pipe sewers only two or three feet deep operate with entire satisfaction even in the northern United States.

19. Subdrains. It has already been stated that sewers should be made as nearly water-tight as possible. Otherwise there would be danger of the sewage leaking out so as to contaminate the adjacent soil. Hence, while it is not possible at any reasonable expense to make sewers *absolutely* tight, they should be built with the utmost care in this particular.

Yet, when due care is used in this respect, the sewer is made unfit for performing another important duty—that of draining away subsoil water so as to dry out unwholesome dampness from the soil, and especially from wet cellars and from under and around houses built on low ground.

In order to secure such drainage, and also, in case of wet ditches, to help remove water from the trenches during construction, it often becomes necessary or advisable to add to the sewer a *subdrain.*

A *subdrain* is a line of drain tile or sewer pipe laid with open joints, in the same trench with the sewer.

To allow connections with cellar drains to be made from both sides of the streets, the subdrain should be placed with its top a few inches below the bottom of the sewer; and to leave a firm foundation for the sewer itself, the subdrain should be placed a little to one side of the sewer.

With the above arrangement, special care should be taken to make the sewer joints tight, and there is some danger of slight leakage of sewage into the subdrain. Such leaks tend to stop themselves as time passes.

It is not safe to connect cellar drains directly with a sewer, even though they are trapped to prevent the sewer air from penetrating into and filling the pores of the soil under houses. In dry times, there may be no water running in the cellar drains; and at such times the water in a trap may evaporate so as to unseal it. Cellar and foundation drains should be connected to the subdrain instead of to the sewer itself.

The general relation of the subdrain to the sewer in the street, and the method of connecting it with the foundation drains, may be seen in Fig. 6.

In construction, the joints of the subdrain should usually be wrapped with muslin to prevent the entrance of mud and sand. The cloth, of course, does not last long; but by the time it rots, the soil around the tile will usually have become recompacted so that there is no longer danger of its getting into the drain. In quicksand, it may sometimes be necessary to fill in fine pebbles or broken stone around the subdrain.

20. House Connections. In Fig. 6 is also shown the method of connecting the sewer itself with the iron soil-pipe which drains the different plumbing fixtures, and which should extend at least 6 feet outside the basement wall. The house connection should be a line of 4-inch vitrified sewer-pipe, laid at right angles to the sewer, with tightly cemented joints, and if possible to at least a 2 per cent grade (that is, with a fall of 2 feet in 100 feet length). Some prefer 6-inch house connections; but these should not be allowed with 8-inch sewers, as the house connection may then allow obstructions to be carried to the street sewer large enough to catch therein and cause stoppages. At the sewer, the house connecton should turn down, by a 4-inch 45-degree elbow, into a 4-inch Y-junction laid so as to slant upward 45 degrees—all as shown in Fig. 7. This slant upward

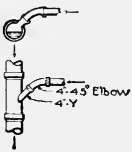


Fig. 7. Junction of House Connection with Sewer.

keeps the Y from affecting the smooth ordinary flow in the sewer.

In case the sewer is more than 12 feet deep below the street surface, the expense of digging down to it in making house connections would be so great that it is usually better, while the trench is open during sewer construction, to put in a *deep-cut house connection*, as shown in Fig. 8. In this case, sewer pipe must be used from the subdrain also, if such a drain is used; and care should be taken to turn the bells of the subdrain connection down so that the plumbers need make no mistake in the connections afterwards.

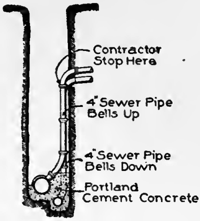


Fig. 8. Deep-Cut House Connection.

In sewer construction, a Y-junction for a house connection (or a deep-cut house connection, if the sewer is over 12 feet deep), should be conveniently located opposite each lot on each side of the sewer; and the ends should be stopped with vitrified stoppers, covered over with sand and then cemented in. Full and accurate records must be kept of the exact locations of these connections, so that they can be found without trouble at any time.

No person should be allowed to cut or break into a pipe sewer for making house connections or any other kind of junction. If there is no Y or T-branch already set for the connection, a full length of pipe should be broken out and the proper Y or T-branch inserted. A skilful workman can readily do this by breaking off one-half the bell of the new pipe, and of that of the old piece into which it must be inserted, and turning the new piece half around after insertion. The joints must then be re-cemented with great care.

21. Manholes. It has already been stated (Art. 16) that manholes must be placed at intervals along sewers, to permit of examination and repairs. These manholes are usually circular brick wells, with Portland cement concrete bottoms and heavy cast-iron covers, as shown in detail in Fig. 9. They must be large enough at the bottom, and for a couple of feet above the top of a pipe sewer, to permit a man to work comfortably. Four feet in diameter is a satisfactory size. Sometimes the manholes are made elliptical at the bottom, with the long axis lengthwise of the sewer; but this form is more difficult to build. Above the point mentioned, the sewer may be drawn in gradually to a diameter of about 2 feet 9 inches, at a point

2 feet 9 inches below the street surface, and thence narrowed more rapidly to about 20 inches diameter at the bottom of the cover casting.

The cover casting may be of any manufacturer's design satisfactory to the engineer, weighing at least 375 lbs. The lid should usually be perforated with 1-inch holes, to permit ventilation of the sewer; and immediately below it, there should be hung a heavy cast-iron *dustpan*, to catch any dirt entering through the perforations.

There should be a ladder of iron rungs built into the walls, as shown in Fig. 9.

The channels in the concrete bottom should be very carefully formed to give smooth, true, circular channels. They are sometimes lined with split sewer pipe. The benches at the sides of the channels should slope down towards the channels, as shown in the figure.

The concrete for the bottom may be made of 1 part Portland cement, 3 parts sand, and 5 parts of broken stone. All the brickwork should be laid with tight *shove joints*, in 1-to-3 Portland cement mortar; and the manhole walls should be plastered both inside and outside with 1-to-2 Portland cement mortar.

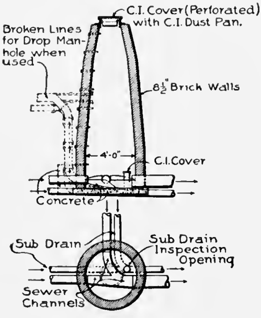


Fig. 9. Sectional Elevation and Plan of Sewer Manhole.

Should sudden drops in the sewer be desirable, they can be made at *drop manholes,* in the manner shown by the broken lines of Fig. 9.

In the case of large masonry sewers, which often are many feet in diameter, the manholes may be joined directly to the masonry of the upper part of the sewer.

Opinions of sanitary engineers differ somewhat as to the distance apart at which manholes should be placed. In general, a manhole should be placed at all junctions of sewers, and at every change of grade or alignment in all sewers but those large enough to be entered readily for cleaning. This means that sewers should ordinarily be perfectly straight between manholes, to facilitate inspection and repairs, all changes in both grade and alignment being made at the manholes themselves.

Also, in any part of the system—such as in the business district—where it is especially objectionable to have the street dug up for repairs, manholes should be placed at least as often as every city block—that is, 300 to 400 feet apart. In the other parts of the system, some engineers leave out every other manhole where the grade and alignment are straight, putting manholes at least every two blocks. The intermediate manholes left out are replaced by *lampholes* (Art. 22) to save cost. In Figs. 4 and 38, the above arrangement of manholes is shown in two actual sewer systems.

22. Lampholes. The lampholes which, to save cost, are sometimes adopted in place of part of the manholes, consist each of a vertical line of sewer pipe, with cemented joints, reaching to the street surface, as in Fig. 10. Usually 8 inches is the minimum diameter for this pipe, which is cemented at the bottom into a regular sewer-pipe T-junction. Some concrete should be placed under and around this tee for a foundation. At the street surface, there should be an iron casting similar to a manhole casting, but smaller, as shown in Fig. 10.

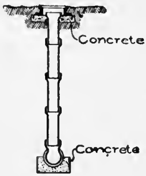


Fig. 10. Lamphole.

The earth, in refilling, needs to be very thoroughly tamped around the lamphole; and the lamphole casting should not be set until the material is thoroughly settled.

The object of the lamphole is to permit of inspection of the sewer, in determining whether it is clean and in locating stoppages. While its name suggests the lowering into it of a lamp, a beam of sunlight reflected into it from a mirror is more convenient.

A lamphole usually costs about \$30 to \$35 less than a manhole.

In Figs. 4 and 38 the above arrangement of lampholes in two actual sewer systems may be seen.

23. Flush-Tanks. Near the upper ends of sewers the flow of sewage is very small, sufficient only to make a shallow, trickling stream, liable not to be able to carry along the solid matter in the sewage so as to prevent deposits. An 8-inch lateral sewer in a residence district in a small town, even if laid at the minimum grade, would usually have an average depth of flow in the upper two and one-half blocks of less than one inch. Hence it is desirable, though not always absolutely necessary, to provide some special means for regularly flushing the upper portions of sewer laterals, to make them self-cleansing.

Again, in low-lying, level districts, it may be necessary, on account of the lack of fall, to lay the sewers at such slight grades that the velocity is insufficient to prevent deposits. Here, too, some special means should be provided for regularly flushing the sewers.

In the case of pipe sewers, such as are ordinarily used for the laterals in all systems, and for most of the mains in separate systems,

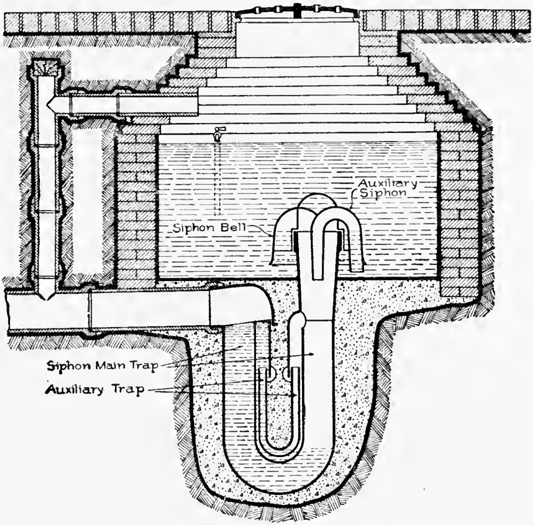


Fig. 11. Sewer Flush-Tank with "De La Hunt" Adjustable Siphon.

the most efficient and reliable means for securing regular flushing is the use of automatic flush-tanks.

A *flush-tank* is a masonry cistern built in the street, above the grade of the sewer, filled by a constantly running stream of water brought by a small pipe from the water-supply mains, and suddenly emptied by automatic devices into the sewer whenever the high-water line is reached.

Flush-tanks usually have a capacity of 150 to 500 gallons, and should approach the larger size named, to secure an efficient flush for two or three blocks. When made separate from manholes, flush-tanks are usually circular and of the general design of the masonry tank shown in Fig. 11. It is usually better, however, to combine the flush-tank with a manhole, as is shown by the masonry tank and manhole in Fig. 12. This permits inspection of the flush-tank and sewer, and is cheaper than to build manhole and flush-tank separate.

The bottoms of flush-tanks are usually of Portland cement concrete, and the walls of brick laid in Portland cement mortar. The

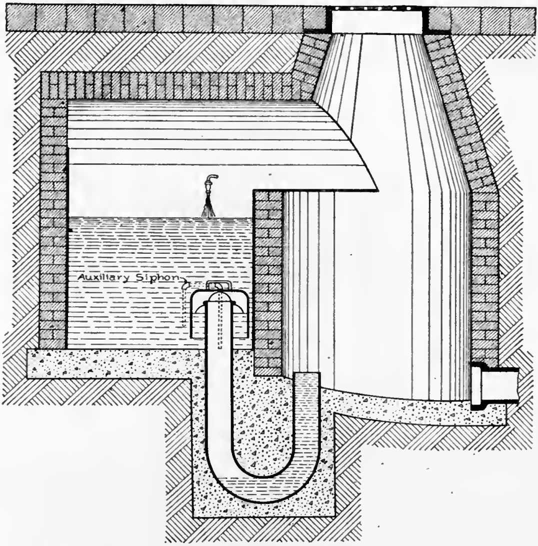


Fig. 12. Combined Flush-Tank and Manhole with Special "Miller" Siphon.

tanks should be plastered inside and outside as described for manholes (see Art. 21). Special care should be used to make flush-tanks absolutely water-tight.

The water is usually brought to the flush-tank by a $\frac{3}{4}$-inch galvanized pipe from the nearest water main. This pipe must be laid below the frost line ($5\frac{1}{2}$ to 7 feet deep, in the northern part of the United States), but should be turned up after it enters the flush-tank so as to discharge above the high-water line, as shown in Fig. 11.

The flush-tank may be prevented from freezing by being connected with the sewer above the high-water line, as shown in Figs. 11 and 12, so as to admit the warm air from the sewer.

It is a quite common practice to place flush-tanks at the heads of all laterals, as illustrated in Figs. 4 and 38. While some engineers dispute the necessity for this, it must be admitted that such an arrangement will be of great benefit, and its adoption is here advised for most cases.

In Fig. 38 the use of flush-tanks is shown at certain half-way points on the long laterals. The necessity for this arose from the fact that the sewers were not to be completed to the north ends of the laterals for some years after the southern portions were built.

The writer of this paper has used flush-tanks with success and great benefit, at intervals of about two or three blocks on sewers laid at grades below those considered necessary to make the sewers self-cleansing, though part of the flush from the intermediate tanks flows some distance upstream at each discharge.

The flush-tanks of a sewer system should be frequently inspected after the sewers are put into operation, and should be carefully kept in working order. The things needing most faithful watching are: first, the automatic discharging apparatus; and, second, the supply of water. The faucet admitting water may readily become choked up, putting the flush-tank out of service, or, on the other hand, may get wide open, wasting thousands of gallons of water every day.

24. Automatic Flushing Siphons. The reliability of flush-tanks in actual use will depend upon the frequency and care with which they are inspected and kept in working order, and especially on the reliability of the automatic discharging apparatus. No discharging apparatus having moving parts should be used in flush-tanks. Such apparatus is too likely to get out of order.

In Figs. 11 and 12, *sewer siphons* are shown for automatically discharging the flush-tanks suddenly whenever they fill to the high-water line. Such siphons have no moving parts whatever to get out of order, and should always be employed with flush-tanks.

In Fig. 11 the four ordinary parts of a flushing siphon are indicated. All four are usually iron castings, and must be air-tight. The *siphon bell* rests upon the *main trap*, which latter, together with the *auxiliary trap*, must be filled with water to the heights of the short legs, before the bell is placed in position. The main trap must be set plumb. The *auxiliary siphon* serves to ensure, at the end of the discharge, the *venting* of the siphon—that is, the free admission of air to the inside of the bell. With clear water, the auxiliary siphon is not always used; but it should be used whenever the siphon is to be used with raw sewage.

In the working of the siphon, the water in the flush-tank confines the air inside the bell and above the water in the main and auxiliary traps, and puts it under increasing pressure as the water rises. When the high-water line in the flush-tank is reached, this pressure becomes so great that the water in the auxiliary trap is forced down to the very bottom of the trap, and the confined air then blows out of the short leg of the auxiliary trap, thus releasing the air-pressure inside the bell, which up to this time has held back the water in the flush-tank. The water in the flush-tank then rushes out into the sewer through the main trap, and by siphonic action will continue to flow out until drawn down to the level of the bottom of the bell. Air then enters the bell through a small *sniff-hole* provided near the bottom of the bell for this purpose, *breaking* the siphonic action—that is, *venting* the siphon.

In case a siphon is used for raw sewage, there is often difficulty in securing satisfactory venting of the siphon at the close of the discharge; but this trouble can be remedied by using an *auxiliary siphon*, as shown in Fig. 11, and as illustrated by broken lines for the "Miller" siphon in Fig. 12.

In the *Miller siphon*, shown in Fig. 12, there is no auxiliary trap; but at high-water line the air-pressure in the main trap becomes so great that a bubble escapes, taking with it enough water from the short leg to start a sudden rush of water from the tank into the main trap, which suffices to establish siphonic action. This greatly simplifies the siphon; and the principle can be relied upon for siphons not larger than about eight inches internal diameter of the main trap. Larger siphons should have auxiliary traps.

In some siphons—as, for example, the *Rhoads-Miller*—the auxiliary trap is cast as a part of the main trap, out of which it opens below the floor of the tank, being entirely buried out of sight and reach in concrete. An objection to auxiliary traps such as shown in Fig. 11, is that they are inaccessible and may in time become stopped up. However, they make the action of large siphons more certain.

25. Hand-Flushing of Sewers. For large sewers, flush-tanks and siphons would have to be extremely large to be effective. Even in small sewers the effect of the flush will not be great for many blocks below the tank. Some engineers doubt the necessity for very extensive use of flush-tanks. When flush-tanks are not properly inspected and regulated (as to the feed faucet), they sometimes waste great quantities of city water. For these reasons, and sometimes to save cost, hand methods are sometimes relied upon for flushing sewers.

The most convenient, economical, and effective hand-flushing device is a connection with a water main by a water pipe of size large enough to flush the sewer very thoroughly. The only labor then required is that necessary for opening and closing the valves on this pipe. Such a flush, continuing much longer than the discharge of a flush-tank, can be made effective through a long stretch of sewer. The objections are the trouble and the danger of neglect inherent in hand work, and the usual greater length of time between flushings. To flush the sewers daily would be very expensive, both as to labor and as to the large amount of water needed.

Occasionally, very favorable local circumstances may permit of the admission at will of large volumes of water for flushing purposes from a stream or lake higher than the sewer.

In some cases, hand-flushing is done by temporarily damming up the sewage itself, and then suddenly releasing it when sufficient head has been secured.

A fire hose run to a manhole from a nearby hydrant may be the resort in other cases. In extreme cases, water has even been hauled to the sewer in tanks, for flushing.

26. Sewer Ventilation. More fear used to be felt of the danger of *sewer gas* (more properly termed *sewer air*, see Art. 1) in communicating disease, than medical knowledge warrants at the present time. Nevertheless, it is very important, not only from the sanitary but from many other points of view, that sewer air should be as pure as possible; and this requires good ventilation of the sewers. Fresh-air currents in the sewers should be maintained in some reliable way.

One method of securing this is to use perforated manhole covers (see Fig. 9). Objection is sometimes made to these as letting objectionable odors out into the street; but with well-designed and well-constructed sewers, well flushed and well ventilated, there will be no cause for complaint. If there are seriously objectionable odors from the manholes, such odors should be considered valuable as notices that the sewers are in dangerous condition, demanding immediate work to make them safe. Sewer air escaping into streets through manhole-cover perforations, is at once so diluted by fresh air as not to be dangerous to the health of passers by.

Another effective means for securing good ventilation is to extend the cast-iron soil-pipes (which form the main drainage pipes in the plumbing systems of houses) untrapped and full size through the roof. Figs. 4 and 35 show the omission of traps on the soil pipe. In Fig. 35, however, the use of a *disconnecting trap*, to disconnect the sewer air from that in the house plumbing pipes, is shown by broken lines. In case this is used, a ventilating pipe for the sewer should be extended up the sides of the house from the sewer side of the trap, and a fresh-air inlet provided on the house side, both as shown by the broken lines in Fig. 35.

The use of perforated manhole covers and untrapped soil pipes extending through the roofs, is all that is required to secure good ventilation of the sewers, the house connections, and the soil pipes themselves. Their use provides a large number of openings at different levels; and the temperature of the air in the sewers is practically always different from that above the ground. Hence air-currents are maintained for the same reason that chimneys cause draughts for fires, and a good circulation of air is maintained.

In the past, experiments in sewer ventilation have been made with tall chimneys, fan blowers, etc.; but such devices are entirely unnecessary, are very costly, and are usually unsuccessful on account of the very large number of openings into the sewer, which limit the air-currents produced by such devices to short distances.

27. Street Inlets and Catch-Basins. In the case of storm sewers and combined sewers, means must be provided for admitting the storm water to the sewers from the streets. For this purpose, either *street inlets*, as shown in Fig. 13, or *catch-basins*, as shown in Fig. 14, may be used. If the water can be allowed to flow one block safely in the surface gutters, the inlets for storm water would need to be only at each street intersection. In a few cases they need to be closer; but in many more cases the storm water can be carried in the gutters for two or even a greater number of blocks without injury, thus greatly reducing the number and cost of storm sewers and of inlets for storm water.

The simplest and least expensive arrangement for admitting storm water is the *street inlet*, which, as shown in Fig. 13, is a mere branch sewer, with a grated opening from the street. Besides costing less, the street inlet is often preferred for sanitary reasons, as it does not retain foul, unsanitary deposits, as does the catch-basin.

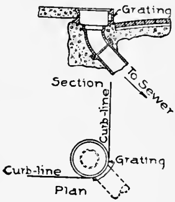


Fig. 13. Street Inlet.

The *catch-basin*, shown in Fig. 14, is designed to catch the sand, dirt, and other heavy street detritus, and prevent their entering the sewer. Unless catch-basins are frequently cleaned, however (which is very seldom the case), they fail almost entirely in this; and as they are usually well filled with more or less foul deposits, they are condemned by many engineers.

When street inlets and catch-basins are left untrapped, as shown in Figs. 13 and 14, they assist in the ventilation of the sewers. This is sometimes objected to on account of the opportunity for the escape of foul odors, and traps are introduced in both, as shown by the dotted lines in Fig. 14, to prevent ventilation of the sewers through the storm inlets. If the sewers are kept in as good condition as they should be, there will be no good ground for such objections.

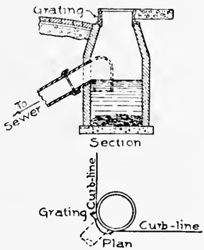


Fig. 14. Catch-Basin.

28. Inverted Siphons. It sometimes becomes necessary or desirable to carry a sewer down below the regular grade line, to pass under some obstacle or depression, and to raise it again to the regular grade line beyond. Such a stretch of sewer will necessarily flow full and be under some pressure. It is called an *inverted siphon*. The necessity for the use of the inverted siphon may be occasioned by some stream, by railway tracks, by another sewer, by a large water main, or sometimes merely by a low stretch of ground which happens to lie at such a level that the sewer cannot be carried across it at the regular grade.

Inverted siphons have often been constructed and operated successfully. It is wise, however, to take certain precautions in their design and construction, as otherwise serious trouble may be experienced with them.

First, as to material, it may be said that ordinary sewer pipe is not well suited to carry sewage under pressure, on account of the great difficulty in making absolutely tight joints, and on account of the brittle and unreliable nature of the pipe as to resistance to bursting pressures. If used under pressure, pipe sewers should be subjected to only a few feet of head, and all joints should be thoroughly encased in impervious Portland cement mortar and concrete, reinforced with imbedded steel bands. Brick masonry is still less suited to with-

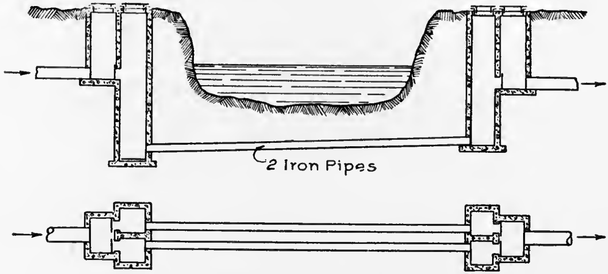


Fig. 15. Sectional Elevation and Plan of Inverted Siphon.

stand bursting pressures. Ordinarily iron pipe should be used for inverted siphons.

Second, it is especially important to insure a current in the inverted siphon sufficiently rapid to prevent deposits. If the flow is light at first, to increase afterwards, as is often the case, it is well to divide the siphon into two or more pipes with valves on each, so that the entire flow can be turned into one at first. If it is easy to add the second pipe in the future, it may often be left out at first. Thus in Fig. 38, the inverted siphon from the 18-inch outlet sewer to the septic tank is at present only an 8-inch cast-iron pipe, with provision for adding a 12-inch cast-iron pipe later.

Third, the design should be such as to permit ready access for inspection and removal of obstructions. The inverted siphon should, if possible, be so planned that the flow of sewage can be diverted for a short time, either into one pipe, or entirely away from the siphon; and the siphon should drain to a low point from which the contents can be removed by gravity through a blow-off or by being pumped out. Where feasible, and especially where it will be very difficult (as under a stream) to dig down to the siphon in emergencies, the siphon should be made absolutely straight in grade and alignment, and a manhole placed at each end.

In Fig. 15 is shown an outline of an inverted siphon designed according to the above principles.

Where the siphon can readily be opened for repairs, as is the case with the one in Fig. 38, such expensive construction need not be resorted to. The one in Fig. 38, which carries sewage across low ground to a sewage tank about seven feet above the surface, is laid at an average depth of about six feet, and neither the grade nor the alignment is straight. It drains, however, to a low point, where a blow-off into a sewer is placed.

29. Outlets for Sewer Systems. We have heretofore discussed the house connection, and the laterals, submains, and main sewers, with their manholes, flush-tanks, and other accessories. We come next to the *outlet*, which, though not considered first here, would be one of the first things a sewerage engineer would have to consider in designing a sewer system.

Where possible, all of the sanitary sewage or combined sewage of the city should be led to one outlet, as the cost of disposing of it properly may be lightened thereby, and as the danger of injunction suits and other legal difficulties arising from damages from impurified or only partially purified sewage may be multiplied with the number of outlets. Often this will be possible by constructing comparatively short lengths of deep sewers where at first sight the topography would seem to make it impossible to secure one outlet. The size of the city, as well as the topography, will affect the number of outlets.

Storm sewage in the separate system can usually be discharged through a number of outlets into nearby natural watercourses.

Great effort should be made to secure an outlet or outlets for the sewer system low enough to drain all parts of the city by gravity. Pumping of the sewage or a material part of it, will mean a continuous expense involving an amount which would be sufficient to pay the interest on a large initial expense to secure a gravity outlet. Besides, there is the danger of such apparatus failing at critical times.

Usually effort is made to secure, if possible, an outlet into a considerable stream or body of water, even if the sewage is to be purified.

30. Sewage Disposal. Heretofore, sewage has been disposed of, in the great majority of cases, by simply emptying it into the largest available stream or body of water near at hand. Such serious contamination of natural waters has resulted from this practice, that at the present time much more attention than formerly is being paid to sewage purification; and usually the outlet plans should be made with the expectation that some method of purification will have to be adopted in the future, if not at present.

Sewage disposal is discussed further on, at much greater length (see Arts. 110 to 124). It will only be said here that the methods at present in favor almost all involve passing the sewage through large tanks, and then through some form of filter.

SEWER MATERIALS AND CROSS-SECTIONS

31. Sewer Materials. Sewers 24 inches in diameter and under, are usually built of *vitrified sewer-pipe.* A 24-inch pipe sewer, laid to a fall of 0.2 feet in 100 feet, will carry the sanitary sewage, under average conditions, of 29,000 people; and hence it is evident that in separate systems, all the sanitary sewers will be made of pipe, except a few main and outlet sewers in large cities. Considerable percentages of storm sewer and combined sewer systems will be pipe sewers also.

Occasionally *cement sewer-pipe* is used instead of the vitrified pipe.

Sewers 30 inches and larger in diameter, are most frequently built of *brick.* Pipe is sometimes used, however, for 30-inch to 36-inch sewers.

Concrete has of late years been growing in favor, to take the place of brick in sewer construction.

Stone was formerly used to a considerable extent for sewers; but on account of its roughness, and the great cost of cut-stone masonry, stone is suited only for backing brick linings in larger sewers. Even here, concrete would now ordinarily be employed, as both cheaper and better.

Occasionally, as in the case of submerged-outlet sewers into bodies of water, or sewers across marshes on soft foundations, *wooden stave pipe* is used for sewers. These pipes are made of pieces of timber, usually about two inches by four inches in size, put together breaking joints in the field, and hooped at regular intervals with iron bands which can be screwed tight. Wood should be used only where it will be wet all the time, to prevent rotting.

Cast-iron pipe, such as is used for water mains, is often adopted for short stretches of sewer under railways or streams where great strength is essential; for inverted siphons; and in cases where absolutely water-tight joints are essential, such as submerged lines in lakes, harbors, and stream crossings, or where there is much ground water.

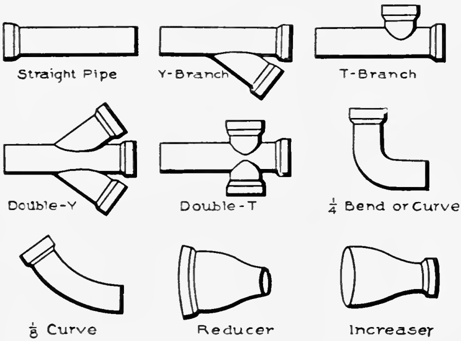


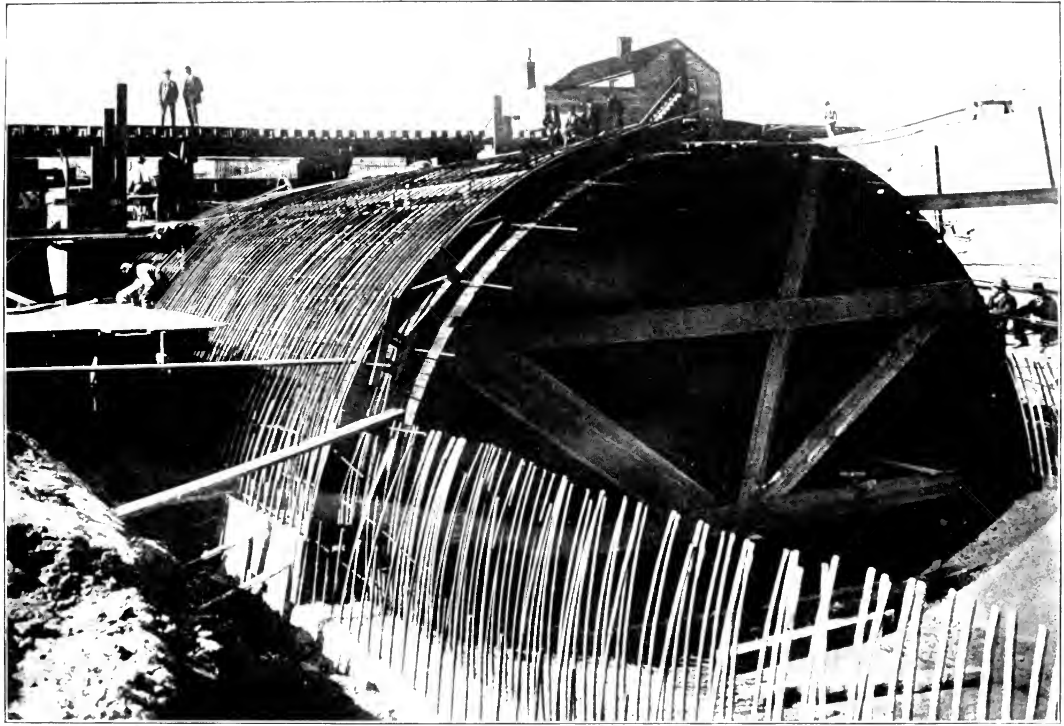
Fig 16. Vitrified Sewer-Pipe and Specials.

32. Vitrified Sewer-Pipe. Vitrified sewer-pipe has many excellent qualities for sewer use. It is hard, impervious, smooth, strong, does not decay or disintegrate, and is not affected by chemicals. It has few joints as compared with brickwork, and these joints are of convenient shape to make practically water-tight. Vitrified sewer-pipe is readily handled and laid in sewer construction. The materials of which it is made are widely distributed, and hence the cost of the pipe is reasonable.

In Fig. 16 are shown the general forms of the straight pipe and also of the special fittings (*sewer-pipe specials*) most commonly used in sewer construction.

In Table I (page 35) are given standard dimensions for straight sewer-pipe.

Vitrified sewer-pipe is made from shale clays, in very much the same way as brick and other clay products. The temperature at



HARLEM CREEK PUBLIC SEWER, ST. LOUIS, MISSOURI which it is burned in the kilns must be very high, as in the case of paving brick, so as to produce an "incipient vitrification," a softening and running together of the particles of clay, which gives, on cooling, a very hard, impervious, and strong structure. Smoothness of interior and exterior surfaces is secured by the use of salt during the process of burning, so as to produce a "salt-glazed," glassy skin.

TABLE I

Standard Dimensions for Sewer Pipe

STANDARD				DOUBLE STRENGTH OR EXTRA THICK			
Inside Diam. Inches	Thickness of Shell. Inches	Depth of Socket. Inches	Weight per Ft. Lbs.	Inside Diam. Inches	Thickness of Shell. Inches	Depth of Socket. Inches	Weight per Ft. Lbs.
8	$\frac{3}{4}$	$2\frac{1}{2}$	22	8	$\frac{7}{8}$	$2\frac{1}{2}$	25
9	$\frac{3}{4}$	$2\frac{1}{2}$	27	9	$\frac{7}{8}$	$2\frac{1}{2}$	30
10	$\frac{7}{8}$	$2\frac{1}{2}$	30	10	1	$2\frac{1}{2}$	34
12	1	$2\frac{1}{2}$	41	12	$1\frac{1}{8}$	3	50
15	$1\frac{1}{8}$	3	60	15	$1\frac{1}{4}$	3	70
18	$1\frac{1}{4}$	3	80	18	$1\frac{1}{2}$	3	100
20	$1\frac{3}{8}$	3	95	20	$1\frac{2}{3}$	3	120
21	$1\frac{3}{8}$	4	105	21	$1\frac{3}{4}$	4	140
24	$1\frac{5}{8}$	4	135	24	2	4	180
27	2	4	215	27	$2\frac{1}{4}$	4	240
30	$2\frac{1}{4}$	4	270	30	$2\frac{1}{2}$	4	300
33	$2\frac{3}{8}$	$4\frac{1}{2}$	320	33	$2\frac{5}{8}$	$4\frac{1}{2}$	340
36	$2\frac{1}{2}$	5	365	36	$2\frac{3}{4}$	5	390

The bells are made large enough to allow an annular space for cement, ranging from $\frac{3}{8}$ inch thick for 8-inch pipe to $\frac{3}{4}$ inch for 36-inch pipe.

Smaller sizes of pipe, down to 3 inches in diameter, are made.

Double-strength pipe is used only in cases requiring unusual strength.

Vitrified sewer-pipe must be carefully inspected, piece by piece, just before being used in the sewer, all poor material being rejected. Some of the points to be noted in making the inspection are as follows:

(1) The pipe should be straight, and true in shape.

(2) The pipe must have a hard-burned, strong internal structure showing incipient vitrification. Small pieces may be chipped out of occasional lengths to test this; and the color will also be a guide after the inspector has become thoroughly familiar with the make of pipe being used.

(3) The hub and socket ends of adjacent pipes should fit together well, leaving at least the spaces for cement given under Table I.

(4) There must not be on the lower half of the interior of the sewer any lumps, blisters, or excrescences. A few may be allowed,

if not too large, if the pipe can be turned so as to bring them to the upper half.

(5) There must be no cracks extending into the body of the pipe, or of such nature as to weaken it materially. On tapping the pipe with a light hammer, if it does not give a clear ring, the presence of invisible cracks may be suspected.

(6) There must be no broken pieces of material size, from either the hub or the socket ends, nor any at all which cannot be turned to the upper half.

Nothing of human construction can be perfect, and sewer pipes are no exception to the rule. Hence the pipe inspector must have good judgment and considerable experience to draw the line properly between important and unimportant defects. In clause 25, Art. 93, of the sewer specifications given hereinafter, some definite rules are laid down to govern inspectors in this particular.

Vitrified pipe can be secured in 2, $2\frac{1}{2}$, and 3-foot lengths. The longer the lengths, the fewer the joints, which is a material advantage.

33. Joints in Pipe Sewers. The joints are the weakest points in pipe sewers, and should be made with the utmost pains to secure as nearly as practicable an absolutely water-tight job. In Fig. 17, the upper joint shown illustrates the form commonly employed.

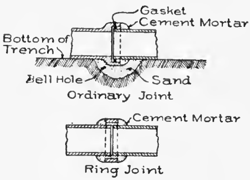


Fig. 17. Joints in Pipe Sewers.

In the bottom of the trench, which should be rounded to fit the under part of the sewer pipe, *bell-holes* are dug for all bells, to permit the joint on the under side of the pipe to be made properly, and to give the pipe a bearing on its full length instead of merely on the bells. Before the spigot end of the pipe to be laid is *entered* into the bell of the last pipe laid, it should be wrapped with a *gasket* of hemp, oakum, or jute, as shown in Fig. 17, so that the inverts of the two pipes will match in a smooth line when the pipe is entered, and so as to prevent the soft cement mortar from being forced up through the joint to project into the pipe. The gasket also assists in making the joint water-tight, especially if there is water in the trench. Disastrous results have often followed the omission of the gasket, which should always be used.

After the pipe is entered and brought exactly to grade, Portland cement mortar, mixed about 1 to 1 or 1 to 2 with sand, should be *calked* into the joint, *to fill it absolutely full,* and should be beveled off on the outside, as shown in the figure. Special care should be taken on the under side of the pipe. *Immediately* after placing the cement, the bell-hole should be packed *full* of sand, so as to support the cement on the under side of the pipe till it has set. It is best to keep the cementing back two or three lengths of pipe from the pipe laying, to avoid danger of the cement being broken in placing the next pipe.

Without the most careful watching of every joint during construction, the workmen are sure to slight the joints. An inspector should be kept constantly on the work.

In the lower part of Fig. 17 is shown the *ring joint,* formerly preferred by some engineers, but now very seldom used. It is more costly than the ordinary form.

Various joints have been invented and used to a limited extent, which include simple beveling of the ends of the pipe without using bells, the use of grooves at one end with corresponding projections at the other end, etc. Sometimes the exterior of the spigot end and the interior of the bells are grooved and made rough in the ordinary form of joint. This is an advantage in holding the cement, and in securing a water-tight job.

34. Cement Sewer-Pipe. Ever since the early use of pipe sewers in the latter half of the nineteenth century, cement pipe has been used to some extent for sewers; and recently there seems to be a revival and extension of its use. Experience has shown that cement is a very suitable material for making sewer pipe, and that cement pipes, when well made, of first-class materials, give excellent satisfaction for sewers, and are durable and not disintegrated by the sewage.

The manufacture of good cement sewer-pipe, however, cannot be successfully carried on by men who do not have the necessary skill, which is to be gained only by experience in this particular work; and even skilled manufacturers will not be successful unless both the cement and the sand used are of first-class quality, nor unless plenty of cement is used. Much poor cement pipe has been made, because these almost self-evident facts have not been understood; and in this way cement sewer-pipe has gained a bad reputation in many localities.

In general it may be said that the sand should be clean, sharp, and coarse, and that it should contain a considerable proportion of fine pebbles, smaller than a cherry-pit. Only the best Portland cement should be used, and the mortar should not be weaker than 1 to 3. The mixing must be very thorough, as also the tamping into the moulds.

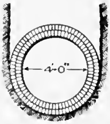


Fig. 18. Circular Brick Sewer, Ingersoll Run, Des Moines, Iowa.

Two general kinds of cement sewer-pipe are made. In one, just coming into use, the pipes are made continuously in the ditch. A form of moulds is used to give the correct shape and size, which can be forced ahead as the work progresses; and there are no joints. It is too soon yet to tell how successful this plan may be.

In the more common form of cement sewer-pipe, the pipes are made in a factory, in pieces of the same length as vitrified pipe. Usually, comparatively little water is used in mixing, in order to permit immediate removal of the pipe from the moulds. While such pipe are *curing* (setting), the omitted water must be supplied by frequently wetting them, or the process of setting and hardening cannot go on properly. Many cement sewer-pipes of this kind are spoiled in the curing.

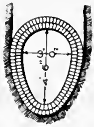


Fig. 19. Egg-Shaped Brick Combined Sewer.

Cement pipe are now made with bells for the joints, the same as vitrified pipe. The manufacture of specials, such as the Y-junctions required in such numbers for house connections, is still in unsatisfactory condition.

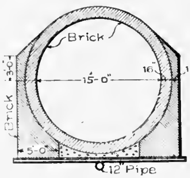


Fig. 20. Circular Brick Sewer with Sub-drain, 64th Street Brooklyn, N. Y.

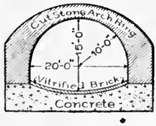


Fig. 21. Section of a Large Sewer in St. Louis, Mo.

The body of a cement sewer-pipe is of much weaker material than that of which vitrified pipe are made; and the thickness of cement pipe should be much greater than the thickness given in Table I for vitrified pipe.

35. Typical Cross-Sections of Large Sewers. In Figs. 18 to 25, inclusive, are shown some typical designs for sewers too large to be constructed of sewer pipe.

In Fig. 18, the common circular form is shown. This form is more economical to construct than any other when good foundations

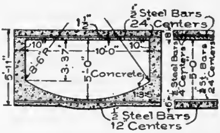


Fig. 22. Ingersoll Run Sewer with Low Headroom, Des Moines, Iowa.

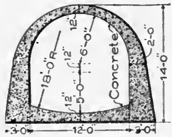


Fig. 23 Dry-Run Sewer, Waterloo, Iowa.

can be had, for the circle gives a larger area and velocity of flow when full than any other shape having the same circumference.

In the case of combined sewers, however, the dry-weather flow of sewage is so very small, in comparison with the size of the sewer, that it makes only a shallow, trickling stream of little velocity, and the sewer will not be self-cleansing. For such sewers, this difficulty can be overcome by the use of the egg-shape of sewer, shown in Fig. 19. This shape has a circular invert having a radius only half that of the top; and the depth and velocity of the dry-weather flow will be the same as in a circular sewer of this smaller radius, while at the same time the capacity in time of flood is equivalent to a much larger circle.

In Fig. 20, a favorite type of design for very large circular sewers

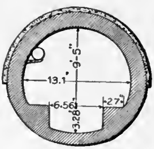


Fig. 24. Old Type of Main Sewers, Paris, France.

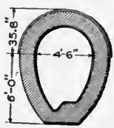


Fig. 25. New Type of Sewers, Paris, France.

is shown. For such large sewers, the upper half constitutes an arch, which exerts heavy pressures or thrusts horizontally outward against the sides of the sewer at the height of the center. To withstand these thrusts, the masses of masonry backing shown in the figure are added. This backing may be of brick, rubble-stone, or concrete masonry.

In the large sewers, too, it usually is not practicable to round the bottom of the trench to fit the circular shape, as is done for smaller sewers; and hence the flat foundation, also shown in the figure, is adopted. In soft materials, it often becomes necessary to drive piles to carry the weight of sewers.

In Fig. 21 is shown the favorite design for large sewers. For reasons given in discussing Fig. 20, the foundation is necessarily made flat; and with this shape of foundation, Fig. 21 will give a larger area and capacity for the same amount of material than Fig. 20, other conditions being the same. Also, Fig. 21 requires less headroom than Fig. 20 for the same capacity—which is often of great importance in the case of these large sewers. The invert of Fig. 21 is not so well suited to prevent deposits as that of Fig. 20; but in the case of these large sewers, there is usually a large flow even in dry weather, so that this point may be of little importance.

In Fig. 23 we have an example of the use of concrete for a large sewer of the general type shown in Fig. 21, and just discussed.

In Fig. 22 we have an extreme case of low headroom, secured by making the top an absolutely flat slab of concrete, reinforced with steel. In this case the bottom of the sewer was necessarily located at a very shallow depth below the street, while the required size of sewer was large.

Finally, in Figs. 24 and 25, are shown two typical cross-sections of the famous sewers of Paris. The large main shown in Fig. 24 acts not only as a sewer, but also as a subway for the water mains and for other purposes. The entire ordinary flow of sewage is confined within the *cunette,* or comparatively small channel shown in the bottom. The ledge on each side serves for the passage of workmen and of cleaning carts, flushing devices, etc. The section shown in Fig. 25 is a later type, and is more nearly self-cleansing. The dirt in the streets is washed into these sewers by the use of hose, and special conveniences for cleaning it out of the sewers are needed.

36. Junction=Chambers for Large Sewers. Where two or more large sewers join, special difficulties present themselves, in providing supports for the partial arches whose supports are cut away in making the junction. It is usually necessary, when the sewers are large, to build a masonry chamber enclosing the entire junction, and with a self-supporting roof spanning all the sewers.

Various designs for such junction-chambers are used, but the most common type is illustrated in Fig. 26. Here a *bell-mouth arch* is used to span the opening, the case being the junction of three of the Chicago intercepting sewers (see Fig. 5). Sometimes *flat roofs* are used, supported by steel beams or made of reinforced concrete.

The bottoms of such junctions are the *mathematical intersections,* executed in masonry, of the lower halves of the sewer channels; and for sewers not too large, the upper halves may sometimes be built in a similar way, or with *vault ribs,* as in the roofs of old cathedrals.

37. Brick Sewers. It has already been stated that brick is the favorite material for sewers too large to be made of pipe, the dividing line usually being drawn at 30 inches to 36 inches diameter. Brick present many advantages for sewer work, including their moderate cost, their durability, and their small size and regular shape, which enable them to be readily handled and used in building sewers of any desired cross-section, with comparatively smooth and true interior surfaces.

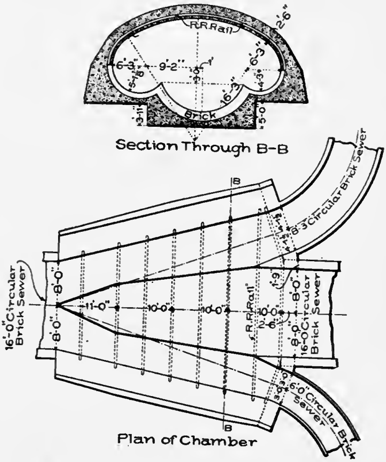


Fig. 26. Junction of Brick Sewers, Lawrence and Sheridan Avenues, Chicago, Ill.

Sewer brick, as those suitable for sewer construction are commonly called, should be harder burned than ordinary building brick, to enable them to stand the wear from the flow of sewage, and to insure against disintegration. They need not, however, be as hard burned as No. 1 paving brick, and hence constitute an intermediate grade between building brick and pavers. Sewer brick should be uniform in size, and of regular, true shape, so as to permit of being laid with thin joints, to form smooth, true surfaces. They should be carefully inspected on the work just before being used, and all defective brick thrown out. The common size for sewer brick approximates $8\frac{1}{2}$ by 4 by $2\frac{1}{4}$ inches.

In the sewer, the brick are laid in rings, as shown in Figs. 18 and 19, with the 4-inch dimension radial and the $8\frac{1}{2}$-inch dimension lengthwise of the sewer. Care should be taken to *break joints* in each ring. The brick should be laid in Portland cement mortar, made of at least 1 part of cement to 3 parts of clean, sharp sand of medium-sized grains. Pebbles should be screened out of the sand so as to permit thin joints. All joints should be filled *full* of mortar, the brick being laid with *shove joints,* to make a practically water-tight job. The outside ring of the invert should be laid against a layer of 1 to 2 Portland cement mortar; and the outside of the arch (or upper half of the sewer) should be plastered with the same mortar, to keep out ground water. Similarly, to prevent leakage of sewage, the entire interior surface of the sewer should be plastered with the same mortar, or else thoroughly washed with at least two coats of liquid cement, after the joints have been carefully pointed and smoothed. Even with the utmost care, it will be found impossible to secure absolute water-tightness; and the difficulties will be especially great when ground water and soft materials are encountered in the trench.

Up to 6 or 7 feet diameter, two rings of brick are usually sufficient. In fact, for the smaller sizes of brick sewers, one ring would be amply strong with firm foundations; but it is difficult to make the sewer sufficiently tight when only one ring is used, because all joints extend entirely through. Sometimes an exterior layer of concrete may be used to meet this objection, at least for the lower half of the sewer; or an outside ring of brick may be used for the invert only. Sewers larger than 6 or 7 feet in diameter usually require three rings of brick; and more are needed for very large sewers, for which the number required must be calculated for each particular case to suit the special conditions.

38. Concrete Sewers. Of late years, concrete has frequently been employed in preference to other kinds of masonry for many purposes, of which sewer construction is one. Its advantages for sewers are many. The following may be mentioned:

First, and foremost, the cost is usually less than the cost of brick masonry.

Second, the concrete exactly fits the irregularities of the excavation, giving better foundations.

Third, sewers built of concrete constitute a solid structure without joints, and hence are less liable to uneven settlement.

Fourth, there are no joints, as in brickwork, to be made watertight, though, on the other hand, it is not easy to make the body of the concrete entirely impervious to seepage.

Fifth, the concrete can be readily moulded to any desired shape of sewer.

Sixth, the concrete can be made by comparatively unskilled workmen, if skilled foremen are employed.

Concrete may be used for foundations, as shown in Figs. 20 and 21; for the backing of brick sewer rings; and in various other combinations with brick; or it may be used for the entire sewer, as in Figs. 22 and 23.

Reinforced concrete, or concrete reinforced with steel rods, to prevent cracks from tension stresses, has opened up of late years entirely new possibilities in sewer construction, of which Fig. 22 is an example.

It has been reported that the concrete invert of the large St. Louis sewer shown in Fig. 21 has shown surface pitting and disintegration from the effects of the sewage. This is a trouble which does not appear to have been experienced elsewhere, and hence is presumably uncommon, and would seem due most probably to poor materials or poor workmanship. Danger from this source could be prevented by lining the concrete sewer with one ring of vitrified paving brick.

FORMULÆ AND DIAGRAMS FOR COMPUTING FLOW IN SEWERS

39. Formulæ for Computing Flow in Sewers. It has already been stated that more than 99.8 per cent of even sanitary sewage is simply ordinary water which has been added to the foul wastes to assist in removing them. Hence the mathematical formulæ for the flow of sewage are the same as those for the flow of water. They may be studied in detail in the instruction paper on Hydraulics.

Two general hydraulic formulæ have commonly been employed in sewer computations, as follows:

(1) *Weisbach's Formula.* The older computations were generally based on Weisbach's formula, which is as follows:

$$v = \frac{\sqrt{2gh}}{\sqrt{1 + e + c\frac{l}{d}}}.$$

In the above formula,

v = Average velocity of flow, in feet per second.
g = Acceleration due to gravity = 32.2 ft. per second.
h = Fall of sewer, in feet.
e = Coefficient of entrance = 0.505.
c = Coefficient of friction in pipe = $0.0144 + \frac{0.0169}{\sqrt{v}}$.
l = Length of pipe, in feet.
d = Diameter of pipe, in feet.

Weisbach's formula has been much used for sewer computations, for the reason that Mr. Baldwin Latham, in the first treatise on Sanitary Engineering worthy the name (1873), published extensive tables of flow, calculated from this formula, which made sewer computations very simple. Hence it was easier for later engineers simply to make use of these tables than to compute new ones of their own.

(2) *Kutter's Formula.* In later hydraulic computations, it has generally been considered that Kutter's formula gives the most reliable results. It is as follows:

$$v = c\sqrt{RS} = \left\{ \frac{41.66 + \frac{1.811}{n} + \frac{.00281}{s}}{1 + \left(41.66 + \frac{.00281}{s}\right)\frac{n}{\sqrt{R}}} \right\} \sqrt{RS}.$$

In this formula,

v = Average velocity of flow, in feet per second.

R = Mean hydraulic radius in feet = Area of cross-section of stream in square feet, *divided by* wetted perimeter, in feet, of length of portion of circumference of channel wet by the stream. (NOTE.—For circular pipe sewers, $R = \frac{1}{4}$ of the diameter when the pipe is flowing either full or half-full.)

S = Slope of the sewer = $\frac{\text{Fall}}{\text{Length}}$.

n = Coefficient of roughness, varying with the roughness of the channel.

For pipe sewers it is common to assume that $n = 0.013$; and for brick sewers, that $n = 0.015$. For cement pipe sewers, the roughness might be considered intermediate between these values of n; but $n = 0.013$ is generally used for them as well as for clay pipe. New and perfectly clean channels would not be so rough as indicated by these numbers; but the growths and deposits which may accumulate in sewers render it wise to adopt the above values for n.

Both the above sewer formulæ give merely the average velocities (v) of flow. *To obtain the discharge in cubic feet per second, we must multiply "v" by the area in square feet of the cross-section of the stream of sewage.*

Kutter's formula gives less capacities for pipe sewers than Weisbach's for the small sizes, up to about 18 inches' diameter. It will be on the safe side to adopt Kutter's formula; and this is now very generally done, though actual gaugings of small pipe sewers either new or in very good condition, may often show greater velocities and capacities than the formula would indicate, when the values of n above given are adopted.

In this paper, Kutter's formula will be adopted as the basis of all calculations of the flow of sewers.

40. Diagram of Discharges and Velocities of Circular Pipe Sewers Flowing Full. Direct numerical computations of flow in sewers from the formulæ given above, would be very laborious and tedious. The work may be very greatly simplified by the use of tables or diagrams. Diagrams are more convenient than tables, and are adopted for this paper. With their aid, computations of flow in sewers are very easy and short.

Fig. 27 is such a diagram, giving the capacities and velocities of circular *vitrified* pipe sewers flowing full. *Cement* pipe sewers would probably have discharges and velocities somewhat less than those shown in this figure.

TO USE THE DIAGRAM

(*A*) *When the diameter of the pipe and the grade are given, to find the discharge and the velocity.*

(1) Look along the bottom horizontal line till the grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the point where the vertical line through the given grade intersects the inclined line marked with the given diameter of sewer. (3) Trace horizontally through this point, interpolating by the eye, if necessary, between the horizontal lines on the diagram; and read the discharge of the pipe running full, on the left side of the diagram in cubic feet per second, or on the right side of the diagram in gallons per 24 hours. (4) If the velocity is desired, it can be determined by noting where the point (found in 2, above) of intersection of the given grade and diameter lines falls with reference to the inclined lines marked with the different velocities, estimating by the eye the decimals of a foot per second.

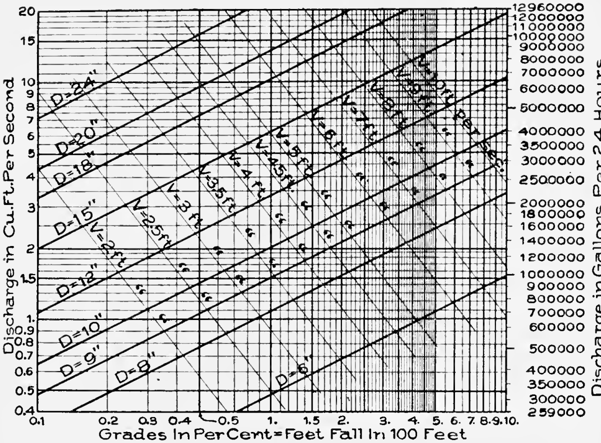


Fig. 27. Discharges and Velocities of Circular Vitrified Pipe Sewers Flowing Full. By Kutter's Formula ($n=0.013$).

(*B*) *When the grade and the required discharge are given, to find the necessary diameter of pipe, and the velocity.*

(1) Look along the bottom horizontal line till the given grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the intersection of the vertical line through this grade with the horizontal line through the given discharge, finding the discharge on the left of the diagram if it is given in cubic feet per second, or on the right if it is given in gallons per 24 hours. (3) Note between which two diameter lines this point of intersection falls, and take the diameter line nearest as that required. (4) Also note the position of the point of intersection with reference to the velocity lines, and so estimate the velocity, interpolating by the eye between the inclined velocity lines.

(*C*) *When the velocity and diameter are given, to find the grade and discharge.*

(1) Find the intersection of the given diameter line with the given velocity line, interpolating by the eye, if necessary. (2) Then vertically downward to the bottom of the diagram from this point of intersection, read the required grade; and horizontally to the left side or to the right side of the diagram, read the discharge, interpolating by the eye in each case, if necessary.

All other cases may be solved by similar obvious methods.

EXAMPLES

Example 1. What will be the discharge and velocity of a 15-inch pipe sewer laid to a 0.2 per cent grade?

Solution. See *A*, above. From the intersection of the vertical 0.2 per cent grade line with the inclined 15-inch diameter line, we read horizontally to the left the discharge of 2.8 cu. ft. per second, or to the right, of 1,850,000 gallons per 24 hours. We further note that the point of intersection of the 0.2 per cent grade line with the 15-inch diameter line falls between the 2.0 and the 2.5 ft. per second velocity lines, and by the eye we estimate the velocity to be 2.3 ft. per second.

Example 2. See *B*, above. What size of pipe sewer laid at a grade of 0.5 per cent will be required to carry an average flow of 200,000 gallons of sewage per day, the maximum rate of discharge being three times the average? (NOTE.—Hence use 600,000 gallons discharge in solving the example.) Also, what will be the velocity?

Answer. Required diameter of sewer, 9 inches; velocity of flow, about 2.3 ft. per second.

Example 3. See *C*, above. If the minimum allowable velocity of flow is 2 ft. per second when a sewer flows full, what minimum grade will be required to produce this velocity in a 12-inch sewer?

Answer. 0.23 per cent minimum grade.

Example 4. If an outlet sewer serves 20,000 people, each person contributes 100 gallons per day, and the maximum rate of flow is 3 times the average, what size of sewer will be required, if its grade is 0.25 per cent?

Answer. 24 inches diameter.

Example 5. If an 8-inch pipe sewer is laid at a 0.45 per cent grade, what will be the discharge and the velocity when it flows full?

Answer. 480,000 gallons per day; 2.1 ft. per second.

Example 6. A storm pipe sewer drains 10 acres, and should be able to carry 1.5 cu. ft. per second per acre. Its grade is 0.5 per cent. What diameter will be required?

Answer. 24 inches diameter.

41. Diagram of Discharges and Velocities of Circular Brick and Concrete Sewers Flowing Full. Fig. 28 is the diagram for circular brick and concrete sewers, corresponding to Fig. 27 for pipe sewers, and is used in the same way.

TO USE THE DIAGRAM

(*A*) *When the diameter of the pipe and the grade are given, to find the discharge and the velocity.*

(1) Look along the bottom horizontal line till the grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the point where the vertical line through the given grade intersects the inclined line marked with the given diameter of sewer. (3) Trace hori-

zontally through this point, interpolating by the eye, if necessary, between the horizontal lines on the diagram; and read the discharge of the pipe runn ng full, on the left side of the diagram in cubic feet per second, or on the right side of the diagram in gallons per 24 hours. (4) If the velocity is desired,

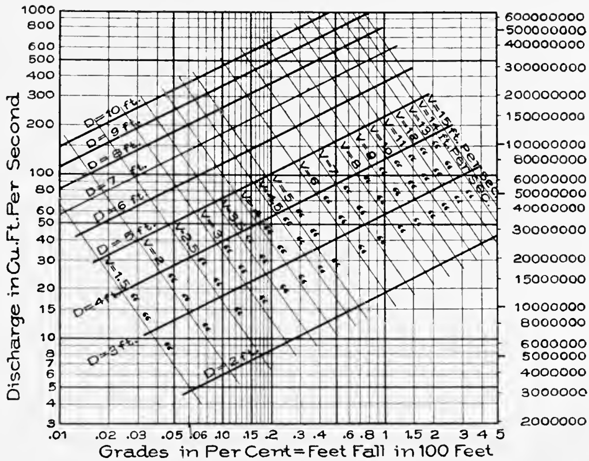


Fig. 28. Discharges and Velocities of Circular Brick and Concrete Sewers Flowing Full. By Kutter's Formula (n=0.015).

it can be determined by noting where the point (found in 2, above) of intersection of the given grade and diameter lines falls with reference to the inclined lines marked with the different velocities, estimating by the eye the decimals of a foot per second.

(*B*) *When the grade and the required discharge are given, to find the necessary diameter of pipe, and the velocity.*

(1) Look along the bottom horizontal line till the given grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the intersection of the vertical line through this grade with the horizontal line through the given discharge, finding the discharge on the left of the diagram if it is given in cubic feet per second, or on the right if it is given in gallons per 24 hours. (3) Note between which two diameter lines this point of intersection falls, and take the diameter line nearest as that required. (4) Also note the position of the point of intersection with reference to the velocity lines, and so estimate the velocity, interpolating by the eye between the inclined velocity lines.

(*C*) *When the velocity and diameter are given, to find the grade and discharge.*

(1) Find the intersection of the given diameter line with the given velocity line, interpolating by the eye, if necessary. (2) Then vertically

downward to the bottom of the diagram from this point of intersection, read the required grade; and horizontally to the left side or to the right side of the diagram, read the discharge, interpolating by the eye in each case, if necessary.

All other cases may be solved by similar obvious methods.

EXAMPLES

Example 7. What size of circular brick or concrete sewer laid to a 0.2 per cent grade will be required to carry a storm sewage flow of ¾ cu. ft. per second per acre from one square mile of drainage area, and what will be the velocity?

Solution. See *B*, above. 1 square mile = 640 acres. The capacity required is 640 × ¾ = 480 cu. ft. per second, which we find on the left of Fig. 28 just below the 500 cu. ft. per second horizontal line, interpolating by eye. We next find the 0.2 per cent grade line at the bottom of the diagram, and locate the point of intersection of this vertical 0.2 per cent grade line with the horizontal 480 cu. ft. per second line already found above. This point of intersection comes nearly on the 9 feet inclined diameter line, and between the seven and eight feet per second inclined velocity lines.

Answer. Diameter of sewer required, 9 feet. Velocity = 7.6 ft. per second.

Example 8. What will be the minimum grade for a 60-inch brick or concrete sewer, if the minimum velocity allowed when flowing full is 3 ft. per second?

Answer. See *C*, above. 0.067 per cent grade.

Example 9. How large a population, contributing 75 gallons per capita per day of sanitary sewage, on the average (the maximum flow being 3 times the average), can be served by a 48-inch circular brick sewer, laid to a 0.06 per cent grade; and what will be the velocity of flow? (NOTE: Find the capacity as in *A*, above; and then divide by 3 times the average per capita amount per day.)

Answer. 89,000 population. 2.4 ft. per second.

Example 10. What will be the grade required to force a flow of 500 cu. ft. per second through a 96-inch circular brick sewer?

Answer. 0.38 per cent grade.

42. Diagram of Discharges and Velocities of Egg-Shaped Brick and Concrete Sewers Flowing Full. Fig. 29 is the diagram for egg-shaped brick sewers, corresponding to Fig. 27 for circular pipe sewers, and to Fig. 28 for circular brick and concrete sewers.

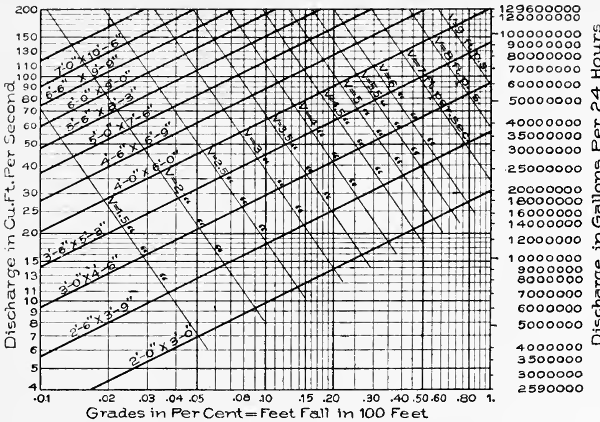


Fig. 29. Discharges and Velocities of Egg-Shaped Brick and Concrete Sewers Flowing Full. By Kutter's Formula ($n=0.015$).

TO USE THE DIAGRAM

(*A*) *When the diameter of the pipe and the grade are given, to find the discharge and the velocity.*

(1) Look along the bottom horizontal line till the grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the point where the vertical line through the given grade intersects the inclined line marked with the given diameter of sewer. (3) Trace horizontally through this point, interpolating by the eye, if necessary, between the horizontal lines on the diagram; and read the discharge of the pipe running full, on the left side of the diagram in cubic feet per second, or on the right side of the diagram in gallons per 24 hours. (4) If the velocity is desired, it can be determined by noting where the point (found in 2, above) of intersection of the given grade and diameter lines falls with reference to the inclined lines marked with the different velocities, estimating by the eye the decimals of a foot per second.

(*B*) *When the grade and the required discharge are given, to find the necessary diameter of pipe, and the velocity.*

(1) Look along the bottom horizontal line till the given grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the intersection of the vertical line through this grade with the horizontal line through the given discharge, finding the discharge on the left of the diagram if it is given in cubic feet per second, or on the right if it is given in gallons per 24 hours. (3) Note between which two diameter lines this point of intersection falls, and take the diameter line nearest as that required. (4) Also note the position of the point of intersection with reference to the velocity lines, and so estimate the velocity, interpolating by the eye between the inclined velocity lines.

SECTION OF CONCRETE SEWER PIPE IN TRENCH, BROOKLINE, MASSACHUSETTS
Metropolitan Sewerage System of Boston.

(*C*) *When the velocity and diameter are given, to find the grade and discharge.*

(1) Find the intersection of the given diameter line with the given velocity line, interpolating by the eye, if necessary. (2) Then vertically downward to the bottom of the diagram from this point of intersection, read the required grade; and horizontally to the left side or to the right side of the diagram, read the discharge, interpolating by the eye in each case, if necessary.

All other cases may be solved by similar obvious methods.

EXAMPLES

Example 11. What will be the discharge and velocity of flow of a 4 by 6-feet egg-shaped brick or concrete sewer flowing full and laid to a 0.4 per cent grade?

Solution. See A, above. Find the 0.4 per cent grade line at the bottom of Fig. 29, and locate the point of intersection of the vertical line through this point with the inclined 4 by 6 dimension line. Then tracing horizontally to the left, we estimate by the eye 128 cu. ft. per second for the discharge. We also note that the point of intersection of the vertical 0.4 per cent grade line with the inclined 4 by 6 dimension line found above, is practically on the inclined 7 ft. per second velocity line.

Answer. Discharge, 128 cu. ft. per second. Velocity, 7 ft. per second.

Example 12. What will be the size of egg-shaped brick or concrete sewer required to carry a storm flow of ½ cu. ft. per second per acre from a drainage area of ½ square mile (= 320 acres), the grade being 0.3 per cent?

Answer. See *B*, above. 4 ft. 6 in. by 6 ft. 9 in.

Example 13. A 6-foot circular sewer and a 5 by 7 ft. 6-in. egg-shaped sewer have nearly the same area of cross-section. If both are laid to a 0.2 per cent grade, find the discharge and velocity of each when flowing full. (NOTE: Solve by Figs. 28 and 29. See *A*, above.)

Answer. Discharge, 165 cu. ft. per second; and velocity, 5.8 ft. per second, for the circular sewer; and discharge 163 cu. ft. per second; and velocity, 5.7 ft. per second, for the egg-shaped sewer.

NOTE: Although the egg-shaped sewer has a slightly smaller velocity when both are flowing full, it has a materially greater velocity than the circular sewer for small depths of flow.

Example 14. If the minimum allowable velocity of flow in storm sewers is 3 ft. per second, find the minimum allowable grades for 2 ft. by 3 ft., 4 ft. by 6 ft., and 6 ft. by 9 ft. egg-shaped sewers, respectively.

Answer. See *C*, above. 0.20, 0.08, and 0.05 per cent, respectively.

43. Diagram of Discharges and Velocities in Circular Sewers at Different Depths of Flow. The diagrams so far given show the discharges and velocities in sewers *flowing full.* It often, however, is necessary to be able to calculate the discharge and the velocity when the sewer flows only *partially* full.

For *circular sewers,* the discharges and velocities, when flowing only *partially full,* can readily be determined by the use of the diagram, Fig. 30, in connection with Figs. 27 and 28.

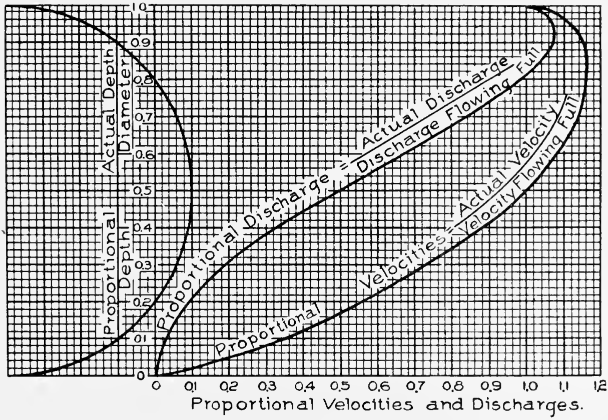


Fig. 30. Diagram Showing Changes in Velocity and Discharge in Circular Sewers for Different Depths of Flow.

TO USE THE DIAGRAM

(*A*) *When the depth of flow is given, together with the diameter and grade of the sewer, to determine the discharge and the velocity.*

(1) By Fig. 27 if a pipe sewer, or by Fig. 28 if a brick or concrete sewer, determine the *discharge* and *velocity* of the sewer *flowing full.* (2) Divide the given depth of flow by the given diameter, to determine the *proportional depth* of flow; and find this proportional depth on the vertical scale towards the left of Fig. 30, interpolating by the eye, if necessary. (3) Find the intersection of the horizontal line through the proportional depth (found in 2, above), first, with the *proportional discharge line,* and, second, with the *proportional velocity line,* in Fig. 30; and read off at the bottom of the diagram vertically below these intersection points, the *proportional discharge* and the *proportional velocity.* (4) Multiply the *discharge and velocity flowing full* (found in 1, above), by the *proportional discharge* and *proportional velocity*

found in 3, above), and the products will be the required *actual discharge and actual velocity, for the given depth of flow.*

(*B*) *When the actual discharge is given, together with the diameter and grade of the sewer, to find the depth and velocity of flow.*

(1) By Fig. 27 if a pipe sewer, or by Fig. 28 if a brick or concrete sewer, determine the *discharge of the sewer flowing full.* (2) Divide the given discharge by the discharge flowing full, to determine the *proportional discharge*; and find this along the bottom of the diagram in Fig. 30, interpolating by the eye, if necessary. (3) Find the intersection of the vertical line through the proportional discharge (found in 2, above) with the *proportional discharge curve* in Fig. 30; and horizontally to the left, read off on the vertical scale near the left of the diagram the *proportional depth* of flow. (4) Multiply the diameter of the sewer by the proportional depth, and the product will be the *actual depth of flow for the given discharge.* (5) The actual velocity can now be found as described above for case *A*.

All other cases than *A* and *B* can be readily solved by similar obvious methods.

EXAMPLES

Example 15. What will be the actual discharge and velocity of flow in a 48-inch circular brick sewer laid to a 0.15 per cent. grade, and flowing 6 inches deep?

Solution. See *A*, above. (1) By Fig. 28, with the sewer flowing full, the discharge would be 30,000,000 gallons per day, and the velocity 3.8 ft. per second. (2) $\frac{6 \text{ inches}}{48 \text{ inches}} = 0.12+ =$ proportional depth of flow, which we find on the vertical scale near the left of Fig. 30. (3) Horizontally opposite the point found in 2, we locate points on the proportional discharge curve and the proportional velocity curve in Fig. 30; and vertically beneath these points we read at the bottom of the diagram, 0.04 = proportional discharge, and 0.40 = proportional velocity. (4) 0.04 × 30,000,000 gallons = 1,200,000 gallons per day = actual discharge for 6 inches depth of flow; and 0.40 × 3.8 = 1.5 ft. per second = actual velocity for 6 inches depth of flow.

Example 16. An 8-inch pipe sewer, laid to a 0.40 per cent grade, is to carry the sewage of 500 people contributing 100 gallons each per day. What will be the average depth and velocity of flow?

Solution. See *B*, above. (1) By Fig. 27, the discharge and velocity flowing full would be respectively 450,000 gals. per day, and 1.9 ft. per second. (2) The actual discharge is 500 × 100 = 50,000 gals. per day, and hence the *proportional discharge* is $\frac{50{,}000}{450{,}000} = 0.11$. We find this proportional discharge along the bottom line of Fig. 30, interpolating by eye. (3) Vertically above the 0.11 proportional velocity, we find a point on the proportional discharge curve; and tracing horizontally to the left, we there read off the proportional depth = 0.225. (4) 0.225 × 8 = 1.8 inches = the actual depth of flow for the given discharge. (5) Horizontally to the right from the 0.225 proportional depth, we find a point on the proportional velocity line; and vertically beneath this point we read off at the bottom of the diagram, proportional velocity = 0.60. Then 0.60 × 1.9 (see 1, above) = 1.1 ft. per second = actual velocity for the given depth.

Example 17. What will be the discharge and velocity of a 12-inch pipe sewer laid to a 0.25 per cent grade when flowing 4 inches deep?

See *A*, above.

Answer. Discharge, 250,000 gals. per day; velocity, 1.7 ft. per second.

Example 18. What will be the depth and velocity of flow in a 15-inch pipe sewer, laid at a 0.2 per cent grade, carrying 1,000,000 gallons of sewage per day?

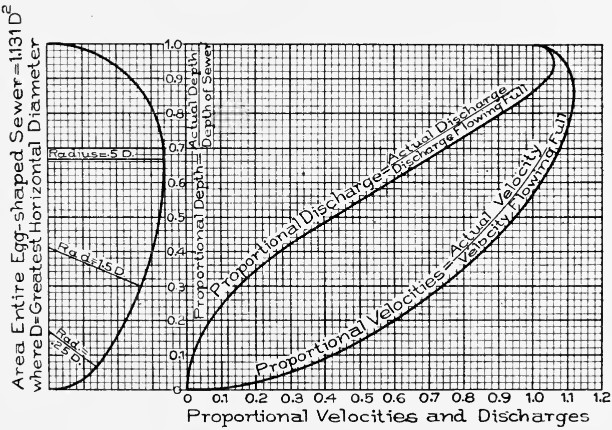


Fig. 31. Diagram Showing Changes in Velocity and Discharge in Egg-Shaped Sewers for Different Depths of Flow.

See *B*, above.

Answer. Depth, 8 inches; velocity, 2.3 ft. per second.

44. Diagram of Discharges and Velocities in Egg-Shaped Sewers at Different Depths of Flow. For egg-shaped sewers, the discharges and velocities, when flowing *partially full,* can readily be determined by the diagram, Fig. 31, used in connection with Fig. 29.

TO USE THE DIAGRAM

(*A*) *When the depth of flow is given, together with the diameter and grade of the sewer, to determine the discharge and the velocity.*

(1) By Fig. 29, determine the *discharge and velocity* of the sewer *flowing full.* (2) Divide the given depth of flow by the given height to determine the *proportional depth* of flow, and find this proportional depth on the vertical scale towards the left of Fig. 31, interpolating by the eye, if necessary. (3) Find the intersection of the horizontal line through the proportional depth (found in 2, above), first, with the *proportional discharge line,* and, second, with the *proportional velocity* line, in Fig. 31; and read off at the bottom of the diagram, vertically below these intersection points, the *proportional discharge,* and the *proportional velocity.* (4) Multiply the *discharge and velocity flowing full* (found in 1, above), by the *proportional discharge and proportional velocity* (found in 3, above), and the products will be the required *actual discharge and actual velocity for the given depth of flow.*

(*B*) *When the actual discharge is given, together with the diameter and grade of the sewer, to find the depth and velocity of flow.*

(1) By Fig. 29, determine the *discharge of the sewer flowing full.* (2) Divide the given discharge by the discharge flowing full, to determine the *proportional discharge,* and find this along the bottom of the diagram in Fig. 31, interpolating by the eye, if necessary. (3) Find the intersection of the vertical line through the proportional discharge (found in 2, above), with the *proportional discharge curve* in Fig. 31, and horizontally to the left, read off on the vertical scale near the left of the diagram the *proportional depth* of flow. (4) Multiply the height of the sewer by the proportional depth, and the product will be the *actual depth of flow for the given discharge.* (5) The actual velocity can now be found as described above for case *A*.

All other cases than *A* and *B* can be readily solved by similar obvious methods.

EXAMPLES

Example 19. What will be the discharge and velocity in an egg-shaped brick or concrete sewer 3 ft. by 4 ft. 6 in., laid to a 0.15 per cent grade, and flowing 12 inches deep?

See *A*, above.

Solution. (1) By Fig. 29, discharge and velocity flowing full = 36 cu. ft. per second, and 3.45 ft. per second, respectively. (2) The proportional depth $= \frac{12}{54} = 0.22$, which we find at left of Fig. 31. (3) We locate the intersections of the horizontal line through the 0.22 proportional depth with the proportional discharge and proportional velocity curves, respectively; and vertically below these points we read off, at the bottom of the diagram, proportional discharge = 0.08, and proportional velocity = 0.63. (4) $36 \times 0.08 = 2.9$ cu. ft. per second = actual discharge; $3.45 \times 0.63 = 2.2$ ft. per second = actual velocity.

Answer. Discharge = 2.9 cu. ft. per second; velocity = 2.2 ft. per second.

Example 20. What will be the depth and velocity of flow in an egg-shaped brick or concrete sewer 5 ft. by 7 ft. 6 in. dimensions, laid to a 0.10 per cent grade, and carrying 30 cu. ft. per second flow of sewage?

See *B*, above.

Solution. (1) By Fig. 29, the discharge and velocity flowing full = 117 cu. ft. per second and 4.05 ft. per second, respectively. (2) Proportional discharge = $\frac{30}{117}$ = 0.26 –, which find at bottom of Fig. 31. (3) Vertically above the 0.26 proportional discharge, we locate a point on the proportional discharge curve in Fig. 31, and horizontally to the left from this point read off the proportional depth = 0.39. (4) $90 \times 0.39 = 35$ inches = actual depth of flow. (5) Horizontally to the right along the 0.39 proportional depth line, we locate a point on the proportional velocity line; and vertically beneath this, we read off, at the bottom of the diagram, proportional velocity = 0.845. Then $4.05 \times 0.845 = 3.4$ ft. per second = actual velocity.

Answer. Depth of flow = 35 inches; velocity = 3.4 ft. per second.

Example 21. What will be the discharge and velocity in an egg-shaped brick or concrete sewer 2 ft. by 3 ft. dimensions, laid to a 0.50 per cent grade, flowing 18 inches deep?

See *A*, above.

Answer. Discharge = 5,900,000 gals. per day; velocity = 4.5 ft. per second.

Example 22. What will be the depth and velocity of flow in an egg-shaped brick or concrete sewer 3 ft. 6 in. by 5 ft. 3 in. dimensions, laid to a 0.08 per cent grade, carrying 25 cu. ft. per second of sewage?

See *B*, above.

Answer. Depth of flow = 39 inches; velocity of flow = 2.9 ft. per second.

GENERAL EXAMPLES FOR PRACTICE WITH FIGS. 27-31

45. The solution of the following general examples will further familiarize the student with the principles thus far explained.

Example 23. A 24-inch sewer is to be laid to a 0.25 per cent grade, and may be made of vitrified sewer pipe or of brick. Compare the discharges and velocities obtained with the two materials. (NOTE: Use Figs. 27 and 28.)

Answer. With sewer pipe, discharge = 7,200,000 gals. per day; velocity = 3.6 ft. per second.
With brick, discharge = 6,000,000 gals. per day; velocity = 3 ft. per second.

Example 24. A combined sewer, laid to a 0.15 per cent grade, drains an area requiring either a 3-foot circular or a 2 ft. 6 in. by 3 ft. 9 in. egg-shaped brick sewer. (These sizes have the same cross-sectional area, and nearly the same discharges and velocities, when flowing full.) The dry-weather flow of sewage will be only 1,000,000 gallons per day. Calculate the dry-weather depth and velocity of flow with each design. (NOTE: Use Figs. 28 and 30, and Figs. 29 and 31.)

Answer. With circular sewer, depth = 6.1 inches; velocity = 1.6 ft. per second.
With egg-shaped sewer, depth = 9.2 inches; velocity = 1.9 ft. per second.

Example 25. In a 10-inch pipe sewer, laid to a one per cent grade, the maximum depth of flow observed was 7 inches; and the minimum, 2 inches. What were the corresponding discharges? (NOTE: Use Figs. 27 and 30.)

Answer. Maximum discharge = 1,100,000 gals. per day;
Minimum " = 120,000 " " "

Example 26. What size of circular sewer laid to a 0.08 per cent grade will be required to carry the sanitary sewage of a city of 100,000 population, with an average flow of sewage of 150 gallons per capita per day, the maximum rate of flow being three times the average?

Answer. 5 ft. 3 in. diameter.

Example 27. What size of egg-shaped combined sewer, laid to a 0.07 per cent grade will be required to carry a storm sewage flow of 0.5 cu. ft. per second per acre from a drainage area of 320 acres?

Answer. 6 ft. by 9 ft.

46. Summary of Laws of Flow in Sewers. The principles discussed in Articles 38 to 44, inclusive, may be briefly summarized as follows:

(1) The laws of flow for sewage are the same as for water.

(2) Kutter's formula is generally considered most reliable for calculating the flow in sewers, though complicated to use directly.

(3) In Kutter's formula, the values of the coefficient of roughness generally used for sewer computations, are $n = 0.013$ for pipe sewers, and $n = 0.015$ for brick and concrete sewers.

(4) Sewer diagrams greatly simplify sewer computations, and are presented in Figs. 27 to 31, inclusive, for circular and egg-shaped sewers, with full instructions for use.

(5) In Fig. 30, the laws of flow for different depths of flow in

circular sewers are shown. An examination of the diagram brings out this important law:

In circular sewers flowing half-full, the velocity is the same as when the sewer flows full; and hence the discharge flowing half-full is just half the discharge flowing full.

(6) Figs. 30 and 31 also show the following important law of flow:

In a sewer of any shape, not flowing under pressure, the maximum discharge and velocity will occur, not with the sewer flowing full, but with it flowing a little less than full.

This is due to the increased friction against the top of the sewer when it flows full. Owing to this law, no sewer can flow full without being under pressure.

(7) In the case of combined sewers having a dry-weather flow very small as compared with the storm flow, egg-shaped sewers give materially greater depths and velocities of dry-weather flow than circular sewers.

CALCULATIONS OF SIZES AND MINIMUM GRADES OF SEPARATE SANITARY SEWERS

47. Minimum Sizes of Sanitary Sewers. In the early construction of sewers, previous to the last half of the 19th century, the laterals and sub-mains were usually made very much larger than the amount of sewage would require, with the idea, apparently, that the bigger the sewer the better. Such badly proportioned sewers were in great danger of stoppages from the inability of the shallow, trickling stream to carry along the solid matter. In fact, the sewers were expected to form deposits, and were purposely made large to hold a large amount of deposit and to enable men to enter for the purpose of cleaning them. Disastrous sanitary experience with such foul sewers made it apparent that there was just as much danger from making the sewers too large as from making them too small, especially in the case of sanitary sewers. Such sewers should be made small enough to give a good depth and velocity of flow.

Sanitary sewers should not be made small enough, however, to cause frequent stoppages by catching articles which have been admitted into them through the house connections. House owners are often reprehensibly negligent in putting into their plumbing fixtures, articles which should be carefully excluded. On this account, the size of house connections should be restricted to 4 inches.

An 8-inch sewer pipe will practically always carry freely, even crosswise, any article which can come lengthwise around the traps and bends in 4-inch soil-pipes and house connections Hence *eight inches should usually be adopted as the minimum size for sanitary sewers.*

Usually the great bulk of the sanitary sewers in a separate system will be of this minimum size, only a limited length of the larger sizes being required for sub-mains and mains. See the sewerage map of Ames, Iowa, Fig. 38.

In the early use of the separate system, many 6-inch laterals were constructed, and, except for occasional stoppages from articles improperly put into the sewers, they have worked well. Some engineers still use six inches as the minimum size.

48. Minimum Grades and Velocities for Separate Sanitary Sewers. In the design and construction of sewers it has been found that certain minimum grades should be adopted to prevent deposits, no sewers being built to lighter grades than the minimum unless special means for flushing, or special facilities for cleaning, are provided. This is to insure sufficient velocity to prevent the settling out of the solid matter in the sewage to form deposits in the sewers.

These minimum grades for separate sanitary sewers are as follows:

TABLE II
Minimum Grades for Separate Sanitary Pipe Sewers

Diameter	Minimum Grade	Diameter	Minimum Grade
4 inches	1.20 per cent	18 inches	0.12 per cent
6 "	0.67 " "	20 "	0.10 " "
8 "	0.43 " "	24 "	0.08 " "
9 "	0.36 " "	27 "	0.07 " "
10 "	0.30 " "	30 "	0.06 " "
12 "	0.23 " "	33 "	0.05 " "
15 "	0.16 " "	36 "	0.045 " "

Caution.—*For the above minimum grades to be satisfactory and safe, there must be enough sewage to give a good depth of flow.*

The flow and velocity in a sewer fluctuate greatly, as illustrated in Article 52, below, the velocity at low flow being much less than when flowing full or half-full.

Experiments have shown that an actual velocity of $1\frac{1}{4}$ to $1\frac{1}{2}$ feet per second is sufficient to prevent deposits of the solid matters usually found in sanitary sewers; but to secure this velocity at low flow requires about 2 feet per second when the sewer flows full or half-full (see Figs. 30 and 31 for the fluctuation of velocity with depth of flow). Hence *the minimum grades for sanitary sewers should usually be those giving a velocity of 2 feet per second when flowing full or half-full, as shown by the diagrams, Figs. 27, 28, and 29.*

It is usually considered that, within a reasonable period in the future, the increased high-water flow each day should be sufficient to fill the sewer half-full or nearly so. However, in numerous cases, sanitary sewers have been observed to work well at the above grades with less depths of flow than this.

Much will depend on the nature of the sewage. Some thick, manufacturing sewages, heavily loaded with solid matter, would require considerably heavier grades to insure self-cleansing.

Where it is absolutely impossible to secure the above minimum grades, special means for flushing, such as automatic flush-tanks placed about three blocks apart, should be used.

49. General Explanation of the Calculation of Amount of Sanitary Sewage. The first thing necessary in computing the size required for any particular sanitary sewer, is to ascertain the amount of sewage it must carry. While this cannot be foretold with exactness, yet, by well-established methods, an approximation sufficiently close for all practical purposes can readily be made.

The *first step* in computing the amount of sewage will be to estimate the future tributary population which may use the sewer. For this, see Art. 50, below.

The *second step* will be to estimate the average amount of sewage contributed by each person per day—that is, the average flow of sewage per capita per day. This, multiplied by the tributary population, will give the total average amount of sewage per day which the sewer must carry.

Two methods are in use for estimating the average flow of sewage per capita per day:

(1) It is often assumed to equal the average consumption of water per capita per day. For this method, see Art. 51, below.

(2) The best method is to compare the local conditions with

actual sewer gaugings of flow in sewers under similar conditions elsewhere. For this method, see Art. 52, below.

50. Methods of Estimating the Population Tributary to Sanitary Sewers. The most important difficulty encountered in estimating the population tributary to sanitary sewers, is the fact that it is the *future* population which must be determined. To know the present tributary population is not sufficient. Two methods will be described:

(1) *The best method of estimating the future population tributary to sanitary sewers is as follows:*

(*a*) On the sewer map, lay out sewers to serve all districts to be served in the future as well as at present.

(*b*) After careful examination of the ground, and study of the conditions, estimate the number of persons tributary to the sewers per 100 feet of sewers in each district when it is built up as fully as can reasonably be expected.

In doing this, five or six persons per family should usually be allowed, and the number of families on both sides of the street for one block in the future estimated. The number of persons per block so obtained should then be divided by the number of hundred feet of sewer per block from center to center of streets.

Thus, if there are 6 lots 50 feet wide per block (=300 feet) on each side of the sewer, and the streets are 60 feet wide (=360 feet center to center of streets), and if it is thought that every lot will eventually contain one residence,

$$\text{Tributary population} = \frac{12 \times 6 \text{ persons}}{3.60} = 20 \text{ persons per 100 feet of sewer.}$$

The tributary population per 100 feet of sewer will usually range from 20 persons in the residence districts of small cities, to 100 persons in thickly built-up business districts. In the congested districts of the largest cities, the population is still denser.

(*c*) *To determine the total population tributary above any point on a sanitary sewer, scale from the sewer map the total number of hundred feet of tributary sewer above that point, including all branches; and multiply the total so obtained by the tributary population per* 100 *feet of sewer.*

Thus, if there are 15,600 ft. of tributary sewers, and the tributary population is 20 per 100 ft., the total tributary population will = 85 × 20 = 1,700 persons. In some cases part of the length of tributary sewers may have to be multiplied by one density of tributary population, and part by another.

(2) In case the future population of an entire city is to be estimated, a different method must be used.

Usually, the past population of the city at different dates is obtained from census reports; and by study of this past growth, and of the present and probable future local conditions as affecting growth, and by comparison with the past growth of larger cities whose conditions were similar, estimates are made of the probable future populations at different dates, for 20 to 50 years in the future.

Usually, also, the past records of the city that is being studied, and of others, are platted as curves on cross-section paper, the ordinates representing population, and the abscissæ dates; and the future estimates are made by prolonging the curve of growth into the future.

51. Use of Statistics of Water Consumption in Determining the Per Capita Flow of Sanitary Sewage. Since about 99.8 per cent of sanitary sewage is merely ordinary water, nearly always taken from the public supply, the total flow per capita of sanitary sewage is usually approximately equal to the consumption of water per capita (that is,

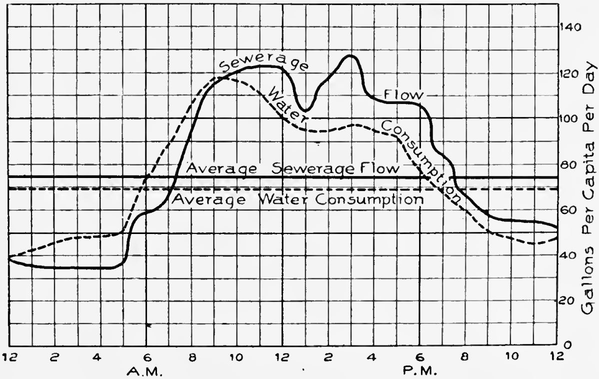


Fig. 32. Typical Gauging of Flow of Sanitary Sewage, Des Moines, Iowa, Friday, July 5, 1895.

per person). In Fig. 32 may be seen how closely sewage flow and water consumption ordinarily correspond.

In many towns, however, there will not be such close correspondence. Sometimes considerable amounts of water may be used for manufacturing or other purposes which divert it from the sewers, making the sewage flow less than the water consumption. More often there will be considerable influxes of ground water through leaking sewer joints, sometimes making the sewage flow several times as great as the water consumption.

However, very extensive statistics of water consumption in a large number of places have been collected, while actual gaugings of flow of sewage are comparatively few. Hence statistics of the water consumption of the town for which sewers are being designed, or of similar towns elsewhere, are often used as the basis for estimating the per capita flow of sanitary sewage. In studying each town preliminary to designing sewers for it, all possible information should be secured relative to its water consumption.

TABLE III

Consumption of Water in American Cities, 1895

City	Population	Daily Consumption per Person, 1895. Gallons
New York	3,437,202	100
Chicago	1,698,575	139
Philadelphia	1,293,697	162
St. Louis	575,238	98
Boston	560,892	100
San Francisco	342,782	63
Buffalo	352,387	271
New Orleans	287,104	35
Minneapolis	202,718	88
Columbus	125,560	127
Atlanta	89,872	42
Nashville	80,865	139

On pages 4 to 10 of the instruction paper on Water Supply, Part I, will be found a detailed discussion of water consumption. From a larger table given there, Table III herewith is condensed, to show how the average per capita water consumption varies in different American cities.

It will be noted that there is a very wide range in water consumption. The excessively low rates usually mean an incomplete water supply, which is likely to be extended later, while the excessively high rates usually mean great waste of water. This can often be greatly reduced by introducing water meters.

Under fairly average conditions the consumption will usually fall between the limits of 40 and 125 gallons per capita per day, as shown in detail in Table IV.

TABLE IV

Water Consumption under Ordinary Conditions

Use	Gallons per Capita per Day		
	Minimum	Average	Maximum
Domestic	15	25	40
Commercial	7	20	35
Public	3	5	10
Waste and Loss	15	25	40
Total	40	75	125

52. Use of Sewer Gaugings in Determining the Per Capita Flow of Sanitary Sewage. It has already been stated that the flow of sanitary sewage is not always equal to the water consumption. In one case of sewer gaugings, the writer found the flow of sewage to be only 50 to 60 per cent of the water consumption, the remainder of the water being consumed for purposes which diverted it from the sewers. In another case of sewer gaugings, the writer found the flow of sewage to be over 500 per cent of the water consumption, the increase being due to infiltration of ground water through sewage joints. Hence, water consumption data alone are not sufficient in making estimates of sewage flow, and data from actual sewer gaugings are needed. Of late years there is an increasing accumulation of data of sewage flow obtained from actual gaugings. Some of these data are given in Table V.

At the Iowa State College, the sewage flow, as given in Table V, below, was 50 to 60 per cent of the water consumption, owing to uses of water which diverted it from the sewers. At Grinnell, on the other hand, infiltration of ground water into the sewers increased the sewage flow to about six times the total water consumption on the same day.

A study of Table V will show, however, that *in general the average flow of sanitary sewage is between the limits of* 50 *and* 125 *gallons per capita per day.*

53. Capacities of Sanitary Sewers Required to Provide for Fluctuations in the Rate of Flow. So far our discussion of flow of

TABLE V
Gaugings of Flow of Sanitary Sewage

Sewer	Date	Duration, Days	Tributary Population	Sewage Flow, Gals. per Capita per Day		
				Min.	Av.	Max.
Compton Ave., St. Louis	1880	6	8,200	65	102	149
College St., Burlington, Vt.	1880	5–8	325	65	115	140
Huron St., Milwaukee, Wis.	1880	–	3,174	—	—	120
Memphis, Tenn.	1881	–	20,000	61	—	140
13 Sewers, Providence, R. I.	1884	1–6	33,825	—	78	—
Asylum, Binghamton, N. Y.	1888	–	1,300	—	—	608
16 Sewers, Toronto, Ont.	1891	3	168,081	—	87	—
Insane Asylum, Weston, W. Va.	1891	2	1,000	40	91	151
Schenectady, N. Y.	1892	1	*10,000	72	86	103
Canton, Ohio	1893	–	40,000	54	129	180
Chautauqua, N. Y.	1894	–	7,000	6	20	30
Iowa State College, Ames, Ia.	1894	7	289	0	32	77
Des Moines, Ia., E. Side	1895	15	8,100	22.5	74	142
Des Moines, Ia., W. Side	1895	13	19,400	23.2	66	175.3
Iowa State College, Ames, Ia.	1900	2	800	54	95	175
Iowa State College, Ames, Ia.	1900	28	800	30	57	130
Marshalltown, Ia.	1900	1	4,200	67	85	111
Grinnell, Ia.	1901	1	2,000	169	186	200
Insane Asylum, Mt. Pleasant, Ia.	1901	1	1,200	32	62	115
Waverly, N. Y.	1905	4	1,796	79	155	194

* Estimated.

sanitary sewage (Arts. 51 and 52) has referred particularly to the *average* flow per capita per day. The flow, however, is not uniform, but fluctuates greatly. First, there is a *seasonal fluctuation.* The flow is apt to be especially high in severe cold weather, when faucets are left running to keep pipes from freezing; in hot weather, when water consumption is high; and in wet weather, when some ground water finds its way into the sewers.

Second, there is a *daily fluctuation.* For example, gaugings show that the flow usually is light on Sundays and holidays, when business is suspended. The flow on Monday is apt to be especially high, on account of wash day.

Third, there is an *hourly fluctuation,* at different times of the day and night. In Fig. 32, an example is shown of the fluctuation of sewage flow throughout one day, as determined by a continuous sewer gauging in the case of a city of 56,000 population. As shown in this figure, the flow of sanitary sewage is usually low through the night, reaching a minimum at about 2 to 3 A. M. It increases rapidly early in the morning, reaching a high point at about 10 to 11 A. M. Although there is usually a temporary drop at the noon hour, the flow continues high until early evening, and then decreases rapidly to its low night value.

A study of the sewer gaugings summarized in Table IV, together with others, shows that the flow of sanitary sewage ordinarily fluctuates from a minimum rate of 30 per cent to a maximum rate of 265 per cent of the average rate. If the gaugings had been extended over longer periods of time, still greater fluctuations of flow would certainly have been found.

It is apparent that the fluctuations in rate of flow will be greater in lateral sewers than in main sewers. To make them large enough to provide for the greatest rates of flow to be reasonably expected, *sanitary sewers should be given the following capacities*:

PROPER CAPACITIES OF SANITARY SEWERS

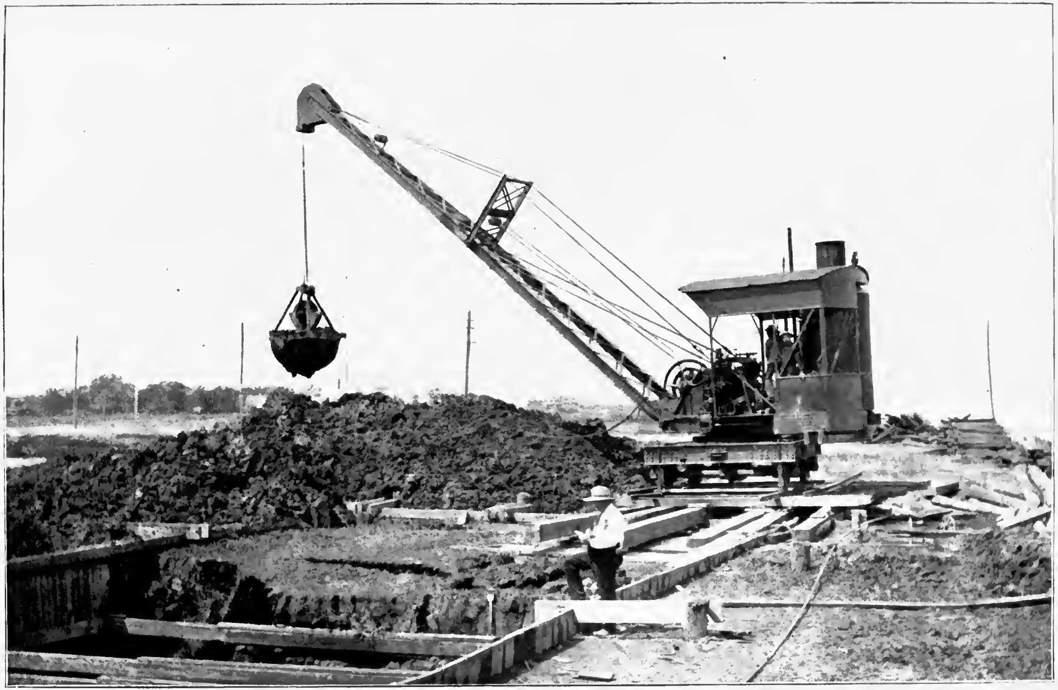
For lateral sewers, 350 per cent of the average flow.
For sub-main sewers, 325 per cent of the average flow.
For main sewers, 300 per cent of the average flow.

Table VI (page 68) is proportioned on the above basis.

54. Ground Water in Sanitary Sewers. In addition to the sanitary sewage itself, provision must often be made in separate sanitary sewers for leakage of ground water into the sewers. The amount of ground water to be allowed for, will depend on the character of the soil, on the height of the ground water with reference to the sewer, and on the care with which the sewer joints are made. *If the joints are made very carefully, the amount of ground water to be expected may range, with the soil, and height of ground water, from* 0 *to* 30,000 *gallons per mile.* This will constitute, say, 0 to 30 per cent of the sewage, but is a steady flow, not requiring the 300 to 350 per cent allowance for fluctuations required for sewage (see Art. 53). Hence, *if the joints are carefully made, the capacity of the sewers need not be increased more than* 10 *per cent for ground water.*

If sub-drains with outlets separate from the sewers are provided for all wet stretches of trench, no allowance whatever for ground water need be made in the size of the sewers.

The infiltration of ground water is apt to be much greater during and immediately after the construction of sewers than later, for the effect of sewers is to lower permanently the level of the ground water.



LOCOMOTIVE CRANE USED IN SEWER TRENCH CONSTRUCTION

Metropolitan Water and Sewerage System of Boston, Mass.

55. Summary of Methods of Computing Sizes of Separate Sanitary Sewers. The methods for computing the sizes of sanitary sewers may be summarized as follows:

(1) Lay out on the sewer map all the sewers required to serve all districts which can reasonably be expected to be included in the system, either at present or within say 30 to 50 years in the future.

(2) By a careful study of the topography, business conditions, manufacturing possibilities, and other future prospects, together with the sizes of blocks and lots, and the widths of streets, determine the probable future tributary population in each district per 100 feet of sewer, allowing usually five or six persons per family.

(3) By a careful study of the statistics of water consumption (Art. 51), and by comparison with actual sewer gaugings (Art. 52), taking into account all local conditions, estimate the average flow of sewage in gallons per capita per day.

(4) Beginning at the upper ends of the sewers, scale from the map and tabulate the total lengths of tributary sewer above successive points in the system, to the outlet. Multiply the number of hundreds of feet in these lengths by the tributary population per 100 feet, and by the average per capita flow of sewage per day, to get the total flow of sanitary sewage at the successive points.

(5) To allow for fluctuations (Art. 53), multiply the above average rates of flow of sanitary sewage by

$3\frac{1}{2}$ for lateral sewers;
$3\frac{1}{4}$ " sub-main "
3 " main "

to get the maximum rates of flow of sanitary sewage.

(6) To the maximum rates of flow so found, add 0 to 30,000 gallons per mile of tributary sewers, to allow for ground water (Art. 54).

(7) Occasionally it may be necessary also, in the case of certain sewers, to make special allowances for manufacturing sewage from large factories, each factory being studied by itself to determine its probable sewage flow. This flow will usually be subject to as much fluctuation as sanitary sewage, and hence must be multiplied by the factors given in 5, above.

(8) On the sewer profiles (see Art. 92), the grades of the sewers at the successive points will be determined and shown. Using these grades, and the total maximum rates of flow of sewage determined

TABLE VI
Sizes Required for Separate Sanitary Pipe Sewers

1	2	3	4	5	6	7	8	9
Diam. of Sewer, Ins	Grade of Sewer, %	Maximum Permissible Av. Flow. Gals. per Day	Maximum Permissible Tributary Population			Maximum Permissible Linear Feet of Tributary Sewer for 20 Persons per 100 Feet		
			Gals. per Capita per Day			Gals per Capita per Day		
			75	100	125	75	100	125
8	0.43	130,000	1,700	1,300	1,000	8,700	6,500	5,200
	0.60	160,000	2,100	1,600	1,300	11,000	8,000	6,400
	0.80	180,000	2,400	1,800	1,400	12,000	9,000	7,200
	1.00	200,000	2,700	2,000	1,600	13,000	10,000	8,000
	1.40	240,000	3,200	2,400	1,900	16,000	12,000	9,600
10	0.30	220,000	2,900	2,200	1,800	15,000	11,000	8,800
	0.40	260,000	3,400	2,600	2,100	17,000	13,000	10,000
	0.60	310,000	4,100	3,100	2,500	21,000	15,000	12,000
	0.80	360,000	4,800	3,600	2,900	24,000	18,000	14,000
	1.00	400,000	5,300	4,000	3,200	27,000	20,000	16,000
12	0.23	350,000	4,700	3,500	2,800	23,000	17,000	14,000
	0.40	460,000	6,100	4,600	3,700	31,000	23,000	18,000
	0.60	560,000	7,500	5,600	4,500	37,000	28,000	22,000
	0.80	650,000	8,700	6,500	5,200	43,000	32,000	26,000
	1.00	720,000	9,600	7,200	5,800	48,000	36,000	29,000
15	0.17	550,000	7,300	5,500	4,400	37,000	27,000	22,000
	0.30	750,000	10,000	7,500	6,000	50,000	37,000	30,000
	0.40	850,000	11,000	8,600	6,900	57,000	43,000	34,000
	0.60	1,000,000	13,000	10,000	8,000	67,000	50,000	40,000
	0.80	1,200,000	16,000	12,000	9,600	80,000	60,000	48,000
18	0.13	800,000	11,000	8,000	6,400	55,000	40,000	32,000
	0.30	1,200,000	16,000	12,000	9,600	80,000	60,000	48,000
	0.40	1,400,000	19,000	14,000	11,000	93,000	70,000	56,000
	0.60	1,700,000	23,000	17,000	14,000	113,000	85,000	68,000
	0.80	2,000,000	27,000	20,000	16,000	134,000	100,000	80,000
24	0.10	950,000	12,000	9,500	7,400	62,000	46,000	37,000
	0.20	1,300,000	17,000	13,000	10,000	87,000	65,000	52,000
	0.40	1,900,000	25,000	19,000	15,000	126,000	95,000	76,000
	0.60	2,300,000	31,000	23,000	18,000	153,000	115,000	92,000
	0.80	2,600,000	35,000	26,000	21,000	173,000	130,000	104,000
27	0.08	1,400,000	19,000	14,000	11,000	93,000	70,000	56,000
	0.20	2,200,000	29,000	22,000	18,000	147,000	110,000	88,000
	0.30	2,700,000	36,000	27,000	22,000	180,000	135,000	108,000
	0.40	3,100,000	41,000	31,000	25,000	207,000	155,000	124,000
	0.60	3,800,000	51,000	38,000	30,000	254,000	190,000	152,000
30	0.06	2,200,000	29,000	22,000	18,000	147,000	110,000	88,000
	0.10	2,800,000	37,000	28,000	22,000	187,000	140,000	112,000
	0.20	4,000,000	53,000	40,000	32,000	267,000	200,000	160,000
	0.40	5,700,000	76,000	57,000	46,000	380,000	285,000	238,000
	0.60	7,000,000	93,000	70,000	56,000	466,000	350,000	280,000
36	0.05	3,200,000	43,000	32,000	26,000	214,000	160,000	128,000
	0.10	4,600,000	61,000	46,000	37,000	307,000	230,000	184,000
	0.20	6,500,000	87,000	65,000	52,000	433,000	325,000	260,000
	0.40	9,300,000	124,000	93,000	74,000	620,000	465,000	372,000
	0.60	11,400,000	152,000	114,000	91,000	760,000	570,000	456,000

in 5, 6, and 7, above, refer to Fig. 27 for pipe sewers, or to Fig. 28 for brick or concrete sewers, and find the sizes of sewers required.

Example 28. In a town in which the blocks are 340 feet, center to center of streets, there are 14 lots per block. The total length of tributary sewers above a certain point on a sub-main sewer in the system (separate sewers), is 16,600. The conditions affecting rate of sewage flow per capita are average. No allowance need be made for ground water or manufacturing sewage. The grade of the sewer is 0.30 per cent. What size is required?

Solution. The tributary population will be $\frac{14 \times 6}{3.4} = 25$ persons per 100 feet of sewer. The average rate of flow may be assumed at 85 gallons per capita per day. Hence the maximum rate of flow for this sub-main sewer will be $166 \times 25 \times 85 \times 3\frac{1}{4} = 1{,}150{,}000$ gallons per day.

Hence, by Fig. 27, for a 0.30 per cent grade, a 12-inch pipe sewer will be required.

Answer. A 12-inch pipe sewer.

56. Table of Sizes Required for Sanitary Sewers. By the methods given in Art. 55, omitting allowances for ground water and manufacturing sewage, Table VI (page 68) has been computed, to reduce the labor of computation of sizes of separate sanitary pipe sewers.

TO USE THE TABLE

Proceed to follow out steps 1, 2, 3, and 4, in Art. 55, just above (which read), thus determining the total estimated future number of linear feet of tributary sewer at successive points, the estimated future number of persons tributary per 100 feet of sewer (which let $= P$), and the estimated average flow of sewage in gallons per capita per day (which let $= F$). Also ascertain the grade to which the sewer is to be built.

(A) *If $P = 20$ persons per 100 feet, and if F lies between 75 and 125 gallons per capita per day, and if no allowance is necessary for ground water or manufacturing sewage,* find in column 7, 8, or 9, or by interpolating between them, according to the value of F, a number close to the calculated number of linear feet of tributary sewer opposite to the given sewer grade, interpolating between the grades, and take the corresponding size of sewer in column 1.

Example 29. For 13,100 linear feet of sewer, 20 persons per 100 ft., 85 gallons per capita per day, and 0.35 per cent grade.

We find that for a 0.35 per cent grade an 8-inch sewer would be considerably too small, as shown by interpolating between the numbers in columns 7 and 8, while a 10-inch sewer would be a little larger than needed.

Answer. A 10-inch pipe sewer.

(B) If P does not $= 20$ persons per 100 feet (the other conditions remaining as in A, above), first multiply the number of linear feet of tributary sewer by $\frac{P}{20}$, and then proceed as in A, just above.

Example 30. For 16,300 linear feet of sewer, 30 persons per 100 feet of sewer, 110 gallons per capita per day, and a sewer grade of 0.25 per cent.

We first find $16{,}300 \times \frac{30}{20} = 24{,}450$ linear feet. Then interpolating between columns 8 and 9, we find that for a 0.25 per cent grade a 12-inch would be considerably too small, while a 15-inch sewer is a little larger than needed.

Answer. A 15-inch pipe sewer.

(*C*) If F (rate of sewage flow) is less than 75 or more than 125 gallons per capita per day, first multiply the number of linear feet of tributary sewer by $\frac{F}{100}$, and then by $\frac{P}{20}$ (where P = persons per 100 feet of sewer), and then find the nearest number in column 8 opposite the given grade.

Example 31. For 22,500 linear feet of sewer, 35 persons per 100 feet, 150 gallons per capita per day, and 0.45 per cent grade.

We first find $22{,}500 \times \frac{150}{100} \times \frac{35}{20} = 59{,}000$ linear feet. In column 8 we find that for a 0.45 per cent grade a 15-inch sewer would be considerably too small, while an 18-inch is too large.

Answer. An 18-inch pipe sewer.

(*D*) *If ground water or manufacturing sewage, or both,* must be allowed for, ascertain the *total average sewage flow*, by multiplying the linear feet of tributary sewer by $\frac{P}{100}$ (P = persons per 100 feet), and this result by F (= gallons per capita per day, of sanitary sewage), and by then adding to this result the total allowance for manufacturing sewage, and $\frac{1}{3}$ the total allowance for ground water. Then find by interpolation in column 3 the nearest number opposite the given grade, and take the corresponding size of sewer.

Example 32. For 15,600 linear feet of tributary sewer, 25 persons per 100 feet, 85 gallons per capita per day, 15,000 gallons per day per mile ground water, 200,000 gallons per day manufacturing sewage, and 0.20 per cent grade.

We find the total average flow of sewage to use is $15{,}600 \times \frac{25}{100} \times 85 + 200{,}000 + \frac{15{,}000}{3} \times 3$ (miles) $= 546{,}000$ gallons per day. In column 3 we find that for a 0.20 per cent grade, a 12-inch sewer would be considerably too small, while a 15-inch is a little larger than is needed.

Answer. A 15-inch pipe sewer.

GENERAL EXAMPLE FOR PRACTICE IN DESIGNING SEPARATE SANITARY SEWERS

57. Working out the following example will materially help the student.

Example 33. Calculate the size of the outlet sewer of the sewer system shown in Fig. 4, assuming that there will be in the future 20 persons tributary per 100 feet of sewer, that the average flow of sewage will be 100 gallons per capita per day, no special allowance for ground water or manufacturing sewage being needed. Also assume that there may be in the future 15,000 feet of sewer extensions not shown in the figure. The grade of the outlet sewer is 0.20 per cent. Assume scale of drawing, 1,500 feet per inch.

Solution. Take a long strip of paper with one edge straight; and on this, mark off with a pencil a scale of feet from the scale assumed above. With this, scale off the lengths of all the sewers shown, except the storm sewers. Add up the lengths scaled, and add 15,000 linear feet of future extensions, to get the total length of tributary sewer. Then use Table VI.

Answer. An 18-inch pipe sewer.

CALCULATION OF SIZES AND MINIMUM GRADES OF STORM AND COMBINED SEWERS

58. Storm and Combined Sewers Calculated by Same Methods. In combined sewers the rate of flow of sanitary sewage is so small in time of storms in proportion to that of the storm sewage, that the sanitary sewage can be neglected altogether in calculating the size. For example, a combined sewer one mile long, with 20 persons tributary per 100 feet, and 75 gallons per capita per day, would have a maximum rate of flow of sanitary sewage at its lower end of $\frac{52.8 \times 20 \times 75 \times 3\frac{1}{2}}{7\frac{1}{2} \times 86{,}400} = 0.43$ cu. ft. per second (there being $7\frac{1}{2}$ gals. in 1 cu. ft., and 86,400 seconds in 1 day, and the maximum rate of flow being $3\frac{1}{2}$ times the average).

If the blocks are 360 feet wide, center to center of streets, this same sewer would have to take the storm sewage from $43\frac{1}{2}$ acres. The amount of this at the time of the maximum storm allowed for, calculated by the methods described below, would probably be at least 20 cu. ft. per second. The sanitary sewage would therefore be only about 2 per cent of the storm sewage. The amount of the latter cannot be foretold nearly so close as 2 per cent. Thus the sanitary sewage would have no appreciable effect upon the size of the combined sewer, and can be neglected.

59. Minimum Sizes of Storm and Combined Sewers. In the case of sanitary sewers, 8 inches was stated to be the minimum allowable diameter (see Art. 47); but in the case of sewers carrying storm sewage, there is much greater danger of stoppages from dirt, sticks, and other debris washed in from the surface during storms. Hence *twelve inches should be the minimum allowable diameter for storm and combined sewers.*

60. Minimum Grades and Velocities for Storm and Combined Sewers. It was stated in connection with sanitary sewers (Art. 48), that the minimum allowable velocities to prevent deposits should be

TABLE VII

Minimum Grades for Storm and Combined Sewers

Shape	Material	Size	Minimum Grades to Give Velocities of 3 ft. per Sec.	Minimum Grades to Give Velocities of 4 ft. per Sec.
Circular	Pipe	12-in. Diam.	0.48	0.88
"	"	15 "	0.34	0.62
"	"	18 "	0.25	0.47
"	"	24 "	0.17	0.31
"	"	30 "	0.13	0.23
"	Brick or Concrete	3-ft. "	0.14	0.25
"	"	4 "	0.10	0.17
"	"	5 "	0.07	0.12
"	"	6 "	0.06	0.10
"	"	7 "	0.05	0.08
"	"	8 "	0.04	0.06
"	"	9 "	0.03	0.05
"	"	10 "	0.025	0.045
Egg-Shaped	"	2 ft. × 3 ft.	0.20	0.35
"	"	$2\frac{1}{2}$ " × $3\frac{3}{4}$ "	0.15	0.26
"	"	3 " × $4\frac{1}{2}$ "	0.12	0.20
"	"	4 " × 6 "	0.08	0.14
"	"	5 " × $7\frac{1}{2}$ "	0.06	0.10
"	"	6 " × 9 "	0.05	0.08
"	"	7 " × $10\frac{1}{2}$ "	0.04	0.07

$1\frac{1}{4}$ feet per second at the minimum depths of flow, which will require grades sufficient to give minimum velocities of 2 feet per second when the sewer flows full or half-full. For sewers carrying storm sewage, however, greater minimum velocities are necessary to prevent deposits, on account of the dirt, pebbles, and other heavy rubbish washed into them from the surface in times of storms. *For combined and storm sewers the minimum allowable grades should be steep enough to give a minimum velocity of 3 feet per second. If practicable without too great expense, 4 feet per second should be secured.*

61. General Explanation of the Calculation of Amount of Storm Sewage. When rain begins to fall upon the area drained by a storm sewer, the water falling in the immediate neighborhood of the outlet at once enters the sewer and begins to be discharged. As time passes and the rain continues, water arrives at the outlet from more and more remote portions of the drainage area, and the discharge at the outlet increases quite rapidly until water is being discharged from all portions of the drainage area at the same time. After that, any further increase is slow, being due only to a per cent of run-off slowly increasing as the saturation of the soil becomes more complete.

The *time of concentration* is the longest time required for water from the remotest points of the portion of the drainage area being considered, to reach the outlet of that portion.

The *general law of the heaviest rainfalls,* the ones which determine the sizes of sewers, is that the heaviest rates for short storms are much greater than the heaviest rates for long storms. *The longer the time, the less will be the average rate of the maximum storm lasting that time.*

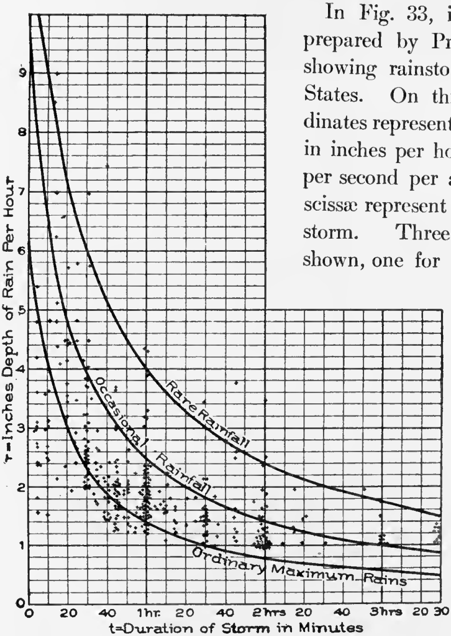


Fig. 33. Rates of Heavy Rainfall in the North Central States, Ohio, Indiana, Illinois, Missouri, Kansas, and Iowa.

In Fig. 33, is given a diagram prepared by Prof. A. N. Talbot, showing rainstorms in the Central States. On this diagram the ordinates represent the rate of rainfall in inches per hour (which = cu. ft. per second per acre), while the abscissæ represent the duration of the storm. Three curves are also shown, one for very rare rainfalls, one for ordinary heavy rains, and one intermediate. On the diagram each + represents o e storm.

The storm causing the greatest rate of discharge in a storm sewer will usually be the maximum rain lasting a length of time equal to the time of concentration. If a time less than this be taken, water will not be discharged at the outlet from all parts of the drainage area at once, and that from near the outlet will have a chance to run away before that from the remotest points arrives. On the other hand, if a time be taken longer than the time of concentration, the *heaviest* rate of the maximum storm lasting this long will be less than the rate of the maximum storm lasting a length of time just equal to the time of concentration; and since the storm is lighter the flow will be lighter.

Not all of the water falling on a drainage area will be carried away in the sewer. During and after the storm, some of the water is evaporated into the air, and some is absorbed into the soil. Some also accumulates on the surface, to flow off into the sewer after the rain has ended. *The engineer determines the percentage of the rain flowing off in the sewer, by estimating the percentage of maximum run-off of the drainage area.*

The general method for calculating the amount of storm sewage for any particular drainage area, is therefore as follows:

(*a*) Calculate the *time of concentration,* or longest time of flow to the point for which the size of sewer is being determined.

(*b*) Calculate the *rate of maximum rainfall* corresponding to the time of concentration.

(*c*) Calculate the *percentages of impervious and pervious areas* on the watershed drained by the sewer.

(*d*) Using the percentages of impervious and pervious areas obtained in *c*, calculate the *maximum percentage of run-off,* or the percentage of the rate of the maximum rainfall which will be running off in the sewer under design at the end of the time of concentration.

(*e*) *Calculate the total maximum rate of flow of storm sewage,* by *multiplying together* the *drainage area,* the *maximum rate of rainfall* corresponding to the time of concentration, and the *maximum percentage of run-off.*

62. Calculation of the Time of Concentration. The time of concentration, which is the longest time required for water falling on the remote portions of the watershed to flow to the point for which the size of sewer is being determined, will be the sum of, (1), the time required for the water from roofs, yards, sidewalks, and pavements to reach the sewers by way of the gutter and street inlets, and, (2), the longest time required for the water to flow through a line of sewers to the point for which the size of sewer is being calculated.

(1) *Time Required for Water from Roofs, Gutters, etc., to Reach the Sewers.* This will usually be between the limits of 5 and 15 minutes, depending on the steepness of the slopes of the surface and of the gutters, on the distance the water must flow to reach the gutters and the distance it must flow in the gutters to reach the street inlets, on the character of the surface (whether it offers obstructions to flow or not), or whether the roofs are connected to the gutters or directly to

the sewers, etc. By looking over the ground carefully, and allowing for the above conditions in a general way, the time may be estimated as closely as the data will warrant, without special calculations. The upper limit of 15 minutes may be used when the gutters have a very light grade, and are two blocks long, and where the roofs discharge into the gutters instead of into the sewer direct.

(2) *Longest Time Required for the Water to Flow through the Sewers.* This is computed by taking the grades and sizes of the different parts of usually the longest line of sewers, and determining the corresponding velocities of flow by the use of the sewer diagrams, Figs. 27, 28, and 29, already given. From these velocities, and the lengths of the several portions of the sewer, the corresponding times required for the sewage to flow through each part can be readily computed, and their sum will be the time required. The designing must be begun at the upper ends of the sewers, so that we may know the sizes of sewer needed in computing the times of flow through each portion.

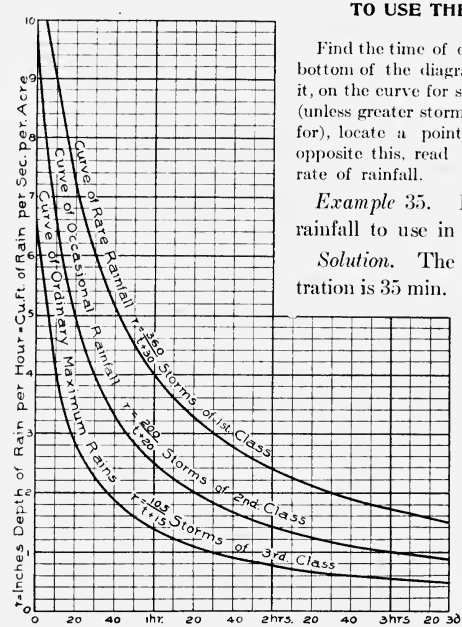
Example 34. Required the time of concentration in the following case: The longest sewer consists of 400 feet of 18-inch pipe sewer, grade 0.5 per cent; 800 ft. of 24-inch pipe, grade 0.3 per cent; 1,200 ft. of 36-inch brick sewer, grade 0.25 per cent; 2,400 ft. of 48-inch brick sewer, grade 0.17 per cent. The roofs discharge into the gutters, through which the sewage must flow 2 blocks at 0.5 per cent grade to reach a street inlet.

Solution:

	Velocity	Time
Estimated for water from roofs and gutter to reach sewer		15.0 min.
In 18-inch sewer, Fig. 27	4.2 ft. per sec.	1.6 "
" 24-inch " " 27	4.0 " " "	3.3 "
" 36-inch " " 28	4.0 " " "	5.0 "
" 48-inch " " 28	4.0 " " "	10.0 "
Answer. Total time of concentration =		35 "

63. Calculation of the Rate of Rainfall Corresponding to the Time of Concentration. In Fig. 34 are reproduced separately the three rainfall curves shown in Fig. 33. Storms of the 1st and 2d classes are rare, and are so very heavy that it would be excessively expensive to build sewers large enough for them. Hence sewers are usually built only large enough to provide for storms of the 3d class.

It is considered less expensive to suffer some damage from rare overcharging of the sewers than to build the greater sizes, though in case very valuable property would be damaged it may be wiser to provide for the heaviest storms.



t=Duration of storm in minutes=Time of concentration=Time required for water to flow from the remotest part of the area drained to the point under consideration on the sewer.

Fig. 34. Diagram Showing Rates of Maximum Rainfall to be Used in Calculating the Size of Storm Sewers.

TO USE THE DIAGRAM

Find the time of concentration at the bottom of the diagram. Vertically over it, on the curve for storms of the 3d class (unless greater storms are to be provided for), locate a point; and horizontally opposite this, read off on the left the rate of rainfall.

Example 35. Find the rate of rainfall to use in example 34.

Solution. The time of concentration is 35 min. Over this we read on the curve for 3d-class storms, 2.1 inches per hour.

Answer. 2.1 inches per hour.

64. Calculation of the Percentages of Impervious and Pervious Areas on the Sewer Watershed. The percentage of impervious area may be calculated in the following manner:

Take a typical unit of area, usually one average block, and divide it into different classes of surfaces, having different percentages of imperviousness, as follows:

(*a*) *Roof Area.* From the average size of buildings, and the average number of buildings per block which will be connected with the sewers or with the gutters, calculate the total roof area in the block. Take this at its full value if the roofs are connected directly with the sewers, but take only 90 per cent if the roofs are connected with the gutters.

(*b*) *First-Class Pavements.* Calculate the total area, per block, of brick, asphalt, stone block, and similar first-class pavements, with tight joints, and take 80 per cent of this area.

(*c*) *Second-Class Pavements.* Calculate the total average area per block, and take 60 per cent.

(*d*) *Third-Class Pavements.* Calculate the total average area per block of good macadam and similar pavements, and take 40 per cent.

(*e*) *Hard-Earth Roads.* Calculate the total average area per block of the traveled, hard-earth surfaces, and take 20 per cent.

(*f*) *Sidewalks.* Calculate the several total average areas per block of 1st, 2d, and 3d-class sidewalks, corresponding to the classes of pavements in *b*, *c*, and *d*, above. If these extend to the gutters, as in business districts, take the same percentages as for the corresponding classes of pavements—namely, 80, 60, and 40 per cent for 1st, 2d, and 3d-class sidewalks, respectively. But if the pavements are separated from the gutters by wide parking, as in the residence districts, take only one-half the above percentages—namely, take 40, 30, and 20 per cent, for 1st, 2d, and 3d-class sidewalks, respectively.

Finally, add together all the reduced average areas per block (a, b, c, d, e, and f) obtained as above explained, and divide the sum by the total area of the typical block. The quotient will give the percentage of impervious area.

The percentage of pervious area is obtained by subtracting the percentage of impervious area from 100 *per cent.*

Example 36. In examples 34 and 35, assume the typical block to be 360 ft. square, center to center of streets, as follows:

Streets, 60 ft. wide; pavements, 30 ft. wide; asphalt on two streets; good macadam on the other two; cement sidewalks, 5 ft. wide, on all four streets.

One alley 20 ft. wide.

Lots, 12 in number, each 50 × 140 ft., each lot containing one house, the houses averaging 30 × 40 ft., the roofs connected with the gutter.

Calculate the percentage of impervious and pervious area.

Solution:

(*a*)	Roofs,	$30 \times 40 \times 12 \times .90 =$	12,960 sq. ft.
(*b*)	1st-Class Pavements,	$2 \times 15 \times 360 \times .80 =$	8,640 "
(*d*)	3d-Class Pavements,	$2 \times 15 \times 330 \times .40 =$	3,960 "
(*f*)	1st-Class Sidewalks,	$5 \times 1{,}210 \times .40 =$	2,420 "
	Total impervious area per block	$=$	27,980 sq. ft.

Total area of one block $= 360 \times 360 = 129{,}600$ sq. ft.

Answer. *Percentage of pervious area* $= \dfrac{27{,}980}{129{,}600} = 21.58$ per ct.

Percentage of pervious area $= 100 - 21.58 = 78.42$ per cent.

Mr. Emil Kuichling, M. Am. Soc. C. E., has calculated the percentages of impervious area in various cities of New York State, and his work has been repeated by Prof. H. N. Ogden,* who finds the percentage to vary with the intensity of population, as follows:

TABLE VIII

Approximate Percentages of Impervious Area in Cities

Population per Acre	Percentage of Impervious Area	Percentage of Pervious Area
5	4	96
10	9½	90½
15	15	85
20	20½	79½
25	26	74
30	31½	68½
35	37	63
40	42½	57½
45	47½	52½
50	52½	47½
55	58	42

Even very heavily populated sections in the largest cities will seldom have more than 80 to 85 per cent of impervious area.

Table VIII furnishes an easy method of making approximate estimates of the percentages of impervious area.

Example 37. In example 36, estimate the percentage of impervious area by Table VIII.

Solution. The typical block contains 129,600 sq. ft.; and $\dfrac{129{,}600 \text{ (sq. ft.)}}{43{,}560 \text{ (sq. ft.)}} = 3$ acres. The 12 houses at an average of 5½ persons per house, would give 66 persons per block = 22 per acre.

* *Sewer Design*, p. 62.

Referring to Table VIII we find by interpolating, $22\frac{3}{4}$ per cent of impervious area, as compared with 21.6 per cent obtained above by the more exact method.

65. Calculation of the Maximum Percentage of Run-Off. Not all of the rain falling on the impervious area of a watershed will run off during the storm. Small amounts are evaporated or absorbed at once, for no city surfaces are absolutely impervious. A larger amount goes to fill up small depressions in the surfaces. A still larger amount accumulates on the surfaces of the watershed, making its way toward the sewer, the amount so accumulated and its rate of movement increasing as the storm continues at the same rate, until finally an equilibrium of flow is established, and the rate of the run-off from the impervious area becomes practically 100 per cent of the rainfall. Thus, the shorter the storm, the less the percentage of run-off from the impervious area; and hence sewer watersheds having the smallest times of concentration are likely to have the smallest percentages of maximum run-off from the impervious areas.

The maximum downpours which determine the size of the sewer, are often preceded by lighter downpours which saturate and partially flood the watershed. Hence *it will probably never be allowable to assume less than 75 per cent as the percentage of maximum run-off from the impervious areas of a sewer watershed, even with very short times of concentration, and comparatively little damage from overcharged sewers.*

With long times of concentration (say 45 minutes or more), and wherever great damage would be caused by overcharged sewers, 100 per cent of maximum run-off from the impervious areas should be assumed.

In the case of long-continued storms, the pervious area becomes gradually saturated, until some run-off occurs from it also. In the case of storms lasting several hours, such as cause the great floods in rivers, this percentage of maximum run-off may be quite high; but for sewers, the times of concentration, and hence the duration of the maximum downpour, are comparatively short—rarely as long as one hour.

For soils of average porosity and for moderate slopes, the percentage of maximum run-off from the pervious areas may be assumed to range from 0, *for* 15 *minutes time of concentration, to, say,* 20 *for* 1 *hour's time of concentration. For porous, sandy soils and flat slopes, assume 0 to 50 per cent, and for very impervious soils and very steep slopes, 125 to 150 per cent of the above percentages of maximum run-offs from pervious areas.*

Example 38. In examples 36 and 37, assume that the territory is a residence district, with moderate slopes and clay subsoil. Estimate the percentage of maximum run-off.

Solution. Since the time of concentration is only 35 minutes, while the damage from overcharged sewers would not be so great as in a business district, we shall assume 90 per cent maximum rate of run-off from the impervious area. For the pervious area, we interpolate roughly between 0 per cent for 15 minutes, and 17 per cent for 1 hour, and assume 8 per cent maximum rate of run-off.

$.90 \times 21.6$ per cent $= 19.4$ per cent from impervious area.
$.08 \times 78.4$ " " $= \underline{6.3}$ " " " pervious "

Answer. Total $= 26$ per cent maximum rate of run-off.

66. Summary of Methods of Computing Sizes of Storm Sewers. We may now summarize the methods of computing the sizes of storm sewers, described above in Articles 61 to 65, inclusive, as follows:

(*a*) Calculate the *time of concentration* (Art. 62), or longest time of flow from the remote portions of the sewer watershed to the point for which the size of sewer is being calculated.

(*b*) Calculate the *maximum rate of rainfall* (Art. 63) corresponding to the time of concentration.

(*c*) Calculate the *percentages of impervious and pervious areas* on the sewer watershed (Art. 64).

(*d*) From the percentages of impervious and pervious areas, and knowledge of the characteristics of the sewer watershed, calculate the *percentage of maximum run-off* (Art. 65).

(*e*) Calculate the *maximum rate of flow of storm sewage*, by *multiplying together* the *area of the sewer watershed in acres*, the *maximum rate of rainfall in inches per hour* (*b*), and the *percentage of maximum run-off* (*d*). The product will be the *cubic feet per second of maximum storm sewage flow.*

(*f*) Knowing the grade of the sewer, refer to Fig. 27, or Fig. 28, or Fig. 29, according to the shape and material of the sewer, and determine the size of sewer required to carry the maximum flow of storm sewage (*e*) when flowing full.

Example 39. In examples 34 to 38, assume that the sewer watershed is 5,280 feet long by 800 feet wide, and that the grade of the circular brick outlet sewer is to be 0.15 per cent. Calculate the required diameter.

(*a*) The time of concentration = 35 min. (see Ex. 34).
(*b*) The rate of maximum rainfall = 2.1 in. per hr. (see Ex. 35).
(*d*) The percentage of maximum run-off = 26 (see Ex. 38).
(*e*) The drainage area $= \frac{5{,}280 \times 800}{43{,}560} = 97$ acres.

$$97 \times 2.1 \times .26 = 53 \text{ cu. ft. per sec.}$$
$$= \text{maximum flow of storm sewage.}$$

(*f*) Referring to Fig. 28, we find, by interpolating between the 4-foot and 5-foot diameters, that for a grade of 0.15 per cent a diameter of 4 ft. 3 in. will be required for a circular brick sewer which can carry 53 cu. ft. per sec.

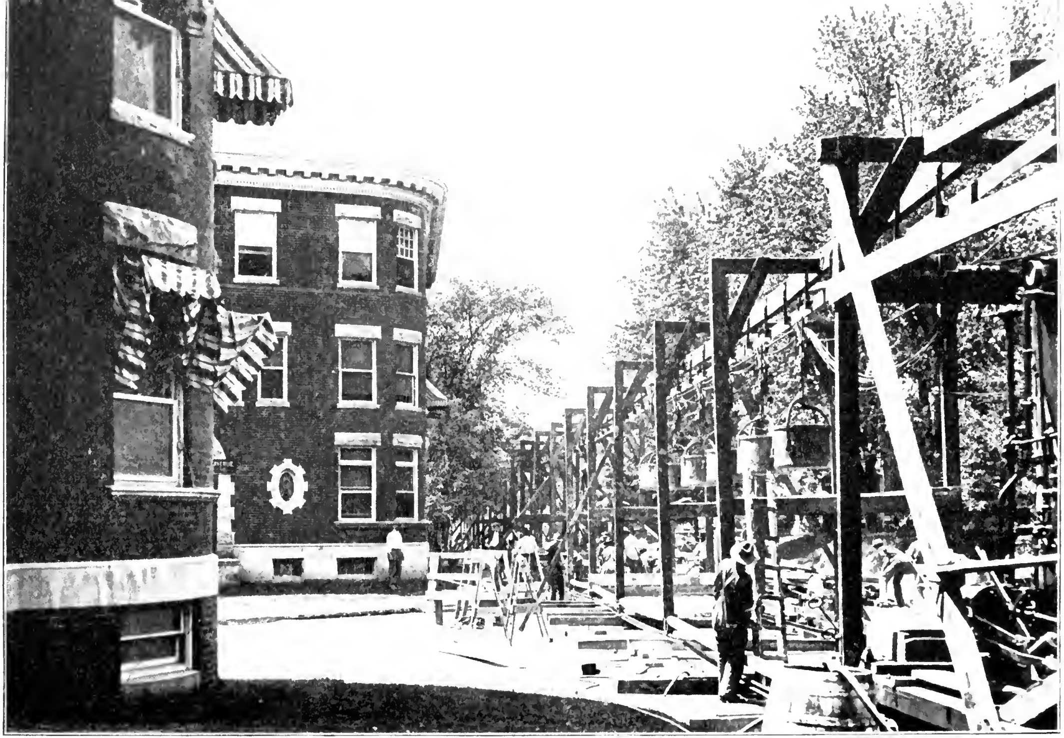
Answer. A 4 ft. 3 in. circular brick sewer.

GENERAL EXAMPLE FOR PRACTICE

67. Before proceeding further, the student should work out the following example in computation of the proper size of sewer:

Example 40. A thickly built-up sewer district, having a population of 35 persons per acre, contains 160 acres. The slopes are very flat, and the soil is sandy and porous. The longest line of sewers is 6,000 feet; and the velocity of flow in the sewers averages four feet per second. The roofs are connected with the gutters, in which the longest flow is two blocks. Calculate the diameter of the circular, brick outlet sewer, laid to a 0.08 per cent grade (NOTE: Use Table VIII.)

Answer. A 6-foot circular brick sewer.



TRENCH MACHINE IN OPERATION ON GORHAM AVENUE, BROOKLINE, MASSACHUSETTS

SEWERS AND DRAINS

PART II

LAND DRAINS AND SUBDRAINS

68. General Discussion of Land Drains. Definitions of sewers and drains were given in Art. 1. Land drains have for their object the reclaiming of wet lands, to render them suitable for cultivation. The reclamation of wet lands also greatly improves the sanitary condition of the vicinity.

There are two principal kinds of land drains—namely, *tile drains*, or lines of agricultural drain tiles laid a few feet beneath the surface of the ground, to remove ground water; and *drainage ditches*, or open channels, made to serve as outlets for the tile drains and to drain ponds and remove surface water.

69. Planning and Construction of Land-Drainage Systems. When a tile drainage system is projected, a competent drainage engineer should at once be engaged to do the necessary surveying, plan the system, and pass on the construction.

The surveying will include the obtaining of data for a complete map of the system; and each drain should be staked out, stakes being set 50 feet apart, and an elevation taken with a good level at each stake. All the work should be checked.

The engineer should then prepare for the landowner a complete map of the system, to a scale of 200 to 400 feet per inch; also a sheet of profiles, including a profile of each drain, showing the depth and grade at all points. Without such map and profiles, knowledge of the system may be lost, and, on some future occasion, when very badly needed, may be unavailable.

The engineer should plan as simple and regular a tile system as possible, adopting long, parallel, straight lines of tile when practicable, with as few junctions as possible.

The grades may be very light in case of necessity, and short tile drains have worked well even at level grades; but the lighter the grade, the greater should be the care used in construction.

The minimum depths should usually be $3\frac{1}{2}$ to 4 feet. Shallower depths do not drain out the soil so thoroughly; and tile, if laid $3\frac{1}{2}$ to 4 feet deep, can be placed farther enough apart to more than make up for the cost of the greater depth.

The lines of tile should usually be placed from five to ten rods apart, depending on the soil—farthest apart in the most porous soil. The outlet should be built with special care; and a masonry wall should be constructed to hold the last length of tile.

For drainage ditches, careful surveys of the entire watershed must be made by a very competent engineer; and fully detailed plans and specifications must be prepared.

70. Contracts and Specifications for Tile Drains. The employer and the tile ditcher should sign a printed contract with detailed specifications, such as given herewith:

CONTRACT

It is hereby agreed between ...., employer, and ...., contractor, that the contractor shall, except for the furnishing of the tile along the ditch and the refilling of the ditch, entirely construct for the employer the following described drains:

....
....
....
....
....
....

It is further agreed that for the above work the employer shall pay the following prices:

....
....
....

It is further agreed that the employer ....furnish board free to the contractor and his helpers during active prosecution of the work.

It is further agreed that the contractor shall begin the work by ....and complete the same by ....

It is further agreed that all the above work and the payments therefor shall be in strict accordance with the specifications given below and with the engineer's maps, profiles, and plans, all of which are hereby made a part of this contract.

Witness the hands of the respective parties, this ....day of ....A. D.....

.......................Employer
.......................Contractor

SPECIFICATIONS

1. *Staking Out the Work.* The work will be staked out by the engineer, and his stakes must be carefully preserved and followed.

2. *Digging the Ditches.* The digging of each ditch must begin at its outlet, or at its junction with another tile drain, and proceed toward its upper end. The ditch must be dug along one side of the line of survey stakes, and about ten inches distant from it, in a straight and neat manner, and the top soil thrown on one side of the ditch and the clay on the other. When a change in the direction of ditch is made, it must be kept near enough to the stakes so that they can be used in grading the bottom. In taking out the last draft, the blade of the spade must not go deeper than the proposed grade line or bed upon which the tiles rest.

3. *Grading the Bottom.* The ditch must be dug accurately and truly to grade at the depths indicated by the figures given by the engineer, measured from the grade stakes. At each grade stake, a firm support shall be erected; and on these supports a fine, stout cord shall be tightly stretched over the center line of the ditch and made parallel with the grade by careful measurements at each stake, using a carpenter's level. Supports shall be kept erected at at least three grade stakes, and the work checked each time by sighting over them. Intermediate supports shall be set and lined in by careful sighting wherever necessary, to support the cord every 50 feet. A suitable measuring stick shall be passed along the entire ditch, and the bottom in all parts made true to grade by measuring from the cord. The bottom must be dressed with the tile hoe, or, in the case of large tiles, with the shovel, so that a groove will be made to receive the tile, in which the tile will remain securely in place when laid.

4. *Laying the Tile.* The laying of the tile must begin at the lower end and proceed upstream. The tile must be laid as closely as practicable, and in lines free from irregular crooks, the pieces being turned about until the upper edge closes, unless there is sand or fine silt which is likely to run into the tile, in which case the lower edge must be laid close, and the upper side covered with clay or other suitable material. When in making turns, or by reason of irregular-shaped tile, a crack of one-fourth inch or more is necessarily left, it must be securely covered with broken pieces of tile. Junctions with branch lines must be carefully and securely made.

5. *Blinding the Tile.* After the tile have been laid and inspected by the employer or his representative, they must be covered with clay to a depth of six inches, unless, in the judgment of the employer or his representative, the tile are sufficiently firm so that complete filling of the ditch may be made directly upon the tile. In no case must the tile be covered with sand without other material being first used.

6. *Risk During Construction.* The ditch contractor must assume all risks from storms and caving-in of ditches; and when each drain is completed, it must be free from sand and mud before it will be received and paid for in full. In case it is found impracticable, by reason of bad weather or unlooked-for trouble in digging the ditch or properly laying the tile, to complete the work at the time specified in the contract, the time may be extended as may be mutually agreed upon by the employer and contractor. The contractor shall use all necessary precaution to secure his work from injury while he is constructing the drain.

7. *The Tile to be Used.* Tile will be delivered on the ground convenient for the use of the contractor. No tile shall be laid which are broken, or soft, or so badly out of shape that they cannot be well laid and make a good, satisfactory drain.

8. *Prosecution of the Work.* The work must be pushed as fast as will be consistent with economy and good workmanship, and must not be left by the contractor for the purpose of working upon other contracts, except by permission and consent of the employer. All survey stakes shall be preserved, and every means taken to do the work in a first-class manner.

9. *Subletting Work.* The contractor shall not sublet any part of the work in such a way that he will not remain personally responsible, nor shall any other party be recognized in the payment for work.

10. *Plant and Tools.* The contractor shall furnish all tools which are necessary to be used in digging the ditches, grading the bottom, and laying the tile. In case it is necessary to use curbing for the ditches, or outside material for covering the tile where sand or slush is encountered, the employer shall furnish the same upon the ground convenient for use.

11. *Payments for Work.* Every............weeks during the prosecution of the work, the contractor may claim and the employer shall pay 75% of the value of the work completed satisfactorily, the engineer being the arbiter in case of dispute as to the amount of work satisfactorily completed. The remaining 25% will be retained until the entire work is completed satisfactorily, as certified by the engineer after a final inspection, at which time the whole amount due shall be paid. Prior to any payment, the employer may require a correct statement of all claims incurred by the contractor for labor, materials, or damages on account of the work; and the employer may withhold payments until proof has been presented by the contractor of release of all liens against the employer on account of such claims.

12. *Duties of Engineer.* The engineer shall have authority to lay out and direct the work, and to inspect and supervise the same during construction and on completion, to see that it is properly done in accordance with the contract. His instructions should be fully carried out.

13. *Failure to Comply with Specifications.* In case the contractor shall fail to comply with the specifications, or refuse to correct faults in the work as soon as they are pointed out by the engineer or other person in charge, the employer may declare the contract void; and the contractor, upon receiving seventy-five per cent of the value of the completed drains at the price agreed upon, shall release the work and the employer may let it to other parties.

71. Benefits of Tile Drains. The advantages of tile drains may be enumerated as follows:

1. Tile drainage, by making the soil firm, enables earlier cultivation in the spring. Low ground drained can be cultivated earlier than high ground not drained.

2. Careful observations have shown that tile drainage makes the soil several degrees warmer in the spring. Scientific tests have

shown this increased warmth to be of the utmost importance in promoting the germination and growth of crops.

3. Tile drainage promotes pulverization of the soil, putting it in good condition to cultivate, and preventing baking and the formation of clods.

4. Tile drainage removes from the pores of the soil surplus and stagnant water, which would drown and destroy the roots of plants.

5. Tile drainage makes certain the proper "breathing" of the soil, or free circulation of air in its pores, which is essential to healthy plant growth.

6. Tile drainage establishes in the soil the proper conditions required for the satisfactory carrying on of the chemical processes necessary to prepare the plant food for its use by vegetation.

7. Tile drainage fits the soil for the vigorous life and action of the soil bacteria which are essential to preserve and increase its fertility and promote the growth of crops.

8. Tile drainage increases the depth of soil which can be reached by the roots of plants and drawn upon for plant food.

9. Because in them the roots of plants can penetrate deeper, where they are protected from heat and drouth and can reach the deep-seated moisture, tile-drained soils stand drouth better than undrained soils.

10. By putting the top 3-feet or 4-feet layer of soil into a porous condition, tile drainage enables soils to absorb rain water instead of discharging it over the surface, and so helps to prevent surface wash and consequent loss of fertility.

11. By causing this porous condition, tile drainage makes the upper 3 or 4 feet of soil into an enormous reservoir to catch the rain water and discharge it only slowly into the streams. Thus tile drainage prevents floods instead of causing them.

12. Tile drainage does away with irregular shaped fields, cut up by sloughs and ditches, and so cheapens cultivation.

Benefits of Large Ditches. Tile drainage is always preferred to open-ditch drainage if the drain is not too large. The advantages of large ditches may be enumerated as follows:

1. By furnishing channels to remove storm water, they prevent, if of ample size, the inundation of low-lying lands by floods and surface water.

2. They have a minor value for draining off the ground water from a narrow strip of land each side.

3. One of their main values is in furnishing outlets for tile drains, and in many places tile drainage is impracticable till outlet drainage ditches have been built.

72. Method of Computing Sizes of Tile Drains. The drained soil above the level of tile drains contains a large percentage of air-space in the pores between the soil particles; and this layer of porous soil acts like a great sponge several feet thick to absorb the rain as it falls. Hence the water reaches the tiles very slowly. It has been found that under average conditions tiles will not be called upon to carry more than ¼-inch depth of water in 24 hours. This equals 6,800 gallons per acre per day, or 4,352,000 gallons per square mile per day. The sizes of tile drains for average conditions may readily be taken from Table IX.

TABLE IX

Number of Acres Drained by Tiles Removing ¼-Inch Depth of Water in 24 Hours

Grades		Diameters of Tile Drains										
Per cent	Inches per rod	3 in.	4 in.	6 in.	8 in.	10 in.	12 in.	15 in.	18 in.	20 in.	22 in.	24 in.
0.03	$\frac{1}{16}$					37	59	109	159	205	254	319
0.05	$\frac{3}{32}$		5	13	28	49	75	131	219	264	332	411
0.10	$\frac{3}{16}$	4	7	19	40	69	109	186	289	373	471	582
0.15	$\frac{9}{32}$	4	9	24	49	85	132	232	355	458	577	713
0.25	$\frac{3}{8}$	5	10	28	56	97	153	264	410	529	667	823
0.30	$\frac{9}{16}$	6	12	33	69	119	188	322	502	648	808	1,008
0.40	$\frac{13}{16}$	7	14	39	79	138	216	371	580	748	942	1,165
0.50	1	8	16	44	89	154	246	416	648	838	1,050	1,300
0.60	$1\frac{3}{16}$	9	17	48	97	169	266	457	710	911	1,154	1,422
0.70	$1\frac{3}{8}$	10	19	50	105	182	287	488	768	988	1,242	1,549
0.80	$1\frac{9}{16}$	10	20	55	114	195	307	526	822	1,059	1,332	1,645
0.90	$1\frac{3}{4}$	10	21	59	119	207	326	558	872	1,123	1,414	1,747
1.00	2	11	22	62	126	218	343	589	917	1,176	1,495	1,838
1.50	3	13	28	75	153	267	419	722	1,123	1,450	1,824	2,256
2.00	4	15	31	88	178	309	485	832	1,297	1,676	2,110	2,594
3.00	$5\frac{15}{16}$	19	39	107	216	377	593	1,020	1,589	1,957	2,592	
4.00	$7\frac{15}{16}$	22	45	123	253	437	683	1,176				
5.00	$9\frac{7}{8}$	25	50	138	280	486	765					
7.50	$14\frac{7}{8}$	30	61	169	344							
10.00	$19\frac{13}{16}$	35	71	195								

Table IX is computed from the form of Poncelet's formula recommended for use with tile drains by C. G. Elliott, drainage expert to the U. S. Agricultural Department, Washington, D. C., who recommends the above sizes to drain ground water only. If surface water is also to be removed, as in the case of ponds without other outlets, the tiles will drain safely only one-half to one-third the number of acres given in the table.

When part of the land in the watershed is rolling, not requiring tiling, count only one-fifth to one-third of such rolling land, in addition to all of the low, flat land, in getting the size of tiles to remove ground water only.

Example 41. What size of tile laid to a 0.1 per cent grade will carry the under-drainage of 160 acres of flat land?

Answer. 15 inches.

Example 42. What size of tile to a 0.2 per cent grade will carry the under drainage of 240 acres, two-thirds rolling?

Answer. 80 acres flat land, *plus* one-third of 160 acres rolling, gives $133\frac{1}{3}$ acres, requiring a 12-inch tile.

Example 43. What size of tile laid to 0.3 per cent grade will be required to remove both ground and surface water from a pond whose watershed includes 40 acres?

Answer. 10-inch. (NOTE.—Double or triple the area for both ground and surface water.)

73. Method of Computing Sizes of Drainage Ditches. Since drainage ditches must carry surface water as well as ground water, their capacities must be larger than those of tile drains for the same number of acres drained. It has been found by experience that they must carry from $\frac{3}{4}$-inch depth for small drainage areas, to $\frac{1}{4}$-inch depth for large drainage areas per day. Their size can be taken from Table X.

Example 44. What width of ditch, having a fall of 5 feet per mile, and a depth of water of 3 feet, will be required to drain an area of 5 square miles (3,200 acres)?

Answer. About 12 feet.

Example 45. What size ditch having a fall of 3 ft. per mile, and 9 ft. depth of water, will drain an area of three townships (69,120 acres)?

Answer. About 22 feet.

74. Method of Computing Sizes of Subdrains for Sewers. Sewer subdrains act like tile land drains to remove the ground water from the soil. Being deeper, they will drain wider strips of land—say averaging 16 rods wide, instead of 8 rods, for ordinary land drains in average soil; but also, owing to the greater depth, the water will reach the tiles more slowly, and this may offset the greater width drained. We may assume roughly that each subdrain may be called upon to remove $\frac{1}{8}$-inch depth of water per day from a strip 16 rods wide, *which is the same thing as $\frac{1}{4}$-inch depth per day from a strip of land 8 rods wide.*

TABLE X

Number of Acres Drained by Open Ditches

Depth of Water, 3 feet.							Depth of Ditch, at least 4 feet.		Depth of Water, 5 feet.				Depth of Ditch, at least 6.5 feet.		
GRADE		AVERAGE WIDTH OF WATER								AVERAGE WIDTH OF WATER						
Per cent	Feet per Mile	4 feet	6 feet	8 feet	10 feet	15 feet	20 feet	30 feet	50 feet	6 feet	8 feet	10 feet	15 feet	20 feet	30 feet	50 feet
0.02	1.0			725	970	1,570	2,240	5,300	18,400	980	1,470	1,900	5,000	7,150	23,800	43,800
0.04	2.1	400	690	1,000	1,360	2,250	4,700	7,470	26,100	1,390	2,090	2,800	7,200	20,400	33,500	62,500
0.06	3.2	492	850	1,260	1,690	2,770	5,770	18,400	31,900	1,710	2,560	5,100	17,600	21,700	40,800	75,500
0.08	4.2	572	980	1,460	1,950	4,820	6,670	21,400	37,400	1,980	2,980	6,100	20,400	30,000	48,800	88,000
0.10	5.3	636	1,100	1,630	2,180	5,360	7,140	23,700	41,400	2,220	5,010	7,600	23,400	33,400	54,500	98,000
0.15	7.8	791	1,330	2,010	2,670	6,600	19,000	30,200	52,100	2,720	6,300	17,100	28,700	40,500	66,700	120,000
0.20	10.6	905	1,560	2,310	4,720	7,870	21,800	35,000	60,300	4,820	7,300	19,500	33,000	47,000	77,000	139,000
0.25	13.2	1,020	1,740	2,660	5,300	17,500	24,600	39,000	67,700	5,370	16,300	21,900	37,500	53,000	86,000	155,000
0.30	15.8	1,100	1,970	2,900	5,850	19,400	26,800	42,700	74,000	5,900	17,900	23,900	40,700	57,000	94,000	170,000
0.40	21.1	1,300	2,290	5,050	6,740	22,200	30,800	49,100	85,700	6,830	20,600	27,700	47,000	67,000		
0.50	26.4	1,475	2,550	5,620	7,500	24,800	31,800	55,300	95,200	7,600	23,000	31,000				
0.60	31.7	1,600	2,790	6,230	16,500	27,200	37,700	60,400		16,700	25,200	33,900				
0.70	37.0	1,720	3,010	6,650	17,800	29,400	41,200			18,100	27,300					
0.80	42.2	1,850	4,850	7,170	19,100					19,000						
0.90	47.5	1,955	5,140	7,550	20,100					20,500						
1.00	52.8		5,400													

TABLE X—(Concluded)

Number of Acres Drained by Open Ditches

Depth of Water, 7 feet. Depth of Ditch, at least 9 feet.								Depth of Water, 9 feet. Depth of Ditch, at least 11.5 feet.				
Grade		Average Width of Water						Average Width of Water				
Per cent	Feet per mile	8 feet	10 feet	15 feet	20 feet	30 feet	50 feet	10 feet	15 feet	20 feet	30 feet	50 feet
0.02	1.0	2,300	4,700	16,600	28,000	48,000	88,500	6,550	27,800	40,800	69,500	127,000
0.04	2.1	4,850	6,740	23,400	35,400	58,000	106,000	18,500	34,400	50,000	83,500	157,000
0.06	3.2	5,920	17,000	29,600	43,400	72,000	129,000	22,600	41,600	61,000	103,000	193,000
0.08	4.2	6,940	19,100	34,200	50,000	83,000	150,000	26,300	48,300	71,000	120,000	221,000
0.10	5.3	7,720	21,800	38,400	56,000	92,600	167,000	30,400	54,000	79,100	132,000	244,000
0.15	7.8	19,400	27,000	47,200	68,500	112,000	202,000	37,300	66,100	96,200	162,000	298,000
0.20	10.6	22,400	31,300	54,200	78,700	130,000	235,000	42,900	76,200	104,000		
0.25	13.2	25,000	34,800	60,500	88,000	146,000		48,000	85,300	125,000		
0.30	15.8	27,400	38,200	66,200	96,500			52,500	93,200			
0.40	21.1	31,700	44,100					60,800				
0.50	26.4	35,400										

Table X, for open ditches, is calculated by the well-known standard Kutter's formula, using a "coefficient of roughness" equal to 0.030. This coefficient of roughness is the value recommended by Kutter for channels in moderately good condition, having stones and weeds occasionally, and agrees with actual gaugings of drainage channels made at the Iowa State College. For ditches in first-class condition, the number of acres given may be increased about 25 per cent. The table has been calculated for ditches having sides with slopes of one foot horizontal to one foot vertical but is approximately correct for other slopes.

The capacity of the ditches has been made as recommended by C. G. Elliott, U. S. Agricultural Department drainage expert, as follows, the ditches to run not more than $\frac{8}{10}$ full for the capacities mentioned:

Above the upper heavy line, $\frac{3}{4}$-inch depth of water per 24 hours
Between the two heavy lines, $\frac{1}{2}$-inch depth of water per 24 hours.
Below the lower heavy line, $\frac{1}{4}$-inch depth of water per 24 hours.

Local conditions may vary the size needed, and it is necessary to consult a drainage engineer in each case.

Hence the sizes required for sewer sub-drains may be taken from Table IX, calculating the number of acres drained by multiplying the total lengths of tributary drain tile, in feet, by 132 *feet* (= 8 *rods*), *and dividing the product by* 43,560 *sq. ft.*

The above method will give a capacity approximating 110,000 gallons per day per mile of tributary subdrains. As sewers are ordinarily distributed, it will give a capacity approximating 1,500,000 gallons per day per square mile of territory served by the sewers.

Example 46. Calculate the size of subdrains laid to a 0.25 per cent grade, required to serve as outlet for 30,000 linear feet of tributary subdrains.

Solution: $\dfrac{30{,}000 \times 132}{43{,}560} = 91$ acres = equivalent area drained for $\frac{1}{4}$-inch depth.

In Table IX, opposite the 0.25 per cent grade, we find that a 10-inch tile would be required.

Answer. 10-inch tile subdrain.

75. Cost of Tile Land Drains and Drainage Ditches. The cost of tile-drain construction in central Iowa in 1904, can be approximated from Table XI. Local prices should be determined before using the table for close estimates of work done elsewhere.

TABLE XI
Cost of Tile Drains

Size of Tile	Price per 1,000 Feet	Weight per Foot	Cost of Hauling 1,000 Feet 5 Miles	Cost of Digging and Laying, per Rod: 3 feet deep or less	Add per foot for additional depth over 3 feet: 3–6 ft.	Add per foot for additional depth over 3 feet: over 6 ft.	Refilling, per Rod
3 in.	$ 16.00	5	$ 3.12	$ 0.35	$ 0.15	$ 0.30	2c.-5c.
4 in.	22.00	8	5.00	0.35	0.15	0.30	2c.-5c
5 in.	30.00	10	6.25	0.35	0.15	0.30	2c.-5c.
6 in.	40.00	12	7.50	0.35	0.15	0.30	2c.-5c.
7 in.	50.00	15	9.37	0.35	0.20	0.35	2c.-5c.
8 in.	60.00	20	12.50	0.40	0.20	0.35	2c.-5c.
10 in.	95.00	30	18.75	0.45	0.20	0.35	2c.-5c.
12 in.	120.00	40	25.00	0.50	0.20	0.35	2c.-5c.
15 in.	250.00	50	31.25				
18 in.	400.00	80	50.00				
20 in.	600.00	100	62.50				
24 in.	800.00	125	78.12				

The cost of hauling given in Table XI is on the basis of \$1.25 per ton, or \$2.50 per day for a man and team, making two trips.

The prices for digging and laying given above include board furnished by the ditcher. If the farmer furnishes board, deduct about 20 per cent. The prices for digging and laying are for average ground, and should be increased for quicksand or very wet soils.

N. B. To all estimates it is wise to add 5 per cent to 10 per cent for contingencies and engineering.

Example 47. What will be the cost of 2,000 feet of 6-in. tile drain, 2½ miles from the tile yard, of which 1,000 feet is 4 feet deep, 500 feet 5 feet deep, and 500 feet 6 feet deep, in average soil?

Answer:

2,000 ft. of 6 in. tile @ \$40.00	\$80
Hauling 2,000 ft. 2½ miles, @ \$3.75	7½
Digging and laying 60.6 rods 4 ft. deep, @ 50c	30½
" " " 30.3 rods 5 ft. deep, @ 65c	19½
" " " 30.3 rods 6 ft. deep, @ 80c	24
Refilling 121.2 rods (by team), @ 2c	2½
	\$164
Add 10 per cent for engineering, etc.	16
Estimated cost	\$180

Cost of Open Drainage Ditches. The cost of open drainage ditches is estimated by the cubic yard.

To calculate the number of cubic yards per foot of length of ditch, multiply the average width by the average depth, and divide by 27. Thus a 7-ft. by 12-ft. ditch contains $\frac{7 \times 12}{27} = 3\frac{1}{9}$ cubic yds. per foot length.

The cost per cubic yard in Iowa varies from 7c. to 18c., depending on the size of the job, the character of the soil, and other local conditions, including the certainty of the contractor getting his money promptly. The larger the work, the less is the cost per cubic yard.

HOUSE SEWERAGE

76. Definitions and General Description. A *house sewer* is a small branch sewer which connects the house with the street sewer. In Fig. 6 a general view of a house sewer is given.

A *soil pipe* is the main drainage pipe of the system of house plumbing, into which the different fixtures discharge. See Fig. 35.

A *trap* is a bend or depression in a pipe or drain, which remains constantly full of liquid, thus shutting off air-connection between the portions of the pipe or drain on opposite sides of the trap. See Fig. 35.

A general idea of an entire system of house sewerage can be obtained from Figs. 6 and 35, which see.

The house sewer and outlet for the cellar and foundation drains, extend from the street sewer to the house as shown in Fig. 6.

The iron soil pipe should begin a few feet outside the house, and extend full size through the roof, the separate fixtures discharging into the soil pipe, each protected by a trap, and all traps being vented, as shown in Fig. 35. The dotted lines in Fig. 35 show alternative plans sometimes adopted for house sewerage.

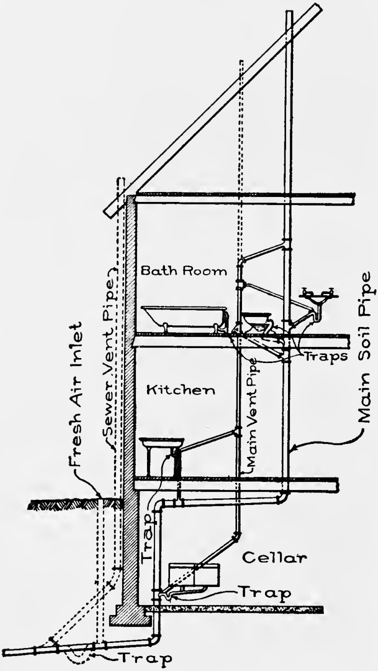


Fig. 35. Diagram of House Sewerage System.

77. House Sewers. House sewers (see Fig. 6) are usually made of vitrified sewer pipe the same as street sewers, and should be constructed with fully as much care. The joints should have gaskets of hemp or oakum, and be carefully cemented, the same as street sewers. (See Art. 33.)

Each piece of pipe should be laid to the exact grade by measuring from a grade string, the same as for street sewers (see Art. 98). The grade should usually be not less than 2 per cent. The house sewer should, if possible, be perfectly straight, both in alignment and in grade, from the house to the house connection at the sewer.

Inspection pipes should be placed just inside the lot line, as indicated in Fig. 6.

House sewers should usually be 4-inch circular pipe. If too large, they are more difficult to keep flushed clean, and they may carry to the street sewer things large enough to cause stoppages, improperly put into the house fixtures. Sometimes 5-inch or 6-inch house sewers are used.

78. General Principles of House Plumbing. The following general principles should be carefully observed in the installation of all house plumbing:

1. The iron pipe should begin a few feet outside the house, as vitrified pipe does not have tight joints and is liable to be broken, where it passes through the foundation wall, by uneven settlement.

2. No pipes carrying sewage should be allowed to be buried under the basement floor, unless placed in masonry-lined trenches with removable covers.

3. All pipes of the plumbing system should be iron or lead, with absolutely tight joints of lead, or screwed, or soldered.

4. In general, no pipes should be built into partitions or walls, where they cannot be gotten at, unless removable panels are placed over them.

5. All fixtures should be completely exposed to view, and should not be enclosed in woodwork. Sinks and washbowls, for example, should be supported on brackets or legs, with clear, open spaces under them.

6. All fixtures should be of durable, smooth, and non-absorbent material, such as porcelain or enameled iron. The least possible woodwork should be used.

7. All fixtures should be located in well-lighted and well-ventilated places.

8. Each fixture must be protected by a good trap. There must be no openings from the plumbing system into the interior of the house not thoroughly protected by traps sure to stay full of liquid.

9. Thorough ventilation of all pipes must be provided for.

10. All pipes must be laid to good grades, without sags, so as to drain completely and quickly.

11. The cellar and foundation drains should be connected with a sewer subdrain, if possible, and not with a sewer, owing to the danger of the water in the traps evaporating in dry weather when no water runs in the drains. If absolutely necessary to connect to the sewer, excessively deep traps should be used, to lessen the danger of evaporation.

79. Soil Pipes. The iron soil pipe begins, as already stated, a few feet outside the foundation wall. At this point a *disconnecting trap* is sometimes placed, as shown by the dotted lines in Fig. 35, in which case a *fresh-air inlet* must be placed on the house side of the trap, as also shown by dotted lines in Fig. 35, to permit complete ventilation of the soil pipe.

The soil pipe should extend full-sized and without any obstruction, a few feet above the roof. It should everywhere be readily accessible, and will naturally be placed in the location most convenient for attaching the fixtures.

The soil pipe is usually 4 inches in diameter, made of cast iron, with air-tight, leaded and calked joints.

80. Traps. The best traps are simply *smooth bends* in the plumbing pipes, giving depressions which stand full of liquid. If the curves are not smooth, or if there are sudden changes in size, the danger of stoppage is increased. The depth from the highest level of the water in the trap to the top of the liquid in the lowest portion, is called the *seal* of the trap. Traps are necessary evils in plumbing systems, as they tend to cause stoppages.

The seals of traps may be forced by any compression or rarefaction of air in the plumbing pipes, such as may be caused by *plugs* of sewage from other fixtures descending the pipes, unless a *vent pipe* is extended from the *crown* or highest point of each trap on the side next to the soil pipe, as shown in Fig. 35.

Traps should be located as closely as possible to the fixtures they are to protect.

81. Ventilation. The vent pipes from the traps mentioned in Art. 80, above, and shown in Fig. 35, serve also to secure ventilation of branch pipes. They should unite in a *main vent pipe*, 2 inches in diameter, as shown in Fig. 35, and this may turn into the soil pipe above the highest fixture, or may extend independently above the roof, as shown by the dotted lines in Fig. 35.

The extension of the main soil pipe unobstructed through the roof, with admission of air from the sewer (or through the fresh-air inlet if a disconnecting trap is used), together with the trap vent pipes and the main vent pipe, as shown in Fig. 35, insure ventilation of all parts of the plumbing system.

COST OF SEWERS, AND METHODS OF PAYING FOR THEM

82. Preliminary Estimates of Cost of Sewers. One of the first things which the sewerage engineer will be asked about sewers for which he has made plans, is what will be their cost. He must be able to answer this question readily, and with close approximation to the actual cost.

Many factors affect the cost of sewers, some of which cannot be exactly foretold. Among the things which can be closely ascertained in advance, are the sizes, lengths, and depths of the sewer, and the amounts of the various kinds of materials required. Among the things which cannot be exactly foretold, are the nature of the soil, the amount of ground water to be encountered, the weather conditions, and the labor conditions.

The competent engineer will thoroughly study all conditions which may affect the cost, before preparing his estimates, and even then will allow a liberal percentage for contingencies.

The engineer should have borings made to determine the character of the soil and the level of ground water, and should learn all he can of previous experience in the town with ditches and other excavations. Even then the actual soil often proves very different from what was anticipated.

After making the preliminary study and plans, the engineer tabulates the sewers by lengths, depths, sizes, and character, together with the manholes, lampholes, flush-tanks, and other items of the system. He then assigns a unit price to each item, after careful study of all conditions, and calculates the total cost.

The data of cost which follow are for average conditions only, and only for the localities named. They will need to be modified by the engineer to meet different conditions.

83. Cost of Pipe Sewers. In estimates of the cost of pipe sewers, the work is usually divided into the following items:

(1) *Trenching and Refilling.* This includes excavating the trench for the sewer, refilling it, and compacting the material after the sewer pipe is laid. Trenching and refilling are usually itemized according to depth, thus:

Trenching and Refilling under 6 feet depth
" " " " 6 to 8 feet depth
" " " " 8 to 10 feet depth
Etc., etc.

The cost of trenching and refilling will vary somewhat also with the diameter of the sewer; but this is often not separately itemized.

For estimates and bids, the lengths in linear feet of each depth of sewer are taken from the profiles, and listed in the tabulation.

(2) *Furnishing Sewer Pipe and Specials.* The pipe are usually specified to be delivered on board cars at the town where they are to be used. The amounts are usually itemized according to the diameters, thus:

Furnishing	sewer	pipe	8	inches	diameter
"	"	"	10	"	"
"	"	"	12	"	"
etc., etc.					

Specials are sometimes itemized separately, and sometimes included in the prices for furnishing pipe, the average distance apart being specified.

For estimates and bids, the total lengths of each size of pipe are ascertained and listed in the tabulation.

(3) *Hauling and Laying Sewer Pipe and Specials.* This includes taking the sewer pipe from the cars, hauling them to the sewer, furnishing cement, sand, and hemp or oakum, and laying the pipe according to the specifications. Some labor in excavating bell holes and a few inches at the bottom of the ditch shaped to fit closely the under side of the pipe, is also included. Hauling and laying are usually itemized according to the diameters of the pipe, thus:

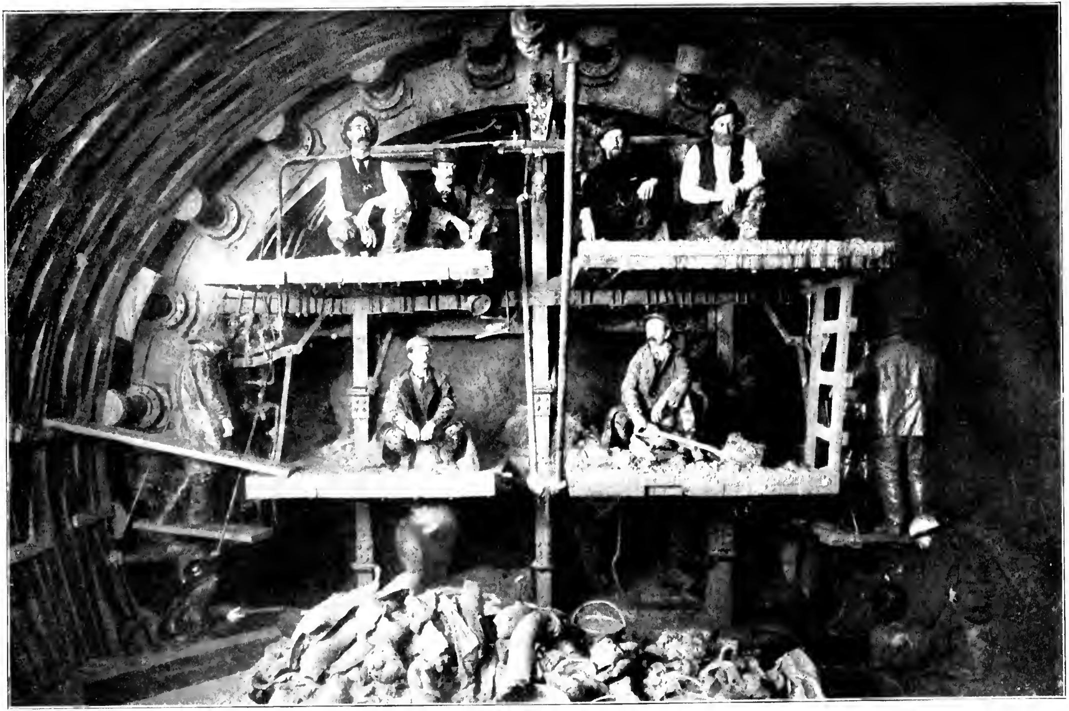
Hauling	and	Laying	sewer	pipe	and	specials,	8	inches	diameter
"	"	"	"	"	"	"	10	"	"
"	"	"	"	"	"	"	12	"	"
Etc., etc.									

The lengths of each size are listed for estimates and bids, the same as sewer pipe.

In Fig. 36 is given a diagram for estimating the cost of pipe sewers and subdrains in the Middle West. It may be used elsewhere by noting local conditions and their variation from the conditions assumed, as follows:

(*a*) If the sewers are to be paid for promptly as the work progresses, in cash instead of in assessment certificates, deduct about 10 per cent.

(*b*) Get actual prices on sewer pipe delivered, and add about 8 per cent for additional cost of specials in the average residence district, and 16 per cent in the average business district.



CONSTRUCTING THE LARGEST SEWER IN THE UNITED STATES

View of men at work in tunneling shield; wooden forms also shown. Sewer lined with brick laid in "Utica" hydraulic cement. View of 39th Street intercepting sewer, Chicago, Ill.

Courtesy of Meacham & Wright, Chicago, Ill.

(*c*) Ascertain the character of the soil, and the likelihood of encountering ground water. If the conditions are very favorable, the cost of trenching, refilling, and pipe laying may be materially decreased, even sometimes to 50 per cent of the figures shown in the diagram; while on the other hand, for very unfavorable conditions, the cost shown for these items will have to be increased, sometimes even to 150 per cent.

Example 48. Estimate the cost of a pipe sewer consisting of 1,200 ft. of 18-inch pipe averaging 16 feet deep, and 2,700 feet of 15-inch pipe averaging 12 ft. deep, under average conditions, together with a 6-inch subdrain.

Solution:

1,200 × 2.35 (from diagram)	=	$3,020	for 18-inch sewer
2,700 × 1.60 (" ")	=	4,320	" 15 " "
3,900 × 0.15 (" ")	=	585	" 6 " subdrain
Answer. Total estimated cost	=	$7,925	

84. Cost of Brick Sewers. The cost of a brick sewer may be estimated by determining separately the cost of the excavation and refilling and that of the brickwork. The number of cubic yards of each of these items is computed for 1 linear foot length of sewer; and the cost per linear foot is estimated by multiplying the results so obtained by estimated costs per cubic yard of excavation and brickwork respectively.

(1) *To calculate the number of cubic yards of excavation per linear foot length of sewer, multiply the average depth of sewer trench by the average width, and divide by 27.*

The *average depth* for a circular bottom will approximate the *average depth from the surface to the invert,* while the *average width* will be *at least as great as the internal diameter plus twice the thickness of the brickwork.*

Thus, for a 2-ring (9 inches of brickwork) circular sewer 6 feet in diameter, with grade line 12 ft. deep, the number of cubic yards excavation per linear foot of sewer is:

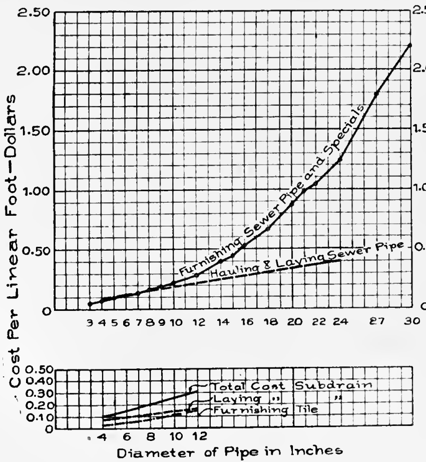
$$\frac{12 \times (6 + 1\frac{1}{2})}{27} = \frac{90}{27} = 3\tfrac{1}{3} \text{ cu. yds. per linear ft.}$$

The cost of sewer excavation and refilling varies usually from $0.20 *per cu. yd. to* $1.20 *per cu. yd., averaging perhaps* $0.50 *to* $0.75 *per cu. yd.*

DIAGRAMS FOR ESTIMATING COST OF PIPE SEWERS

Prepared from data collected for average conditions in the Middle West, 1906; ditches braced and partially sheathed; amount of water, moderate; common labor, $1.75 a day; payments to contractor, partly in assessment certificates.

For especially favorable or unfavorable conditions, the cost may vary 50 per cent either way, except for furnishing.



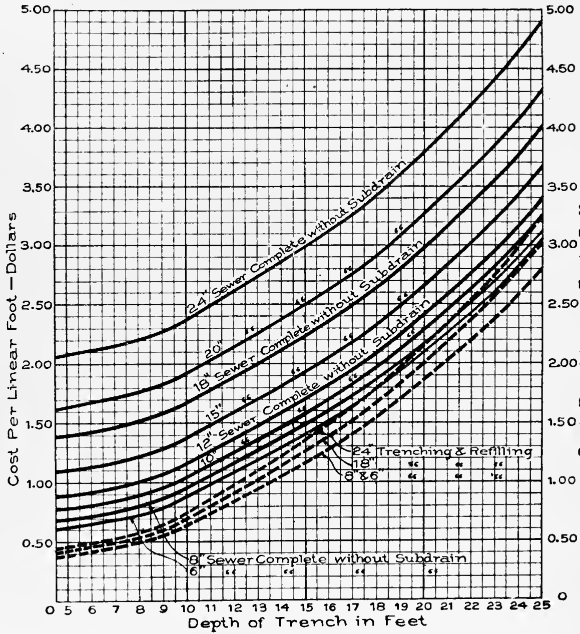


Fig. 36. Cost of Pipe Sewers.

Thus, for average conditions, fairly favorable, the cost of excavation for the 6-foot sewer, 12 feet deep, referred to above, would be $3\frac{1}{3} \times .60 = \$2.00$ per linear foot.

The favorable conditions for low cost per cubic yard, are, large sewers; neither great shallowness nor excessive depth; little water; soil firm enough not to require much bracing, yet not hard enough to require to be picked; and the use of excavating machinery. The opposites of these conditions give the unfavorable conditions.

(2) *The number of cubic yards of brickwork per linear foot* of brick sewers, may be taken from Tables XII and XIII, which are taken mainly from Gillette's *Handbook of Cost Data.*

TABLE XII

Cubic Yards per Linear Foot of Brick Masonry in Circular Sewers

Diameter	One Ring	Two Rings	Three Rings
2 ft. 6 in.	0.125	0.283	
3 " 0 "	0.147	0.327	
3 " 6 "	0.169	0.371	
4 " 0 "	0.191	0.415	
4 " 6 "	0.213	0.418	
5 " 0 "	0.234	0.502	0.802
5 " 6 "	0.256	0.544	0.867
6 " 0 "	0.278	0.589	0.933
6 " 6 "		0.633	0.998
7 " 0 "		0.677	0.063
7 " 6 "		0.720	0.128
8 " 0 "		0.764	1.194
8 " 6 "		0.807	1.260
9 " 0 "		0.851	1.325
9 " 6 "		0.895	1.390
10 " 0 "		0.938	1.456

TABLE XIII

Cubic Yards per Linear Foot of Brick Masonry in Egg-Shaped Sewers

Dimensions	One Ring	Two Rings	Three Rings
ft. in. ft. in.			
2–0 by 3–6	0.128	0.286	
2–6 " 3–9	0.154	0.341	
3–0 " 4–6	0.182	0.396	
3–6 " 5–3		0.451	0.725
4–0 " 6–0		0.506	0.808
4–6 " 6–9		0.561	0.891
5–0 " 7–6		0.617	0.974
5–6 " 8–3		0.673	1.056
6–0 " 9–0		0.729	1.140
6-6 " 9–9		0.785	1.223

The cost of brick masonry in sewers usually varies from \$8.00 *to* \$14.00 *per cubic yard, averaging perhaps* \$9.50 *to* \$12.00.

Thus, under average conditions, the cost, per linear foot, of the brick masonry of the two-ring, 6-foot circular brick sewer mentioned above, would be about 0.589 cu. yds. (from Table XII) × \$10.50 per cu. yd. = \$6.17 per foot. It will depend upon the grade of brick used, their cost per 1,000, the cost and proportions of cement and sand in the mortar, the wages of brick masons, the size and depth of the ditch, etc.

Example 49. Estimate the cost, under fairly favorable conditions, as to excavation and brickwork, of a 10-foot, 3-ring, circular brick sewer 1,875 ft. long, averaging 10 ft. deep.

Solution:

$$\text{Cu. yds. excavation per foot} = \text{about } \frac{10 \times 13}{27} = 5$$

(allowing 13 ft. width of trench, to provide a little extra room for bracing).

Since the conditions are fair, assume \$0.60 per cu. yd. as cost of excavation and refilling.

The brickwork = 1.456 cu. yds. per linear foot (Table XII); and since the conditions are fair, we shall assume a cost of \$9.50 per cu. yd.

Then the estimate will be as follows:

Excavation and Refilling,	5 × \$0.60 =	\$ 3.00 per lin. ft.
Brickwork	1.456 × 9.50 =	13.83 " " "
Total		\$16.83 " " "

1,875 × 16.83 = \$31,556 for total cost, to which, however, it may be wise to add, say, 5 to 10 per cent for contingencies unforeseen.

Answer. About \$33,500.

85. Cost of Concrete Sewers. The cost of concrete sewers may be estimated by a method precisely similar to that described in Art. 84, above, for brick sewers—namely:

(1) *Compute the cubic yards of excavation per linear foot of sewer* $\left(= \frac{\textit{average depth} \times \textit{average width}}{27}\right)$, *and multiply by the estimated cost per cubic yard, which will be from* \$0.20 *to* \$1.20, *usually* \$0.50 *to* \$0.75.

(2) *Compute the number of cubic yards of concrete per linear foot of sewer*

$$\left(=\frac{\textit{total area of concrete in square feet in a cross-section of the sewer}}{27}\right)$$

and multiply by the estimated cost of the concrete per cubic yard, which will be from \$6.50 *to* \$12.00, *usually from* \$7.50 *to* \$9.50.

(3) *In the case of reinforced concrete sewers, compute the number of pounds of steel reinforcing per linear foot of sewer, and multiply by* \$0.04 *to* \$0.05 *per lb.*

The details of designs for concrete and reinforced concrete sewers vary so much that no tables can be given, as for brick sewers, showing the cubic yards of concrete per linear foot of sewer.

The cost of the concrete will depend upon the costs of cement, sand, and broken stone or gravel, and on their proportions; on the size and depth of the trench and its freedom from water; on the cost of labor, etc.

86. Cost of Manholes, Combined Manholes and Flush-Tanks, Flush-Tanks, Lampholes, and Deep-Cut House Connections. Under these headings the following data of cost will be found valuable:

Manholes. Under average conditions, the cost of brick manholes of the design shown in Fig. 9, will be *about* \$40 *for* 8 *ft. depth of sewer.* For greater depths, *add about* \$3 *per foot of additional depth.*

Combined Manholes and Flush-Tanks. Under average conditions, the cost of these may be estimated at \$80, *plus* \$4 *per foot of additional depth of sewer over* 8 *ft.* This is for about 500 gallons' capacity of the flush-tank part.

Flush-tanks of 500 gallons' capacity, under average conditions, may be estimated to cost *about* \$60 *each.*

Lampholes, such as shown in Fig. 10, may be estimated at *about* \$10, *plus* \$0.35 *per foot of additional depth over* 8 *feet.*

Deep-cut house connections (see Fig. 8) may be estimated at \$2.00 to \$3.00 *each,* according to the depth of the sewer.

87. Engineering and Contingencies. In estimates of the cost of a sewer system, it is necessary to allow for unforeseen contingencies and for the cost of the engineering work. From 5 per cent to 20 per cent is usually added to the estimated cost on these accounts, depending upon the certainty or uncertainty of the knowledge of all the conditions.

EXAMPLE FOR PRACTICE

88. *Example* 50. Estimate the cost of the sewer system shown below, the conditions being assumed to be average. (NOTE: See Articles 84 to 87, inclusive.)

PRELIMINARY ESTIMATE OF COST OF SEWER SYSTEM FOR

Item	Approx. Quantity	Cost: Unit	Cost: Total
4-ft. brick sewer, 2 rings, 8 ft. average depth	850 ft.		
3-ft. " " 2 " 10 " " "	625 "		
24-in. pipe sewer, 9 ft. average depth	3,780 "		
18 " " " 11 " " "	1,740 "		
12 " " " 14 " " "	2,640 "		
8 " " " $10\frac{1}{2}$ " " "	46,800 "		
Manholes 12 " " "	68		
Comb. M.H.& F.T.10 " " "	18		
Lampholes 11 " " "	38		
Total of above			
Engineering and Contingencies, 10 per cent of above.			
Total estimate of cost			*

* Answer. About $82,500.

89. **Methods of Paying for Sewers.** This is another question which comes up early in determining whether a city can or will build or extend a sewer system.

Three methods are in common use in paying for sewers, as follows:

(1) *The City as a whole may pay the entire cost.* When this plan is followed, all or part of the money may be raised by selling bonds, or all or any part may be raised at once by taxation.

In some States, cities are given a right to levy a *sewer tax* of a certain rate for a certain number of years in advance, and to anticipate the proceeds of this tax by issuing *sewer warrants*.

Often, when it comes to the construction of sewers, the City will be found to have already issued bonds to the highest legal amount, to build waterworks, an electric light plant, etc., so that no money for sewers can be raised from bonds.

(2) *The entire cost of the sewers may be assessed against the property abutting upon or adjacent to the sewer.* Here the legal principle is that the assessment must be in proportion to the benefit received. Property abutting directly upon the sewer receives the greatest benefit, and must be assessed for most of the cost. Sometimes the benefit will be in proportion to the number of feet frontage of the lots abutting on the sewer; and sometimes the benefit per unit lot is considered to be the same in all parts of the city, a large unit size of lot being adopted in the residence part of the city, and a much smaller size in the business section, with often an intermediate size between these two.

The "assessment" is levied upon the completion of the sewer, when the entire cost can be ascertained. Due notice to all property owners assessed must be given, so that they can present objections if they desire. Usually all property owners who desire are allowed to spread the payment of their assessments in equal installments over a considerable period of years, in which case *assessment certificates* are issued to cover the payments. The contractor is often required to take these certificates in payment for the sewer.

(3) *The cost of the sewers may be divided between the City and the property directly abutting upon or adjacent to the sewer.* This seems the fairest way; since, in the first place, the entire city receives benefit from improved sanitation, attractiveness to investors, etc., from a sewer constructed anywhere within its limits; and since, in the second place, any system of sewers for a city should be planned to give outlets of proper size to all parts of the district, which enlarges and deepens the sewers on many streets. On the other hand, the property along the sewer is benefited much more than the rest of the city, and should accordingly pay a much larger proportion of the cost.

The City Council usually has the right to decide what percentage of the cost is to be paid by the City and what by the property along the sewers.

PREPARATION OF PLANS AND SPECIFICATIONS FOR SEWERAGE SYSTEMS

90. Sewer Reconnaissance. When a sanitary engineer is called upon to prepare plans and specifications for a sewerage system, the first thing which he should do is to make a *reconnaissance* or general study of the entire city and its surroundings, with special reference to its sewerage conditions.

He visits the city and obtains copies of the best *maps* procurable. If these maps do not show the contours or elevations of the surface at different points, he obtains the best procurable information as to such elevations, and enters it upon the maps. Often the elevations of *street grades* will prove sufficient, if better and more detailed information is lacking. If *street profiles* are available, they will of course be of great value.

With maps thus prepared for the purpose, *he rides or walks over all parts of the city,* making himself thoroughly familiar with its *topography* and other features. Some of the information thus obtained may be entered upon the maps. He will note the *present density of population in different sections, and the prospects for future growth.* The presence or absence of *manufacturing industries,* and the future prospects in this line, are of importance. Statistics of the *past growth* of the city will be obtained. Full information regarding the character of the *water supply* and the amount and fluctuations of the *water consumption,* and the distribution of the *water mains* throughout the city, will be of great value. The local *labor conditions,* and the probable *local cost of cement, sand, brick, sewer pipe,* and other needed materials, must be ascertained. All possible information should be secured regarding the *ground water* and the *character of the soil* in different sections of the city. Information about old excavations and about wells can usually be secured, and will give much light on these points.

From his general study of the conditions, including especially the topography, the engineer must decide whether the system of sewerage shall be a *separate* system, or a *combined* system (see Articles 10 to 13, inclusive).

The question of the *outlet* will be one of the most important controlling points to be decided, and the engineer must carefully examine all possibilities in this line. The *number of outlets should be as small as feasible, one outlet being secured if possible.* The outlet must be low enough to drain thoroughly all portions of the district it serves, and should be chosen with a view to safe and satisfactory disposal of the sewage.

Sewage disposal is one of the very important points to be considered. In the past, most cities have simply discharged their sewage into the nearest available body or stream of water which it was considered could be used without causing damage or injunction suits on account of the pollution. At the present time, cities are being compelled more and more to provide means for purifying the sewage (see Articles 110 to 124); and the engineer, in choosing the outlet and planning the sewers, should always consider it probable that in the not distant future the city will be compelled to use some method of purification, and his plans should be so made as readily to permit this in the future, even if the city builds no sewage purification works at first.

During the reconnaissance, the engineer must constantly be recording the significant information he secures, in a neat and systematic manner in *a standard notebook*, which he keeps for the purpose. *Loose-leaf notebooks* of pocket size have many advantages for this purpose. In the same notebook, he should make all his preliminary computations.

On completing the reconnaissance, the engineer usually makes a *preliminary report* to the city officers, stating the conditions he has found, and his conclusions as to the general features of the system he has decided to recommend as best. He also usually presents at this time some rough estimates of cost.

The city then decides whether or not to adopt the general recommendations of the engineer, and whether to go on with the preparation of plans and specifications.

91. Surveys for Sewer Plans. After the reconnaissance, if it is decided to go ahead with the plans, the next step will be to make the necessary *surveys*. These may usually be divided into three principal parts as follows:

(1) *Surveys of Sewage Disposal Site.* In case a sewage disposal plant is to be built, a survey of the site must be made to secure the data needed for the design. Usually this will include data for a *contour map* of the entire tract, and borings or pits to determine the character of the soil.

(2) *Surveys for the Outlet Sewer.* Transit and level lines must be run, and profiles prepared, to determine the best route for the outlet sewer. Data must be secured for an accurate map and profile of the final location of this sewer.

(3) *Surveys for the Street Sewers.* Usually, existing plats can be found sufficiently accurate to give the dimensions necessary for constructing the *general sewerage map*, without special surveys. Small errors on these plats will not affect the general design, and will not be of much importance in view of the accurate surveys which must be made later during construction. Sometimes a few measurements with tape-line and transit must be taken in special localities. Usually the main part of the surveys for the street sewers consists in running *lines of levels* along all the streets on which there is possibility of planning sewers, in order to secure the data necessary to make the *sewer profiles* of all the sewers.

These levels should be referred to the *city datum*—that is, the reference level above which all city elevations are given. If such a datum has not already been adopted, one should be established, and marked by a *permanent bench-mark*. A six-inch iron pipe set six feet in the ground, filled and surrounded with concrete, makes a good, permanent bench-mark. The top, not quite filled with concrete, projects a little above the ground, and a copper bolt is set in the concrete at the top, the top of the bolt constituting the bench-mark. The pipe should have a hinged iron cap to protect the bolt.

In running the level, no effort should be made to trace out the main lines of sewers and their branches, but *each street should be surveyed by itself.* A zero point should be taken at some definite point (such as the center line, or one of the side lines, of a cross-street) at one end of the street, and *station points* 100 feet apart determined by continuous measurements with a steel tape. These stations should be numbered continuously from the zero point, intermediate points being located, in the usual way, by *plus* distances from the preceding station. Thus station 9 + 72 is 972 feet from the zero point.

The exact plus of each side line of each cross-street, and of points opposite other important things, should be determined and recorded in the notebook, to give measurements to be used in preparing the profiles, and in checking the map.

All lines of levels must be checked. At the end of each street, the leveling can be extended across to an adjacent street, and checked with the line of levels on that street.

Numerous bench-marks should be established around the city, located on permanent points, such as the tops of the foundation walls of buildings.

92. Sewerage Plans. From the data obtained by the surveys, the sewerage plans must be prepared. These will usually consist of a large number of separate sheets, the following being a list of the sheets of one particular set of plans, for a separate system of pipe sewers.

1. Index Sheet. (Giving the contents of all other sheets.)
2. General Sewerage Map.
3. General Map of Sewage-Disposal Plant.
4. Detailed Plans of Septic Tank. (For the Sewage-Disposal Plant.)
5. Detailed Plans of Filter Beds. (For the Sewage-Disposal Plant.)
6. Plans of Standard and Drop Manholes, and Lampholes.
7. Plans of Combined Manholes and Flush-Tanks.

8 to 33. Profile Sheets. (Showing profiles of all the sewers.)

In other cases, separate sheets may be needed for many other things, as, for example,

Details of Brick Sewers, of different sizes.
" " Concrete Sewers, " "
Plans of Flush-Tanks.
" " Catch-Basins.
" " Street Inlets.
" " Sewage Pumping Station.
Etc., etc.

For the sake of convenience and of neatness and system, *all the sheets of a set of sewerage plans should be made of a standard size (one or two can be made larger and folded to the standard size), and they should be bound together in regular book covers,* 18 inches by 24 inches being a convenient standard size of sheet for most cases.

Fig. 37 is a photographic view of such a cover containing a set of sewerage plans. The cover protects the sheets from injury, and is so arranged that any sheet can readily be removed and replaced A cover like that shown costs about $1.50.

The original drawings were all made on tracing cloth, except the profiles, which were made on transparent profile paper. Thus all the sheets can readily be reproduced by the process of blue-printing, and only the blue-print sheets are used on the work or by the City, the engineer retaining the original tracings in his office, where they can be kept safe.

In such a set of plans, the sheets should be numbered in order (see Figs. 38 and 39); and a *standard title* (see title of Fig. 38) should be adopted for all sheets which will require few changes of the different sheets.

Sewerage Map. In Fig. 38 is shown a reduced copy of an actual *sewerage map* of a separate system of sewers for a small town. The original size of the map shown was 36 inches by 24 inches, so that folding it once reduced it to the 18-inch by 24-inch size.

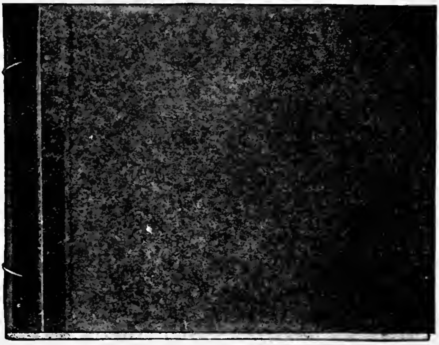


Fig. 37. Standard Cover for Sewerage Plans.

The original scale of the map shown was 200 feet per inch; but for larger places, 300 feet or even 400 feet per inch may be sufficient, since large-scale maps of all the individual sewers appear on the profile sheets.

The lines of sewers in a system such as shown in Fig. 38, *ought to be restricted as far as possible to the streets on which the lots front.* Sewers on cross-streets add to the mileage of sewers without serving additional lots, and are useless except for connecting other sewers.

The manholes, lampholes, flush-tanks, etc., should be numbered systematically, something as shown in Fig. 38, no two structures of the same kind having the same number. This avoids danger of duplication where the same structure is shown on two or more sheets, as is often the case.

Sewer Profiles. In Fig. 39 is shown a sample profile sheet from an actual set of plans.

The original profile was made on "Plate B" transparent profile paper, so that the profiles can be reproduced easily by blue-printing, the same as the other drawings. The sheets were cut to the standard size, 18 inches by 24 inches, to bind with the other drawings.

The profiles should be made in systematic order of the streets, each *street completed before beginning the next,* instead of trying to follow up the main lines of the sewers and their branches.

The profile sheets show large-scale maps of the individual

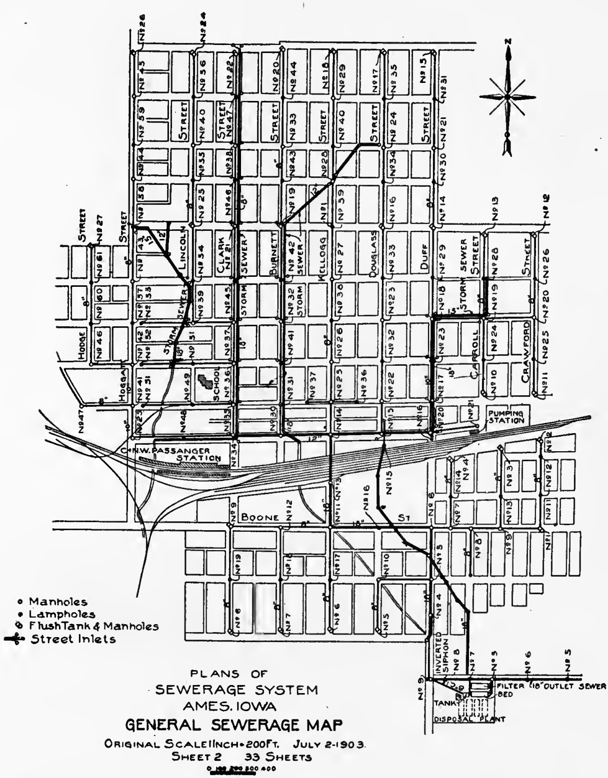


Fig. 38.

sewers immediately below their profiles, to permit the exact location of manholes, etc., and of the sewer itself in the street.

93. Specifications for Sewers. Besides the plans, it will be necessary for the sewerage engineer to prepare precise instructions regarding all matters of importance not fully shown by the plans, likely to come up during the construction of any part of the sewerage system. Such instructions are called *Specifications.*

An ordinary set of sewer specifications will consist of three parts:

(1) A *Notice to Contractors,* or form of advertisement for the city officers, to use in advertising for bids.

(2) A *Form for Proposal,* with suitable blanks, on copies of which, furnished by the city, all contractors are required to make their bids.

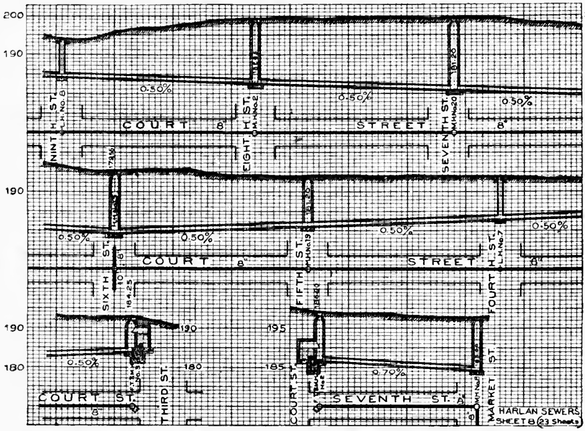


Fig. 39. Typical Sewer Profile Sheet.

(3) *The Specifications Proper.* These again will consist of two main divisions:

(*a*) General clauses, relating to payments, guarantees, etc., and to general features of the work.

(*b*) Specific clauses, specifying the exact details of different parts of the work.

A copy of an actual set of specifications for the construction of a separate system of pipe sewers, with a sewage-disposal plant, is given herewith:

CITY OF____________________, ________

SPECIFICATIONS

FOR

SEWERS AND SEWAGE-DISPOSAL PLANT

NOTICE TO CONTRACTORS

The Incorporated City of—————————,————., will receive sealed bids until———, ———, at ————————; (1) for the construction of a sewage-disposal plant, consisting of a sewage tank of about ———gals. capacity, and —— sand filter beds, each of about——sq. ft. area; and (2) for the construction of sewers as follows: about —— ft. of 18-inch, —— ft. of 15-inch, —— ft. of 12-inch, —— ft. of 10-inch, and —— ft. of 8-inch, with suitable appurtenances, all in accordance with plans and specifications prepared by ——————, Engineer, ——————, and now on file in his office and with the City Clerk. All bids must be accompanied with certified checks, approximately in the amount of 5 per cent of the bid, made payable without recourse to the City of——————,———. The City reserves the right to reject any or all bids, to waive defects, and to accept any bid. All bids must be in sealed envelopes, marked on the outside "Sewerage Bids," and addressed to——————, City Clerk.

INSTRUCTIONS TO BIDDERS, AND GENERAL SPECIFICATIONS

(1) *Items.* The items of work intended to be covered by these specifications are those required for the entire completion of the System of Sanitary Sewers for the City of————————, ——————, according to the plans prepared by——————, Engineer, and include the following:

(*a*) The construction of a Sewage-Disposal Plant, including a sewage tank of about ——— gallons capacity, and —— sand filter beds, each of about —— sq. ft. area, and including all valves, sewer pipes, outlets, etc.

(*b*) The construction of Sewers as follows:

18-inch........................	Ft.
15-inch........................	"
12-inch........................	"
10-inch........................	"
8-inch........................	"
Manholes	"
Lampholes......................	"
Combined Manholes and Flush-Tanks,	"

together with subdrains as directed by the City.

(2) *Application.* These general specifications and instructions to bidders shall apply to all items of workmanship or materials enumerated above or hereinafter mentioned.

(3) *Definitions of Terms.* Wherever the word "City" is used in these specifications, it shall be understood to mean the Incorporated City of———, ———., acting through the Mayor and Council, or their duly authorized representatives. Wherever the word "Contractor" is used in these specifications, it shall be understood to mean the person or firm employed to do all or any part of the work or furnish all or any part of the material for the Sanitary Sewerage System. Wherever the word "Engineer" is used in these specifications, it shall be understood to mean the Engineer employed by the City to design or supervise the construction of all or any part of the Sanitary Sewerage System.

(4) *Bids.* All bids must be on blanks furnished by the City for the purpose. The blanks can be obtained from ————————, City Clerk ——————, ———., or from ——————, Engineer, ————, ———.

All bids must be enclosed in sealed envelopes addressed to————, City Clerk,————, ———., and plainly marked on the outside with the words "Sewerage Bids."

Each bid must be accompanied with a certified check approximately in the sum of 5 per cent of the bid, and made payable without recourse to the City Treasurer, —————————,—————.

The City reserves the right to reject any or all bids, to waive defects, and to accept any bid.

(5) *Certified Checks.* The certified check mentioned above will be forfeited as damages to the Incorporated City of —————————, —————., unless the Contractor enters into contract and furnishes bonds satisfactory to the Mayor and Council within 12 days after the contract has been awarded to him. Certified checks not so forfeited shall be returned to the bidders as soon as the contract is signed and satisfactory bonds are furnished.

(6) *Bond.* A bond satisfactory to the Mayor and Council shall be furnished by the Contractor, approximately in the amount of 50 per cent of the contract price.

(7) *Time.* The Contractor shall begin work within 3 weeks after the contract is awarded to him, and shall entirely complete the work on or before ——————————————, ———————.

(8) *Sub-contracts.* No sub-contracts shall be awarded to parties unacceptable to the City.

(9) *Progress of the Work.* The work shall be prosecuted at a rate to enable its completion within the time specified; and should the Contractor fail to do this, the City may, after giving ten days' written notice, take over the work and complete it at the Contractor's expense.

(10) *Penalties.* Should the Contractor fail to complete the work at the time specified, he shall forfeit to the City a sum equal to all damages to it resulting from the failure to complete the work at the time specified.

(11) *Delays.* No claims for damages shall be made against the City on account of delays in delivery of materials or performance of work; but should there be unduly prolonged delays in the delivery of any materials or the performance of work on the part of the City, the Contractor shall be entitled to corresponding extension of time.

(12) *Obstructions.* The Contractor shall carry on the work in such a way as to obstruct the city streets as little as possible, and so as not at any time entirely to shut off passage of teams and pedestrians at any place. He shall provide temporary crossings satisfactory to the City for this purpose wherever necessary.

(13) *Precautions.* The Contractor shall take all necessary precautions to prevent injury to the public or to his workmen or to stock, such as providing crossing plank, fencing off his work, keeping lanterns burning at night, etc. He shall hold the City harmless against all claims for damages.

(14) *Plans and Specifications.* The City's plans and these specifications shall be a part of the contract, and all materials and workmanship shall be in accordance with them.

(15) *Supervision.* All materials and workmanship shall be subject to the supervision and inspection of the City and of its Engineer or other authorized representative. Instructions as to the details of the work shall be carried

LARGEST SEWER IN THE UNITED STATES

View of 39th Street Intercepting Sewer, Chicago, Ill., showing a portion of the wooden forms and brick lining. Length of sewer, 12,000 ft.; diameter, 24 ft. 10 in. external, and 20 ft. internal; bricks laid in "Utica" hydraulic cement, of which 60,000 barrels were used. This sewer is one of a system of similar sewers paralleling the shore of Lake Michigan, to prevent egress of sewage into the lake. Its level being 20 ft. below that of **the lake, the sewage is pumped up to higher levels, and then flows westward by gravity into the new outlet afforded by the Drainage Canal.**

out, and rejected materials and work shall be promptly removed at any time discovered.

(16) *Quality of Materials and Workmanship.* All workmanship and materials shall be of the best quality.

(17) *Quantities.* The quantities named in the notice to contractors, the form of proposal, or in these specifications, are approximate only. The City shall have the right to vary them; and, if so varied, the total contract price shall be increased or diminished at the rates named per unit in the contract.

(18) *Extra Work.* No extra work shall be done without written orders from the City or its specially authorized representatives placed in charge of the work. In case extra work becomes necessary, it shall be done by the Contractor if so ordered, and shall be paid for by the City on the basis of actual cost, *plus* 10 per cent; but no extra work will be paid for unless ordered in writing by the proper authority at the time undertaken.

(19) *Changes in Plans.* The City shall have the right to make changes in plans. In making such changes, the unit prices named in the contract shall be used, as far as possible, in calculating the changes in price on account of changes in the plans, and where these do not apply, the changes in price, unless a special agreement between the City and the Contractor as to prices is made at the time the changes are ordered, shall be calculated on the same basis as extra work.

(20) *Claims.* The Contractor shall guarantee the payment of all just claims for materials or labor in connection with his contract. Preliminary to the payment for any work, he shall, if required by the City, present evidence satisfactory to the Mayor and Council that all bills for materials and labor have been paid, and any or all payments may be reserved until such evidence has been presented. If the payment of any just claim shall be deferred more than four weeks after written notice has been given concerning it to the Contractor, the City may proceed to pay such claim out of any money due the Contractor.

(21) *Payments.* Payments shall be made as follows:

(NOTE: Fill in, in this blank, whether the payment is to be made in cash, in sewer warrants, sewer certificates, or otherwise. Also whether payments are to be made monthly as the work progresses, or reserved until completion, the former plan being usual for cash payments, and the latter for payments in certificates.)

All payments shall be on estimates prepared by the Engineer and approved by the Council, of materials delivered and work performed; and in case of all payments made prior to the completion of the contract, 15 per cent of the estimate shall be reserved until the final payment on completion of the work.

No payment shall be considered as releasing the Contractor from obligation to remove and make good defective work and materials when discovered at any time.

Two per cent of the total cost may be reserved by the City for one year after the completion of the work, and any part of this reserve may be used to make good defects developed within that time from faulty workmanship and materials, provided that notice shall first be given the Contractor, and that he may promptly make good such defects himself if he desires.

(22) *Guarantee.* The Contractor shall guarantee the workmanship and materials for one year, and keep the system in repair after completion, as provided in clause 21 above.

(23) *Risks.* All materials and work will be at the risk of the Contractor until the final acceptance of the same.

(24) *Cleaning Up.* On completion of each part of the work, all rubbish and unsightly materials must be removed and disposed of as directed by the City, and the streets and grounds left in neat condition. For the sewers, each two blocks must be cleaned up immediately on completion, and on the completion of the entire contract shall be further put in good shape if needed.

MATERIALS

(25) *Vitrified Sewer Pipe.* All sewers shall, unless special permission be given to use cement sewer pipe, be constructed of first-quality salt-glazed, vitrified clay sewer pipe, of the hub-and-spigot pattern, of standard thicknesses and dimensions of hubs. The dimensions of hubs shall be sufficient to leave an annular space for cement of at least $\frac{3}{8}$-inch thickness for 8-inch and 10-inch pipe, and $\frac{1}{2}$-inch thickness for larger diameters.

Pipe may be furnished in lengths of 2, $2\frac{1}{2}$, or 3 feet. All pipe and specials shall be sound and well burned, with a clear ring, well glazed and smooth on the inside, and free from broken blisters, lumps, or flakes which are thicker than $\frac{1}{6}$ the nominal thickness of the pipe and whose largest diameters are greater than $\frac{1}{8}$ the inner diameter of said pipe; and the pipe and specials having broken blisters, lumps, and flakes of any size shall be rejected unless the pipe can be so laid as to bring all of these defects in the top half of the sewer. No pipe having unbroken blisters more than $\frac{1}{4}$ inch high shall be used, unless these blisters can be placed in the top half of the sewer. Pipes or specials having fire-checks or cracks of any kind extending through the thickness shall be rejected.

No pipe shall be used which, designed to be straight, varies from a straight line more than $\frac{1}{8}$ inch per foot of length; nor shall there be any variation between any two diameters of a pipe greater than $\frac{1}{24}$ the nominal diameter.

No pipe shall be used which has a piece broken from the spigot end deeper than $1\frac{1}{2}$ inches or longer at any point than $\frac{1}{4}$ the diameter of the pipe; nor which has a piece broken from the bell end if the fracture extends into the body of the pipe, or if such fracture cannot be placed at the top of the sewer. Any pipe or special which betrays in any manner a want of thorough vitrification or fusion, or the use of improper or insufficient materials or methods in its manufacture, shall be rejected.

(26) *Sewer-Pipe Specials.* All T- and Y- junction curves, etc., required shall be furnished and set without extra charge, and shall conform to the pipe specifications as to quality. Y's for house connections may be required every 25 feet on the average, and shall be closed by vitrified stoppers cemented over sand.

(27) *Drain-Tile.* All drain-tile shall be best-quality vitrified agricultural drain-tile in one-foot lengths. All junctions and inspection openings shall be made with suitable T- and Y- junctions and curves, furnished and set without extra charge.

(28) *Brick.* All brick used on the work shall be sound, partially vitrified, well-shaped brick, equal to No. 2 paving brick.

(29) *Cement.* All cement used shall be ——, ——, ——, ——, ——, ——, or —— Portland Cement, perfectly fresh, and not damaged in any particular. It shall be subject to the Standard specifications of the American Society for Testing Materials, and will be rejected if it does not meet these requirements. All cement shall also be subject to close inspection as it is used on the work, and damaged cement will be rejected and must be promptly removed.

(30) *Sand.* All sand shall be clean, sharp, and coarse. All sand for mortar for sewer joints or brick masonry must have all pebbles screened out.

(31) *Broken Stone and Pebbles.* The aggregate for concrete shall consist of either broken stone or screened pebbles passing a 2½-inch ring for ordinary concrete, and a 1½-inch ring for the septic tank. The materials must be sound and hard and durable. The sand must be screened out of pebbles used; but the fine materials need not be screened out from broken stone, a reduction being made in the amount of sand used, approximately equal to the amount of stone dust.

(32) *Cast Iron.* All cast iron shall be good, tough, gray iron, free from defects. Castings shall be smooth and free from blowholes or other flaws.

(33) *Cast-Iron Water-Pipe.* All cast-iron pipe shall be cast of the hub-and-spigot pattern, of standard weights for water-pipe for light pressures. The pipe shall be well coated.

(34) *Valves.* All valves shall be iron body, brass-mounted, hub-end, double-gate, water valves, well coated, of the —————— or of equal make acceptable to the Engineer.

(35) *Valve Boxes.* All valve boxes shall be ——————— extension boxes with 5¼-inch shafts, or some equal make acceptable to the Engineer.

MORTAR AND CONCRETE

(36) *Mortar.* All mortar for brickwork or other masonry shall be made of one part of Portland cement to three parts of sand; and all mortar for sewer joints, of one part of cement to one of sand, both ingredients being measured loose and thoroughly mixed. All mortar shall be mixed fresh as used, and any mortar which has begun to set shall be thrown away and not used at all on the work.

(37) *Concrete.* All masonry shown on the plans to be made of concrete shall be constructed with Portland cement, sand, and either broken stone or screened pebbles passing a 2½-inch ring, in the proportions 1-3-5 for ordinary work, and 1-2-3½ for the septic tank, the cement being measured packed as it comes in sacks or barrels, and the sand being measured loose as thrown into the measuring box with shovels. The proportions shall be determined by suitable measuring boxes, or by the use of wheelbarrows. In case of hand-mixing, the sand and cement shall first be thoroughly mixed dry until the color of the mixture is uniform. They shall then again be mixed with water, and then again with the freshly wet aggregate, each mixing being very thorough, and sufficient to secure perfect mixture of the materials. If a machine mixer is used, it shall be of a make acceptable to the Engineer, and shall be so used as to give very thorough mixing. Just enough water shall be

used to make the concrete slightly quake when thoroughly rammed, the water freely flushing to the surface under the ramming.

In depositing, the material shall be deposited in layers not exceeding 6 inches in height, and thoroughly rammed. Where work is left for the night, the layers shall be racked back. Where fresh concrete is deposited on work which is already set or begun to set, the surface shall first be thoroughly cleaned and wet, and washed with a coat of liquid neat cement. After the concrete is deposited, great care shall be taken not to disturb it until the work is thoroughly set. The work shall be protected from the sun, and shall be wet from time to time, until it is thoroughly set.

TRENCHING, PIPE-LAYING, REFILLING, ETC.

(38) *Excavation.* The excavation shall be made exactly to line and grade as indicated by stakes set by the Engineer. At the bottom, the trench shall have a clear width at least one foot greater than the external diameter of the body of the pipe. The last four inches shall be excavated only a few feet in advance of the pipe-laying, by men especially skilled, measuring from an overhead line set parallel to the grade line of the sewer. The bottom of the trench shall be rounded to fit the pipe; and holes shall be dug for the bells so as to give a uniform bearing, and permit the proper construction of the sewer joints on the under side of the pipe. The earth taken from the trench shall be deposited neatly at the sides, in such manner as to obstruct the streets as little as possible; and a clear space of two feet next the trench shall be left on the side on which the Engineer places his stakes. Great care shall be taken to preserve and not to cover up the Engineer's stakes.

(39) *Sheathing.* Wherever necessary to prevent caving of the banks or injury to adjacent pipes or buildings, the Contractor shall, at his own expense, brace and sheath the trenches sufficiently to overcome the difficulty to the satisfaction of the Engineer. If such bracing and sheathing is left permanently in the trench by order of the Engineer, it shall, on refilling, be cut off one foot below the surface and shall be paid for by the City at the price named in the contract; but otherwise the Contractor will receive no extra compensation for it.

(40) *Water in Trenches.* In general, all water encountered in trenches must be drained away through the sub-drains or pumped or bailed out, and the trench must be kept dry for the pipe-laying. In no case shall the sewers be used as drains for such water, and the ends of the sewer shall be kept properly blocked during construction. All necessary precautions shall be taken by the Contractor to prevent the entrance of mud, sand, or other obstructing material into the sewers or subdrains; and on completion of the work, any such materials which may have entered must be cleaned out and the sewers and subdrains left clean and unobstructed.

(41) *Refilling.* In refilling, earth free from stones shall be carefully placed by hand under and around the pipe and to the height of two feet above the top of the sewer, and thoroughly and carefully rammed in layers of not more than six inches' depth.

The remainder of the refilling shall be carefully done. Scrapers may be used if desired. The refilling shall be thoroughly flooded by the Contractor according to the direction of the Engineer, the City furnishing the water free at the hydrant; but the refilling shall be carried on in such a way that water is taken only as directed by the Waterworks Superintendent, and so that not more than —— gallons of water shall be required in any one day.

Where the trench is not flooded, it shall be left neatly rounded off on top to a height of twice as many inches as the top width of the trench in feet; and the City may from the 2 per cent reserve make good any settlement below the street surface within one year from the date of completion, notice being first given the Contractor, who may promptly do the work himself if he desires.

All surplus material shall be removed to such point within the limits of the sewer district as may be designated by the City; and in case of deficiency of material, it shall be supplied by the Contractor. The street surface shall be left in neat, sightly condition.

(42) *Foundations.* In case the material encountered should be such as not to be suitable for foundations for the sewer, the Engineer shall direct the character of foundations to be constructed, and this shall be paid for by the City as extra work.

(43) *Protection to Buildings.* The Contractor shall take all necessary precautions to protect building and other structures adjacent to the sewer trenches from injury on account of his work, and shall be responsible for all damages to such structures.

(44) *Existing Sewer and Water Mains.* Wherever existing sewers or water mains are encountered in the work, all necessary precautions shall be taken to prevent injury to them; and in case of an injury, it shall be made good by the Contractor without additional compensation. In case any sewer, drain, or water main should be encountered whose present grade should require changing on account of the new sewers, the work necessary for this shall be performed by the Contractor according to the directions of the Engineer, and shall be paid for as extra work.

(45) *Pipe-Laying.* In pipe-laying, each piece must be set exactly to grade by measuring from the invert to a tightly stretched cord set parallel to the grade line, according to stakes or marks given by the Engineer, and supported at least every 25 feet. In making each joint, a gasket of oakum or hemp freshly dipped in cement grout must first be used and packed into place, so as to make the inverts match exactly, giving a smooth, true flow-line. The joints shall afterwards be tightly packed full and beveled off with 1 to 1 Portland cement mortar; but the cementing must be done at least two pipe lengths behind the pipe-laying. The bell-holes must then be immediately packed with sand to hold the cement in place. Great care must be taken to leave no projecting cement or strings of gaskets on the inside of the sewer, and to make all joints as nearly water-tight as possible. Especial care must be taken in forming the joint on the under side of the pipe.

(46) *House Connections.* At points indicated by the Engineer opposite each lot, and at such other points as may be indicated by the Engineer, 4-inch Y's shall be laid, with the branch tilted up at an angle of about 45°. These shall be furnished and laid without extra charge, up to an average of one in each 25 feet.

At points indicated by the Engineer, deep-cut house connections shall be put in according to the plans. The City shall pay for these the regular contract price.

In both ordinary and deep-cut house connections, the connection shall be closed by a vitrified stopper filled over with sand and lightly cemented.

(47) *Subdrains.* Wherever directed by the City, drain-tile subdrains of diameters directed by the Engineer shall be constructed. Each drain shall be laid just at one side of the sewer, at a depth below the sewer invert equal to the external diameter of the subdrain, *plus* three inches. Each joint shall be wrapped twice with a 4-inch strip of muslin at the time laid. The subdrains shall be laid carefully to line and grade; and wherever the Engineer may direct, 4-inch Y's stopped with brick shall be placed. In general, these Y's will be placed at the same points as the house connections on the sewer.

(48) *Subdrain Outlets.* Wherever directed by the Engineer, subdrain outlets shall be constructed, also as directed by the Engineer, and shall be paid for by the City on the basis of cost as determined by the Engineer, *plus* 10 per cent.

(49) *Measurements.* All measurements of sewers, subdrains, etc., shall be in horizontal lines from center to center of manholes and junctions.

MANHOLES AND OTHER APPURTENANCES

(50) *Manholes.* Manholes shall be constructed as shown on the plans and provided in these specifications, the exact location being indicated by the Engineer. All joints in the brickwork shall be shove joints, being filled full. Especial care shall be taken in forming the channels in the concrete bottoms, and wooden templates or half-sewer-pipe shall be used for this work, as directed by the Engineer. Drop manholes shall be constructed as shown on the plans without additional charge over the price bid, which shall be considered an average price.

(51) *Combined Manholes and Flush-Tanks.* Combined manholes and flush-tanks shall be constructed as shown on the plans and as specified for manholes in clause 50. The siphons shall be carefully set, and the cost of furnishing and setting shall be included in the price bid. The Contractor shall provide and set the water connection and bibbs from a point one foot outside the outside wall, on such side as the Engineer may direct.

(52) *Siphons.* Siphons shall be used as shown on the plans, guaranteed by the manufacturers, and tested after being set before acceptance. For the 8- and 10-inch sewers, 6-inch siphons shall be used, and 8-inch for all sewers larger than 10 inches.

(53) *Lampholes.* Lampholes shall be constructed as shown on the plans and provided in these specifications, the exact locations being indicated by the Engineer. The refilling shall be carefully placed and thoroughly rammed by hand in layers not exceeding 6 inches, around and to a distance of three feet each side of each lamphole. Special pains shall be taken to keep the lampholes truly vertical.

SPECIFICATIONS FOR SEWAGE-DISPOSAL PLANT

(54) *Grading.* All grading shall be done as shown by the plans. The bottom of the filter beds and bottom and sides of the septic tank shall be shaped to true surfaces by hand. All slopes shall be neatly dressed.

Should there be a deficiency of earth for the embankments, the Contractor may borrow from neatly-shaped borrow pits located on adjacent city land, where directed by the Engineer, leaving a smooth, uniform surface. Should there be surplus material, it shall be deposited along the edge of the lake, as directed by the Engineer.

(55) *Concrete Moulds.* The Contractor shall provide moulds of plank not less than two inches in thickness, thoroughly braced at intervals sufficiently close together to avoid distortion of the moulds. These planks shall be dressed on their edges and on the faces next to the wall. The moulds shall not be removed until the walls have become thoroughly set.

(56) *Facing of Concrete Walls.* In the construction of concrete walls, care shall be taken to keep all pebbles or stones away from the faces of the walls, so that the face shall be smooth and free from cavities or exposed stones or pebbles. The upper surface of the roof shall be floated with 1-2 thin mortar applied when the roof is made, and all cavities in other concrete surfaces filled and smoothed with 1-2 mortar.

(57) *Cement Wash.* On completion of concrete walls and floors, and after removal of the moulds and pointing up defects, all interior surfaces of floors and walls and roof, and the upper surface of the roof, shall be given two good coats of thin, neat Portland cement grout applied with a whitewash brush, time being left between applications for the first coat to set hard.

(58) *Alternating Siphons.* The alternating siphons shall be provided of the make shown on the plans, and set by the Contractor, strictly according to the directions of the manufacturer as given through the Engineer. Any imperfections affecting the working of the siphons when they are tested shall be corrected by the Contractor,who must guarantee their satisfactory working.

(59) *Filters.* The pebbles for the bottoms of the filters shall be screened clean of sand and properly graded, the 2-inch layer of fine pebbles being small enough to hold up the sand placed over it. All sand shall be clean and coarse, but the pebbles need not be screened out. In placing pebbles and sand, care shall be taken not to injure or disturb the drain tile, and the top surface of the sand shall very carefully be made level. Drain tile shall be laid carefully to line and grade.

(60) *Pipe-Laying.* All sewer pipe and cast-iron pipe shall be carefully laid to line and grade, with gaskets and tight joints, all as provided in the regular sewer specifications.

(61) *Sodding.* All earthwork slopes of the tank and filters shall be neatly sodded.

(62) *Bulkheads.* All bulkheads shown on the plans shall be constructed of Portland cement concrete, with moulds, and with care as to facing the same as provided for the concrete work of the septic tank.

(63) *Reinforcing.* The reinforcing shown on the plans is .... corrugated bars of not less than 50,000 lbs. per sq. in. elastic limit; but other forms of bars having equal elastic limit, equal net area, and a mechanical bond acceptable to the Engineer, may be used. The net area of any bars used must be increased to make good any deficiency in the elastic limit.

....

....

....

For brick sewers, the following specifications are suggested by Folwell in his book on *Sewerage:*

"For brick masonry in straight walls or sewers, none but whole, sound brick shall be used. For manholes, flush-tanks, and similar work, a limited number of half-brick may be used, not to exceed ⅓ of the whole in any case. Unless the Engineer direct otherwise, each brick shall be thoroughly wetted immediately before being laid. It shall be laid with a full, close joint of cement mortar on its bed, ends, and side at one operation. In no case is mortar to be slushed in afterward. Special care shall be taken to make the face of the brickwork smooth; and all joints on the interior of a sewer shall be carefully struck with the point of a trowel or pointed to the satisfaction of the Engineer. Where pipe-connections enter a sewer or manhole, "bull's-eyes" shall be constructed by laying rowlock courses of brick around them, the cost of such construction being included in the regular price bid for the sewer or appurtenances. Around pipe more than 15 inches in diameter, 2 rowlock courses shall be laid.

"Brickwork in sewers shall be laid by line, each course perfectly straight and parallel to the axis of the sewer. Joints appearing in the sewer shall in no case exceed ¼ inch in width. Sewers shall conform accurately in section and dimensions to the plans of the same. All inverts and bottom curves shall be worked from templates accurately set; the arches are to be formed upon strong centers accurately and solidly set, and the crowns keyed in full joints of mortar. No centers shall be drawn until the arch masonry has set to the satisfaction of the Engineer, and refilling has progressed up to the crown. They shall be drawn with care, so as not to crack or injure the work. The extrados is to be neatly plastered with cement mortar ½ inch thick, the arches being cleaned and wetted just before plastering. The end of each section of brick sewer shall be toothed or racked back; and before beginning the succeeding section, all loose brick at the end shall be removed and the toothing cleaned of mortar. All brickwork shall be thoroughly bonded, adjacent courses breaking joints at least ⅓ the exposed length of the brick.

"If there should be any distortion of the sewer before acceptance, this shall be corrected by tearing down and rebuilding. No local patching will be allowed, but when repairs are necessary a section shall be removed at least 3 feet long and including the entire arch, or the entire sewer if the defect is in the invert. Leakage of ground water into the sewer shall be similarly corrected, unless it can be prevented by calking the joints with oakum saturated in cement, with wooden plugs, or other material acceptable to the Engineer."

FORM OF PROPOSAL

——————————, ————.

To the Mayor and Council of the Incorporated City of——————, ————.

Gentlemen:

—— have carefully examined the plans and read the specifications prepared for your proposed sewage-disposal plant and sanitary sewers by————, Engineer, and —— agree to furnish all the materials and perform all the labor required for the completion of the proposed work for the following prices:

Item	Approximate Quantity	Unit Price	Total Price
Sewage Disposal Plant, complete......			
Sewers, complete, including Y's, except subdrains, manholes, lampholes, and flush-tanks.			
18-inch........................			
15-inch........................			
12-inch........................			
10-inch........................			
8-inch........................			
Subdrains, complete			
10-inch........................			
8-inch........................			
6-inch........................			
Deep-Cut House Connections, complete.			
Manholes, complete.................			
Combined Manholes and Flush-Tanks, complete.....................			
Lumber Left in Trenches (per M., B. M.)			

All the above shall be strictly in accordance with the plans and specifications.

In case —— bid is accepted,—— agree to begin work within three weeks after the acceptance of —— bid, and to entirely complete the work on or before————————————,————.

——— further agree to enter into contract and furnish bond satisfactory to the City Council within 12 days after acceptance of —— bid.

Respectfully submitted,

————————————

————————————

94. Form for Sewerage Contract. Besides plans and specifications, the sewerage Engineer is sometimes called upon to furnish a *Form of Contract* to be signed by the Contractor and the city representatives, though this, more properly, should be the work of the City Attorney. The following simple form of contract has been used successfully with specifications such as those given above:

This Article of Agreement, made this ——— day of ———— A.D., ——, by and between ————————, of ————————, ————, party of the first part, and the Incorporated City of ———, ————, acting through its Mayor and Council, party of the second part,

WITNESSETH:

The party of the first part agrees to furnish all material and perform all labor required for the entire completion of sanitary sewers, subdrains, and other appurtenances, on streets in the said City of —————, ———, as follows:

(NOTE: In this space place a list of the sewers included in the contracts by streets, giving the sizes on each street of both sewer and subdrain, and the points at which each size begins and ends.)

All the above sewers are to have manholes and other appurtenances as shown by the plans and specifications.

The party of the first part further agrees that all the above labor and materials shall be strictly in accordance with the sewer plans and specifications prepared for the party of the second part by ——————, Engineer, said plans and specifications identified by the signatures of the parties hereto, being hereby made a part of this contract.

The party of the second part agrees to pay to the party of the first part for the above labor and materials, the following prices:

Sewers, complete, except subdrains, manholes, lampholes, and flush-tanks,			
	24-inch	$	per lin. ft.
	20 "		" "
	18 "		" "
	15 "		" "
	12 "		" "
	10 "		" "
	8 "		" "
Subdrains, complete,			
	24-inch		" "
	18 "		" "
	15 "		" "
	12 "		" "
	10 "		" "
	8 "		" "
Manholes, complete		$	each
Lampholes, complete			"
Combined Manholes and Flush-Tanks, complete			"
Flush-Tanks, complete			"
Lumber ordered left in trenches		$	per M., B. M.

The payments shall be made in——and paid to the party of the first part in accordance with the provisions of the specifications, 2 per cent being reserved for one year to guarantee the work.

In Witness Whereof we have hereunto set our hands and seals the date and place first above mentioned.

SEAL ——————————

Party of the First Part

The Incorporated City of ————, by

SEAL ——————————Mayor,

——————————

Party of the Second Part

95. Form of Bond for Sewerage Contract. The Contractor for a piece of sewerage work is usually required to furnish to the City a bond, which is frequently for a sum equal to about one-half the amount of the contract. The simpler the form of the bond, the better. The following form has been used successfully:

BOND

KNOW ALL MEN BY THESE PRESENTS, that we,—————————, of ——————————, ———, Principal, and

————————————————————————————————

————————————————————————————————

————————————————————————————————

Sureties

are held and firmly bound to the Incorporated City of ————————, ————, in the penal sum of ———————————————— Dollars (————————), lawful money of the United States of America.

NOW, THE CONDITION OF THIS OBLIGATION is that whereas the above-mentioned ——————————, of ——————————, ———, has entered into contract with the Incorporated City of ————, ———, dated ————., A. D. ———, to furnish all labor and materials required for the entire completion of about ——— feet of sanitary sewers, subdrains, and other appurtenances for the said City of ——————, ——————, now, if the said ——————————, shall well and truly perform all the obligations of his said contract, strictly according to the terms thereof, then shall this bond be null and void, but otherwise it shall be and remain in full force and effect.

————————————

Principal

————————————

————————————

Sureties

CONSTRUCTION OF SEWERS

96. Letting the Sewer Contract. After the plans and specifications have been completed and accepted by the City, the next step will be to let the contract for the work.

First. The work should be advertised, if possible, three or four weeks in advance, in at least two good engineering or trade journals. It must often, by law, be advertised also in at least one local journal. For a form for the advertisement see pages 112 and 113.

Second. On the day and at the hour specified in the advertisements, the City Council meets to open the sealed bids which have been submitted on the blank "forms for proposals" furnished by the City for the purpose.

Third. If the bids are satisfactory, the contract is awarded to the *lowest responsible bidder.*

Fourth. A contract for executing the work in accordance with the plans and specifications, is signed by the Contractor and by the City.

Fifth. The Contractor furnishes a bond satisfactory to the City.

In all these steps, there is need of great care on the part of the city authorities to make sure that all provisions of the law are complied with, and they should be fully advised at all times by a competent attorney.

97. Organization of Engineering Force during Construction of Sewers. It is not common for the Consulting Engineer who prepares the sewerage plans and specifications, to be constantly on the ground or even in the city during construction. He makes only occasional visits for inspection and consultation.

The actual work of sewer construction is usually directly supervised either by the City Engineer, or by a *Resident Engineer* employed especially for this purpose.

It will be necessary for the resident engineer in charge of the construction of a sewerage system of some magnitude, to have an office and an adequate equipment of drafting apparatus, surveying instruments, etc. He will have employed under him:

Draftsmen and clerks, in the office.
Instrument men and rodmen, to do the surveying.
Inspectors, constantly on all work, to insure its being properly executed.

The resident engineer himself will supervise these employees, visit all parts of the work frequently, and constantly exercise general supervision over all its features.

98. Laying Out the Sewer Work. After checking up the bench-marks on the original survey, it will be necessary for the engineering force to stake out the sewers, keeping somewhat in advance of the actual construction.

The stakes are usually placed a uniform distance to one side of the true line, so as not to be disturbed by the digging of the trench. This distance, and the side on which the stakes are placed, should be the same for all parts of the work, to avoid confusion and mistakes.

The stakes should usually be set about 25 feet apart.

The manholes should usually be located first, in accordance with the profile sheets; and the sewers should be run as straight lines, center to center of adjacent manholes. All discrepancies from the original measurements should each be adjusted, if possible, between the two manholes between which each was found; and such discrepancies should not be carried on to affect all the rest of the work.

There are two methods of giving grades for sewers.

(1) The best method is to set the grade stakes nearly flush with the surface, at a uniform offset to one side of the trench, ascertaining

the distance of the top of each stake above grade by carefully checked levels. By measuring from these stakes, a grade cord, supported on cross-frames every 25 feet, is stretched parallel to the grade line of the sewer, over its center line. For this method of giving grades, see Fig. 40.

(2) Another method is to set grade stakes at the bottom of the trench. This method is adapted only to very large sewers.

99. Trenching and Refilling. Sewer trenching and refilling may be done either by machines or by hand. *Excavating Machines* for sewers are of two types:

(1) *Machines which themselves do the excavating.* These are just coming into use, and are becoming more and more successful.

(2) *Machines which simply carry away the excavated material,* usually dumping it over the completed sewer further back. This type has the advantage of not piling up the dirt in the busy street. It carries, on overhead cableways or trestles, buckets which can be lowered into the trench, and in which the excavated material is placed by hand.

Machines of both types are suited best to comparatively extensive work; and under favorable conditions they lessen the cost materially.

Most sewer trenching, however, is done by *hand.* For such work the men are organized in gangs, the number of men in each gang varying from 20 to 80. Each gang has a foreman, and a water boy, and sometimes a sub-foreman. A pair of pipe-layers may work with each gang, or, if the trench be deep, one pair of pipe-layers may work part of the time with one gang and part with another.

The details of sewer trenching and refilling as ordinarily carried out, are specified quite fully in clauses 38, 40, and 41 of the sample sewer specifications given in Art. 93 (which clauses now read carefully). All details there specified should be enforced by the Inspector and the Engineer.

In clause 41, Art. 93, referred to above, the method specified for compacting the refilling is by flooding with water. While this is the cheapest method, where the water is available, and while it gives good results if properly done, it may be found necessary sometimes, in the case of paved streets, to adopt the more expensive method of *tamping.* For thorough tamping, there should be from 1 to 2 men tamping, to 1 shoveler, and the rammers used should weigh 4 to 6 pounds each. The soil refilled should be moistened if dry, and should be tamped in about 4-inch layers. It is possible by very thorough tamping to compact the soil more thoroughly than by flooding.

100. Sheathing. Except for shallow ditches in very solid earth, it is usually necessary to brace the sides of sewer trenches to prevent their caving in. Such bracing is called *sheathing.* The most common methods of sheathing are illustrated in Fig. 40.

The horizontal members of the sheathing are called *rangers,* and the rangers are held the right distances apart by *sewer braces* of wood or iron. The iron braces are shown in Fig. 40. The rangers are usually about 12 feet long. Behind the rangers are placed the vertical planks of the sheathing, either a few feet apart in firm material, forming *skeleton sheathing,* or in contact with each other in caving material, forming *close sheathing.* The sheathing plank are 2 inches thick and are usually about 10 feet or 12 feet long. The rangers may be 2-inch planks in favorable soil, or 4 by 4 or even 4 by 6 inches in poor soil.

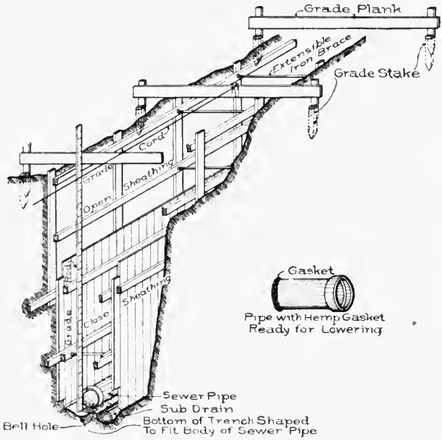


Fig. 40. Diagram Showing Construction of Pipe Sewer.

The sheathing plank are usually driven by hand, with wooden mauls.

Sometimes, for large sewers, heavy *sheet piling* may be driven by pile-drivers, to take the place of ordinary sheathing.

Ordinary sheathing is removed from the trench as the refilling proceeds. In case of special danger to near-by water mains, conduits, or foundations, on account of possibility of the banks caving before the refilling is finally settled, the Engineer may order the sheathing to be left permanently in the trench. In such case, the Inspector makes record of the exact amount of lumber left in the trench, and the City pays for it.

101. Pipe-Laying. The pipe-laying is usually done by two men, though, with large pipes, another may be needed. These men excavate the last few inches of the trench, as well as lay the pipes.

The laying of every pipe, and the making of every joint, should be carefully watched by an Inspector, who should faithfully enforce the specifications.

For specifications for pipe-laying, see clause 45, Art. 93 (which clause now read carefully).

All the sewer pipe should be carefully inspected before being used, and those pieces rejected which do not meet the specifications. See clause 25, Art. 93. The Inspector should see that no rejected or poor pipe is used.

The Inspector should see that every pipe is laid exactly to grade by measurement from the grade cord (see Fig. 40).

The Inspector should also see that house-connection Y's are placed opposite each lot on each side of the street, at the proper points; and he must exactly locate each such connection by measurements fully recorded in his notebook.

102. Construction of Brick Sewers. For specifications for the construction of brick sewers, see reference to Folwell in Art. 93, p. 122. (Read carefully.)

The construction of a brick sewer is shown in Fig. 41.

It will be the duty of the Inspector to inspect all brick before they are used, rejecting the poor ones, and to fully enforce the specifications for construction. He must also see that the templates are set truly to line and grade, that the house connections are set at the proper places and heights, and accurately located in his records.

In the case of large brick sewers, more trouble is to be expected with foundations than in the case of pipe sewers. Sometimes soft soil or quicksand may make it almost impossible to shape the material in the bottom to fit the outside of circular sewers. In such cases, special foundations, such as shown in Fig. 20, may have to be put in through the treacherous material. Other forms of special foundations are often used.

The Engineer should make full record of all such features of the work.

103. Records of Sewer Construction. *Daily Reports.* The resident Engineer in charge of the construction of a sewerage system, should require, from all members of his engineering force, daily reports, on suitable blank forms, showing the exact work on which each was engaged. Another set of exact reports should show the work accomplished by the Contractor each day, and the materials and labor used on each part of the work.

Data of Sewer Construction. The information from these daily reports should be entered in a permanent book, showing all features

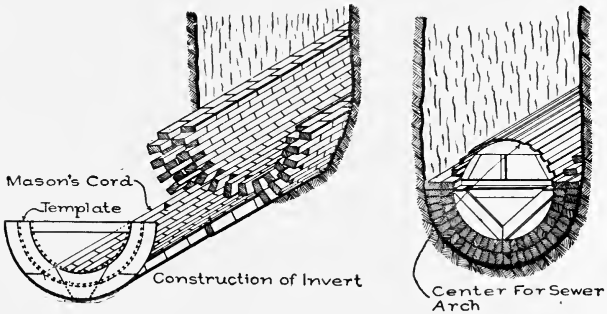


Fig. 41. Diagrams Showing Construction of Brick Sewer.

of the progress of the work, and giving data for itemized estimates of the cost.

Sewer Record Book. In another permanent book, a complete, final record of all the sewers should be entered.

On the left-hand page may be given in order the numbers of the stations of the sewer survey, running from the bottom to the top of the page, together with the surface elevations, the grade elevations, and the rate of grade.

The exact character of the soil should also be shown, with exact levels for computing any rock excavation. Notes should be made of the level and amount of any ground water encountered.

On the right-hand page should be made a large-scale sketch of the sewer, showing its exact location with reference to the street lines and the lot lines, and the exact location of manholes and other accessories. This sketch should also show the location of all house connections, with exact measurements (such as the station and *plus* of each connection) by which to locate all such connections.

On the right-hand page may also be entered the exact limits of sheathing left in trenches, and the amounts of lumber in such sheathing, as well as the exact limits and character of all special sewer

Fig. 42. Construction of Dry-Run Concrete Sewer, Waterloo, Iowa.

foundations, of changes of grade where other conduits are crossed, and of all other extra work.

Final Sewerage Map and Profiles. On completion of the system, the resident Engineer should make a complete final sewerage map, and complete final profiles of all sewers, both corrected by any changes from the original plans adopted during construction.

Plat of Sewer Connections. For small towns, at least, large-scale plats of the different streets should be prepared, showing the exact location of all house connections.

MAINTENANCE OF SEWERS

104. Sewerage Systems should be Carefully Maintained in Good Condition. Too often it appears to be considered that when a sewerage system is completed all further care of it can be neglected with impunity. This is a great mistake. The sewerage system may become a source of danger to the public health, instead of a means of safety, unless it is given proper care and attention.

105. Sewer Ordinances, Permits, and Records. Every city having sewers should pass a carefully prepared *Sewer Ordinance*, prescribing in detail the conditions under which citizens are permitted to use the sewers.

One provision of the Sewer Ordinance should be, that all property owners desiring to make sewer connections shall first secure a *Sewer Permit*. For this and for the application for it, blank forms are provided, which are to be filled in by the applicant, giving full description of the connection. The permit will require the work to be done according to the city regulations.

Every house sewer should be connected with the sewer at a regular house connection. No cutting into the sewer whatever should be permitted, as there is great danger of such cutting ruining the sewer.

Full *Sewer Records* should be kept by the proper city officers, showing full details of all connections with the sewers. This is too often neglected, to the great detriment of the City, which finds itself without means of ascertaining what people or how many are using the sewers, and perhaps putting injurious substances into them.

106. Plumbing Regulations, Tests, and Licenses. The city should also prescribe by ordinance strict *Plumbing Regulations*, setting forth in full detail the requirements for good plumbing (see Articles 76 to 81 inclusive). All property owners should be required to do all plumbing in strict accordance with these regulations.

The work should be carefully *inspected and tested* by a City Inspector, to see that it fully complies with the ordinance. The *water test* is applied by stopping up the outlets of the soil-pipe and of the various fixtures, and filling the pipes with water, when defects will be shown by leaks. In the *smoke test*, the pipes are blown full of smoke; and in the *peppermint test*, oil of peppermint is poured into them. In neither case must it be possible to detect any of the odor in the interior of the house.

Plumbing regulations usually require that plumbing shall be done only by plumbers holding *plumbers' licenses* granted by the City. The proper city officers have blank forms for making applications for such licenses, as well as for the licenses themselves. The plumber making application for a license should be required to show proof of proficiency, and should be placed under bond to comply fully with the sewer ordinance and the plumbing regulations, and to protect the City from damages on account of his work. The plumber may also be made subject to fines for violating the sewer ordinance and regulations, and to revocation of his license.

107. Regular Sewer Inspection. In sewer maintenance, besides the work of granting sewer permits, and inspecting house plumbing and the making of connections with the sewers, the entire sewerage system should be gone over regularly and carefully by a Sewer Inspector, once every two weeks if possible.

The Inspector, in this work, should open all manholes and lampholes, and carefully examine the sewer to make sure that it is keeping clean, well-ventilated, and reasonably free from offensive odors. He should also examine carefully the working of all flush-tanks, to make sure that they are operating satisfactorily. He should also examine all catch-basins, to make sure that they are cleaned frequently enough.

Small defects found on these periodical inspections should be remedied at once, and full notes made of more extensive work found to be necessary.

108. Flushing and Cleaning of Sewers. In many sewerage systems, it is found impossible to prevent absolutely the formation of deposits in the sewers, which must then be removed by hand-flushing, or by direct cleaning of the sewers.

Flushing is ordinarily preferred to hand-cleaning methods where the water for the purpose is available, and where it is readily possible to remove the deposits in this way. For the most common methods of hand-flushing, see Art. 25.

In hand-cleaning, large sewers may be entered by the workmen themselves to remove the deposits. In small sewers, lines are often floated down from one manhole to the next below; and by means of these lines, various cleaning devices are dragged through the sewer, or back and forth in it, to remove the deposits. Sometimes, for small sewers, a ball, a little smaller than the sewer, with a line attached to haul it back in case of stoppage, is allowed to float down the sewer, from manhole to manhole. The sewage is dammed back by it, and spurts out on all sides under pressure, thus scouring and cleaning the sewer.

For large sewers, discs or gates, traveling on carriages, or boats, may be used, working on the same principle. Many forms of such apparatus have been devised. A notable example of the use on a large scale of traveling sewage-scouring gates is in connection with the Paris sewers, Fig. 24.

109. Cleaning of Catch-Basins. In Art. 27, catch-basins were described; and it was stated that unless they are frequently cleaned they become filled with filth and soil and debris from the street, and fail utterly in their purpose, which is to keep such materials out of the sewers. Moreover—which is still worse than this—uncleaned catch-basins are unsanitary, and are sources of foul odors. Hence catch-basins, when used, should be regularly cleaned, and the City should have a regular arrangement for this work, and should provide labor-saving apparatus for the work, such as hoisting apparatus or special pumps for lifting the material from the catch-basins to the wagons.

SEWAGE DISPOSAL

110. Sewage Disposal Definitions. There is some confusion as to the meaning which should be given to the term *sewage disposal*, there being a tendency to treat it as meaning the same thing as sewage purification. It seems wise to hold more closely to the strict meaning of the words.

Sewage Disposal refers to the means adopted for disposing of, or getting rid of, sewage.

Sewage Purification is treatment of sewage to rid it of its foul impurities and render it harmless.

111. History of Sewage Disposal. In ancient times the only method used for disposing of the sewage of cities was to empty it into some stream or other body of water. This method, called *dilution*, is still in use more than any other, owing to its cheapness. From time immemorial, however, the cesspool has been used to receive the sewage of private houses, and we now know that a considerable percentage of purification is effected in cesspools, by bacterial action of the same nature as that now utilized in the modern *septic tank.*

By the middle of the nineteenth century the construction of sewerage systems had increased to such an extent that the streams in thickly settled countries became badly polluted by sewage, and it became necessary to turn attention to methods of purification. In England, especially, much work was done, and much success was attained with purification by *land treatment*, or *irrigation.* A great deal of work was done, also, in the same country, with methods of *chemical treatment.*

In 1887, the Massachusetts State Board of Health in this country began extensive experiments in sewage disposal, which soon demonstrated the great value of *intermittent sand filtration.*

About 1896 the *septic tank* came into prominence in both England and the United States. At about the same time, also, the *contact bed* was developed in England, and soon after copied in the United States, where it did not prove much of a success.

Within the last few years, *sprinkling filters* have come into use for conditions which require a large amount of sewage to be purified on a small area.

There is, at present, much activity in sewage purification, both as regards actual construction of plants, and as regards continued experimentation and research.

112. Importance of Sewage Disposal. The importance of sewage disposal at the present time is very great. All cities and nearly all villages find sewers indispensable, yet neither law nor justice will permit them to cause damage to the property or danger to the health of other communities or persons by discharging in their midst foul, unpurified sewage. More and more sewage purification plants are being required in connection with sewerage systems. Communities which disregard the rights of others in this respect are more and more finding that they must face damage and injunction suits.

113. Variable Composition of Sewage. Prior to taking up methods of sewage purification, it will be necessary to learn something about the composition of sewage; and the first thing to be noted is that the composition is extremely variable. Even in the same sewer, sanitary sewage is much stronger in the daytime, when the flow is heavy, than at night, when the flow is light. In fact, the composition will vary from minute to minute. Manufacturing and storm sewage, also, vary greatly in character at different places and times.

A sample of sewage for analysis, therefore, should consist of a mixture of several small amounts, taken systematically at different times, with great care to get a truly average portion each time.

114. Chemical Analyses of Sewage. Chemical analyses of sewage are *indirect*—that is, it is impossible to determine *directly* the amount and kind of polluting organic matter. Hence the chemist determines a number of things, harmless in themselves, from which he can judge in a general way of the amount and kind of the polluting organic matter. The things usually determined in a chemical analysis, and their meanings, are as follows:

Chlorine. This is in the form of common salt, in itself harmless. In sewage it indicates the strength of the original sewage, but not whether or not it has been purified.

Albuminoid Ammonia. This indicates the amount of undecayed organic matter, containing nitrogen, in the sewage.

Free Ammonia. This indicates the amount of decaying organic matter, containing nitrogen, in the sewage.

Nitrites. These indicate a further step in the process of decay (which is also the process of purification).

Nitrates. These indicate purified organic matter, containing nitrogen.

Oxygen Consumed. This gives an indication of the total unoxidized organic matter in the sewage.

Solids on Evaporation. These indicate the total foreign matter in the sewage, whether organic and therefore dangerous, or mineral and therefore probably not dangerous.

Loss on Ignition. This is intended to indicate the total organic matter which can be burned out of the solids on evaporation by heating them to a red heat; but if the water has a high mineral content, the loss on ignition does not appear to give a very reliable indication of the organic matter.

115. Bacterial Analyses. Bacterial analyses of sewage, as usually made, are quite simple, consisting simply of determinations of the total number of bacteria, without regard to their different kinds, in one cubic centimeter of the sewage. Most of these bacteria are perfectly harmless; but where many bacteria can flourish, disease germs might at any time flourish also.

116. Sample Analyses of Sewage. In Table XIV are presented a few random samples of chemical and bacterial analyses of sewage from sewage purification plants.

The sewage analyzed at Fort Des Moines was weak; that at Ames, stronger, but hardly of average strength; and that at Mt. Pleasant, of about average strength.

For each place, the *raw sewage* represents the *unpurified* condition; the *septic tank effluent*, the *partially purified* condition; and the *filter effluent*, the *purified condition* of the sewage. The purification at Fort Des Moines and at Ames was very good, and that at Mt. Pleasant poor.

TABLE XIV

Sample Analyses of Sewage from Purification Plants

Parts per 1,000,000

Place and Kind of Sewage	Chlorine	Albuminoid Ammonia	Free Ammonia	Nitrites	Nitrates	Oxygen Consumed	Solids on Evap.	Loss on Ignition	Bacteria per Cu. C. M.
Ft. Des Moines, Ia.									
Raw Sewage	16.0	6.0	2.5	Trace	0	20.6	490	130	713,000
Tank Effluent	18.0	6.5	8.5	0	0	15.9	524	166	582,000
Filter "	16.0	0.4	0.5	Trace	12.0	6.3	460	120	1,100
Ames, Iowa.									
Raw Sewage	60.0	7.5	20.0	0	0	96.2	950	230	495,000
Tank Effluent	61.0	5.0	21.0	0	0	99.6	978	226	849,000
Filter "	80.0	0.6	0.1	Trace	4.8	20.2	1,150	250	900
Insane Asylum Mt. Pleasant, Ia.									
Raw Sewage	150.0	5.0	28.0	0	0	64.6	2,412	440	1,680,000
Tank Effluent	157.0	4.0	28.5	0	0	105.0	2,072	278	1,210,000
Filter "	150.0	4.0	16.0	Trace	0	25.3	2,062	270	519,000

117. Methods of Sewage Purification. The principal different methods of sewage purification are as follows:

Irrigation. In this method, the sewage is used to irrigate crops, on a *sewage farm.* The method is very efficient with sufficiently porous land; but the large area required, and the difficulty experienced by cities in successfully operating sewage farms, restrict the use of this method in the United States almost entirely to the arid regions, where the soil requires irrigation anyhow, and where water for irrigation is scarce and valuable. The method is also used to a considerable extent in Europe, notably in Paris, in Berlin, and in Birmingham and several other English cities.

From 5,000 to 25,000 gallons of sewage per acre per day may be purified by irrigation, depending upon the porosity of the soil. Porous, sandy soils are the best. A very high degree of purification can be attained by irrigation.

Chemical Precipitation. In this method, certain chemicals (usually lime, alum, or iron) are added to the sewage, to precipitate the suspended organic matter, in *precipitation tanks.*

On account of the great cost of the chemicals and labor required, and the great difficulty of satisfactorily disposing of the large amount of *sludge* precipitated, the chemical treatment of sewage is now very seldom adopted, though it was quite popular twenty-five years ago. Only 25 to 50 per cent efficiency can be attained.

Settling Tanks. These are for a preliminary treatment, sometimes given sewage before filtering it, in order to get rid of part of the solid matter in the sewage, which otherwise might tend to clog the filters. Some bacterial purification also occurs in settling tanks.

Septic Tanks. These tanks are larger than settling tanks, and hold the sewage and sludge (or solid matter settling in the tank) long enough for bacteria to act and to effect partial purification. This is also, usually, a treatment preliminary to filtration. For further discussion, see Art. 120.

Intermittent Sand Filtration. In this method the sewage is discharged *intermittently,* upon the surface of sand filters. The sewage may or may not have first a preliminary treatment in tanks. This is a very efficient method, and is one of the most common at present in use. For further discussion, see Art. 122.

Contact Beds. These are filters of coarse material (say $\frac{1}{8}$-inch to one-inch size) with water-tight walls and bottoms, which are alternately filled with sewage, allowed to stand full for a certain *contact period,* emptied, and allowed to stand empty for a certain *aëration period.* This method was quite popular a few years ago, but proved inefficient and troublesome in many places, and is now largely out of favor.

Sprinkling Filters. These filters, also, are made of coarse material; but the sewage is *sprinkled* continuously upon the surface, so as to trickle slowly over the pieces of filter material, the outlet drains being left open all the time. This method has lately come into favor as requiring much less area than sand filters, though not so efficient, and is now the method commonly recommended where circumstances render it advisable to adopt high rates of filtration, though at some cost of efficiency. For further discussion, see Art. 123.

118. Methods of Sewage Disposal Now Most Commonly Used. These are:

(*a*) *Dilution,* where purification is not required.

Where, on the other hand, purification is required, the usual methods are:

Preliminary Treatment by
(*b*) *Septic Tanks,* or by
(*c*) *Settling Tanks,*
followed by
Final Treatment by
(*d*) *Intermittent Sand Filters,* or by
(*e*) *Sprinkling Filters.*

119. Dilution. In this method of sewage disposal, the sewage is simply discharged into the ocean, or into a lake, a river, or other body of water, dilution by the water being relied upon to prevent the creation of a nuisance by the sewage. In the water, bacterial processes of purification by decay start up, which, after a sufficient time, break up the organic compounds in the sewage, and finally render it harmless.

Dilution has, over other methods, the one advantage of *cheapness;* and this is still sufficient to decide in its favor in the majority of cases, when the body or stream of water utilized is sufficiently large to prevent a nuisance. Chicago furnishes a most notable example of large expenditure to secure sufficient dilution for its sewage, having already expended over $50,000,000 in building the great "Drainage Canal" (see Fig. 5) to take water from Lake Michigan for this purpose.

Most cities in the United States dispose of their sewage by dilution, and the sewerage Engineer should always give the possibilities along this line full consideration. He will usually advise adoption of the method (at least for the present) where it is cheaper, and where it is certain that a nuisance will not be created, nor serious damage or danger to other communities or persons result.

120. Septic Tanks. These are simply large tanks, in which the sewage is held long enough for most of the solid matter to settle out, and long enough for certain species of bacteria to act both upon the liquid sewage and the sludge.

Theory of Action of Septic Tanks. The bacteria which flourish in septic tanks belong to the general class known as *anaërobic* bacteria. This term means bacteria which do not need the oxygen of the air to live. In ordinary decay of organic matter, anywhere, both these and other bacteria are the active agents, and some of the germs are found in all sewage. In septic tanks the conditions are favorable to the enormous development of anaërobic bacteria, since the sewage is still, and there is usually no free oxygen in the sewage, and since, moreover, there is abundance of the organic matter which forms the food of these particular bacteria. In septic tanks, the bacteria act upon the sludge, to partially liquefy it; and they also act upon the organic matter in both the solid and the liquid state, to partially purify it.

Efficiency of Septic Tanks. In practice it is found that septic tanks remove only 25 to 50 per cent of the organic matter in the sewage, and that the bacteria in the effluent are very high in number (see Table 14).

Essentials of Septic Tanks. The only essentials of septic tanks are: (1) That the sewage shall be introduced and taken out in such a way as to insure a uniform distribution through the entire cross-section as it passes through the tank; (2) that the outlet shall be so arranged that neither the floating scum on top nor the layer of settled impurities at the bottom shall be permitted to escape; (3) that the tank shall be large enough to hold the sewage sufficiently long for the bacterial action, but not so long that the bacterial action proceeds too far, which might cause excessive offensive odor, and unfit the sewage for filtration. A capacity of 12 to 24 hours' flow of sewage is usually considered to be the proper size.

Septic tanks are commonly used preliminary to filtration of the sewage.

In Fig. 43 is shown the general arrangement of a typical sewage-disposal plant for a small city. It consists: (*a*) of a *septic tank* of 80,000 gallons' capacity, in which the sewage is first received; (*b*) of *two sand filters*, each containing 13,000 sq. ft. of area, through which the sewage is filtered after first passing through the septic tank; and (*c*) of a *sludge area* provided for drying the sludge after it is taken from the septic tank, preparatory to hauling it away.

Usually *a septic tank should be nearly emptied of sludge about once a year.* The sludge area may be simply a prepared earth area, on which the sludge can stand and drain. Usually the sludge area is at a lower elevation than the bottom of the septic tank, and the sludge

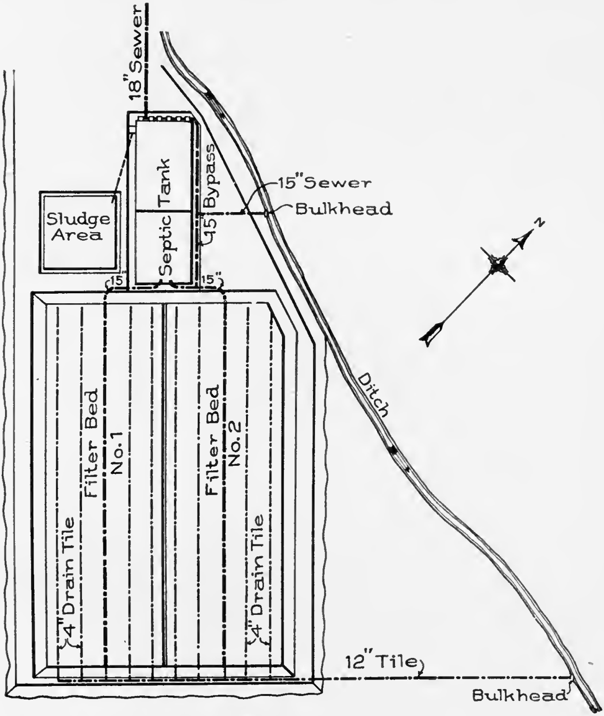


Fig. 43. Plan of Sewage-Disposal Plant, Carroll, Iowa. Original Scale, 1 Inch = 40 Feet.

is allowed to run out upon it through an iron pipe, by gravity. Otherwise a centrifugal pump may be provided for pumping out the tank.

In Fig. 44 detailed plans are given of the septic tank whose general location is shown in Fig. 43. The tank shown is made entirely of concrete, reinforced with steel rods. Even the flat roof is 5 inches of reinforced concrete. It consists of two parts as shown, a *septic tank proper*, and a *dosing chamber.*

The *septic tank proper* holds 60,000 gallons, and is divided longitudinally into two compartments, one twice as large as the other, to permit the size used to be varied to suit the amount of sewage flowing. Entering at the left-hand end of the tank, as shown in Fig. 44, the sewage passes into the tank through 6 openings, and, striking a baffle wall, the currents are forced down and spread out to give a uniform distribution of the flow. At the opposite end of the septic tank proper, the sewage must pass up under another baffle wall, and then flows into the dosing chamber over six weirs, opposite the six inlets. In falling from the weirs the sewage is aërated.

The *dosing chamber* holds 20,000 gallons, and is provided with two *alternating siphons*, one connected with each sand filter bed.

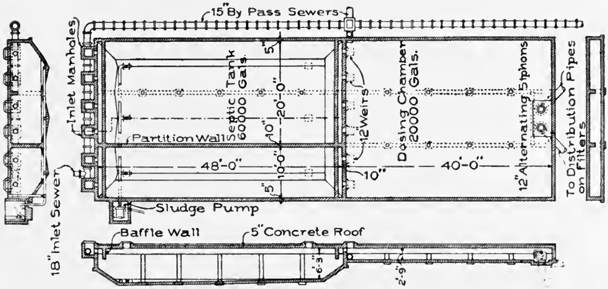


Fig. 44. Plan and Sectional Elevation of Septic Tank, Carroll, Iowa. Original Scale, ¼-Inch = 1 Foot.

These are similar to the flushing siphons described in Art. 24, but are so arranged that they discharge in rotation. Whenever the dosing chamber fills to the high-water line, one of these siphons discharges the entire 20,000 gallons within a few minutes upon the surface of its filter. The distribution of the sewage upon the filters is thus automatic. There are other types of automatic distributing apparatus. Those having moving parts are more liable to get out of order than are siphons.

In Fig. 45 is given a view of the above concrete septic tank during construction.

Many other designs of septic tanks are used successfully. In some, a house is built over the sewage.

In Fig. 46 is given an interior view of the dosing chamber of one of the septic tanks at Ames, Iowa. The alternating siphons appear in the view.

121. Settling Tanks. Settling tanks differ in no essential way from septic tanks, except in point of size. Settling tanks are made much smaller than septic tanks, and hence do not afford time for so complete bacterial action, and must be emptied of sludge frequently, instead of only once a year. Hence a complete, convenient, and inexpensive means of cleaning out and drying the sludge is even more important than in the case of septic tanks.

Fig. 45. Septic Tank at Carroll, Iowa, under Construction.

122. Intermittent Sand Filters. With the exception of irrigation under favorable conditions, intermittent sand filtration furnishes the most efficient means of purifying sewage which is in common use. In this method, the sewage is discharged *intermittently* upon the surface of sand filters $2\frac{1}{2}$ to 4 feet deep. The area of filter needed will usually be one acre to every 100,000 to 150,000 gallons of sewage per day. Any good, clean, coarse mortar sand will answer for the filter. The filter is usually underdrained by lines of agricultural drain-tile placed 5 feet to 20 feet apart; and the bottom of the bed is often covered with a layer of graded pebbles or broken stone, to make the drainage more nearly perfect.

Theory of Action of Sand Filters. In each cubic foot of sand are many millions of particles of sand, whose aggregate surfaces may amount to thousands of square feet, and these particles have many millions of intervening pores. Upon the surfaces of the sand grains, the bacteria of purification become established in innumerable billions,

Fig. 46. Interior of "Dosing Chamber" of Septic Tank at Ames, Iowa, Showing "Alternating" Siphons.

and they work upon the organic matter in the sewage slowly trickling past them. In sand filters the bacteria are of the general class known as *aërobic* bacteria, or those which require oxygen to live. Hence the application of sewage must be *intermittent,* to allow each dose to

Fig. 47. Cross-Section of Intermittent Sand Filter.

penetrate down into the sand out of sight, and draw air into the pores after it, before the next dose is applied.

Efficiency of Sand Filters. Sewage-disposal plants having sand filters should remove 85 to 98 per cent of the organic matter from the sewage, and 98 to 99.8 per cent of the bacteria.

In Fig. 47 is given a cross-section of one of the intermittent sand filters shown in Fig. 43. Each of these filters is 200 feet long by 65 feet wide, by 2 feet 9 inches average depth. A large sewer-pipe from one of the alternating siphons passes down the center on top of each bed, with 4-inch openings each side every 10 feet for distributing the sewage evenly over the surface. The sand is 2 feet 6 inches deep, and is underlaid with a layer of graded pebbles 0 to 6 inches deep. Lines of 4-inch agricultural drain-tile 13 feet apart are provided to remove the filtered sewage.

In Fig. 48 is given a view of a similar sewage filter under construction. In this case considerable grading had to be done out into a lake to get room for the filters.

Fig. 49 is a view of a completed plant, consisting of a septic tank, with intermittent sand filters. The purified effluent from this plant is as clear and odorless as spring water.

123. Sprinkling Filters. These are made of coarse material, say $\frac{1}{8}$ inch to 1 inch in size. Sewage flowing upon such coarse material would pass through the large pores too quickly to receive much purification. Hence the sewage must be sprinkled upon the top surface in drops to insure its simply trickling over the surfaces of the pieces of filter material. There are many devices for distributing the sewage in this way, including, principally, traveling perforated arms, and spray nozzles. All the devices need constant, intelligent care to keep them in order.

The material of which sprinkling filters are made may be pebbles, crushed stone, crushed coke, or any hard, durable material, crushed to the proper size.

Sprinkling filters possess the great advantage over other types, of the very high rate of filtration possible, and the small filter area consequently required. Rates of 1,000,000 to 2,000,000 gallons per acre per day have been proposed. They are not so efficient, however, as sand filters.

Theory of Action of Sprinkling Filters. In the case of sprinkling filters, owing to the coarseness of the pieces of filtering material, and the fact that the sewage is applied in drops, and runs over the pieces of the filtering material in films, without filling the pores, sufficient air remains constantly in the pores of the filters to keep alive the aërobic bacteria of purification. Hence the application of the sewage

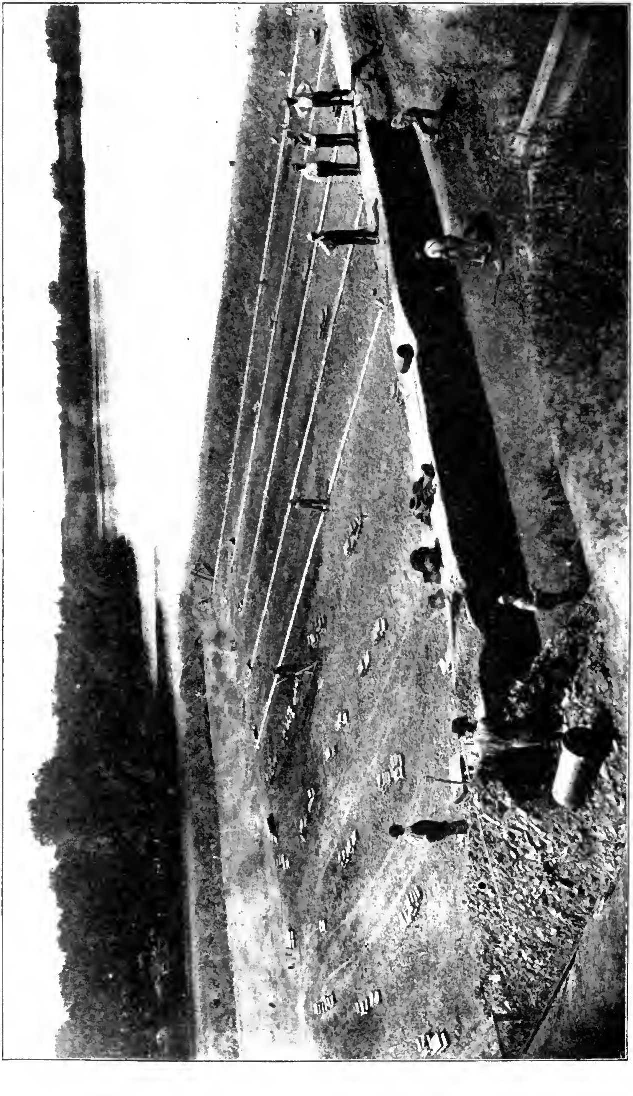


Fig. 48. "Intermittent" Sand Sewage Filters at Fairmont, Minn., under Construction.

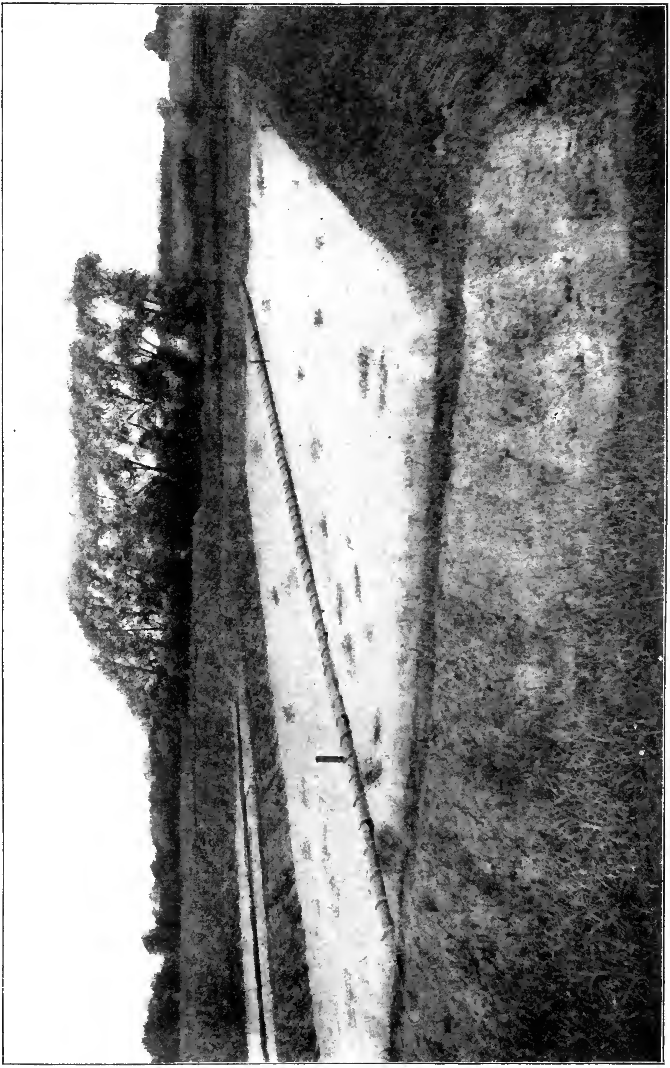


Fig. 49. View of Completed Sewage-Disposal Plant at Ames, Iowa, Consisting of a Septic Tank and Two "Intermittent" Sand Filters.

need not be intermittent as in the case of sand filters. However, the germs do not have time and opportunity to work so thoroughly upon the organic matter as in sand filters.

Efficiency of Sprinkling Filters. This is not nearly so high as for sand filters. Fine, black particles of partially purified organic matter often cloud the effluent to such an extent that settling tanks must be provided for clarification.

Sprinkling filters are suited best to large cities, and to cases where the highest efficiency of purification is not essential.

124. Maintenance of Sewage-Disposal Plants. Sewage-disposal plants, like other forms of apparatus, will not run themselves. For large cities, where men must be constantly employed to care for the

Fig. 50. Iowa State College (Ames, Iowa) Sewage Filters in Winter.

large plants, little trouble is experienced in securing proper care; but for small cities, sewage-disposal plants are often almost entirely neglected.

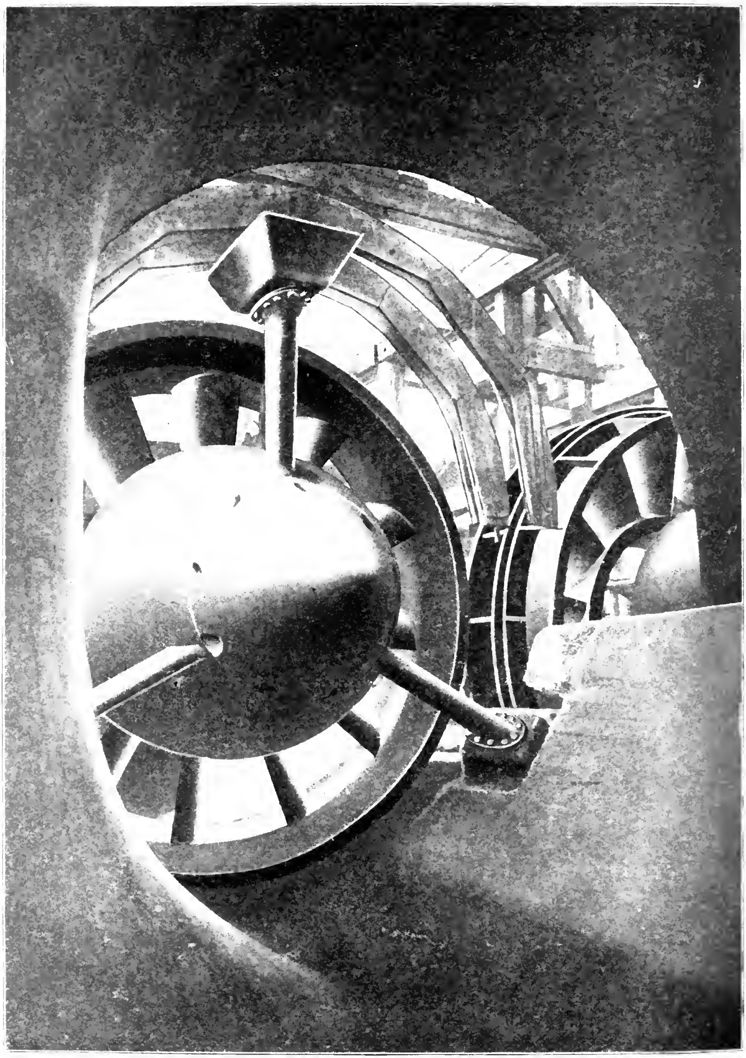
Every sewage-disposal plant should be visited at least once a day by an intelligent man, who should make sure at every visit that everything is operating properly, and who should remedy any trouble found.

Care of Tanks. Septic tanks require cleaning out about once a year. After the sludge is thoroughly dried, it should be hauled away and ploughed under for fertilizer. Besides this, the only care needed is to make sure that no passages are stopped up, that valves are arranged properly, and that siphons or other automatic apparatus work properly.

Care of Filters. Sand filters require to be raked or harrowed or dug or ploughed up, to loosen the surface, at intervals of a few days to two months, depending on the clarity of the sewage and on the rate of application of sewage. This is to keep the surface of the sand loose, so that the sewage can penetrate it. Otherwise the filter will become so water-tight that the sewage will continually flood it, which will drown the aërobic bacteria of purification.

At the approach of winter, the surfaces of sand filters should be ridged up with a plough, or by hand, into a succession of ridges and furrows. Ice, which forms only in very severe weather, will then be supported on the ridges, and will leave hollows underneath in the furrows for the next dose (see Fig. 50).

In case of sprinkling filters, the distributing devices require constant care, and the filter material may need occasional loosening up, or even washing or renewal. Sprinkling filters are not well adapted to very cold climates.



DEFLECTOR RING AND HUB OF SCREW PUMP, CHICAGO SYSTEM OF INTERCEPTING SEWERS

Erected at 39th Street Pumping Station.

PLUMBING

PART I

Plumbing occupies an important position among the trades as an application of Sanitary Science.

Sanitary science is defined by an eminent authority* as "that body of hygienic knowledge, which, having been sufficiently and critically examined, has been found so far as tested to be invariably true. Its phenomena are natural phenomena; its laws are natural laws; its principles are scientific principles."

The same authority defines the sanitary arts as "those methods and processes by which the applications of the principles of sanitary science are effected," and would include plumbing with other practical arts of construction involved in sanitary engineering and architecture.

Having thus noted the position occupied in this broad field by the matters under consideration, we may define plumbing as *the art of placing in buildings the pipes and other apparatus used for introducing the water supply and removing the foul wastes.*

Historically, the plumber is primarily one who works in lead; but this definition would be a misnomer applied to the handicraftsman of to-day. While in time past, and even within the memory and practice of men now working at the trade, it suited the occupation designated as *plumbing*, the term "plumber" survives the transition from lead to iron more by reason of established usage than from its fitness to indicate the workman of the present.

Two score of years ago, traps and soil, waste, and supply pipes were in many localities almost wholly of lead; and much of the larger pipe was hand-made. Lead was then everywhere more frequently used for all these purposes than it is anywhere in the country now. To-day, first-class plumbing is possible in any type of building without employing a vestige of lead, and that, too, with fixtures and fittings regularly on the market. Lead, however, is still used to a marked extent in plumbing, principally for traps, pipe connections, calked joints, water-service pipes, tank linings, flashings, etc. Its retention for these secondary purposes is due generally to superior fitness; yet

* *The Principles of Sanitary Science*, by Wm. T. Sedgwick.

in some instances it is because of the style of connection provided on certain fixtures, or for other reasons independent of the merits of the metal. On the whole, its loss of prestige has been slow and impartial. Indeed, those manually skilled in the manipulation of lead have often opposed the adoption of other materials sufficiently to retard substitution of the better.

Lead has unequaled merit for plumbers' use in specific instances; and if the trade has suffered by injudicious substitution of other material during its rapid evolution in recent years, time will adjust the error as the fitness of lead becomes apparent. For service lines in the ground, no other material lasts longer or gives more satisfaction than lead, provided the use of lead is safe with the particular water which flows through it. For cold-water lines inside buildings, it answers well. Wood tanks properly lined with lead are, in many cases, the best for indoor storage.

Lead pipe is not self-supporting in any position, in the sense that iron or brass may be considered so; and the providing of reasonably permanent support for lead work is an expensive item. Lead pipe costs more than iron or brass, in every case; and the cost increases proportionally with the extra weight necessary for all but very light pressures; while ordinary merchant's iron pipe, or seamless brass pipe of iron-pipe size, will withstand the pressure of any municipal or private supply in America.

Lead does not serve well for hot water. The contraction while cooling appears not to equal the expansion from heating; hence the pipe deteriorates at the hottest points, usually showing weakness first near the reservoir in the kitchen, especially at bends, and finally crystallizing beyond repair at those points. So much trouble has been experienced with stove and range connections of lead, that lead pipe for this purpose has been entirely abandoned. The wish to install something better suited than lead for hot-water service, is in large measure responsible for the general adoption of other material. Hot and cold supply lines that are dissimilar in material, in diameter, in joints, and in fastenings, are so unsymmetrical and out of harmony in every way that no mechanic is willing to install them for a slight real or fancied betterment.

With reference to the action of frost, lead pipe has an advantage in that the diametrical expansion of the water when freezing does not burst the pipe at the point frozen, unless it has been repeatedly swelled from the same cause. Lateral extension of the core of ice in the portion frozen, crowds the water which it cannot compress; and, as the ice is frozen to the wall of the pipe, the weakest place ruptures. Sometimes a faucet ball will be driven in, and occasionally a coupling collar will be stripped of its threads; but usually room is made for the extra volume of the water by the pipe swelling to an egg-shape and bursting at one point. Such a break can be repaired by wiping a single patch or joint on the original pipe.

Frost breaks in lead pipe nearly always occur on the house side of the point frozen, because the water in the street end is easily driven toward the main. Air-chambers on the house service would often obviate the bursting of lead pipe; but where the type of faucets or a limited pressure does not require their use in order to prevent reaction, plumbers frequently omit them, under the impression that air-chambers can serve no other good purpose.

With iron pipe, frost breaks are more serious. Diametrical expansion splits the pipe at the point frozen every time freezing occurs; and lateral extension of the ice staves in the faucet stems, etc., quite as frequently as would happen with lead pipe under the same conditions. Of late years, the improvement in types of buildings, more careful provision against frost on the part of plumbers, and the vigilance of the Weather Bureau in giving warning of approaching cold snaps, have made insignificant the amount of damage by frost in both kinds of pipe.

Lead pipe, as a rule, requires less trench work on ground lines than iron pipe, because drilling, even if very poorly aligned, will often suffice to get the pipe in place. There are numerous instances, however, where longer stretches of iron pipe have been placed in drilled holes than would be practicable with lead at the same excavating cost. It is well to remember that any small line of house service in the ground should be placed deeper, so far as immunity from frost alone is concerned, than is necessary for the protection of large pipes in the same locality, because the volume of contents in house pipes is small, the wall surface of the pipe relatively large, and the flow of the water not so regularly maintained.

The action of natural waters on lead has been a matter of wide discussion by able men. The subject of possible contamination of water supply through the agency of lead conduits, is too broad, however, for full consideration here, and will therefore be but briefly touched upon. This trait of lead has been voiced against its use, with more or less effect; but known cases of poisoning from this source have been exceedingly rare. Galvanized-iron pipe charges the water with salts of zinc when the water contains certain impurities; and most other kinds of pipe are also more or less open to objection at times by reason of their injurious effect on the water, the staining of fixtures, etc. Some of the salts of lead formed by the agency of water conveyed through lead supply pipe, are protective. Others, without doubt—fortunately of rare occurrence is actual practice—are corrosive. Sulphate or phosphate of lime, in solution, will part with its acid in passing through lead pipe, the acid combining with a new base (lead) and forming sulphate or phosphate of lead as the case may be. Chloride, sulphate, nitrate, borate, and other compounds of lead, may be similarly formed. These incrust the pipe; and such of them as are practically insoluble in water protect the lead from further attack, thus preserving the quality of the water. Carbonate, sulphate, and phosphate of lead, which doubtless form most frequently in lead water pipes, belong to the protective class. Of course, not all the compounds mentioned are encountered in any one source of supply. Chemical compounds designed to produce an insoluble incrustation have sometimes been purposely placed in solution, and allowed to stand in systems of lead supply pipe where it was known that the water to be commonly used would otherwise be dangerously corrosive. In view of the possibility of such precautionary measures, the deleterious effect of lead on many water supplies, and the consequent menace to health if lead were used indiscriminately, could hardly alone to any appreciable extent result in the substitution of pipe of other material.

Lead has been thus dwelt upon at the outset, because the industry of plumbing itself derived its name from this metal (*Plumbum*, Latin for "lead"). A discussion sufficient to define broadly the present and past status of the metal in the plumbing business, is certainly apropos in this connection. To many persons, the term "Plumbing" suggests lead and lead work generally, without regard to its distinctive forms, some of which are quite foreign to the ordinary trade meaning. To those acquainted with the building practices of Europe, visions of lead-covered roofs and spires, rainwater heads, etc., in addition to manifold other uses of the metal not common in America, may come to view in the mind's eye when "plumbing" is mentioned. To American plumbers of the past generation, "plumbing" suggested stacks of hand-made lead soil and waste pipe; hand-made lead traps; lead "safe" pans cumbersomely boxed-in under fixtures; ridiculously small lead ventilation pipes; lead drip-trays; lead supply pipes (sometimes also hand-made); all "wiped" joints and seams; and blocks, flanges, braces, boards, and boxes galore, jutting out in profusion, for supports, covering, etc.

In reality, we in America have now but little of what the name "plumbing" would lead the uninitiated to expect. Stacks of plain or galvanized wrought-iron pipe, or of plain, tarred, or galvanized cast-iron pipe, of weight to suit the height of building and to serve as main soil, waste, and ventilation pipes, with sundry lead bends and ends for fixture connections—these, with galvanized wrought-iron or brass pipes for supply, constitute the "roughing-in" stage of a job of plumbing; while painted or bronzed main lines exposed to view, galvanized-iron and nickel-plated brass pipe, with fixtures, partitions, etc., make up a view of the finished work, conveying little idea of the functions and importance of the unseen portions. Finished work in an unpretentious dwelling or storehouse, when properly charted, is fairly easy for even the house-man to understand. In large apartment and office buildings, department stores, etc., however, the plumbing, ventilating, gasfitting, heating, and automatic sprinkler pipes and electric conduits, make, in any but the finished state, a maze of pipe beyond the understanding of any except engineers well versed in those lines of work. In the completed work, the details are concealed. The toilet rooms present an orderly perspective of closets, lavatories, or other fixtures, as the case may be, with simple connections according with the customary finish, kind, or purpose of the pipe.

This apparent harmony, proportion, and simplicity in the result, coupled with a memory of sundry glimpses of a confusion of pipes in the rough state, has, it is to be regretted, propagated in many minds, a sense of false security regarding plumbing, based on the assumption of the plumber's evident ability to produce order and perfect service out of what in the "roughing-in" stage looked chaotic to a hopeless degree. The bulk of plumbing work, however, is not of the "skyscraper" class, nor is it handled by the same type of skill and superintendence. Any feeling of confidence or sense of security on the part of the public, is treacherous if based on the assumption that only by a degree of skill in direct proportion to the size of the job can satisfactory plumbing service be provided in residential and other small buildings. There is evidence of a somewhat indifferent state of the public mind regarding the plumber and his work, induced by the reasons stated and also by lack of due consideration and appreciation of conditions wrought by progress in other trades.

Plumbing, in its advancement, is merely keeping pace with the allied lines on which it is dependent. Their progress has created new conditions to be met; and as the future plumber will hail from the ranks of the populace, the light in which the public regards the plumber and the importance of his trade will have no uncertain bearing on the character and earnestness of those who take up the calling. The rank and file of apprentices have already too long been attracted merely on the score of a promising means of livelihood. There is ample reason to begin a plumbing career with all the pride felt by followers of any other vocation. It is altogether improbable that any individual will be found with so much education or such promising ability as to give rise to just grounds of fear that plumbing will not offer him sufficient scope to acquit himself with dignity.

The advent of tall buildings, the general increase in the height and other proportions of buildings in cities, and the changes in material and in design of fixtures, together with the abnormal demand resulting from the decreased cost, natural growth, and gradual awakening through education to the value of sanitary conveniences, have brought about a condition of affairs which the old-line plumbers were incapable of coping with, and which the old apprenticeship system was inadequate to provide men capable of dealing with in a creditable manner. The plumbing of one large building involves as much work as hundreds of the average small jobs put together. The handling of such work under the conditions that have prevailed, has developed a deplorable state of so-called "specialism." Men engaged in "roughing-in" a large job are likely to tell you with entire truthfulness that they have no idea what types of closets or other fixtures are to be used; that they know nothing of the principles or merits of plumbing fixtures, and do not need to; that they never connected a fixture in their whole career; that the finishers do that kind of work. By further inquiry one would find the "finishers" utterly at sea in the work of "roughing-in," and accordingly ignorant of the whys and wherefores that govern the success of a job as a unit. These men, called "plumbers," are exceedingly skilful and rapid within their limitations; but it is easy to infer the fate of a job intrusted to such hands alone, and in practice it has been proven that others of metropolitan practice, and merely lacking in variety of experience, were not capable of creditable results on general residence work of the ordinary class.

When the largest jobs were completed in a comparatively short time, and when much of the training which went to make up the plumber's accomplishments was credited to the manual practice necessary to master the working of lead and solder, a period of service in shop and job practice, coupled with oral instructions from the journeyman, served fairly well to make a plumber out of raw material within the period allotted by the American abridgment of the apprenticeship term. On the work of to-day, however, there would be great chances of an apprentice serving such a term without seeing anything of more than from two to five jobs. He would be lucky if it fell to his lot to get even a little experience in each of the natural divisions of those jobs; and again fortunate if those jobs happened not to have the same general layout or to employ identically the same make of fixtures, for there are many shops which seem to have the faculty of securing work from certain particular sources, and which are equally likely for one reason or another to be recommending and using, where possible, one particular make of goods to the exclusion of other kinds just as good or better. These and kindred features now met with on every hand in practice, are stumbling-blocks—prohibitive, in fact, of anyone learning the plumbing trade within any period of time that can sensibly be prescribed for the acquiring of a trade or profession.

For more than a decade, the often-avowed reluctance of journeymen to teach apprentices has been held responsible for the trend of these affairs affecting the practice of the industry; but in the light of what has been said, it is easy to determine what it was that really introduced the Plumbing Correspondence School and Plumbing Trade Classes. It was *necessity*. Trade journals have done and are still doing good work in this line; but their best efforts, added to the opportunities of practice, were insufficient. There was no other satisfactory solution than the Correspondence School—no other route to the acquisition of principles and acquaintanceship with the accumulated information as to the relative merit or fitness of certain materials, designs, systems, etc., and as to the conditions under which this or that would serve well, while it might act just the reverse under other circumstances.

Under the present *régime*, it is not only apprentices and those who intend becoming such, but journeymen as well, that need to seek aid in the schools. The citizen at large, also, serves his own interest in informing himself in a general way at the same fountain, so as to be able to discriminate for himself in matters pertaining to plumbing. Furthermore, any real plumber would prefer that his customer should be familiar with the work in hand. Fewer misunderstandings occur when such is the case, and there is a keener appreciation of good work on one hand and a corresponding effort to merit approval on the other. There is, too, in favor of the plumber, when the customer is informed, an absence of those niggardly tactics of trying to secure much for little, of sacrificing quality and future satisfaction by reducing first cost below the safe limit. The well-informed customer never makes you feel that all plumbing is alike to him and a necessary evil to be paid for at rates far in excess of its value.

With the foregoing introduction in mind let us look further into the subject and see what "Plumbing" really is. Whether we are actual or self-nominated apprentices, journeymen, masters, or the prospective customer himself, a view of the matter will be beneficial, if only in the sense of refreshing memory.

There was a time when sanitary conveniences, crude in comparison with the present, were considered mere luxuries. Under the present views of life and the conditions of living, we may with greater propriety consider these erstwhile luxuries as actual necessities, though they are often luxurious to a degree that dwarfs into insignificance other appointments which even then were granted to be essentials. Plumbing is, therefore, neither in fact nor in opinion, a matter of simple luxury for the rich and delicate, but is, rather, an important subject of deep salutary interest on the one hand and of business acumen on the other—a matter of essentials deeply affecting the best interests of our own health and that of our neighbors, with which mere sentiment has no ground for association. The time when it was thought sufficient to fan out the mosquitoes in summer and break the ice in winter at the family rain barrel in order to wash our faces and hands, has passed. A dwelling job may now embrace almost the entire range of plumbing fixtures. There is therefore no better example from which to build a word-picture of Plumbing.

PLUMBING FIXTURES

Bathtubs. Bathtubs are a prime factor in plumbing. They are of various types:—(1) Wooden cases, with sheet-metal lining, usually copper, on the order shown in Fig. 1; (2) all copper, and steel-clad, suitably mounted, as shown in Fig. 2; (3) cast iron, enameled, with a vitreous glaze fused on the iron, as in Figs. 4 and 5; (4) solid porcelain, potter's clay properly fired, with vitreous glaze fired on, as in Fig. 3; and (5) marble, variegated or otherwise, cut from the solid block. Their cost ranges in the order mentioned.

The relative merit of the different materials and types is not so easily designated. Porcelain and marble baths are large, very heavy, and imposing-looking; and therefore are often selected on the score of massiveness, with a view to harmonizing with the dimensions and finish of the house. One would suppose the mass of material in such baths would have the effect of cooling the water to an annoying extent; but careful tests have revealed no appreciable difference in the effect of thin as compared with thick bathtubs on the warmth of water, and but little in their pleasantness of touch to the person. The bath of most pleasant touch was that of indurated wood fiber, which, however, had but little commercial success, on account of its lack of stability.

Most baths are made in from two to five regular sizes, ranging from 4 to 6 feet in extreme length. The general shapes are the *French* (Fig. 3); the *Modified French* (Fig. 4); and the *Roman* (Fig. 5). The various French patterns have the waste and supply fittings at the foot, which is modified in form to accommodate them. The waste water travels the length of the tub to reach the outlet, and generally leaves scum and sediment on the interior while emptying. Baths of the French type are suited to corner positions, or to positions in which one side runs along the wall; but the ideal position for a bathtub, in the interest of cleanliness, is with the foot end to the wall,

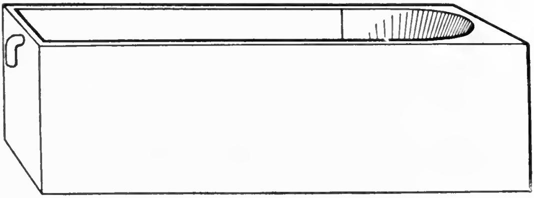


Fig. 1. Wooden Case Bathtub, with Sheet-Metal Lining.

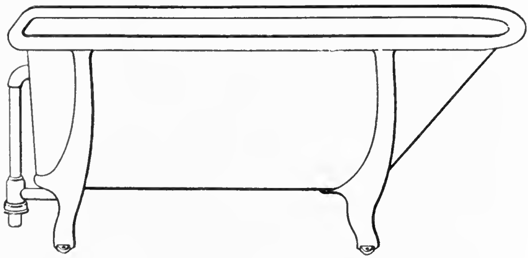


Fig. 2. All-Copper, Steel-Clad Bathtub.

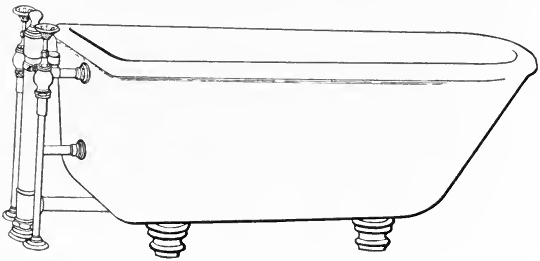


Fig. 3. Solid Porcelain Bathtub, French Type.

thus permitting entrance from either side. A medium size is best suited to the usual provision for supplying hot water for bath purposes; and is also preferred by many because the feet reach the foot, enabling a person, when submerging the body, to keep his head

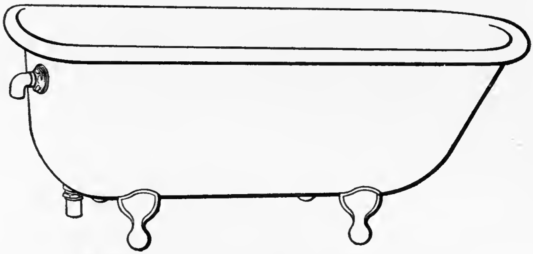


Fig. 4. Enameled Cast-Iron Bathtub, Modified French Type.

out of water, with his shoulder resting on the slant at the head of the tub. Where the house supply is pumped by hand, the medium size of any kind of bath is advisable.

The rims of baths vary from $1\frac{1}{2}$ to 5 inches in width. The larger rims are easy on the person in getting in and out of the bath, and are often used in lieu of a bath seat. In iron baths with rims large enough, the fittings are generally passed through the rim, as illustrated in Fig. 6, thus giving them additional stability and making

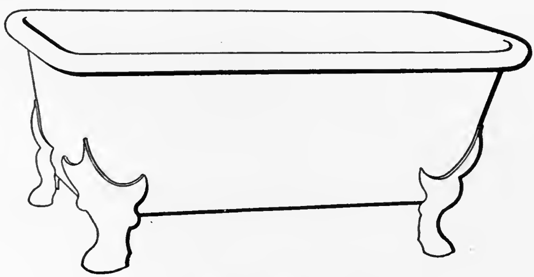


Fig. 5. Enameled Cast-Iron Bathtub, Roman Type.

the stated fixture length include the whole space necessary for its installation. This style of bath fitting is shown in Fig. 7.

Nominal sizes of baths now include the whole length of the fixture proper. Formerly many awkward mistakes resulted from lack of uniformity, one not always knowing whether to consider the nominal size as inside measurement only or including twice the rim width. In cast tubs, actual measures vary slightly from the nominal, because of the furnace effect when heating to enamel. The variation, however, is not sufficient to be considered in noting the space required, or to require any advance in roughing-in measurements.

Roman baths have ends alike, with the fittings at the center of one side, as illustrated in Fig. 8, and the waste outlet at the center of width and length. In general, they empty with better effect, and may be placed in either right or left corner or free of all the walls;

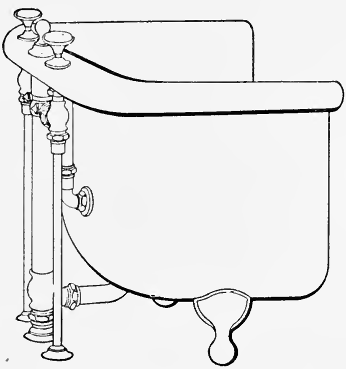


Fig. 6. Fittings Passed through Rim of Enameled-Iron Bathtub, to Give Additional Stability.

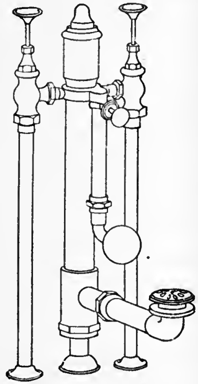


Fig. 7. Style of Bath Fitting Intended to Pass through Rim of Tub.

but the best position, everything considered, is with the fitting side near the wall, and not against either end of the room.

Any finish for iron bathtubs, other than plain paint, should be put on at the factory; iron surfaces cannot be ground and the successive coats of paint dried on in place, properly or cheaply.

Waste fittings and the outlets of baths have always been made too small. Slow emptying takes valuable time, and results in the adherence of scum, which necessitates careful cleansing of the bath before it is used again.

The fittings of baths are not interchangeable unless the obliqueness of the tub walls and the depth and drilling agree. The styles of fittings are universally applicable, except that double bath-cocks

CONSTRUCTION OF "HAPPY HOLLOW" SEWER, LOUISVILLE, KENTUCKY

View showing arch and invert, with longitudinal and transverse reinforcing bars; also, in right background, one of the concrete manholes, the top of which is at the new grade that will be formed by filling in after completion of the sewer.

(Fig. 9) are never placed on Roman baths. All double cocks are provided with detachable coupling and sprinkler, which, fitted to hose, provide a means of spraying the body. Independent spray, needle, shampoo, and overhead shower fixtures, simple and in combination, with or without curtains, are made for use with the various tubs, the tub serving as a receptor for the falling water.

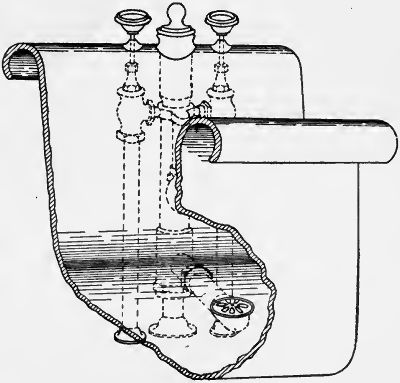


Fig. 8. Showing Central Location of Fittings and Waste Outlet in Roman Bathtub.

The cheapest serviceable bath fittings are a Double Cock and Connected Waste and Overflow. These are shown in Fig. 10. Bell Supply and Waste fittings, a special type of which is shown in Fig. 11, are singularly popular, the water being retained by a ring valve attached at the bottom of the overflow pipe, and operated by means of a knob projecting above and through the top of the waste standpipe. This takes the place of the ordinary plug and chain used with the simple overflow. The supplies are made and fitted in combination with the waste arrangement, with the valve handles projecting above the rim of the bath, the two supplies being delivered into a common yoke-piece, where they mix and flow through a common passage to the bell-piece fitted through the vertical wall near the bottom of the bath. With the usual slotted-bell delivery, these fittings are a nuisance in one respect. Water cannot be drawn into a vessel through the bell for any ulterior purpose; and as no vessel of considerable capacity can be filled at the lavatory faucets, or at a sitz or a foot bath, the sink faucets are the only resort unless a

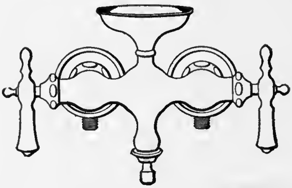


Fig. 9. Double Bath-Cock. Never Used on Roman Bathtubs.

slop sink is available. Nozzle-delivery bells, which afford some relief in this respect, are made; and hand sprays used in conjunction with them avoid the expense of special shower fixtures, which would otherwise be essential if shower or spray were desired at all.

A modification of these fittings, termed "Top-Nozzle Supply and Waste" (Fig. 12), overcomes this objection to the strictly "Bell Supply" type. It has a high nozzle delivery projecting into the tub, and is fitted for spray attachment. The inward projection is much less than with a double cock, which, in a short bathtub, would occupy much needed space. The noise of falling water, obviated with the bell placed low, is the same as with the double cock; and the mixing space, intermediate between that of a cock and the regular bell delivery.

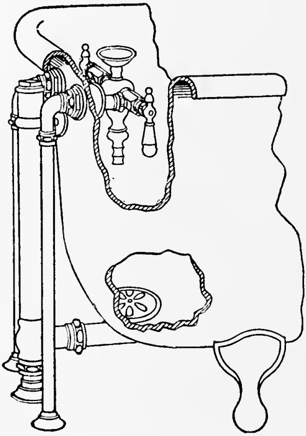


Fig. 10. Common Type of Double Cock and Connected Waste and Overflow.

An element of danger is inherent in a bell-supply outlet placed so low down as to be submerged when the tub is in use. If the supply is opened when the tub contains dirty water, and the pressure of water is lowered by accident or by opening faucets elsewhere, it is quite possible that the fouled water will be drawn back through the bell or nozzle into the supply pipes, thus, perhaps, contaminating the water for domestic use. For this reason, cocks which discharge near the top edge of the fixture, above the level of the water, are increasingly used at present.

For private use, where both children and adults are to be regularly served, the bathtub is the only fixture answering the requirements. As the physical conditions of the members of the family are, or should be, mutually known, and the tub will be regularly cleansed between baths, any possible chance of communicating humors of the skin through the bath can be guarded against. For institutions and general public use, the tub bath is open to serious objections, some of which apply as well to private use. The water for a tub bath is at its best when first drawn into the tub; and the person, before bathing, is certainly in condition to pollute it more or less. As the bathing process nears completion, these conditions are exactly reversed. Tubs used by the public may not be carefully cleansed between times of use, and the bather is ignorant of the condition both of the tub and of the person who used it previously. In institutions for the insane and feeble-minded, unscrupulous attendants have been known to bathe several persons in the same water. Large pools are better, but still not ideal; nor are they always suitable or practicable.

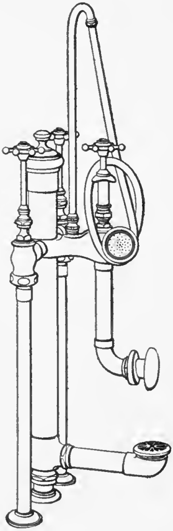


Fig. 11. Bell Supply and Waste Fittings.

Shower Baths. Shower or rain baths are commonly installed in barracks, gymnasiums, and schools, and are no longer unusual in private dwellings. Some of the objections to the tub bath, which have been stated, are entirely avoided by the shower fixture with its supply of running water. Those who have studied the hygienic effects produced by the action of jets or streams on the surface of the body, urge very strongly that the impact results in stimulating the proper action of the skin. This is the opinion of most persons who have had experience with such apparatus.

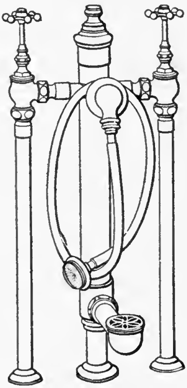


Fig. 12. Top-Nozzle Supply and Waste Fittings.

The older forms of showers, which direct the water vertically upon the head of the bather, are not so desirable as those in which the outlet is inclined and placed at about the level of the shoulders, thus avoiding wetting the head unless desired. Indeed, all the essentials of a bath of this form are met by a water-supplied rubber tube discharging at about the level of the waist over a tight floor or pan provided with a drain.

Aside from the shower baths that may be provided in conjunction with a bathtub, one type of which is shown in Fig. 13, many designs are fitted to floor-pans, called *receptors,* usually having a curtain, as in Fig. 14, thus providing for private installations a great variety of complete showering and spraying appointments. The receptors may be enameled iron, porcelain, or marble. A cement or asphalt floor, sloping to a drain, is simple and effective.

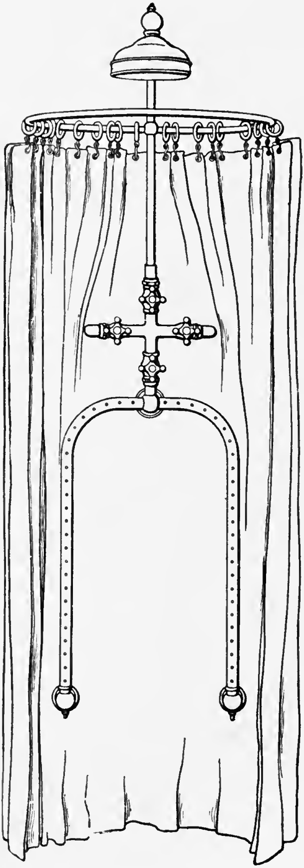


Fig. 13. Type of Shower Bath Provided in Conjunction with Bathtub.

In lieu of the full curtain and regular receptor capable of providing six to eight inches' depth of water, and having tub-like supply and waste fittings in addition to the shower features, a shallow base of marble provided with a drain and having three marble sides, such as is shown in Fig. 15, can be provided with any preferred type of shower fittings. The overhead douche, already noted, set at an angle, with flexible joint for adjustment, as seen in Fig. 16, so that the body can be played on without wetting the hair, is not often fitted to private shower fixtures, as it requires considerable additional space. A rubber cap for the head enables one to use the vertical shower with a fair degree of satisfaction.

A point concerning shower fixtures and relating to the safety of the user, to which special attention should always be given, is that of the valve arrangement. If the design renders it at all possible, as sometimes is the case, one is apt inadvertently to scald himself by at first turning on hot water alone. The chances of injury in this way increase with elaborate combinations, if not carefully guarded against by the designers; and we should not take it for granted that they have provided such safeguards. As a rule, reliable makers do embody ample mixing chambers, thermometers, etc., in such apparatus,

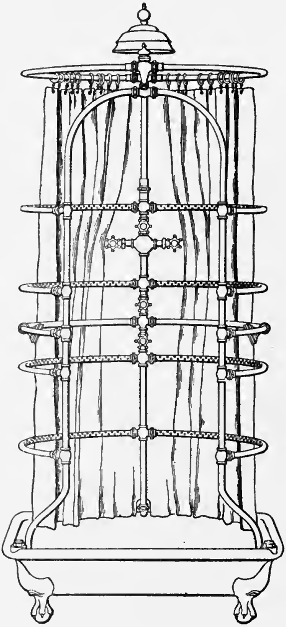


Fig. 14. Shower-Bath, with Curtain Fitted to Receptor.

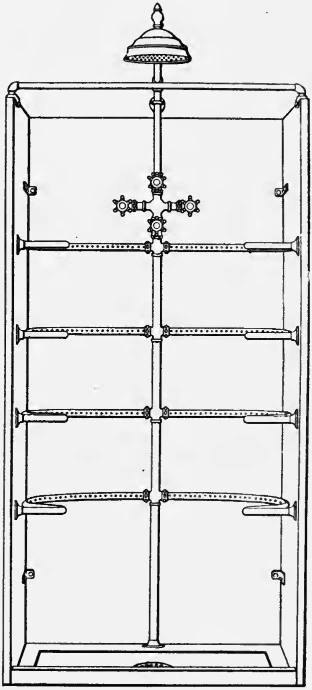


Fig. 15. Shower-Bath with Three Marble Sides and Shallow Marble Base.

where necessary, and they regulate the control of hot-service valves, or in some other way render the improper use of them unlikely.

Sitz Baths. These are primarily for bathing the hips and loins in a sitting posture, but may be fitted with special features as ordered. Porcelain and enameled iron are the usual materials. The fixtures approximate in dimensions 15 inches in height at front and 26 inches at back, and are 26 to 30 inches wide. In the back, at a proper height, in a complete fixture, like that shown in Fig. 17, is a horizontal slit accommodating fittings for a "Liver Spray"—a wide wave-like spray of water, either hot, cold, or of intermediate temperature, as suits the person. In the bottom, in conjunction with the outlet, is a hot or cold douche, equally under control of the user. In the center of the douche, and operated independently, is a Bidet jet. These provisions are entirely separate from and independent of the regular supply fittings, but one waste fitting is used in common for all. The simple sitz bath has the regular Bell Supply and Waste, like those used on the bath, the dimensions being diminished to suit. For the extraordinary features, these fittings are merely adapted in a way to give the user convenient control. For all but the simplest fixtures, the control appliances are invariably fitted through the rims, the valve handles being provided with proper indices to guide the user. Bidet jets in combination with sitz-bath fittings, have to a great extent curtailed the use of separate Bidet fixtures. Bidet jets have often been added to a water-closet, but a satisfactory application cannot be made to a closet. Separate Bidet fixtures are now rare, but are furnished by

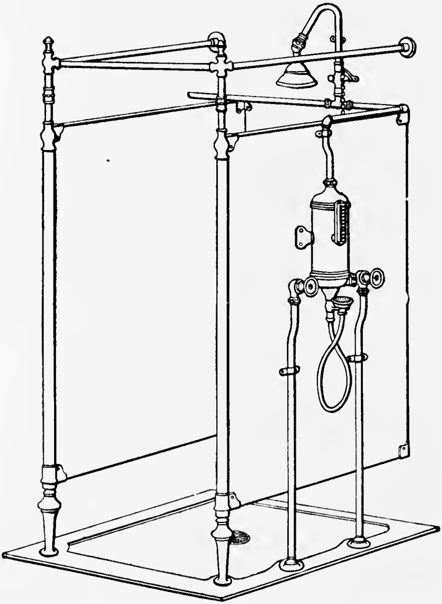
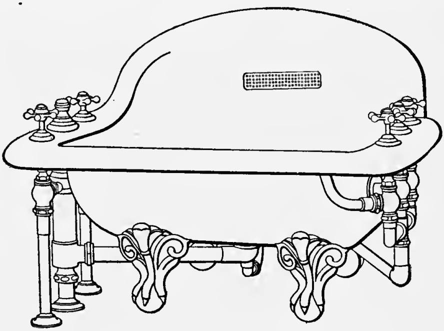


Fig. 16. Shower-Bath Fittings with Overhead Douche Set at an Angle on a Flexible Joint.

fixture makers; and in isolated cases, where frequent or regular use is necessary, are preferable to any combination with a fixture used for other purposes.

The sitz bath is conveniently used for a foot-bath, thus making this fixture doubly useful. Indeed, the sitz bath is a more comfortable means of bathing the feet than is the foot-bath itself. Children's bath-tubs, small, and elevated by legs to the height of a lavatory, are made, but no well-defined demand exists for them. Greater convenience to the nurse, the use of less water, and quicker filling and emptying, are the only points in their favor.

Foot-Baths. The foot-bath is a small rectangular tub with proper feet and rim, furnished with supply and waste of the regular bath pattern, diminished to suit. The sizes average say 12 inches deep, with 20-inch sides. The feet make the total height about 18 inches. Fig. 18 gives a good idea of the usual enameled-iron foot-bath fixture. Enameled iron and porcelain are the usual materials. They require even less water than the sitz bath, but, as before said, are not so convenient for the purpose as the sitz fixture, and are not installed except in the most spacious and elaborate bath-rooms. The foot-bath would serve admirably as a child's bath, except that it is too near the floor.


No. 17. Sitz Bath, with Complete Fittings.

Bidet Fixtures. The majority of leading fixture makers do not now catalogue these. They consist essentially of a pedestal like a closet pedestal, with bowl and rim contracted in the center, giving an outline something like the figure 8. Proper fittings to operate the jet and waste are provided. Porcelain is the material. As mentioned before, Bidet jets are furnished in combination with receptor shower fixtures, as well as with sitz baths.

Drinking Fountains. Drinking fountains are now frequently used in stores, schools, and residences, the various fixtures adapted to such installations being readily obtainable. The basins or drip-slabs for public indoor fountains, are often cut to order by the manufacturer; and the cooling and faucet arrangements are provided by the plumber. Porcelain, enameled-iron, and marble fountains of stock designs are made. For schools, trough-like basins, either with open spouts for continuous streams, or with self-closing faucets, as shown in Fig. 19, are frequent. The fixture shown in Fig. 20, consisting of solid porcelain, in which the recessed drain-slab and the high back constitute a single piece, is of recent design, presents an excellent appearance, and has the advantage of being easily kept in immaculate condition. The three deep waste outlets, above each of which is a faucet, afford facilities to many users in a short space of time.

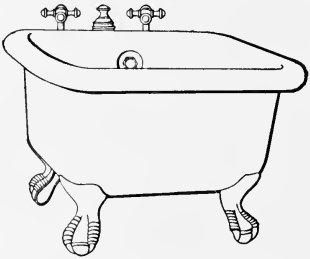


Fig. 18. Common Type of Enameled-Iron Foot-Bath.

One device which serves well for common use, is the ordinary lavatory, provided with a stiff perforated bottom fitting extending well up toward the top of the bowl. This, with a proper faucet on the slab, and a cup-chain fitted to the extra faucet-hole, makes a useful but not attractive fixture.

Recessed porcelain and enameled fountains designed to be placed in wall niches, and having concealed connections, as suggested by Fig. 21, are neat, and require very little room outside the finished wall line. Countersunk slabs with strainer waste, with back either integral or separate, as design or material dictates, are made in marble and porcelain. Marble fountains are adaptable to any location, because the slab and back can be cut to any shape or dimensions preferred. The fountain proper, faucet, cup, and pipe waste connection, with strainer, are all that is supplied by the makers.

A type of fountain shown in Fig. 22, is provided with a flowing jet of water from which one can drink without placing the lips in contact with any metal surface. The small central bowl or cup is constantly submerged and cleansed in the stream of water which passes outwardly over it, thus avoiding the danger incident to the common use of the same drinking cup by many persons. The surface

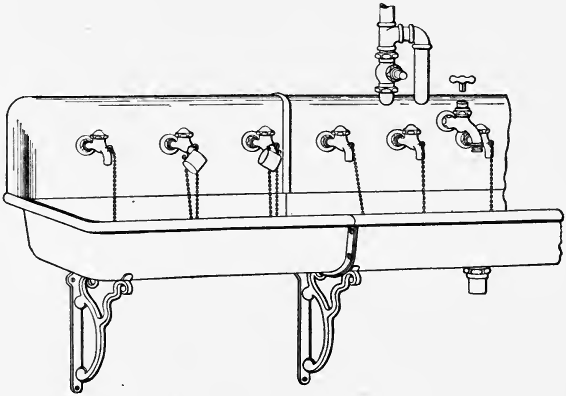


Fig. 19. School Drinking Fountain—Enameled Iron, with Self-Closing Faucet.

does not afford lodgment to possible germs of disease, which are most liable to transmit contagion when allowed to become dry and adhere to a surface.

Lavatories. Lavatories are made from porcelain, enameled iron, marble, and onyx, in numerous patterns. The number of designs is so large that they are best understood if considered in the classes into which they may be divided. In marble and onyx fixtures, the slab, back, and bowl are necessarily separate pieces. In any but very accurate fitting and erecting, the unavoidable joints soon, if not from the beginning, invite the accumulation of dirt. Poor workmanship, settling, abortive countersinks, and

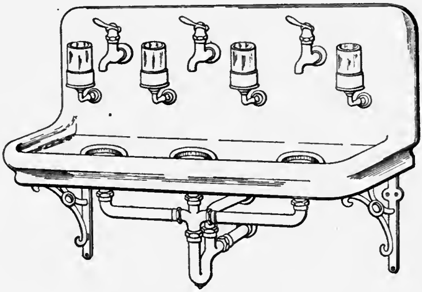


Fig. 20. Porcelain Drinking Fountain, Recessed Drain-Slab and High Back in One Piece.

faucet bosses not cut free within the countersink, have in many cases brought slab types of basins into unjust repute, or, at least, have given basis for strong talking points against them, which have been effectively so used. If made and installed in the most approved manner, these styles, properly cared for, offer little

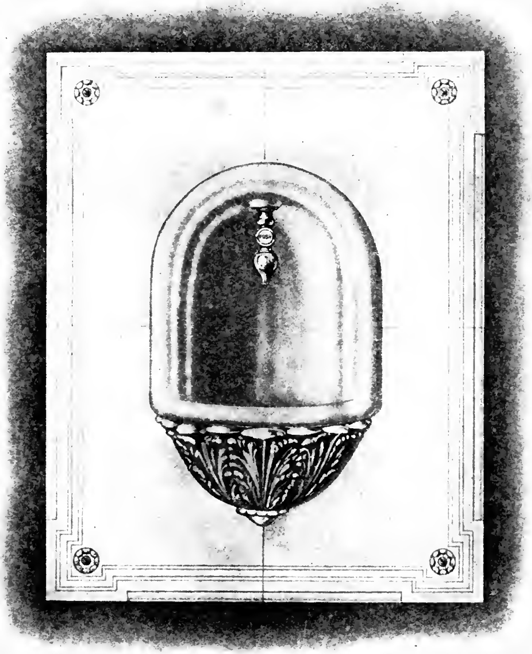


Fig. 21. Porcelain Recessed Drinking Fountain.

reason for severe criticism. One fact, however, must be borne in mind when comparing marble with other materials used for plumbing fixtures—namely, that marble is *not an impermeable stone.* Nearly all marbles (excepting only the very hardest and most dense) are quite absorbent, and depend upon the surface finish given to the slab to resist the entrance of liquids into the body of the stone. As soon as the surface becomes roughened by wear, the greasy and acid wastes penetrate into the pores, and the marble becomes permanently discolored. Only a limited observation of the bad condition of marble floors or urinal slabs which have been subjected to use for a few years, is necessary to confirm this statement.

Ordinary Tennessee, Veined Italian, Hawkins County Tennessee, and Statuary Italian marble, range in cost in the order mentioned. Fancy imported marbles and onyx are much more expensive. Tennessee marble varies in color from grayish brown to very dark reddish brown, uniformly intermixed with light specks. The Hawkins County marble is bright reddish and white-mottled. All the ordinary materials are cut in stock sizes, and may also be had to order, like the more costly, in any size and shape desired.

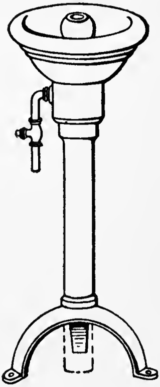


Fig. 22. Drinking Fountain. No Cup Necessary.

The type with apron or skirting, shown in Fig. 23, has legs, and the slab is supported continuously by the skirting. In those supported by brackets or leg-brackets, the strength of the slab is depended upon for support between the bearings. Legs, brackets, and all other metal trimmings should be in keeping with the character and cost of the stone slab. If brackets are properly spaced, the weight is so balanced as to leave very little sagging strain on the center of the slab. A shelf of marble, or a mirror with marble frame, or both, may be fitted above the back as a part of the fixture.

Porcelain and enameled-iron lavatories have bowl, back apron, and soap-cup in one piece. The pedestal of the lavatory illustrated in Fig. 24 is separate, of course, and no back is required, but the general features of integral construction are shown. There are no joints to open. The only injury possible to them is the marring or fracture of the glaze or enamel. Porcelain and iron lavatories, unlike those of marble, are adapted to pedestal support; and some very desirable patterns are therefore made in these materials only. Neither pedestal nor wall lavatories are suitable for use, except where the wall or wainscoting is of marble, tile, or some other waterproof material.

Fig. 23. Brazilian Agate Slab Lavatory, with Apron and Legs.

To provide for leaving the floor clear and free of obstruction, lavatories supported on brackets or hangers, as indicated in Fig. 25, with supply, waste, and ventilating pipes fitted on or into the wall, are best. If found practicable, a neater job results if all pipes leading to and from pedestal lavatories are carried through the pedestal. A supply and waste run to the floor is generally far easier and cheaper to secure than the fitting of all pipes to the wall.

The purchaser seeking iron or porcelain fixtures, has no choice of styles beyond that which the market regularly affords. If he prefers the workable materials, he should insist upon certain features of design which are essential to the best service. Abrupt edges and sharp corners should be avoided; the slab ought to be at least $1\frac{1}{2}$ inches thick, and the back not less than 12 inches high; the general dimensions must be as liberal as space will allow or the service demands (not less than 22 by 32 inches for a 14 by 17-inch bowl); the countersinking must be deep, $\frac{3}{16}$ to $\frac{1}{4}$ inch; the faucet bosses must not join the general border level at all; the faucets must not be less than 12 inches apart, nor so near the bowl that it will be difficult to secure them to the slab; nor may they be placed so close to the back as to make repairing troublesome with any type of Fuller faucets; the joint surface of the bowl must be ground to fit the slab, and provided with not less than four well-drilled anchor-holes for clamps to secure it.

Fig. 24. Lavatory on Pedestal.

Round bowls were formerly quite generally in use, but are now almost relegated to memory. The width of slab needed for a roomy, round bowl is too great; and at best the arms of the user must be cramped in a somewhat vertical and awkward position, while the smaller sizes are very uncomfortable in this respect. The sudden opening of the faucet when the bowl is empty, is likely to ricochet water with annoying results. This is caused by the water striking the curved bowl surface at a tangent, and is not peculiar to the circular bowl; the oval or crescent, or, indeed, any shape of bowl that presents a curved surface to which the faucet stream is tangent, favors the same result; the ovals in integral fixtures are the most annoying. Marble and onyx have an advantage over porcelain and enameled lavatories so far as ricocheting is concerned. The opening in the slab is not so large as the bowl, and thus a horizontal overhanging ledge is formed all around, above the bowl, which generally intercepts the water in a way to keep it off the floor and person. Porcelain and enameled fixtures have not this virtue. The bowl surface, being integral with the slab, is uninterrupted and continuous; hence ricocheting is more violent with them than is possible with the separate bowl.

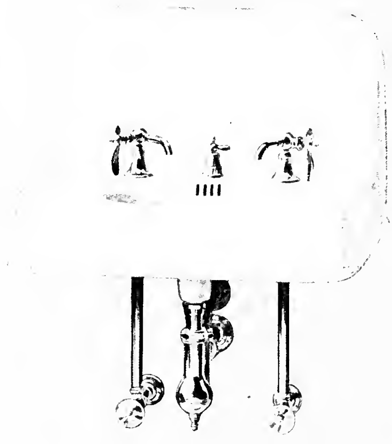


Fig. 25. Lavatory Supported on Brackets.

Oval bowls are now in general use on all types of lavatories. They employ slab space to the best advantage, and are the most convenient for use. The crescent or kidney shape, illustrated in Fig. 26, is, however, as far superior to the simple oval bowl as the oval is to the round. It permits the forearms to lie in a natural and most convenient position when dipping water to lave the face. This form of bowl should be accompanied with a scalloped or recessed front. The D-shaped bowl, and other bowls embracing the prime feature of the D-shape, while not so graceful in appearance, are, without exception, to be preferred, on the score of utter absence of ricocheting when the faucets are properly placed. The D-shape, a transverse section of which is shown in Fig. 27, has a semi-oval front, with the end lines continued parallel some distance past the major axis, and with a straight-line back nearly vertical. This form gives a nearly flat surface in the bottom between the back wall and major axis, on which surface the stream strikes and breaks when the bowl is empty. A depth of water is quickly formed under the stream, which checks any spraying or spattering.

The traps used for lavatories are lead or brass (either cast or tubes), or combinations of these materials, plain or vented or of anti-siphon design. One trouble with lavatory trap ventilation, is the difficulty of obtaining a vertical rise directly above the trap. These vent connections should be carried as nearly vertical as possible, as high at least as the bottom of the lavatory slab, before any horizontal run is made; otherwise the choking of the waste pipe would float solid matters into places from which gravity would not dislodge them. In the absence of water-wash in the vent pipe, these solids would obstruct the vent and defeat its purpose. This danger is not given due attention by many plumbers. The patent and horn overflow bowls, with plug and chain, are the cheapest effective means of controlling the overflow and waste from the bowl. The standing waste, of essentially the same design as the waste fitting for a bathtub, with the body fitting projecting through the slab at the rear of the bowl, is perhaps the most satisfactory waste and overflow arrangement. Various schemes for operating basin stoppers by means of levers and swivels, are employed; but none of them has come into more than limited use.

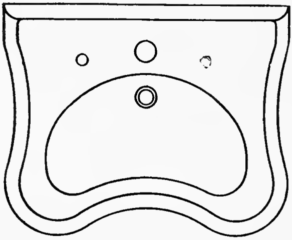


Fig. 26. Plan of Lavatory Slab with Crescent or Kidney-Shaped Bowl.

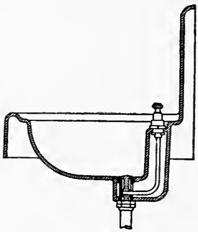


Fig. 27. Transverse Section of D-Shaped Lavatory Bowl.

Basin faucets, aside from special designs, are made on three general operating principles—(1) screw-compression; (2) eccentric action without springs; and (3) self-closing. They are also made in two types—with regular and low-down nozzles. All of these are represented in Fig. 28. The regular type has the nozzle some distance above the base flange, and screws into, or is cast on, the body. The low-down type has its nozzle with a flat bottom, hugging the slab as closely as practicable. The objection to the low-down is the inaccessible narrow space between the nozzle and slab, which becomes filthy and is difficult to clean. High, projecting nozzles obstruct the space over the bowl, especially when washing the hair, but are otherwise most satisfactory. The high nozzle gives trouble with patterns of faucets that separate in the body for repairs, such as the Fuller type, which closes rapidly with pressure. The fault, however, is often that the slab is so shallow as to necessitate the faucets being placed too close to the back to turn without removing the nozzles. If these are cast on, removal of the whole faucet is required before it can be separated. Some faucets are made with union joint in the body, thus avoiding such trouble; but these are not widely used.

The false economy which often dictates the purchase of a small slab, generally also prevails in the selection of its trimmings. Compression faucets close against the pressure, and are slow in action, causing practically no reaction. They are generally responsible for the omission of air-chambers on supplies of medium pressure. On account of their slow action, they are suitable for high pressures although but little weight is given this fact by the trade. The features essential to good, lasting service in the compression faucet, are: a cross-handle, a stuffing box, a raised seat, and a swivel disc. Self-closing faucets of various patterns are made with a view to preventing waste of water, the intention being to compel the user to hold the faucet open only as long as water is needed, and to insure automatic closing when it is released. There are none such except the crown-handled, that an ingenious person cannot find means to hold open at will; yet, withal, self-closing faucets are of great value in reducing wastage. A rabbit-eared faucet can be kept open by placing a ring over the handles while squeezed together; the telegraph bibb, by weighting down or tying up the lever; and the T-handled, while not so easily controlled, can be tied open by a lever secured to the handle. The crown-handled design can be operated with ease by the hand of the user, but does not readily lend itself to unauthorized control by means of a mechanical stop. Self-closing faucets require strong and well-designed springs to close them against the force of the water. They have sometimes come into disrepute through leakage for lack of adequacy in this feature of their construction.

HEADINGS CONTROLLING SUPPLY OF WATER FOR THE SMALLER LATERALS FROM THE MAIN IMPERIAL CANAL

View five miles northwest of Calexico, California.

Lavatory supports should have positive means of leveling the slab, such as set screws, screw-dowels, or whatever adjustment the kind of lavatory and support may be best suited to. Lavatory brackets are generally at fault in having limited bearing at the bottom of the wall-face. This point of the bracket is where all the strain is thrown against the wall, and the effect is noticeable if the upper end springs away ever so little. Full-length brackets are not open to this criticism, but they interfere with the washboard or other finish next the floor.

Sinks. These are made in four general classes according to the purpose to be served—namely, Kitchen, Pantry, Slop, and Factory or Wash-Sinks. The materials used are:—Porcelain; enameled,

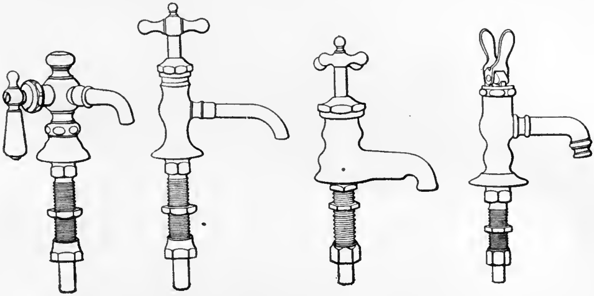


Fig. 28. Common Types of Basin Faucets.

galvanized, and painted cast iron; enameled, galvanized, and painted wrought iron; brown glazed ware; copper; slate; soapstone; various compositions; and occasionally wood. Porcelain and enameled cast iron are most used, galvanized and painted sinks being confined principally to factory use. Sinks of extreme length, in one piece, as shown in Fig. 29, or sectional, 6 to 8 inches deep, with supply and faucets over the center line or at the side, belong to the factory class. These are usually provided with a flat rim, rest on pedestals, and are not over 24 inches wide. There are also roll-rim patterns, with bracket support and iron back, and with faucets fitted through the back. These are generally 8 inches deep and about 20 inches wide.

Kitchen sinks vary in size according to general requirements. Common sizes are 18 by 30 inches and 20 by 30 inches. The depth ranges from 6 to 7 inches. There are two types of iron sink—flat-rim, with outlet at end; and roll-rim, with outlet in center. Neither style of outlet is always desirable as to connection; but the center outlet drains more directly. The flat-rim type is not provided with legs. Cast legs were formerly furnished, being attached to the sink by slipping into dovetails. When legs are desired for this type, the plumber provides gas-pipe legs, with or without a top frame. Iron splash-backs are provided for flat-rim sinks, but not of the deep pattern in which air-chambers may be cast. Plumbers drill these sink rims to attach brackets or legs, and sometimes also to secure to them hardwood capping or drainboard. Hardwood drainboards are generally provided by the plumber's carpenter. Hardwood splash-backs, set free of the wall to permit circulation of air behind the fixture, are also provided. Sometimes marble splash-backs are provided. Marble is best, but is not in keeping with a flat-rim sink. The back may extend to the end of the drainboard, or merely cover the length of the sink. Omitting the back behind the drainboard, as represented in Fig. 30, is often thought desirable. The drainboard should be free of the wall when the back is not extended. Iron sinks, with roll rim on front and ends, are furnished with drainboards suited to attach to either or both ends. These may be added as an after-consideration, or changed from side to side at will, if there is but one drainboard, or removed entirely, without marring the looks or service of the sink. This interchangeability commends itself to both plumber and customer.

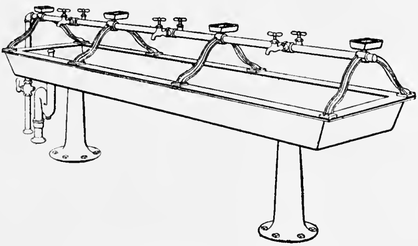


Fig. 29. Long Wash-Sink for Factory Use.

Roll-rim sinks, with the end recessed to receive a drainboard, are also made, which give good service, but in any subsequent change of location require setting in the original relative position.

Wooden drainboards, with an iron end to attach to sink, and enameled-iron drainboards, are furnished if ordered.

Open strainers are most frequently fitted to sinks, in which case the sink cannot be then used for washing dishes, but merely serves as a support for dishpans and other vessels and as a catch-all for drippings from the drainer. Hence the open-strainer sink must be large enough to accommodate suitable washpans, etc., while one fitted with a plug-strainer should be relatively small if it is designed to use the sink proper as a washpan.

The use of wooden sinks in large installations, such as hotel kitchens and restaurants, is not unusual, the theory of their use being that less breakage of crockery occurs, by reason of the softness of the material. The argument against the use of wood is not given due weight in this connection. The well-recognized objection to any porous, absorptive material which retains moisture and is subject to decomposition, is especially to be considered in the use of wood for greasy wastes. For the reason mentioned, wood is never a suitable material for this use.

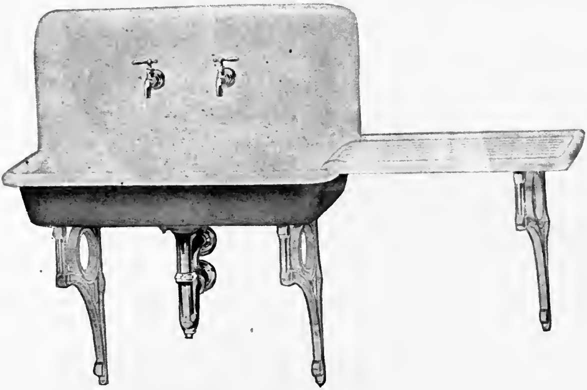


Fig. 30. Enameled-Iron Kitchen Sink Supported on Brackets. Splash-Back Omitted behind Drainboard.

Rubber mats are essential for both sinks and drainboards having enameled or glazed surfaces, in order to avoid accidental injury to the articles cleansed. As a matter of fact, the average dwelling has but one sink, which serves both kitchen and pantry purposes. Dual service is not always satisfactory, however, as no sink can be well adapted to both uses for a large family. A plug-strainer sink should also be provided with an overflow.

Porcelain and iron sinks have generally been supplied with loose backs; but sinks of one piece—that is, with sink and back integral—are now obtainable. Sinks with integral apron or skirting all around, to be placed free of the wall, are suitable for installation where the wall is waterproof.

Sinks are built from slabs of natural stone as desired, and may be with or without drainboard or skirting. They are generally provided with a high splash-back. These sinks are not limited to the patterns of a moulding room, and easily keep pace with the desires of the purchasers. Selection is confined to a choice of material, as every desirable type of fixture is easily supplied.

In the use of any natural stone, such as slate or soapstone, for plumbing fixtures, and especially for sinks, it should not be forgotten that angles and rectangular corners are with difficulty maintained entirely free from deposit. Although the flat surface can be readily scoured, it is always difficult to clean the sharp angles and corners satisfactorily. The difficulty is increased by the fact that some plastic jointing material, such as putty or cement, must be used in putting together the fixture; and small fragments of this material project into the angles and render the corners rough. Stone and porcelain sinks are heavy, and require careful packing for shipment.

Air-chambers may be cast in iron sink-backs. The ordinary sink-back is not well suited to the convenience of the plumber where supplies to any fixtures pass up behind the sink. The faucet-holes cannot be changed, and slots for pipe are not provided at the top edge. Sawing these gaps after the goods are enameled, leaves the fixture with an unfinished appearance. The proportion of shank to the handle of faucets of the Fuller pattern used on sink-backs, must be such that the handles will turn straight back.

A popular fixture of comparatively late design, adapted for small dwellings and now made in the cheaper materials, is the kitchen sink in combination with a single laundry tray, an example of which is shown in Fig. 31. In this, the drainboard serves as a cover for the tray when the sink is in use. Sinks have also been supplied in combination with lavatories, one sink being placed in the center or at the end of a battery of lavatories.

A pantry sink (Fig. 32) should always be provided with a drain-board. It is a smaller fixture than the kitchen sink, and is nearly always of the plug-strainer and overflow type. Its faucets are generally of the high-nozzle type, like those for shampoo purposes, but of smaller capacity and better adapted to rinsing than are kitchen-sink faucets. Indeed, the pantry sink proper need not necessarily differ at all from sinks used for other purposes. Every feature of its trimmings and setting is intended to best serve the butler's needs.

The waste matter from the butler's sink is not like that from the kitchen sink; hence the waste pipe is not necessarily so large, nor is a grease-trap so badly needed. Grease in considerable quantities finds its way into kitchen-sink waste pipes. It floats on the stream of waste water as it travels through the pipe, and, being always next the interior surface, either adheres thereto on contact, or by a reduction in temperature is chilled and congealed, thus clinging to the pipe walls. Successive layers of grease are in this way accumulated, and the bore of the pipe is finally reduced so much that solid matter easily completes the stoppage. Forcing out, and then filling the pipe with boiling lye water, and again flushing with hot water, will usually remove most of the obstruction. Sometimes the lye loosens the grease in chunks, which clog the pipe seriously at the first favoring point, and the pipe must then be cleaned manually.

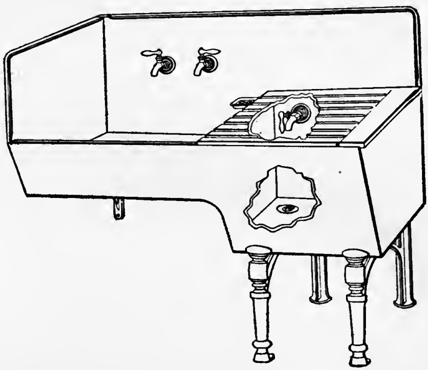


Fig. 31. Kitchen Sink and Single Laundry Tray Combined.

When once choked with grease, the pipe must ultimately be opened and cleaned by hand, often at material expense when long lines are deep underground. To avoid this trouble, various traps (of which two examples are shown in Fig. 33) have been designed to separate and collect the grease, either by flotation or by chilling—generally by the former. Traps to collect the grease by flotation were formerly improvised by the plumber, being placed in the drainpipe just outside the building. This location left too much pipe subject to choking between the grease-trap and the sink; and the trap itself often became a generator of bad odors in warm weather.

The grease-traps now commonly furnished are placed in the kitchen under the sink, and frequently serve as the regular trap for the fixture. The grease is easily removed by lifting out the container or by skimming from the top. Hinged bolts with thumb-nuts secure the covers so that they can be easily and quickly opened and securely closed.

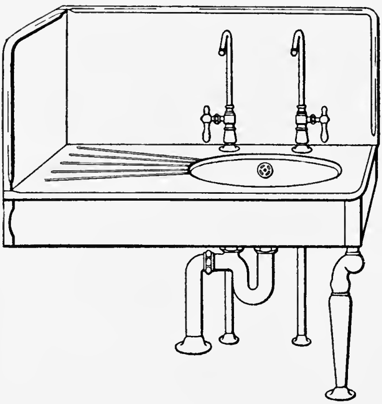


Fig. 32. Pantry Sink.

Traps which chill the grease are not used so much as those acting by simple flotation, but they do the work perfectly. The chilling process is accomplished by means of a water jacket through which the cold-water supply passes. The water entering low, surrounds the wall of the pot trap within, and passes out high up on the opposite side (see fixture at left in Fig. 33). Circulation—or, rather, change of water—in the jacket, is dependent on the amount of water used at the fixtures.

The usual slop sink is 18 by 22 inches and about 12 inches deep. Generally it is furnished mounted on a trap standard, as in Fig. 34, which serves the double purpose of support and waste-trap.

Care should be taken before installing a fixture placed upon a trap standard, to examine carefully whether the seal of the trap is provided for by suitable interior partitions. It is not uncommon to find defects in the casting, if of iron or brass—or in the porcelain, if of that material—which would seriously affect the maintenance of the water seal. In fact, it is desirable in connection with slop sinks, as with all other fixtures, that the trap be of such a form as to show clearly, even after being set in place, the position of the various portions which constitute the trap and maintain the water seal.

The waste pipe is never less in diameter than 2 inches, and is usually 3 or 4 inches. The outlet is invariably through an open strainer.

Slop sinks are made in all the materials common to other fixtures except natural stone. These sinks are to the chambermaid what the kitchen sink is to the cook. The shape and liberal-sized waste are well adapted to removing slop and scrub water. In the complete fixture, the sink is provided with an elevated tank and flushing rim,

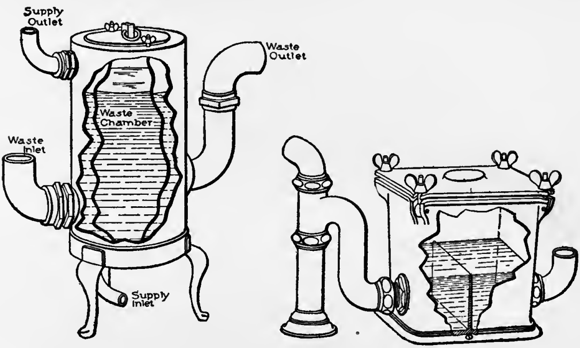


Fig. 33. Types of Kitchen Sink Traps for Separating and Collecting Grease.

to cleanse the fixture walls; also with hot and cold supplies for drawing water, rinsing mops, etc. The supplies usually connect between the valves, and terminate with a long spout with pail-hook and brace. The spout supports the pail over the center of the sink while filling. The ordinary slop sink is provided with hot and cold faucets; and as the rims of the cheaper kinds are plain flanges, no tank flushing is possible.

Laundry Trays. These are made in all the materials used in other plumbing fixtures. Wood trays were formerly common but their unfitness because of absorption and odors, coupled with the increase in cost of lumber and the lessening in cost of the better materials, has effectually driven them out of the business.

The same inherent objection to the use of wooden covers may be urged as to the use of that material for the body of the fixture.

Trays are made singly and otherwise, but generally used in sets of two or three, except in the combination with sink already described. They are supported by a center standard or a metal frame, as best suits the material used.

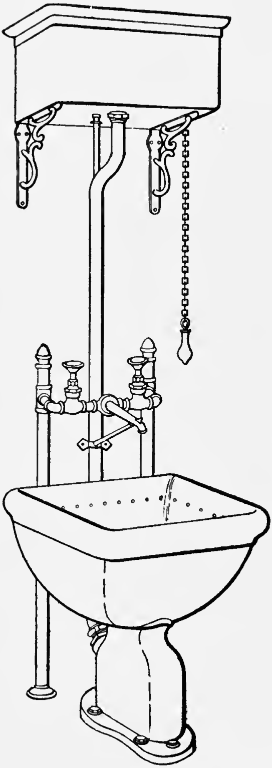


Fig. 34. Slop Sink Mounted on Trap Standard.

Some means of attaching wringers are provided, if possible. The waste is usually 2-inch. One trap answers for a set of trays. The size approximates 26 by 30 inches at top, with 15 inches' depth. The walls are all vertical except the front, which inclines about 30 degrees, making the width at bottom considerably less than at top. Some makers furnish one tray with each set, designed to serve as a washboard, the interior of the front wall being corrugated like the surface of a portable washboard. The inclination of the front is about right for scrubbing, whether the tray or an ordinary board is used, and the supports place the top of trays convenient to the work.

All trays were formerly made with faucet-holes in the back; and the plumber furnished a hinged cover. Side-handle faucets were necessary to allow the cover to close, as holes for top-handle faucets would be so low as to make useless too much of the space above them. The faucet-holes were seldom fitted water-tight. Holes are not now made in trays unless ordered, and the side-handle wash-tray bibb is disappearing. They were always annoying. If placed with the handles right and left as intended, the seat could not be examined, and no reaming or dressing of the faucet seat could be done without removing the faucet. When placed with the faucet handles facing each other, they were wrong-handed and too close together. It was awkward to supply air-chambers—especially so when all the faucet holes were equidistant from the top. When placed for one line of supply above the other, one line of holes was too low. These objections combined brought about the practice of omitting the covers, putting the supplies over the trays, and using regular sink faucets. Overflows are provided only when so ordered.

Enameled backs with air-chambers and faucets are supplied with roll-rim enameled-iron trays. A complete set of three trays, with all

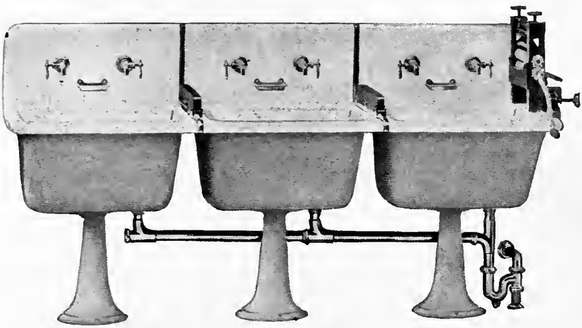


Fig. 35. Set of Three Laundry Trays, with complete attachments and Fittings.

attachments and fittings, is shown in Fig. 35. Flat-rim trays are made with or without faucet-holes, and are intended to have a hardwood frame to secure them rigidly. The wood frame and cover can be had with the fixture, but the plumber often supplies them. Nickel-plated or plain brass wastes and traps are furnished for trays, but the plumber can provide lead or cast-iron waste, if wanted.

Water-Closets. Types of water-closets are innumerable, and are separable into classes according to principles of action. Porcelain and painted or enameled iron are the materials used. Porcelain is more fragile, but has the better finish and is susceptible of a greater variety of design and ornamentation. The all-vitreous body of water-closet china of to-day is far superior to the glazed clay ware of the past, which, depending only on surface impermeability, soon cracked badly, thus permitting of absorption, the forerunner of odors which no plumber's skill could prevent. Enameled iron has not so durable a surface, but will stand rough usage, and has the advantage of very seldom cracking from frost even though the water in the trap freezes.

The greater relative advantage and durability of the porcelain closet over the best qualities of enameled-iron fixtures, should not be overlooked. There is less adherence of the foul wastes to a porcelain surface than to the enameled surface. It is also a fact that enamel is subject more or less to abrasion by the use of harsh scouring materials, as well as to decomposition by uric acid and water-closet discharges, and is therefore not a very durable material. These statements can be confirmed by observation of closets which have been in use for a number of years.

Iron closets of the better forms are used most in public places, stores, warehouses, etc. The pan closet, of iron, with earthenware bowl, is not now installed. For these, a trap was placed under the floor. The pan, operated by the same lever as the flushing valve, retained water, partially sealing the body from the bowl. The flush was by the swirling of a stream which entered tangentially under the rim. The bowls were round, as is necessary in all hopper closets thus washed, for water will not swirl in an oval bowl.

The objection to the pan water-closet is principally due to the fact that the outer bowl or container is a receptacle of filth which can never be properly cleansed. When the pan deposits its contents in the lower portion of the fixture, a considerable amount of the filth is spattered upon the walls and is not subject to the cleansing effect of the stream of water which scours only the upper bowl. When the closet is operated, the odors from this concealed surface permeate the room in an objectionable manner.

Tall round hoppers with swirling supply are yet frequently used in outhouses and other exposed places. No other form of closet will stand such locations under like conditions. The waste-trap is not placed immediately under the hopper, as in other forms, but down below the freezing depth—five feet as a rule. The supply valve is also placed below freezing, and is operated by a pull or by seat-action. These closets are *continuous* or *after-wash*, according to the style of valve used. Such an outfit is the simple frost-proof closet of the market. Tall oval hoppers with valve and slotted spud attached, swirl or rather direct the water sideways in both directions, but not effectively. The tank supply is also inefficient when delivered through a slotted spud under the common flanged rim. Short oval and round hoppers, with valve or tank supply operated by a pull or by seat-action, fitted to "S," "$\frac{3}{4}$ S," and "$\frac{1}{2}$ S" or "P" traps, for lead or iron pipe floor connection, make up several hundred closet combinations, each differing in some respect from the others. These are the poorest types of water-closet.

A sectional view of the *Combined Hopper and Trap* pedestal of to-day is shown in Fig. 36. It is made in one piece, in both porcelain and enameled iron. This form resulted from the separate hopper and trap fixtures before mentioned. The combined form has oval bowl and flushing rim for tank supply.

The *Wash-out* closet is a modification of the combined hopper and trap, being formed with a dipping bed under the mouth of the bowl, which retains enough water to keep soil from sticking to the surface. The water-bed makes it necessary to discharge the contents at either front or rear of bowl. The back-outlet wash-out is most repulsive to view; in them the drop-leg, which the flush never washes thoroughly, is always in view, so that its filthy condition suggests cleansing by hand. The front-outlet wash-out, shown in section in Fig. 37, is of more inviting appearance; but the drop-leg, although hidden, is there just the same.

Both the Wash-out and the Combined Hopper and Trap types have one fault in common. The trap almost always contains the soil from one usage. When the contents of the trap are flushed out after using, sometimes a similar mass refills it. Of course, two or three consecutive flushes would leave comparatively clean water in the trap, but this is not to be expected in regular usage.

On certain occasions the wash-out may serve a useful purpose on account of the water-bed. The stools of children or the sick may thus be easily observed at the will of the physician or at the discretion of those in charge, while such is impossible where the soil is submerged at once.

Pneumatic Siphon closets of various types have been put on the market. A good example of the type requiring two traps with an air-space between, is shown in Fig. 38. A specially constructed flushing tank is connected with the air-space between the traps The falling of the flush water creates a partial vacuum in the bottom compartment of the tank, which induces siphonage of the bowl contents.

To maintain a plenum in the flushing compartment of the tank while the flush water is flowing down and into the closet, the air between the traps is extracted, being drawn up through the air-pipe into the tank. Atmospheric pressure in the room simply presses the water out of the bowl and upper trap when the pressure below it is sufficiently reduced. This water, in motion, added to that of the lower trap which has been drawn above its normal level in response to the vacuum, is sufficient to form the long leg of an ordinary siphon; and thus both traps would be entirely emptied were it not for the vent

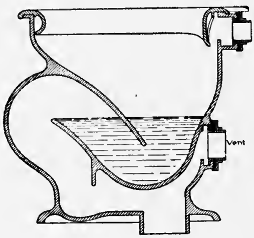


Fig. 36. Section of Combined Hopper and Trap Closet.

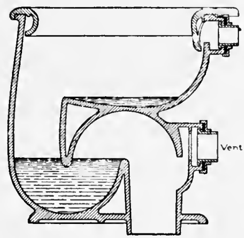


Fig. 37. Section of Front Outlet Wash-Out Closet.

in the crown of the lower trap breaking the siphonage in time to save a water seal for the lower trap.

The upper trap with water visible in the closet bowl in repose, is supplied by the *after-fill,* thus establishing conditions for the next action. The lower trap of such closets must be back-vented, and it is essential that the upper trap have no back vent.

The proper action of the tank is necessary to operate a pneumatic closet. A closet constructed on any other principle can be flushed with a bucket, by hand, if its tank is out of order. When a pneumatic closet, however, gets contrary, pouring water into the bowl simply fills or overflows it. The outlet is air-bound, and no passage of water to the soil pipe can take place until the barrier of air between the traps is removed.

The closets now accorded first place and generally used in the best work, are of the *Jet-Siphon* type, illustrated by the sectional view, Fig. 39. These use more water than is necessary to flush other kinds of closets, because a portion of the water is employed to produce the siphonage. A channel leading from the flush-water inlet to the bottom of the trap, conveys a stream of water to the trap leg, and injects it upward therein. The water in the channel has considerable velocity, and, being discharged into the water in the trap, imparts its energy to the whole mass, which, aided by the rise due to the incoming water from the flushing rim, moves upward at an increased speed depending on the ratio of mass and jet. When the water in the trap has been lifted in this way to an extent where sufficient of it can fall over the weir into the out-leg of the trap, a siphonic movement begins, and true siphonage finally takes place, the cessation of which depends upon the lack of sufficient water to continue it. Before the closet tank is emptied, siphonage often sweeps out the trap thoroughly; and what water falls back into the bowl when the siphon breaks, together with the incoming jet and flush, causes a second siphonage.

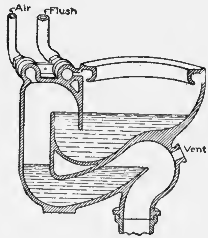


Fig. 38. Section of Pneumatic Siphon Closet, with Two Traps and Intervening Air-Space.

Accuracy in pointing the jet and in shaping the surfaces of its environment, are essential. If the surface above the jet-hole favors interference by the water flowing from the bowl, siphonage will be delayed and abortive, and may not take place at all. So, also, if the jet is not directed so as to maintain approximate concentricity in its travel through the mass of water, its energy is not expended to advantage, and failure is likely.

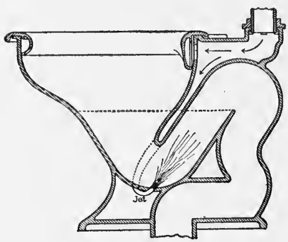


Fig. 39. Section of Jet-Siphon Closet.

There is no excuse for iron closets not siphoning perfectly. The iron pattern can be altered until it gives the best effect in practice, after which all closets cast from it should do the same. With porcelain ware, however, every closet made requires the same skill in design; and notwithstanding how perfectly the closet may be formed and the jet-hole cut, shrinkage in the kiln during the drying and burning process is apt to warp the wall and change the product so that it will not act properly. Closets of both materials, apparently perfect, often fail when first tried after installation, owing to foreign matter or fragments of enamel, clay, or iron lodging in the jet and changing its action. Usually these obstructions are easily removed by the plumber.

The jet principle has been added to the Combined Hopper and Trap closet before mentioned, producing in it a siphonic action resulting in very much improved service over that of the simple form. With the jet-action, the Combined Hopper and Trap is generally termed a *Wash-Down Siphon.* The so-called "jet" is applied in two ways. In some makes, the flush rim has an extra large and specially formed fan-wash feature, which directs down the back wall of the bowl a sluice-like stream. This stream, in addition to wetting the paper and forcing it down into the water, where it will be promptly carried out, sweeps round the curve of the bowl outlet in such a way as to lend its force to the water in the trap to produce apparent and not infrequently true siphonage.

Another form of the wash-down siphon is provided with a channel from the flush inlet, down outside the back wall of the bowl, to near or even below the water-level in the bowl, where the jet enters through a slit. The action is much the same as with the special fan-wash mentioned, but is generally superior in siphonic effectiveness.

Jet-siphon closets are not provided with vent openings in the closet proper, except for the local bowl ventilation. Wash-out traps are, or should be, vented. The simple hopper and trap should be vented in the trap. Wash-down siphons, generally, are not vented, but it is permissible to vent them low down in the outlet leg of the trap.

All closets for indoor use should have flushing rims. In all earthenware closets and in some forms of iron closets, the rims are made integral; but the iron rims are, as a rule, separate pieces, forming a water channel around the bowl. The bottom, inner edge of the iron rim hugs the wall of the bowl as closely as practicable, and the bulk of the water falls through regularly spaced serrations. Various provisions in the shape of barriers opposite the flush inlet, perforated race-way shelves along the rim above the exit openings, etc., are made to insure the rim filling and flushing properly all around.

All kinds of closets were formerly made without regard to the kind of seat to be used. Boxed-in cabinet seats, self-supporting, were universal. These gave way to seat and frame, with wall and leg support. To-day closets are commonly made with base flanges designed to support the weight of the person, and are provided with lugs or seat-shelf for attaching the seat directly to the bowl, as seen in Fig. 40. Metal post hinges are best in every way, if well made and strong. The competition goods, however—made to sell rather than use—*are* so light as neither to keep the seat in place nor to aid in holding it together under the severe strain. The hinged wood-cleat seats bolted to the closet are strong, but are objectionable because they cannot be kept dry or clean under the cleat.

Fig. 40. Closet with Base Flange Support, and with Lugs for Attaching Seat.

Closets are operated with pull or push-button tanks requiring the attention of the user; and are also made of the seat-action type. Children are likely to be forgetful, and visitors to public toilet rooms indifferent, to such an extent that automatic closets are desirable for public places and schools.

Closets are fitted with two styles of tanks—one placed about 7 feet from the floor and serving with a flush pipe never more than $1\frac{1}{2}$ inches in diameter; and the other placed low down, as close to the bowl as connections will permit. Examples of the *high-tank* and *low-tank* arrangements are shown in Figs. 41 and 42, respectively. The low tanks are wider and deeper than the high style, but do not extend out from the wall so much. The low position delivers the water at much less velocity than the elevated style, and, to secure the utmost speed and the volume necessary, the flush connection is never less than 2-inch in a low-tank closet. The rim and jet channel are proportionately larger in bowls intended for use with low tanks. High tanks are about 17 by 9 by 10 inches. Sheet lead and sheet copper are used for closet-tank linings. Some kinds of water, through galvanic action, attack the soldering of the seams in copper-lined tanks with more effect than where lead alone is used. Generally, however, copper-lined tanks give satisfaction if the copper is heavy enough (12 to 16 oz.) and properly put in. Some makers lock-seam the linings water-tight, and solder on the outside before placing the copper in the wood case.

On account of the greater depth of low tanks, swelling of the wood case has, doubtless, been the cause of most of the trouble experienced with this type. When put together in the factory, the wood is very dry, and after being used for a short time, increases in height as a result of swelling from dampness. If the lining be tacked to the wood at bottom and top, injury is sure to result. If tacked at the top only, the copper will soon be supporting the water without help except where the connections are attached. It is now the practice to omit fastening the lining. Very great care has been found necessary with ball cocks for low tanks, in order to secure proper after-fill, the flush connection being too short to aid much in resealing the bowl with its drainings.

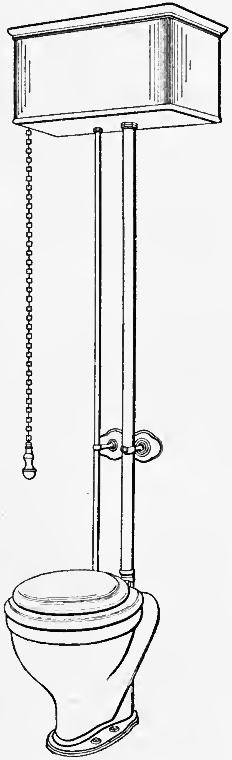


Fig. 41. High-Tank Arrangement of Closet Fixtures.

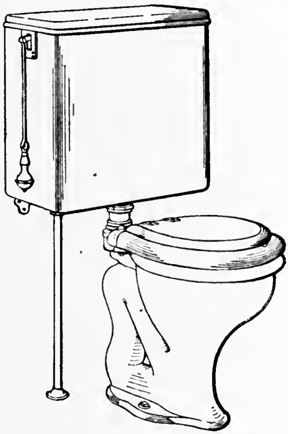


Fig. 42. Low-Tank Arrangement of Closet Fixtures.

CONCRETE HEADWORKS, GATES CLOSED, HUNTLEY PROJECT, MONTANA

This project calls for the irrigation of 26,000 acres of Crow Indian Reservation land by means of water diverted from the lower Yellowstone River.
Photo by Reclamation Service.

Low tanks flush with much less noise than high ones, and permit placing the closet under windows and low ceilings. Low ones require more width on account of the tank, and more depth from the wall to the front, as the seat and lid must be placed far enough forward to be thrown back and remain leaning against the front of the tank. Low tanks are provided with ventilated covers; while the high pattern, which is out of children's reach, is left open at the top. The fewer working parts in a tank, the less likely it is to get out of order.

A type of seat-action closet very seldom placed in private houses, is that with closed metal tank, as represented in Fig. 43. Depressing the seat opens a valve in the supply, and the water passes up through a flush pipe into a closed tank. The air in the tank is compressed until the air-pressure counterbalances that of the water. When the seat is released, the supply valve closes; and a valve is opened, establishing communication between the closet and the tank. The compressed air then expels the water in the tank, flushing the closet just as a large supply with corresponding pressure would do without a tank. Closed-tank closets depend on pressure. The space occupied by the air in the tank is inversely proportional to the pressure; hence, even in heavy pressure, considerable of the tank's capacity is yet occupied by air when equilibrium is established; and the less the pressure, the smaller the amount of water it is possible to get into the tank. They are therefore not fit for very light pressures, though they sometimes serve well in the basement of a building where failure would be certain on the upper floor.

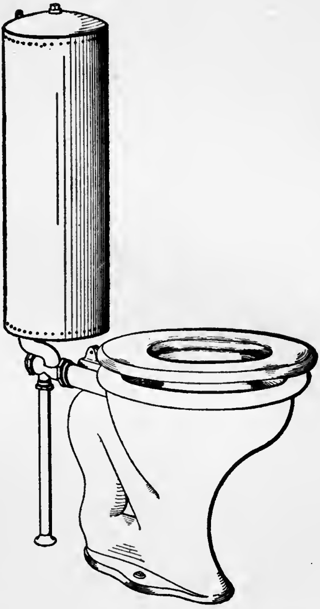


Fig. 43. Seat-Action Closet with Closed Metal Tank.

Condensation on metal tanks is annoying. Open tanks of porcelain and iron are used more or less, but sweating is hard to overcome. Zinc paint and ground cork finishes have been employed with some satisfaction; and drip-cup collars discharging into the flush just under the tank have served in this capacity, but nothing overcomes the sweating so well as a tight wood case, insulated metal cases not excepted. Some makes of the pressure-tank closet require too much weight on the seat for successful operation by a child, and children would as a rule leave the seat too soon to allow the tank to fill reasonably well. The flush pipe of pressure closets is from a few inches to four feet in length. The after-fill is accomplished by projecting the flush connection into the tank an inch or more, and drilling a $\frac{1}{4}$-inch hole or less through it near the bottom of tank. The rapid flow ceases when the water-level falls to the upper end of the inward-projecting flush connection, and the after-fill drains into and down the flush slowly.

The flush fittings of an open tank consist essentially of a valve to admit water to the flush pipe; an overflow always open to the flush pipe; and a lever and connection, with chain and pull or button, to open the flush valve. A simple example of these is the siphon gooseneck, with flush-valve disc on one end and lever connection at the other. Prongs extend below the disc to guide and keep it in place. The overflow is through the gooseneck. Lifting the gooseneck an instant permits enough water to flow down the flush to start the siphon through it when the pull is released. The tank then siphons to the lower end of the gooseneck arm.

Where shortness of flush pipe or form of closet requires a decided after-fill, this is secured by special provision in the flush fittings, or by leading some of the supply delivered by the ball cock into the overflow.

The supply fittings of a closet tank consist merely of a ball cock of suitable form. For light pressure, simple leverage suffices. For heavy pressure, the inlet in the valve would have to be too small, or the ball too large and stem too long, for a small tank, if simple leverage were employed. Therefore compound-leverage cocks are usually substituted where the pressure contended with is over 30 pounds. There are ball cocks made in which the buoyancy of the ball merely operates a small secondary valve in a way to establish the initial pressure over a disc of larger upper surface than that of the under side which covers the main water inlet of the cock. The disc is thus effectually seated, regardless of the pressure; and a 4-inch ball may be arranged to close almost any size valve against any pressure.

When the cock is attached through the bottom of the tank, no precaution against sound is necessary. When the cock is fitted in high up, a pipe from the delivery is extended to near the bottom of tank for the purpose of muffling the sound of the water as it fills the tank. An unmuffled delivery and a high-tank flush make considerable noise when the closet is flushed, and are suggestive and very embarrassing to sensitive people. Silent action is therefore the goal for which many strive. Silence at the expense of thoroughly washing the closet surfaces and flushing out the contents, is not desirable; some noise is necessary to the rapidity of action essential to thorough scouring and evacuation.

Tanks requiring the flush valve to be held off the seat during the entire flush, are now no longer installed. Perfect silence in the flush pipe of a high-tank closet has been obtained by a type of flush fittings that permits the pipe to hang full of water. The flush valve being opened, water begins to flow into the closet immediately. When the valve closes, no air having access at the upper end of the flush, the pipe remains filled. The flush valve of such a closet must close absolutely water-tight to prevent continual dribbling into the bowl.

Of late years, direct-flushing valves of many forms have been a feature of water-closet design. These valves make the individual closet tank unnecessary. Direct-flushing closets, a type of which is shown in Fig. 44, have the same advantage as the low tank in the matter of being placed where high closets cannot conveniently be arranged. A check to their more general adoption has been the lack of large supplies in residences and other buildings.

The possibility that the house system of water supply may be contaminated from the water-closet if the water supply is directly connected to the water-closet fixture, should not be overlooked. Although this contamination is more likely to take place in the operation of the older types of closets, such as the pan closet and the plunger type, it is not of rare occurrence in connection with later types, especially the so-called *frost-proof* fixture. If the pressure is materially lowered in the street main by accident or otherwise, it sometimes happens that water may be drawn back into the house system by siphonage from a water-closet or like fixture, thus of course incurring the possibility that germs of disease may be brought into the water supply used for domestic purposes. The use of a tank into which the water is first drawn, obviates this danger.

The ordinary dwelling or storehouse supply can be made to operate successfully by placing an accumulating chamber on the branch to the closet, and having a check-valve on the street side of it, so that the water cannot flow back when the pressure falls as a result of drawing at other points. In such cases the pipe between the accumulator and the closet must be the usual $1\frac{1}{2}$-inch size. Closets thus fitted are really only pressure-tank closets with the flush controlled by a direct-flushing valve to be operated at will instead of automatically by seat-action.

In all tank installations, the direct method is easily employed by carrying the proper size flush main directly to the closets, independently of the supply for other fixtures. This is recommended in buildings having numerous closets. One tank, with large flushing main, will serve all the closets, and thus the individual tanks and equipment are not needed. Furthermore, no trouble is then experienced in providing suitable space for the small tanks. The flushing valves may, if desired, be placed out of sight, and only the operating lever brought to view in a convenient position. A flushing valve has been made which, like the secondary-valve ball cock, works on the old Jennings diaphragm principle, using a "time" filling cup to establish the initial pressure over the diaphragm. Releasing the pressure over the diaphragm by means of the operating lever, opens the main channel and causes the closet to flush while the time chamber fills again.

In this country and most others, the height of closets has always been uniformly 16 to 17 inches to top of seat. It is claimed that this height results in an unnatural position, and individual opinions against it have been voiced from time to time with little effect. Lately, however, more earnest attention has been given the subject of height, and there has been designed a closet considerably lower than usual, with the top sloping down toward the back. This form, it is said, induces the user to assume an upright position of body, relatively more closely conforming to that of the limbs, and favoring unrestricted action of the intestines. It remains to be seen whether this form will result in any general departure from the old lines.

Closets often also serve as urinals, especially in private houses. For limited service, this is not to be considered an actual abuse of the fixture, though general use of distinct urinal fixtures is indispensable.

Range Closets. Batteries of individual closets are usual in office buildings and many other such structures; but in schools and in many public places open to all classes, ranges divided into stalls or compartments have been considered a satisfactory solution of the problem.

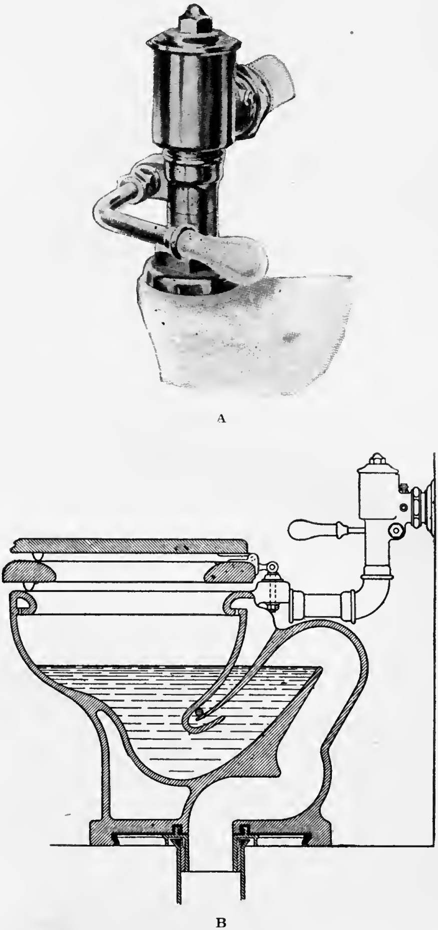


Fig. 44. Direct-Flushing Closet Dispensing with Necessity of Tank. *A* Shows Hand-Flushing Valve; *B* Complete Fixture with Sectional View of Siphon Closet.
Courtesy of the J. L. Mott Iron Works.

The objections to the range type of fixture are inherent in the design. The fouling surface of a trough fixture is much greater than that of the number of individual closets to which the fixture corresponds, and certain parts of this surface are not subject to an adequate flushing action. A certain portion of the surface, much larger relatively than that in individual fixtures, is exposed to spattering with the filth, and is alternately wet and dry. It is also true that the method of applying the water for scouring purposes is much less satisfactory than with single closets. A further objection to the range fixture is that in general its material is less desirable for the purpose than the earthenware or porcelain used for closets. On account of these deficiencies, for some ten years past, individual closets have been used in public schools in certain cities which have given the most attention to this branch of sanitation, and their use is being extended.

Range closets have automatic flushing tanks acting at any required interval between flushes. The tanks are, as a rule, without moving parts, and give good service without much attention after the supply is once set to flush at the interval desired. Whether the users of a closet are indifferent or irresponsible, does not change the result of abuse; and the range type of closet overcomes many annoyances attending the use of ordinary individual closets in unsuitable places—institutions for the insane and feeble-minded, for example. Ranges, like seat-action closets, are not dependent on the user, who may forget to pull a chain or push a button and thereby leave the closet foul.

Various forms of ranges are now operated on the siphon eduction principle. Siphonic eduction is accomplished in three ways—first, by the double trap and air-pipe to the tank indicated by the sectional view, Fig. 45, and operating exactly like the individual pneumatic closet already described; second, by a siphon outlet-end in which the water falls over a central weir that maintains the proper depth of water until the flush begins, and causes siphonage by breaking up and filling the channel as it passes through a constricted bend below. The latter method is shown in section in Fig. 46. Still another type of range is made to siphon by jet-action, just as the individual jet-siphon closet does, the trap providing a retaining weir which holds the water at the proper level in the range between flushes.

There are wash-out ranges with sloping weirs at the outlet to retain enough water to keep soil from sticking. These are open troughs, and the plumber provides the trap. Some siphon ranges are of the open-trough pattern, but the trap or the siphon outlet is a part of the fixture. All open-trough ranges can be supplied with a ventilating section from which a large vent pipe may be carried to a stack in which a draft is insured by a hot flue or some other means. Such ventilation changes the air in the room; and by having lids to all the seats, odors from the entire trough may be uniformly removed by

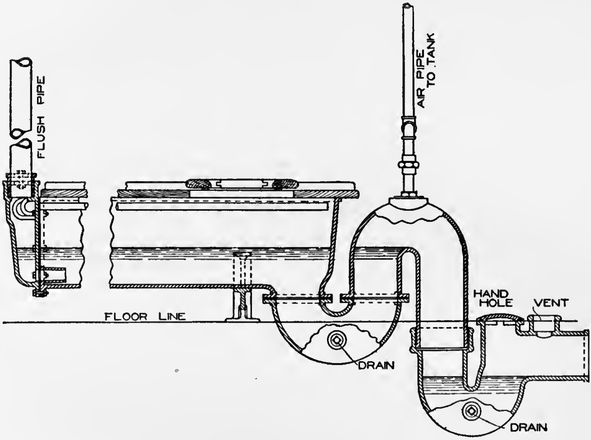


Fig. 45. Section of Range Closet, with Double Trap and with Air-Pipe to Tank to Cause Siphonic Eduction.

leaving up one lid only, at the end opposite the vent pipe. Some forms, having individual flushing-rim bowls cast integral with the section, are supplied by one general flush pipe, as indicated by the plan and elevation shown in Fig. 47. In these, each bowl is separately water-sealed, as the normal water-level is above the general conduit into which the bowls discharge.

Other forms, which receive the entire flush at one end, are water-sealed between the seat holes. The seat-openings, instead of converging like flushing-rim bowls, diverge downward, so that, as the water-level recedes in the sections during flushing, soil falls away from the surface by gravity instead of grinding against it. Therefore, so far as cleanliness is concerned, the type with diverging surfaces but without the scouring effect of flowing water in the openings is, in operation, the practical equivalent of the flushing-rim type with converging surfaces. The open-trough ranges, including the jet-siphon type, have perforated wash-down pipes along the sides and ends, which, however, have little value. The open troughs are made in cast sections as long as convenient, joined by flanges with rubber gaskets and bolts. Suitable feet or chairs for supports are furnished with these fixtures.

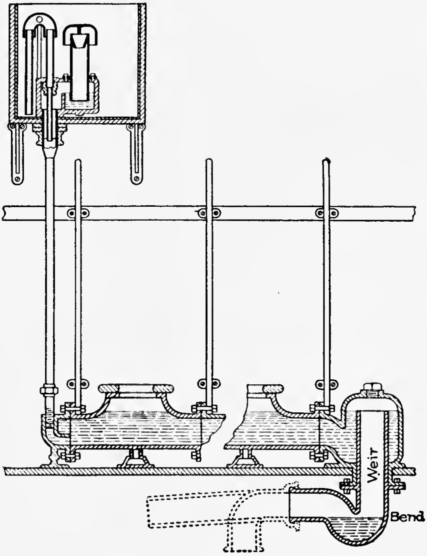


Fig. 46. Section of Range Closet, with Siphon-Outlet End.

Cast partitions, partitions and backs, and full compartment partitions, with slat doors and indicators, are furnished to order in any style or combination desired. For example, the range for a schoolroom may consist altogether of 24-inch sections or divisions, except one intended for the teachers' use made 30 inches and fitted with door and full-length partitions to give a thoroughly private compartment. Ranges are usually made of cast iron, and almost invariably finished with enameled interior and painted exterior. Bowl or section ventilation is provided for where possible. Wood seats and covers are generally used; but enameled-iron top frames with hinged seats and covers, and rigid enameled seats, are also made.

The lower trap of a double-trap range must be ventilated. All soil-pipe stacks into which ranges discharge, and fixtures connected to them, must be well protected against siphonage, because the volume of water discharged at one time by a range is sufficient to siphon traps that would retain their seals under most other conditions.

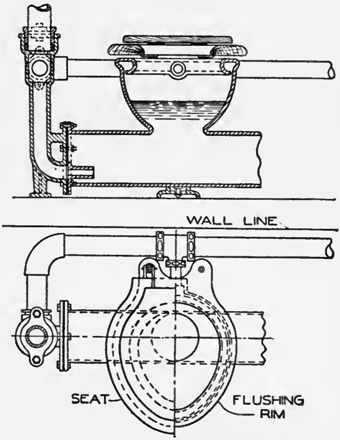


Fig. 47. Sectional Elevation and Plan of Range Closet Seat with Flushing-Rim Bowl Supplied from General Flush-Pipe.

Urinals. Sectional urinals are made of the same materials and finish, and with much the same types of design, as range closets. They are generally installed in the same classes of buildings as range closets; but such urinals will often be found in the same toilet-room with individual closets. Roll-rim enameled troughs, with back and with simple perforated wash-down flush pipes on the back, are available.

Single urinals are usually of porcelain, although some have been made of iron. The common types are plain or lipped, made in flat-back and corner designs. Flat-back types of both de-

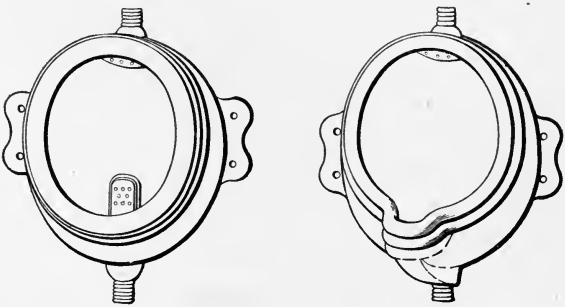


Fig. 48. Flat-Back Types of Single Urinals.

signs are shown in Fig. 48. All have flushing rims. Direct-flushing valves of the same type as used on closets, adapted to the purpose, and cocks of various types, are the means of flushing generally provided for a single urinal. When two or more are placed in one toilet-room, an automatic tank with branched flush pipe is employed. These tanks are of greater variety than those used with range closets. The tilting bucket, pivoted within a tank case, which empties itself periodically by means of the flow of water changing the center of gravity to the unsupported side and tipping it just before it overflows, is a familiar type of automatic urinal-flushing tank. The standard tank with immovable parts, which siphons automatically, is also prevalent. Examples of these types are illustrated in section in Fig. 49.

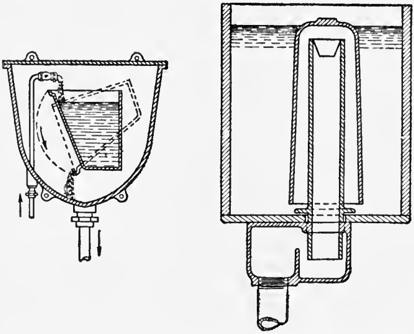


Fig. 49. Automatic Urinal-Flushing Tanks. Tilting-Bucket Type at Left; Self-Siphoning at Right.

Another design consists of a tank with common siphon, fitted with a ball cock which *opens*, instead of closing, as the water in the tank lifts the ball. The interval between flushes is governed by a small bibb cock, which may be turned on more or less so as to take greater or less length of time for the water in the tank to reach the ball. When water begins to lift the ball, the ball cock also admits water. From this point the tank fills

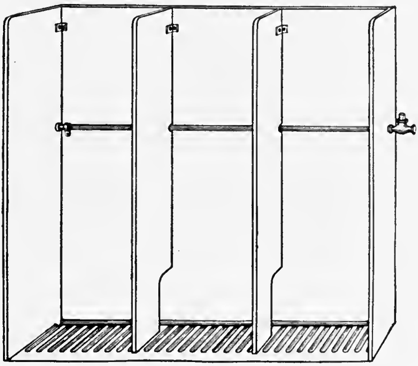


Fig. 50. Urinal Stalls of Slate or Marble, Flushed by Perforated Pipe, with Channeled and Guttered Floor.

rapidly. The higher the ball is lifted, the faster the tank fills, so that by the time the water-level reaches a point where water begins to flow over the neck of the siphon, it is coming into the tank rapidly enough to more than keep pace with the overflow necessary to start the siphon. True siphonage, however, empties the tank much faster than the supply can fill it; and the tank is soon empty, leaving the small bibb cock to admit water again slowly to where this action can be repeated.

Individual urinals which siphon by admitting additional water to that which normally stands in the fixture, and various other types, will be best understood from a study of dealers' catalogues. In good work, marble backs and partitions usually enclose the urinals on three sides. Marble and slate stalls of various construction, with channeled and guttered floor, as shown in Fig. 50, all washed by perforated pipes fixed along the surfaces, are frequently used in lieu of specific urinal fixtures. A thick base of slab material is sometimes used, the gutter and drain-hole being cut in it. Cast-iron gutters, galvanized or enameled, with an outlet-end adapted to a soil-pipe connection, are supplied by the makers.

In describing the fixtures and trimmings that have been noticed, only salient features of form and principles of design have been considered. Sufficient guidance to insure intelligent comparison of merits and skilful discrimination in selection, has been given. Catalogue detail and illustration, and a view of the aetual goods described therein, should, with what has now been given, insure the fullest understanding of the fixture branch of Plumbing.

HOUSE WATER SUPPLY

While the plumber is apt to give more attention to supply pipe, and to methods of installing it in buildings to secure specific service, *water supply* embraces also, in its broadest sense, the source and quality of water and the means of conveying it to the building. Plumbers generally have little dealing with water supply outside of the house walls. Custom has fixed certain arbitrary sizes in ordinary work, to such a degree that the average plumber has generally ignored information on the flow of water through pipes. Indeed, he is so rarely in actual need of this knowledge, that it appears a burden to acquire and to fix permanently in his mind the simplest formula bearing on the subject. Enough information to determine approximate deliveries and point the road to further research, will not be out of place in behalf of those who may need simple directions.

The laws of gravity are the basis for the science of hydraulics, of which a prime factor of every problem is *velocity.* There is no exception to the rule that all bodies falling freely, descend at the same rate—in round numbers, 16 feet for the first second, at the end of which the acquired velocity is one of 32 feet a second. This is the basis on which are formulated the laws of falling bodies, which, exhibiting what is known as *velocity of efflux,* together with loss by friction, must be considered when calculating the flow of water.

There are three kinds of velocity—*uniform, accelerated,* and *retarded.* It is the last, and its cause, friction, that plumbers should be most interested in, as velocities calculated merely from the laws of falling bodies do not take account of friction, change of course, etc., which must be allowed for as causes diminishing the delivery of water through pipes. Briefly stated, the mysterious-looking Torricellian formula $\sqrt{2gh} = V$, means only that velocity is found by extracting the square root of the product of the head multiplied by 2×32, g standing for the force of gravity, and h for the height. For example, a stream filling a 1-inch pipe, with 25 feet head of water, would have a velocity calculated thus: $2 \times 32 \times 25 = 1{,}600$; and the square root of $1{,}600 = 40 =$ Velocity, friction not considered.

The shape of the orifice through which water enters a pipe, has much to do with the amount of water that will enter it. Friction against the sides of the pipe, and change of direction due to bends and connections, occasion great variation from the theoretical flow. Not only is the character of the pipe surface and fittings to be considered as initial causes varying the delivery, but velocity, the all-important factor, must be reckoned with in every instance. With a velocity of 10 feet per second in a pipe of comparatively smooth interior surface, the friction loss in pounds on one square foot of surface will be about $\frac{1}{2}$ pound. If this velocity is increased or diminished, the factor of friction will vary accordingly, always in proportion to the square of the velocity. Suppose the velocity to be 20 feet instead of 10 feet per second; we then have, 10 squared equals 100, and 20 squared equals 400. The square of these velocities is as 1 to 4, and as we assign a $\frac{1}{2}$-pound loss to ten feet velocity per second, on a stated amount of surface, the friction due to doubling the velocity should be four times a $\frac{1}{2}$ pound = 2 pounds, showing that doubling the velocity increases the friction four-fold; trebling it increases friction nine-fold, etc.

A column of water weighs .43 pound per square inch of base, per vertical foot. Therefore a vertical pipe 100 feet high, with 1-inch sectional area, filled with water, would contain 43 pounds, and a gauge at the bottom would show 43 pounds pressure. If the pipe were only $\frac{1}{4}$ inch, or were 40 inches in diameter, the gauge would show the same pressure for the same vertical height—namely, .43 pound per square inch per vertical foot. A head of water expressed in feet, may be changed to pounds by multiplying the feet of head by .43. Pressure is made to read in feet of head by multiplying pressure per square inch by 2.3. A *head of water* is the number of vertical feet from level of source of supply to center of outlet or point of delivery.

Diameter of the pipe has nothing to do with static head or pressure; but its relation to the size of the orifice from which the water is to be drawn has much to do with the amount of pressure lost by friction. If a faucet and supply pipe are of the same size, and we *double* the size of the pipe, the velocity of the water flowing through it is reduced three-fourths; and the friction is, under these conditions, but one-sixteenth what it was in the original size. Moreover, as in drawing similar amounts of water under the same head through a one-inch and a two-inch pipe, the amount of friction surface presented is twice as great in the one-inch as in the two-inch pipe, the friction in the one-inch can be shown to be 32 times as much as in the two-inch pipe.

With the formula given, one can roughly approximate by finding the theoretical delivery and deducting a liberal percentage for friction, according to size, length of pipe, and head or pressure. The subject, however, is vast and tedious, introducing intricate calculations in higher mathematics when considered in detail with a view to extreme accuracy of results, and is a branch properly belonging to hydrodynamics, rather than suited to presentation at length here. Two tables are given, however, which with the rules for use, will be of value to those who fail to make further research.

Table I shows the pressure of water in pounds per square inch for elevations varying in height from 1 to 135 feet.

Table II gives the drop in pressure due to friction in pipes of different diameters for varying rates of flow. The figures given

TABLE I

Head in feet	Pressure pounds per square inch	Head in feet	Pressure pounds per square inch	Head in feet	Pressure pounds per square inch
1	.43	46	19.92	91	39.42
2	.86	47	20.35	92	39.85
3	1.30	48	20.79	93	40.28
4	1.73	49	21.22	94	40.72
5	2.16	50	21.65	95	41.15
6	2.59	51	22.09	96	41.58
7	3.03	52	22.52	97	42.01
8	3.46	53	22.95	98	42.45
9	3.89	54	23.39	99	42.88
10	4.33	55	23.82	100	43.31
11	4.76	56	24.26	101	43.75
12	5.20	57	24.69	102	44.18
13	5.63	58	25.12	103	44.61
14	6.06	59	25.55	104	45.05
15	6.49	60	25.99	105	45.48
16	6.92	61	26.42	106	45.91
17	7.36	62	26.85	107	46.34
18	7.79	63	27.29	108	46.78
19	8.22	64	27.72	109	47.21
20	8.66	65	28.15	110	47.64
21	9.09	66	28.58	111	48.08
22	9.53	67	29.02	112	48.51
23	9.96	68	29.45	113	48.94
24	10.39	69	29.88	114	49.38
25	10.82	70	30.32	115	49.81
26	11.26	71	30.75	116	50.24
27	11.69	72	31.18	117	50.68
28	12.12	73	31.62	118	51.11
29	12.55	74	32.05	119	51.54
30	12.99	75	32.48	120	51.98
31	13.42	76	32.92	121	52.41
32	13.86	77	33.35	122	52.84
33	14.29	78	33.78	123	53.28
34	14.72	79	34.21	124	53.71
35	15.16	80	34.65	125	54.15
36	15.59	81	35.08	126	54.58
37	16.02	82	35.52	127	55.01
38	16.45	83	35.95	128	55.44
39	16.89	84	36.39	129	55.88
40	17.32	85	36.82	130	56.31
41	17.75	86	37.25	131	56.74
42	18.19	87	37.68	132	57.18
43	18.62	88	38.12	133	57.61
44	19.05	89	38.55	134	58.04
45	19.49	90	38.98	135	58.48

are for pipes 100 feet in height. The frictional resistance in smooth pipes having a constant flow of water through them is proportional to the length of pipe. That is, if the friction causes a drop in pressure of 4.07 pounds per square inch in a $1\frac{1}{4}$-inch pipe 100 feet long, which is discharging 20 gallons per minute, it will cause a drop of $4.07 \times 2 =$

TABLE II.

Gallons discharged per minute.	½ in.		¾ in.		1 in.		1¼ in.		1½ in.		2 in.		2½ in.		3 in.	
	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.	Velocity in feet per second.	Friction loss in pounds.
5	8.17	24.8	3.63	3.3	2.04	.84	1.31	.31	.91	.12						
10	16.3	96.0	7.25	13.0	4.08	3.16	2.61	1.05	1.82	.47	1.02	.12				
15			10.9	28.7	6.13	6.98	3.92	2.38	2.73	.97	1.53	.27				
20			14.5	50.4	8.17	12.3	5.22	4.07	3.63	1.66	2.04	.42				
25			18.1	78.0	10.2	19.0	6.53	6.40	4.54	2.62	2.55	.67	1.63	.21	1.13	.10
30					12.3	27.5	7.84	9.15	5.45	3.75	3.06	.91				
35					14.3	37.0	9.14	12.04	6.36	5.05	3.57	1.25				
40					16.3	48.0	10.4	16.10	7.26	6.52	4.09	1.60				
45							11.7	20.2	8.17	8.15	4.60	2.02				
50							13.1	24.9	9.08	10.0	5.11	2.44	3.26	.81	2.27	.35
75							19.6	56.1	13.6	22.4	7.66	5.32	4.90	1.80	3.40	.74
100									18.2	39.0	10.2	9.46	6.53	3.20	4.54	1.31
125											12.3	14.9	8.16	4.89	5.67	1.99
150											15.3	21.2	9.80	7.00	6.81	2.85
175											17.1	28.1	11.4	9.46	7.94	3.85
200											20.4	37.5	13.1	12.47	9.08	5.02

8.14 pounds in a pipe 200 feet long; or $4.07 \div 2 = 2.03$ pounds in a pipe 50 feet long, acting under the same conditions. The factors given in the table are for pipes of smooth interior, like lead, brass, or wrought iron.

Examples.—A $1\frac{1}{2}$-inch pipe 100 feet long connected with a cistern is to discharge 35 gallons per minute. At what elevation above the end of the pipe must the surface of the water in the cistern be to produce this flow?

In Table II we find the friction loss for a $1\frac{1}{2}$-inch pipe discharging 35 gallons per minute to be 5.05 pounds. In Table I we find a pressure of 5.2 pounds corresponds to a head of 12 feet, which is approximately the elevation required.

How many gallons will be discharged through a 2-inch pipe 100 feet long where the inlet is 22 feet above the outlet? In Table I we find a head of 22 feet corresponds to a pressure of 9.53 pounds. Then, looking in Table II, we find in the column of Friction Loss for a 2-inch pipe that a pressure of 9.46 corresponds to a discharge of 100 gallons per minute.

Tables I and II are commonly used together in examples.

A house requiring a maximum of 10 gallons of water per minute is to be supplied from a spring which is located 600 feet distant, and at an elevation of 50 feet above the point of discharge. What size of pipe will be required? From Table I we find an elevation or head of 50 feet will produce a pressure of 21.65 pounds per square inch. Then if the length of the pipe were only 100 feet, we should have a pressure of 21.65 pounds available to overcome the friction in the pipe, and could follow along the line corresponding to 10 gallons in Table II until we came to the friction loss corresponding most nearly to 21.65, and take the size of pipe corresponding. But as the length of the pipe is 600 feet, the friction loss will be six times that given in Table II for given sizes of pipe and rates of flow; hence we must divide 21.65 by 6 to obtain the available head to overcome friction, and look for this quantity in the table, $21.65 \div 6 = 3.61$, and Table II shows us that a 1-inch pipe will discharge 10 gallons per minute with a friction loss of 3.16 pounds, and this is the size we should use.

In calculating the contents of pipes, cylinders, and cisterns, where it is usual to correct the area found as a result of squaring the diameter by multiplying by .7854, before dividing by 231 for U. S. gallons, multiplication by the decimal may be omitted, and dividing by 294 instead of 231 will then give the same result.

EXAMPLES FOR PRACTICE

1. What size pipe will be required to discharge 40 gallons per minute, a distance of 50 feet, with a pressure head of 19 feet?

Ans. $1\frac{1}{4}$-inch.

MODESTO IRRIGATION CANAL, CALIFORNIA

On line of Southern Pacific Railway.

2. What head will be required to discharge 100 gallons per minute through a $2\frac{1}{2}$-inch pipe 700 feet long?

Ans. 52 feet.

TYPES OF WATER SUPPLY

There are various ways in which it may be necessary to obtain the water supply for a building. The usual course in cities and towns is to employ the Municipal Water Works service. This, of course, settles the supply feature, and the plumber simply provides the house and yard pipe, $\frac{5}{8}$-inch or larger main, according to the character of the work. If of lead, the pipe must be of strength according with the pressure. Any of the light-weight grades of lead supply will stand 1,000 pounds per square inch for a short time; and the usual strength used on 50 to 80-pound pipe will not burst under 1,400 to 1,600 pounds when new and unstrained. Under constant pressure, the enormous strain possible from water-hammer, and general deterioration from use, make it advisable to employ pipe which, when new, is 20 times as strong as that necessary to contain the pressure. No attention is necessary as to the strength of zinc-coated or tin-coated iron pipe; it will stand any pressure ordinarily encountered.

The two general methods of supplying buildings with water are: (1) the *direct* system; and (2) the *indirect* or *tank* system. The direct method, generally employed in cities, places each fixture connected with the supply under the same pressure as the street main, unless a reducing valve is introduced, thus often subjecting the work to needless high pressure and always to the widely varying conditions and quality of service incidental to such use. In the direct system it is good practice, where at all practicable, to pipe and fit the work generally for pressure not exceeding 50 pounds per square inch, and then use a reducing valve to maintain such pressure as is required.

The indirect method is almost always necessarily employed in isolated work; and even where municipal service is available, it is generally better for ordinary domestic purposes. With the indirect system, the connection with the street main is carried directly to a tank placed in the attic, or at some point above the highest fixture, as shown in Fig. 51. The supply to tank is regulated by a ball-cock which automatically shuts off the water when the tank becomes full, and opens and refills it again when water is drawn out. All the plumbing fixtures are supplied directly from the tank, and are therefore under a constant minimum pressure depending on the distance the fixtures are situated below the tank. The tank storage is a matter of great convenience during repairs to street mains, aside from its ad-

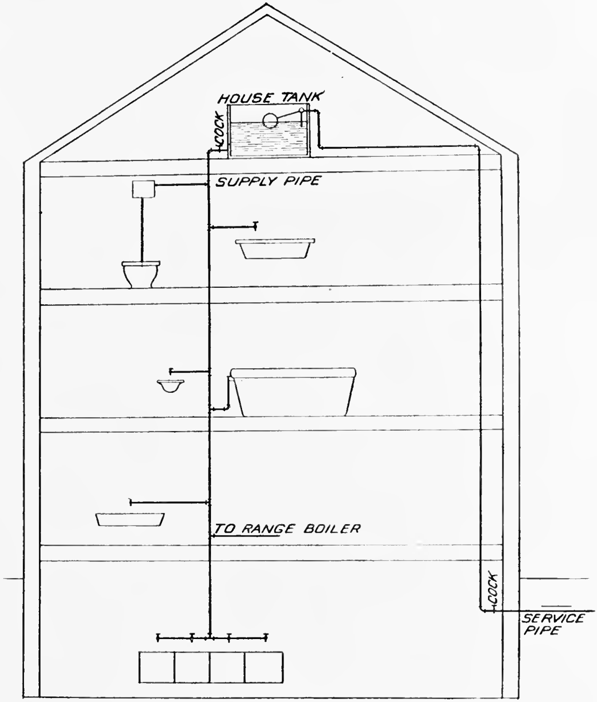


Fig. 51. Indirect or Tank System of House Supply.

vantages of uniform pressure, reduced expense of fitting and maintaining low-pressure work, etc.

In municipalities where the pressure in the main is not sufficient to carry the water up to the house tank in the attic, and in elevated situations, an automatic, electrically-operated rotary or other suitable form of pump is often installed to lift the water. A screw pump like that shown in Fig. 52 is especially adapted to this use when equipped with an electric motor to start and stop automatically by means of a float in the tank operating an electric switch as shown in the engraving.

Where steam pressure is available, steam-operated pumps are very frequently used, and are invariably arranged for automatic service whether there are engineers regularly in attendance or not. A device that may be attached to steam pumps for this purpose is shown in Fig. 53. When the high-water line in the tank is reached, the float closes a valve in the pump discharge pipe, thus promptly increasing the pressure in it so as to actuate a piston through a pipe connection from the pump discharge to the regulator beneath the piston head. The regulator is shown complete, in detail, partly in section, in Fig. 54. Raising the piston shuts off the steam supply to the pump at the governor valve. When the water line in the tank is lowered, the float falls and the ball valve opens, relieving the pressure in the pump discharge pipe and allowing the steam governor valve to open by the action of the coun-

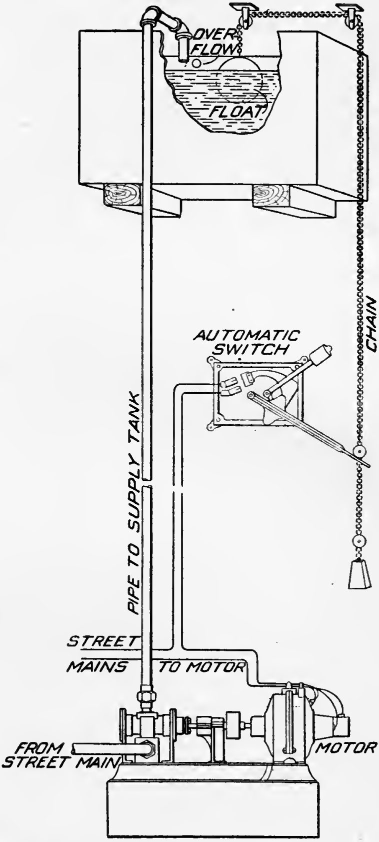


Fig. 52. Electrically-Operated Pump for Lifting Water to Tank. Automatically Started and Stopped by Means of Float Operating Electric Switch.

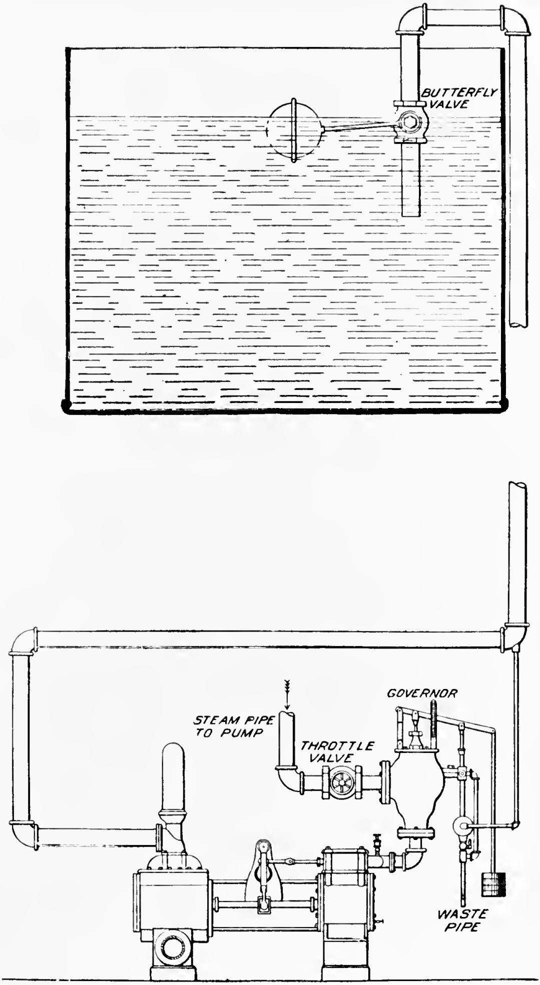


Fig. 53. Steam Pump Equipped with Regulator Operated by Float in Tank, Securing Automatic Service.

terweights attached to the lever arm, as shown; and the pump then works regularly until the lifting of the float by the rising water again closes the valve in the pump discharge and repeats the action described.

Outside of corporations, the supply may be from an elevated

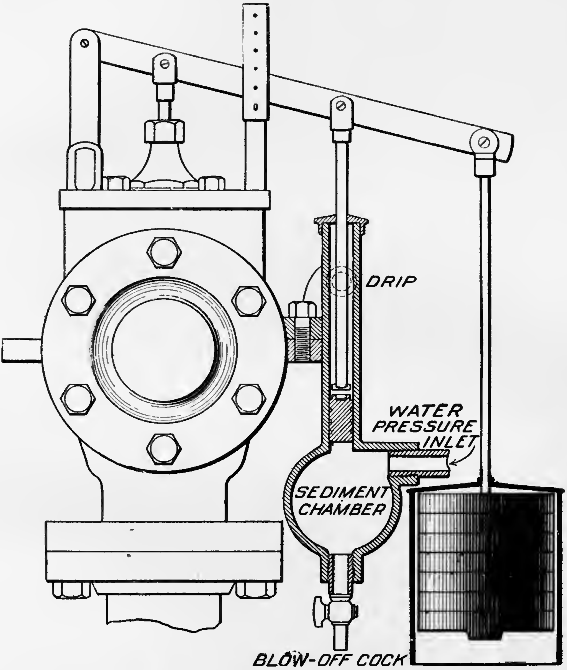


Fig. 54. Steam Pump Regulator (Shown Partly in Section) Automatically Operated by Valve Controlled by Float in Supply Tank.

spring or stream, or from wells, cisterns, or other sources below the level of use. If the natural supply is high enough, it may be conveyed into a tank of sufficient height without intermediate apparatus. Tanks inside the dwelling or house are best, ordinarily.

Tanks for cold-water storage are made of various materials and in different shapes and sizes, according to the special uses for which they are required. For indoor use, copper-lined or lead-lined wood-case tanks without safe-pans, and wrought-iron or cast-iron tanks with safe-pans to catch the condensation, constitute the list generally favored by reason of superior fitness. Within limited dimensions, a durable and satisfactory tank-case can be made of heavy, well-fitted, and well-seasoned plank bolted together with iron rods and nuts, as shown in Fig. 55. For large sizes, heavy wood stays with tie-rods one-third of the way from each end, are added. With copper linings, but few nails should be used; and they should be so placed as to be covered by the copper, the joints being soldered by soaking the best quality of solder into the seams. The locking of the seams is shown greatly exaggerated in the engraving.

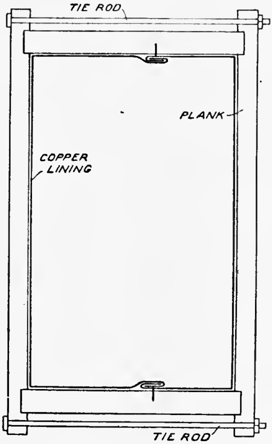


Fig. 55. Plan of Storage Tank in Case Made of Planks Bolted Together.

Cast-iron sectional tanks, like the form shown in Fig. 56, can be had in almost any size or shape. They are made up of plates planed

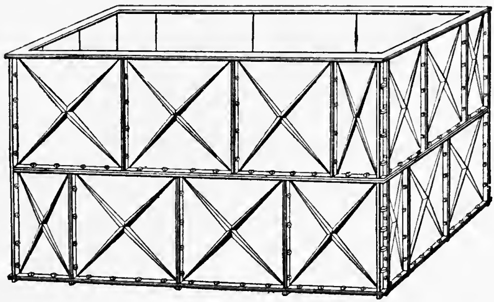


Fig. 56. Cast-Iron Sectional Tank.

and bolted together, the joints being made water-tight with cement. The sections are in convenient sizes, so that they can be handled easily, and conveyed without difficulty through small doorways or other openings to any part of the house. These tanks are easily set up, and are practically indestructible. Open and closed wrought-iron tanks, plain or galvanized, are often used, but are not so easily handled; and the larger sizes require to be riveted together and calked in place.

Lead-lined tanks are most frequently used for ordinary house plumbing. The linings were formerly wiped-in without exception. Sweating the lead together with a torch flame is however, quite as durable, and is much cheaper. To *sweat-in* a lining, take the exact length and breadth of the tank, trying at different points to be sure of allowing for any variations. Then cut out the bottom lining just the shape of the tank bottom, one and one-half inches larger each way, less twice the thickness of the lead. This allows three-quarters of an

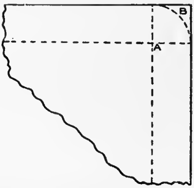


Fig. 57. Marking Off Bottom Sheet of Lead for Tank Lining, Leaving Edge to be Turned Up.

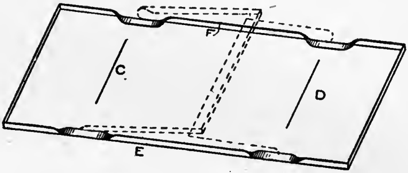


Fig. 58. Bending Bottom Sheet of Lead Ready to be Put in Tank.

inch to turn up all around; and the bottom will just fit when the side pieces are in place. Mark off the bottom all around, as shown by the dotted lines in Fig. 57; and turn up the edge. With the intersection of the lines *A* as a center, and the termination of one of them as a starting point, describe the line *B*, and cut off the corner outside of it. Then *work* the corner up square without a kink. If the lead is heavy, a little heat will make it work better. After working-up, the lead at the corners will be much thicker than along the sides; this may be needed in stretching out, at some of the corners.

When the edges and corners of the bottom are formed, clean the edges and about three-eighths of an inch down the outside all around, and rub the clean part with sperm candle. Next make a mark, say three feet from one end on each side, as at *E* and *F*, Fig 58. Then, on lines *C* and *D*, push the edges down inside, and fold the ends over as indicated by the dotted lines.

The bottom is now ready to be put in the tank, but it must wait until the sides and ends are in. If the sides and ends are light enough to be handled after joining like a ring, cut out a strip half an inch longer than will exactly go around the tank inside, equal to its depth plus the thickness of the tank wood for a flange at the top, as shown at J, Fig. 63. Then clean a half-inch of the under side and edge of the end that is to show in the seam, and three-quarters of an inch of the side that comes in contact with it, at the other end. The lead may then be propped up in the position shown in Fig. 59, by means of trestles and poles or in any other convenient manner; and the seam may be set, as shown, upon a board of hardwood, and the solder sweated into the lap by means of the torch and blowpipe. Solder for this kind of work should be three-fifths tin and two-fifths lead. A hardwood board is used because it will not smoke and burn like soft wood.

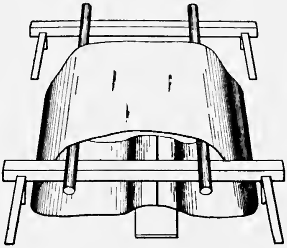


Fig. 59. Side and End Sheet of Lead Propped Up to Enable Seam to be Set and Soldered.

When the seam is made in this way, it shows inside the tank, and a good joint where the bottom seam crosses it can be made with ease, while one is never quite sure of the result if the seam crossed is on the other side.

Another method is to cut the lead the exact length that will go around the tank, clean the edges, butt them together over a hardwood board, as shown in Fig. 60, and *burn* them together instead of soldering. This can be done by using, instead of solder, a well-cleaned strip of lead about half an inch wide. Sperm candle will also answer as flux for burning. A piece of steel

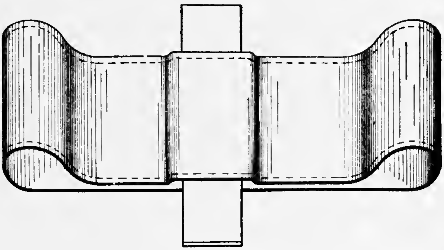


Fig. 60. Another Method of Joining the Two Ends of the Lead Sheet. The Ends are Butted against Each other Over the Hardwood Board and Fused Together.

or iron is best to place under the seam when burning, as more heat is required to do the work. An old crosscut saw blade, fastened to a board, serves well for such seams. The bottom edge of the side lining should be cleaned $1\frac{1}{4}$-inches wide, as shown at *H*, Fig. 61, which indicates how the cleanings on the bottom and the side and end lining come together in the tank. It is a good plan to run the soil brush around the bottom edge of the lining, as shown at *O* and *P*, Fig. 61. The soil keeps the solder from sweating too deep, and enables the seam to fill quickly. Further than this, however, soiling, as in the preparation for wiping, is not necessary for sweated seams.

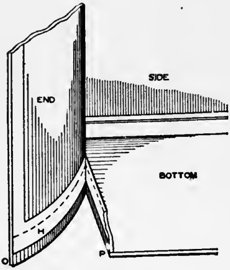


Fig. 61. Method of Joining End and Side Linings to Bottom Lining.

When the side lining "loop" is ready, lift it into the tank, square it out, flange over at the top, and secure the flange with brass, copper, or galvanized nails. Next, mark distances in the tank corresponding to those at *E* and *F* in Fig. 58. Then catch the bottom at the folded edges (Fig. 58), and lower it into the tank. As the ends are folded, there is room to stand inside the tank at the ends. Pull the folds upright so that marks *E* and *F* can be seen, and slide the bottom back or forward until *E* and *F* correspond with the marks made on the side lining. The ends may then be pushed down in place, and will be found to fit exactly if the measures have been properly taken.

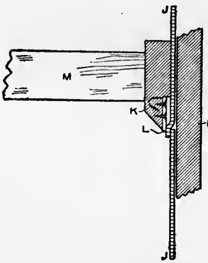


Fig. 62. Method of Keeping Lead in Place While Making Upright Seam in Tank.

After dressing down the bottom and pressing the turned-up edges against the sides and ends, sweat the bottom to the sides in the same way as the other seam was made, being sure that the solder "takes" well to both pieces of lead.

When a tank is large, handle the sides and ends in two or more pieces, always having the seams that are to be made in place come at the ends of the tank, as the ends are stiffest and best to brace against.

Fig. 62 shows the method of keeping the lead in place while making the upright seam in the tank, *I* being the tank wood, *JJ* the lining, *K* the straight edge, and *M* the brace. *K* is a piece of hardwood fastened to a strip of steel (a piece of an old framing square), as shown in the cut, the wood being about four inches wide by two feet long, and the steel *L* sticking half an inch out from the beveled edge of the wood. This steel edge keeps the lead from buckling under influence of the flame while blowing the seam, and is much better than a wood straight-edge, as it can be applied at the proper place with no fear of its burning or annoying the operator by smoking from the heat.

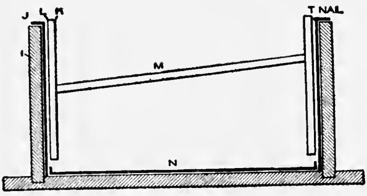


Fig. 63. Section Showing Lead Lining in Place, and Method of Bracing for Making Upright Seams.

Fig 63 shows the lining in place, and the method of applying the brace and straight-edge to the seams that are to be blown upright in position. Letters and parts in Figs. 62 and 63 correspond, *N* in Fig. 63 being the bottom.

Unless the supply is regular and abundant, and the storage by gravity, outside tanks of ordinary capacity, if of wood, are expensive and troublesome from leakage due to shrinkage of staves above the water-line and from necessity of painting; if of iron, from change in character of water, freezing, cost of boxing, delivery to, and discharge from, in a frost-proof manner, etc.

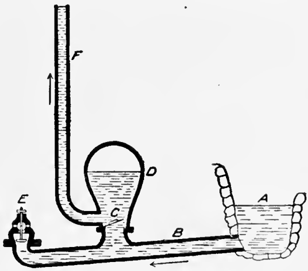


Fig. 64. Illustrating Principles of the Hydraulic Ram.

A spring supply will answer if of sufficient elevation to store water by gravity; or a waterfall above or below the house level may be handled with a hydraulic ram if 5 to 15 per cent of the water regularly available will suffice.

Hydraulic Ram. A ram uses the energy of a fall to elevate part of the water passing through it—one-sixth or less, according to the fall and the height to which the water is to be delivered. Four feet of fall is about as little as can be utilized to advantage; and fifty feet of liberal-size drive-pipe, even though it has to be coiled with uniform fall, is necessary to give the water momentum enough to get the best results.

Fig. 64 illustrates the elementary principles of a simple ram. *A* represents the source or spring; *B*, the drive (supply) pipe; *C*, a valve opening upward; *D*, an air-chamber; *E*, a valve tending to close downward by gravity; and *F*, the discharge pipe. In action, the water passes through the ram and out at a waste valve *E*, which is open downward until sufficient velocity is attained to lift and close the waste exit. There being then no other means of egress, the check-valve *C*, opening upward to the discharge pipe, is forced open; and the energy of acquired momentum delivers water into the air-chamber *D* and discharge pipe *F*, until the pressure on the waste valve falls too low to hold it up (closed). The check-valve *C* then closes, and retains the water in the discharge; and the waste valve *E* falls open by gravity, leaving a comparatively unrestricted exit through which the water continues to waste with increasing force until the velocity in the drive pipe is again sufficient to repeat the impulsive delivery. Rams are made with large air-chambers, to cushion the initial strain of impulse, and should have a delivery pipe at least one size larger than the ram opening, especially if working under light fall or high delivery.

Cisterns are seldom so deep or situated so low that ordinary house force-pumps within doors cannot be used. The distance of the cylinder above the lowest level from which water may need to be pumped, is limited in all pumps alike—33 feet 9 inches atmospheric lift under *perfect* conditions, and about 25 feet under the most perfect practicable pump arrangement. Indeed, the velocity of flow into the cylinder at any point above 20 feet is so slow that in practice the cylinder should be well within a twenty-foot limit in vertical distance from the water; and the closer the better. A foot-valve strainer at the end of a cistern suction pipe will keep the pipe filled and avoid frequent exhausting of the air before water can be obtained. When a foot valve is used, means of draining the suction to below frost line, when necessary, must be provided.

REVIEW QUESTIONS.

PRACTICAL TEST QUESTIONS.

In the foregoing sections of this Cyclopedia numerous illustrative examples are worked out in detail in order to show the application of the various methods and principles. Accompanying these are examples for practice which will aid the reader in fixing the principles in mind.

In the following pages are given a large number of test questions and problems which afford a valuable means of testing the reader's knowledge of the subjects treated. They will be found excellent practice for those preparing for Civil Service Examinations. In some cases numerical answers are given as a further aid in this work.

REVIEW QUESTIONS

ON THE SUBJECT OF

WATER SUPPLY

PART I

---

1. How does the use of water vary month by month and day by day?

2. How do surface and ground waters compare generally as to quality?

3. What is a fair amount of water consumption per capita for various purposes?

4. If rain is falling at the rate of 4 inches per hour and the run off is one-half as fast, what will be the flow in cubic feet per second from a drainage area of 10 square miles?

5. If the least annual run-off of a drainage area of 10 square miles be equal to 8 inches in depth, how many people will this provide for if the consumption averages 100 gallons per head per day, assuming there is storage capacity sufficient to utilize all the run-off for the year?

6. What storage capacity will be required in the above case if all the 8 inches runs into the reservoir in 5 months, leaving 7 months' demand to be met from the reservoir?

7. What conditions make it possible to secure artesian wells?

8. In what sort of material are we likely to find the most ground water available?

9. About what rate of consumption for fire purposes would be expected in a city of 25,000 inhabitants?

10. What causes the occurrence of springs?

11. What causes water to flow through the ground?

12. What are the most important uses of a public water supply?

13. What are the advantages and disadvantages of timber dams?

REVIEW QUESTIONS

ON THE SUBJECT OF

WATER SUPPLY

PART II

---

1. Calculate the necessary thickness of a cast-iron pipe to carry a water pressure of 175 lb. per sq. in.

2. If cast-iron pipe costs $30.00 per ton, what will be the cost of one mile of 8-in. pipe designed for a 250 ft. head?

3. Under what conditions are masonry conduits the most suitable forms of conduit for carrying water?

4. Compare the masonry conduit with iron pipe in regard to cost, durability, and the conditions under which they are the best form of construction.

5. When may conduits of vitrified clay pipe be used to advantage?

6. What is the function of a distributing reservoir?

7. Under what conditions is it desirable to employ reservoirs of earth; of masonry; of steel in the form of tanks or towers?

8. What capacity must a tank have to store water sufficient for one hour's fire use at a reasonable maximum rate in a town of 8,000 inhabitants?

9. What is the use of puddle in reservoir walls?

10. What precautions are to be observed in the construction of reservoir embankments?

11. What are the advantages of covered reservoirs?

12. Determine the thickness of a standpipe at points 10 feet apart from the top downward whose dimensions are: height 120 ft.; diameter 18 ft.

REVIEW QUESTIONS

ON THE SUBJECT OF

IRRIGATION ENGINEERING

---

1. Define the term *gravity works* as applied to irrigation, and enumerate the sources of water supply.

2. Explain fully the effect of slope and of area of cross-section upon the location of a canal. What should determine the minimum and maximum velocity of flow?

3. Describe fully the nature of irrigation, and compare the design and control of an irrigation system with the design and control of a domestic water supply system.

4. Describe fully the principles governing the design and location of masonry weirs and dams.

5. Describe the different types of current meters, and explain their use in determining mean velocity of flow.

6. Describe fully the principles governing the location of an impounding dam, and describe the method of conducting surveys to determine the capacity of the reservoir.

7. Explain the effect of evaporation upon storage water supplies, and the allowance to be made therefor.

8. Define the term *duty of water* as applied to irrigation. Explain fully how and why the duty of water may determine the financial success or failure of an irrigation project.

9. Define the term *precipitation*, and explain the relation between available precipitation and the necessity for irrigation.

10. Explain the Chezy formula as modified by Kutter.

11. Define the term *lift irrigation*, and enumerate the sources of water supply.

12. Define the terms *wetted perimeter*; *hydraulic radius*; *coefficient of friction*.

13. Define the term *weir dam*, and fully explain the principles governing its location and construction.

14. Describe the effect of evaporation upon run-off; and explain how evaporation is affected by topography, soil, humidity, and temperature.

15. Explain the effect of absorption and percolation upon storage supplies, and the allowance to be made therefor.

16. Explain how the variations in run-off will affect the design of the spillway of a reservoir.

17. Explain the method of measuring flow of streams by weirs, and state the Francis formula for the flow of water over weirs.

18. Describe fully the units of measure for water duty and flow.

19. Describe the effect of altitude upon precipitation.

20 Enumerate the parts forming the headworks of a canal, and describe fully the functions and construction of each part.

21. Explain fully the methods of measuring rainfall.

22. Describe the rating of the current meter.

23. Explain fully why the duty of water is rising in some portions of the West.

24. Explain how the topography of the country will affect the location of a canal.

25. Explain the necessity for drainage upon side-hill location, and describe the various ways in which it may be accomplished.

26. Define the term *run-off*, and explain how it will be affected by topography, soil, and temperature.

27. Explain the principles governing the flow of water in open channels. Explain the relation between velocity of flow and cross-sectional area of a channel.

28. Describe the field operations in determining the mean velocity of flow in a stream.

29. Define the term *storage works;* explain their functions, and fully explain the principles governing their capacity, location, and construction.

30. What depth of water upon the surface is necessary to water the average soil thoroughly, and how many waterings should be given in a season?

REVIEW QUESTIONS

ON THE SUBJECT OF

SEWERS AND DRAINS

PART I

---

1. Explain what is meant by the *separate* system of sewerage, and name its advantages and disadvantages as compared with the *combined* system.

2. Name the different materials used for sewers, and state the conditions to which each kind of material is adapted.

3. What is a *subdrain*, and when and why should subdrains be used?

4. What should be the minimum size of separate sanitary sewers, and why? Of storm sewers, and why?

5. Why is the egg shape of sewer sometimes used? For what kind of sewers is it advantageous?

6. What nations of antiquity built the first known sewers? When did the scientific design and construction of sewers become general?

7. Give the principal objections to the use of cesspools.

8. What difficulties are encountered in designing and constructing the junctions of large sewers, and what designs are generally adopted for junction chambers?

9. What are the general facts as to the fluctuations in the flow of sanitary sewage at different hours of the day and night?

10. How would you determine the probable flow of sanitary sewage per capita per day, for any particular sewer system? Between what limits would the per capita per day flow probably lie in different systems?

11. How large capacities should the different kinds of separate sanitary sewers have as compared with the average flow of sewage in them?

12. What velocity of flow, when running full or half-full, should a separate sanitary sewer have to be self-cleansing? What velocity would such a sewer have at the time of ordinary low flow?

13. What should be the minimum grade of a 10-inch separate sanitary sewer? What velocity would this grade give when the sewer flows half-full?

14. How does the velocity of flow in a sewer running half-full compare with the velocity running full? Is the velocity of flow in a sewer ever greater than when the sewer runs full; and if so, when?

15. What should be the minimum grade of a circular storm sewer 4 feet in diameter, and what velocity would this give when the sewer runs full?

16. What are the purposes of *manholes* in sewer systems, and how far apart is it proper to locate manholes?

17. Describe and discuss the principal features of designs for manholes, and of the construction of manholes.

18. What are the purposes of sewer *flush-tanks*, and where should flush-tanks be provided in a sewer system?

19. Give a general description of the principal features of a *sewage siphon*, and explain how such a siphon operates to flush the sewer.

20. Explain the proper method of making the connection between a house sewer and the street sewer.

21. What diameter of circular brick sewer, laid to a 0.35 per cent grade, would be required to carry 65 cubic feet per second of storm sewage.

22. What size of egg-shaped brick sewer, laid to a 0.25 per cent grade, would be required to carry 42 cubic feet per second of sewage?

23. What size of separate sanitary lateral pipe sewer, laid to a 0.47 per cent grade, would be required to carry the sewage of 1,000 tributary population, the average flow of sewage being 85 gallons per capita per day?

24. Give a general summary, in your own language, of the methods of calculating the sizes of the different sewers in the design of a separate sanitary sewer system.

25. State the general law of the heaviest (or *maximum*) rainfalls.

26. What is the *time of concentration* for a storm sewer, and what use is made of it in calculating the size of a storm sewer?

REVIEW QUESTIONS

ON THE SUBJECT OF

SEWERS AND DRAINS

PART II

---

1. At 15 cents per cubic yard, what will be the cost of a drainage ditch 1 mile long, 6 feet average depth, and 20 feet average width?

2. What is a *trap*, what are the purposes of traps, and where should they be used? What kind of traps are best?

3. What size and kind of pipe should be used for house sewers, and at what grades should they be laid?

4. What average width of drainage ditch, carrying 5 feet depth of water, will be required to take the drainage of 50,000 acres of land, the grade being 5.3 feet per mile?

5. Estimate the cost, complete, under average conditions, of a 3-ring circular brick sewer, 9 feet in diameter, the average depth to the invert being 16 feet, and the length being 4,900 feet, with 12 manholes.

6. Explain the proper arrangement for the ventilation of a system of plumbing. Why is it necessary to connect the high point of each trap on the sewer side to the main ventilation pipe?

7. State in your own language the principal advantages of tile land drains; of open-ditch drains. Compare tile drains and open ditches for draining land.

8. What size of sewer subdrain, laid to a 0.35 per cent grade, will be required for outlet for 26,000 feet of tributary subdrains, under ordinary soil and ground-water conditions?

9. What is a *soil pipe;* of what size and material should it be made, and how high should it be extended?

10. Give a general explanation, in your own language, of the principal factors affecting the cost of sewers in different cities. How much variation from the average cost may these factors cause?

11. What is a sewer *reconnaissance?* Explain how such a reconnaissance is made, and its objects.

12. What is a *Notice to Contractors?*

13. Estimate the cost complete, under average conditions, of a 20-inch pipe sewer 6,000 feet long, with an 8-inch subdrain, and 20 manholes, the average depth being 12 feet.

14. Give a general explanation and discussion, in your own language, of the principal methods of paying for sewers.

15. Give a general description, in your own language, of the surveys required for sewer plans.

16. What, in a general way, are *sewer specifications?*

17. Explain the organization and duties of the engineering force during the construction of a sewer system.

18. What is a *Sewer Ordinance?* A *Sewer Permit?*

19. Explain the proper construction of a *sewerage map.*

20. Explain the proper construction of *sewer profiles.*

21. What other plans than the sewerage map and the sewer profiles should an engineer provide in making complete plans for a sewer system?

22. What records should be kept by the engineer during the construction of a system of sewers?

23. What is a *Form of Proposal,* and for what purpose should it be required to be used?

24. Explain how and why the composition of sewage varies even in the same sewer.

25. What things are usually determined in making chemical and bacterial analyses of sewage; and what, in a general way, is the meaning of each thing usually determined in such analyses?

26. Write, in your own language, a specification for laying sewer pipe.

27. Explain the general steps in letting a sewer contract.

28. What is *sewer sheathing?* Explain how it is used in sewer construction.

29. Describe *intermittent sand filters,* and explain their action on sewage.

REVIEW QUESTIONS

ON THE SUBJECT OF

PLUMBING

PART I

---

1. Under what conditions is a spring of water available for house supply?

2. What methods are adopted of supplementing municipal service in case of insufficient pressure?

3. What are the essential requirements of a good laundry tray?

4. What do you consider the poorest types of water-closets?

5. What are the general methods of supplying buildings with water?

6. How far should the bottom of a cistern be below the cylinder of an ordinary house suction pump?

7. How is *siphonic eduction* effected in the case of range closets? Illustrate by diagram.

8. Describe the part played by lead in modern plumbing.

9. What is a *pneumatic siphon* closet? Give diagram.

10. How can an open-trough range closet be satisfactorily ventilated?

11. How can supply to tanks be automatically regulated?

12. When is a hydraulic ram available for house supply?

13. Describe the various kinds of bathtub supply and waste fittings.

14. What are the advantages or disadvantages of the *combined hopper and trap* and the *wash-out* types of closets? Illustrate by freehand sketches.

15. Classify bathtubs (1) according to material; (2) according to shape. Illustrate the shapes by freehand sketches. Discuss the relative merits of the different classes of bathtubs.

16. What size of pipe will be needed to discharge 150 gallons a

minute, at a distance of 100 feet under a pressure head of 40 feet? (Use the tables.)

17. How is flushing of urinals effected?

18. What is a *hydraulic ram?* Illustrate by diagram.

19. Describe the different types of lavatories.

20. Under what conditions will the house supply of water be in danger of contamination from the water-closet? How can the danger be obviated?

21. What difficulties are inherent in the use of wooden tanks for outside storage of cold water?

22. Describe the different types of drinking fountains, and their uses.

23. What are the objections to the range type of water-closet? What conditions are absolutely essential to their satisfactory working?

24. Describe the process of *sweating-in* the lining of a lead-lined tank.

25. What is the cause of "sweating" on tanks and piping? How can it be prevented?

26. Discuss the relative merits of the different materials of which water-closet bowls are made.

27. What part does friction play in determining details of pipe and fixture installation?

28. What is meant by a *jet-siphon* closet?

29. Describe the different types of urinals.

30. What different kinds of tanks are made for cold-water storage indoors?

31. Describe, with diagrams, the different methods of tank arrangement in water-closets.

32. How are the effects of grease in sink water avoided? In answering, refer briefly to the different kinds of sinks.

33. What head will be required for the discharge of 150 gallons a minute through a 2-inch pipe 500 feet long? (Use the tables.)

34. Name and describe the parts of a complete shower-bath fixture.

35. Give a definition of *Plumbing* and of *Sanitary Science.*

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