SEWERAGE
AND
SEWAGE DISPOSAL

A TEXTBOOK

AMERICAN SEWERAGE PRACTICE

THREE VOLUMES

BY

METCALF AND EDDY

VOL. I —DESIGN OF SEWERS
747 pages, 6 × 9, 328 illus., 72 tables

VOL. II —CONSTRUCTION OF SEWERS
564 pages, 6 × 9, 181 illus., 81 tables

VOL. III —DISPOSAL OF SEWAGE
877 pages, 6 × 9, 234 illus., 210 tables

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A TEXTBOOK

BY

LEONARD METCALF

AND

HARRISON P. EDDY

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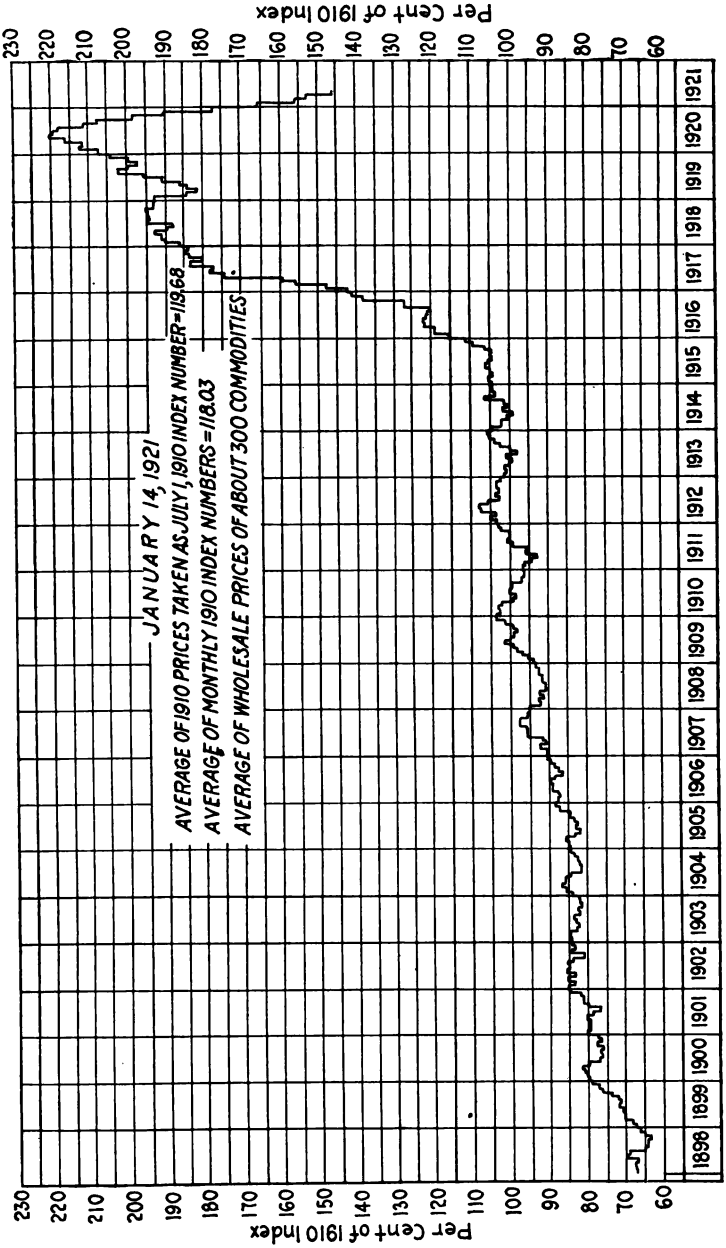
PREFACE

Since the three volumes of "American Sewerage Practice" were published in 1914–1915, the authors have been urged to prepare a single-volume abridgment of them for class use in engineering schools. The authors were reluctant to do this, because of two conditions. The first is the great variation in the time devoted to sewerage engineering in different schools. This has the result that a textbook so brief as to verge on the superficial from the viewpoint of one class may be too detailed for another. The second condition is the fact that some of those features of sewerage engineering which in practice it is most important to master are apparently held by those laying out college courses to deserve little attention from students. For example, a wise estimate of the quantities of sewage and storm water to be handled is the basis of design of a sewerage system, yet the methods of preparing such estimates and actual practice in their use receive relatively little attention in college courses. Furthermore, the best arrangement of the main sewers in a sewerage plan may depend largely upon the treatment, if any, that the sewage must receive before disposal and upon the place of disposal, subjects involving a knowledge of the nature of sewage and the changes which it undergoes under natural conditions or those involved in specific methods of treatment. Yet some college classes in sewerage engineering receive no instruction in sewage treatment.

As the requests for a textbook continued, the authors finally decided to prepare a book giving the information which they consider it desirable for the young student to acquire before taking up work in this field. In a sense, they have felt under obligations as professional engineering specialists to do their part in smoothing the way for the first steps taken by engineering students in this branch of engineering.

This book can be used in two ways. The text can be read and mastered within the time allotted to the subject even when the course is very short. Where more time is available, the authors believe that the student will gain a much better grasp of the fundamentals of the subject by devoting most of this additional time to the solution of problems. For example, a student should, if possible, test his ability to predict the growth of population by forecasting the 1920 population of cities by the methods described in Chapter II, and checking his figures by the 1920 census returns. He should determine the carrying capacity of sewers of different cross-sections on the same grades, and the sizes needed when the plan remains the same, but the grades are different. In short, he should test his grasp of the qualitative information which must necessarily form the larger part of a sewerage textbook, by applying this information to quantitative problems based on assumed conditions or data from places with the local conditions of which he is familiar. He should also lose no opportunity to see sewerage construction in progress and sewage treatment works in operation.

There is little in this textbook not covered in much greater detail in "American Sewerage Practice." The three volumes of that treatise have elaborate indices, so that it seemed unnecessary to insert references to the treatise in this textbook. The student is assumed to have access to textbooks on hydraulics, masonry and reinforced concrete. No cost data are given, because the great changes in costs induced by the war make it necessary to employ older data with a great deal of caution. They must be supplemented, too, by practical knowledge of current changes which very few undergraduate students possess. A study of fluctuations in commodity prices since the war, based upon Dun's and Bradstreet's characteristic numbers, will be helpful to the student, however, in connecting up, approximately, pre-war and post-war costs, and in bringing home to him the necessity of recording with his cost data the unit prices paid to labor and for materials upon the construction work under consideration. In this connection the accompanying diagram of fluctuations in commodity prices, in percentages of 1910 prices, as determined by Dun's index numbers, prepared by the authors for their office use, may be of interest. Dun's index numbers represent the weighted average wholesale quotations of cost per pound, on the first business day of each month, of about 300 commodities, the actual cost of each commodity being weighted according to its annual per capita consumption in the United States. The reasons for making the percentage comparison with the prices of 1910 as a base were that the figures of that year approximated the average condition over the period covered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by a number of different records, were uninfluenced by the war, and yet were near enough the war period to approximate, though they were slightly below, the costs prevailing just before the war.

Reference to sewage disposal for detached residences and institutions has been omitted in this book because the principles involved are fully explained. The practice in their application varies not only with the local conditions, the size of the project and the influence upon design of the necessity for simplicity, but also with the standards set by the local health authorities. Standards have been developed by the U. S. Public Health Service, by the State Colleges, and by State Boards of Health of various states

The authors are engineers, not teachers, hence this text book reflects the engineer's rather than the teacher's viewpoint. They will appreciate the more, therefore, any suggestions for increasing its usefulness to undergraduate students and their instructors. Constructive criticism is invited, because through it the experience of the authors and other engineers in designing, building and operating sewerage and sewage treatment works, can be made more helpful to the students who will eventually carry the responsibility for such undertakings.

The authors take pleasure in giving recognition to Mr. John M. Goodell, for many years Editor of "Engineering Record," who assisted them in the preparation of "American Sewerage Practice" and who has done the greater part of the work of condensing its three volumes into this book. Credit is also due to the junior partners and members of the staff of Metcalf and Eddy, and to the publishers, for their helpful assistance.

As the book reflects the past experiences and current opinions of many practicing engineers, it should be of service to the student.

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14 BEACON STREET,
BOSTON, MASS.
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SEWERAGE AND SEWAGE DISPOSAL

A TEXTBOOK

CHAPTER I

GENERAL FEATURES OF SEWERAGE AND SEWAGE-TREATMENT WORKS

Sewerage provides the means for removing the liquid wastes from a building to some more or less remote place of disposal. In its simplest form, it is merely the extension of the discharge pipe of the plumbing system to a neighboring body of water, to a cesspool,[1] or to some plot of land which will absorb the wastes. Each of these methods of disposal may be objectionable under some conditions. The water into which the sewage is discharged may be contaminated by the sewage so as to lose part or all of its usefulness to others; the cesspool may overflow and become an offensive breeding place for flies; and the land may be made foul by the sewage discharged upon it.

From the technical and sanitary viewpoints, the construction and operation of these small isolated sewerage systems calls for the same kind of knowledge involved in the design and operation of the municipal sewerage plants. In taking up the study of sewerage and sewage treatment for the first time, the clearest conception of the field to be covered can be formed from an examination of the works in a city, because of their greater size.

Definitions.—At the beginning of such an examination, a clear conception of the technical terms employed in sewerage engineering is helpful. These terms are used very loosely by the general public and engineers should avoid such carelessness, in order that what they say and write may be free from ambiguity. The Committee on Sewerage and Sewage Disposal of the American Public Health Association, recommended certain definitions which will be quoted in this chapter:

[1] Cesspools are usually shallow pits in the ground, lined with stone set without mortar, from which the liquids percolate into the ground and mingle with the ground water. They are usually not wholly satisfactory except when they are surrounded by porous material, such as sand, and great care should be taken that the liquids leaching from them shall not contaminate wells in the neighborhood.

"*Sewage* is a combination of the liquid wastes conducted away from residences, business buildings and institutions, together with those from industrial establishments; and with such ground, surface and storm water as may be present.

"*Domestic sewage* is that from residences, business buildings or institutions.

"*Industrial wastes*[1] are the liquid wastes resulting from the processes employed in industrial establishments.

"*Surface water* is that portion of the precipitation which runs off over the surface of the ground.

"*Storm water* is that portion of the precipitation which runs off over the surface during a storm and for such a short period following a storm as the flow exceeds the normal or ordinary run-off.

"*Ground water* is that which is standing in, or passing through, the ground.

"*A sewer* is a conduit for carrying off sewage.

"*A common sewer* is a sewer in which all abuttors have equal rights of entrance and use.

"*A house connection* is a pipe leading from a building to a common sewer.

"*A lateral sewer* is a sewer which does not receive the sewage from any other common sewer.

"*A sub-main*[2] or *branch sewer* is a sewer into which the sewage from two or more lateral sewers is discharged.

"*A main or trunk sewer* is a sewer into which the sewage from two or more sub-main sewers is discharged.

"*An outfall sewer* is a sewer extending from the lower end of the collecting system to a point of final discharge into a body of water, or to a sewage-treatment plant.

"*A separate sewer*[3] is a sewer intended to receive domestic sewage and industrial wastes without the admixture of surface or storm water.

"*A combined sewer* is a sewer intended to receive domestic sewage, industrial wastes and surface and storm water.

"*An intercepting sewer* is a sewer generally laid transversely to the general sewer system to intercept all the sewage collected by the sewers of a separate system or the dry-weather flow of sewage and such additional surface and storm water as may be determined from a combined system.

"*A relief sewer* is a sewer designed to carry a portion of the flow from a district already provided with sewers of insufficient capacity, and thus prevent overtaxing the latter.

"*A storm-water overflow sewer* is a sewer designed to carry the excess over a certain volume of combined sewage from a main or intercepting sewer to an independent outlet.

"*A drain* is a conduit for carrying off storm water, surface water, subsoil or ground water.

"*A storm drain* is a conduit for carrying off surface water and storm water.

[1] In the opinion of the committee, the term "industrial wastes" is preferable to "trade wastes."

[2] The committee is of the opinion that preference should be given to the term "submain sewer" rather than to "branch sewer."

[3] In the opinion of the committee, the use of the term "separate sewer" is preferable to "sanitary sewer," which latter term should be discontinued.

"*A land drain* is a conduit for carrying off subsoil or ground water, and for draining land.

"*A sewer system* is the collecting system of sewers and appurtenances, together with such small pumping stations as may be required to lift the sewage from low-level districts.

"*A combined system* is a system of combined sewers.

"*A separate system* is a system of separate sewers.

"*Sewerage works* comprise the sewer system, main pumping stations, treatment works, means of disposal of effluent and sludge, and all other works necessary to the complete collection, treatment and disposal of sewage."

Before continuing the committee's definitions it is desirable to supplement those already given with others.

Plumbing inside a building, as the term is ordinarily used, comprises the pipes and fixtures used to supply water and to remove it when used. Plumbing connects with both the sewerage and water-supply systems of a city, just as the capillaries in the human body connect the veins and arteries.

The main or *house drain*, or *house sewer*, better characterized as the house connection, is the pipe connecting the sewers or drains within the house with the common sewer in the street. It should not be too small. The wastes removed from a store, apartment house or office building are usually much the same as those from houses. Sometimes large quantities of dirt and grease are washed from an automobile into the house connection from a garage; bundles of rags, towels, and occasionally old hair brushes and even larger objects get into the pipes and stop them if they are not large enough to give such rubbish free passage.

Because of the misuse of plumbing neither the main drain nor the house connection should be less than 4 in. in diameter and 5 or 6 in. (the most common size) is a preferable size for the house connection, which is more difficult to clean than the main drain.

The main trap is a small inverted siphon sometimes placed in the main drain near the building wall to keep air from the sewer system out of the building. The bend of the trap is filled with water while the house is occupied and this water acts as a seal against the passage of air through the trap.

There has been a great deal of controversy among engineers regarding the use of a main trap. When sewer systems were comparatively novel and often badly designed, so that solids collected in them and gave off foul odors while decomposing, there was a general belief that sewer air was dangerous to health and should be excluded from buildings by the use of the main trap. In the course of time sewers were better designed and built and examinations of the air within such sewers showed no traces of injurious gases or bacteria. Meanwhile the plumbing within buildings was improved and building codes compelled the trapping of all fixtures and the venting above the roof of all stacks. Many sanitary engineers held, therefore, that it was desirable to ventilate the sewers through the main drains and stacks, by omitting the main traps. At first there was great opposition to this, particularly by physicians serving as health officers, but the opposition has practically died out with the spread of definite knowledge regarding sewer air. Traps are now used in cities where all sorts of wastes are discharged into the sewers and there is danger of odors coming from some of these wastes before they leave the sewer system. But in residential districts where the sewers are in good condition, it is customary to omit the main traps.

Stacks are vertical pipes rising from the main drain.

A soil pipe, strictly speaking, is any pipe through which pass the wastes from a water closet, but in the plumbing trade the term also designates cast-iron pipe of any size used in the plumbing system.

An inverted siphon is a portion of a sewer which drops considerably below the average grade and then rises again, in such a way as to run under pressure.

A waste pipe is one receiving wastes from other fixtures than water closets.

A dead end is the upper extremity of a sewer, above the point where the first house connection or other sewer enters it.

Roof water is the storm water running off roofs.

Street wash is the water flowing from the surface of a street into a sewer or drainage system. Street wash may be rain water or water used in flushing the streets from hydrants or sprinkling carts, or melted snow. The first flow from a street after dry weather is often grossly polluted and liable to cause offensive conditions if not removed promptly.

Run-off—in a narrow sense—is the portion of the storm water that reaches the sewers and drains. The term is also used to denote the portion of the rainfall that passes away in brooks and streams from a designated area.

Infiltration or *leakage* is the ground water that finds its way into sewers. In sewerage engineering "leakage" usually has an opposite meaning to its significance in water supply, for it designates the water entering a conduit, rather than the liquid escaping from it. Infiltration sometimes presents problems difficult for the engineer to solve. The authors have known storm and ground water to leak into sewers to such an extent that the normal flow of 300,000 gal. per day during dry weather was increased to 3,000,000 gal. by heavy rainfall, although all storm and ground water was theoretically excluded from the sewers.

Underdrains are drains to remove ground water from engineering works above them, such as sewers, pavements, filters and foundations.

Pipe sewers are those constructed of relatively short lengths of pipe, jointed together.

A cradle is a support for a sewer, usually constructed of wood or concrete, which distributes the weight of the sewer over a wider portion of the bottom of the trench or other support than the sewer itself would cover.

The invert of a sewer is technically the lowest point of its interior at any cross-section; the term is also applied loosely to the whole inside bottom surface of a sewer.

The crown of a sewer is the highest point of its interior at any cross-section.

Sewer Appurtenances.—There are a number of appurtenances of sewers which are so generally used that they are enumerated here. The Committee on Sewerage and Sewage Disposal of the American Public Health Association recommended the following definitions in 1917:

"*A manhole* is a shaft, or chamber, leading from the surface of the ground to the sewer, large enough to enable a man to gain access to the latter.

"*A lamphole* is a small vertical pipe or shaft leading from the surface of the ground to the sewer, for admitting a lantern or reflected light for purposes of inspection.

"*A wellhole* or *drop manhole* is a vertical shaft in which sewage is allowed to fall from one sewer to another at a lower level.

"*An inlet* is a connection between the surface of the ground and a combined sewer or drain for the admission of surface or storm water.

"*A catch basin* is a chamber inserted on an inlet to prevent the admission of grit and other coarse material into the sewer or drain.

"*A flush tank* is a tank in which water or sewage is accumulated to be quickly discharged later, for the purpose of flushing the sewer.

"*A regulator* is a device for controlling the quantity of sewage admitted to an intercepting sewer.

"*An outlet* is the end of a sewer or drain from which its contents are finally discharged.

"*A storm overflow* is a weir, orifice or other device for permitting the discharge from a combined sewer of that portion of the storm flow in excess of that which the sewer is designed to carry."

Other appurtenances which are quite generally used and need definition here are the following:

Fittings are forms of pipe in which the cylindrical form between the spigot and socket ends is modified, in such a way as to accomplish a specific purpose, such as the joining of two or more sewers, or changing the diameter or direction of the sewer. They are best defined by the diagram of their outlines, Fig. 1. Some of them,

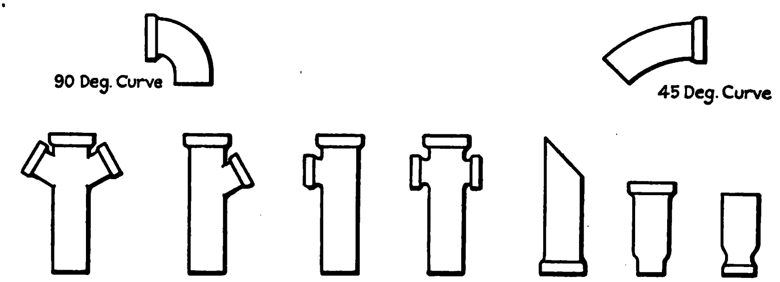


FIG. 1.—Vitrified pipe fittings.

such as the *slant*, are also used in connecting pipe sewers with masonry or concrete sewers. Such fittings were formerly called "specials" because each engineer designed his own forms, but as they are now standard products "fittings" is a better term for them.

A chimney is a vertical pipe, usually encased in 6 in. or more of concrete, which is sometimes run up from every branch or slant of a sewer in a deep trench, to afford an easy connection for a house sewer. These chimneys usually terminate at a uniform depth below the surface.

A manhole casting comprises a heavy cast-iron frame set on top of the manhole masonry and a cover or lid, closing the manhole, fitting rather loosely on the top of the frame. If it is desired to ventilate the sewer through the manhole, as when main traps are used, the cover may have holes through it and it is then sometimes called a "perforated" or "ventilating" cover. The present tendency is to minimize the number of perforated covers, as they permit the entry of storm water and earth, grit, and other foreign material into the sewer.

General Conditions Governing Sewer Systems.—Experience indicates that in order to avoid frequent clogging, laterals should not be smaller than 8 in. in separate sewerage systems, except in very small towns. In most cases the smallest size suitable for a storm-water drain or combined sewer is 12 in. The size of the sub-main and main sewers should be selected so that they will be able to carry off the domestic sewage determined by the methods explained in Chapter II, the industrial wastes estimated after making an investigation of the local conditions, and, in some cases, the storm water determined by the methods explained in Chapter III.

To save needless expense both sewers and house connections should be laid no deeper than necessary, but in order to carry off the sewage from the laterals the sub-main sewers must be deeper than local house drainage alone demands. The grades of all sewers should be sufficient to give an adequate scouring velocity to the sewage, as explained in Chapter IV. Anything more than this in relatively flat country involves useless increase in cost; anything less, increase in cost of maintenance. The grades must be flat enough to allow the main sewers to discharge into the outfall sewer without pumping, if this is practicable; but it is sometimes necessary, in order to avoid excessively deep sewers, to construct small pumping stations where the main sewers reach considerable depths and to lift the sewage at these small stations to such a height that it will be able to flow away by gravity through sewers at reasonable depths. The elevations and grades of the outfall sewers are fixed by the local conditions. The network of lateral, sub-main and main sewers must be adjusted to them. It is in making such connections of lateral, sub-main and main sewers that drop manholes often prove useful.

The main stems of a sewerage system are the outfall sewers and the larger main sewers. In small cities there may be but one outfall sewer, while in large cities there may be several, sometimes discharging independently and sometimes radiating like the ribs of a fan. With combined sewers, the volume of storm water is generally so much greater than that of house sewage and industrial wastes that the capacity of the large sewers is determined by it. The only influence of the volume of house sewage and industrial wastes in such a case is to modify the shape of the invert of some sewers, so as to afford a self-cleansing channel for the dry-weather flow, a subject explained in Chapter IV.

Where one sewer joins another, particularly when they are large, the angle between the axes of the two currents at the junction should be as small as possible in order to prevent any checking of the velocity of flow. For the same reason, the elevation of the surface of the sewage in both sewers at the junction should be as nearly the same as practicable.

The cost of outfall sewers, usually a large item in the cost of the sewerage system, may be reduced by various expedients. Sometimes there are several points along a main sewer of a combined system where storm water can be discharged into short channels or conduits leading to neighboring bodies of water. In London part of the storm water entering the outfall sewers is pumped from them into the Thames. These expedients, by relieving the outfall sewer of excessive quantities of storm water, enable its size and hence cost to be kept within reasonable limits. Another expedient is to run the combined sewers to neighboring bodies of water and provide intercepting sewers near their outlets which will take from them all the dry-weather sewage and conduct it to a more distant point where its discharge will be less objectionable. In this way the stronger dry-weather flow only is carried to the more distant disposal point, the more dilute storm-water flow finding its way directly to the nearest water course.

Intercepting sewers are used not only as explained in the previous paragraph but also to collect all or part of the sewage above a given elevation, either to permit flow by gravity to the place of treatment or disposal, or to prevent the accumulation of so much sewage at lower levels that the intercepting sewers there would be excessively large.

Respective Usefulness of Separate and Combined Sewers.—A combined sewerage system requires but a single sewer in a street, thus saving a portion of the space required by the two conduits of the separate system. This is important in streets congested with underground structures. Usually the quantity of domestic and industrial sewage is not greater than the margin of error in estimates of the quantity of storm water to be removed. Therefore the inclusion of the sewage with the storm water does not require a combined sewer appreciably larger than the drain for serving the same district adequately. Even if it does require an appreciably larger section, the additional cost of building the combined sewer as compared with the drain will be practically negligible in most cases. Hence the cost of a combined system is less than that of both a separate system and a drainage system, where complete underground removal of both storm water and sewage is necessary.

It may be undesirable to allow storm water from urban districts to contaminate neighboring bodies of water, not only because of sanitary objections but sometimes because of the disagreeable appearance of accumulations of refuse or deposits of sediment forming on the bottoms of these water courses, floating matter and sleek. Such conditions in any case make necessary an extensive drainage system which might as well be made to remove sewage, thus avoiding the additional expense of dual systems.

Where a separate system of sewers is paralleled by a drainage system and roof water cannot be allowed to flow over the surface of the ground, a system of collecting drains is required outside the buildings in addition to the usual inside plumbing, or two systems of plumbing are necessary within the buildings, one to take care of the sewage and the other to remove the roof water. This is not only expensive to the property owner but it introduces a dangerous condition. There have been sewerage and drainage systems administered so badly that the surface water was discharged by incorrectly laid connections into sanitary sewers, forcing the sewage back into the cellars and out through manholes upon the streets; in other cases sewage has been discharged into drains resulting in unnecessary pollution of waters into which they emptied.

Notwithstanding these disadvantages of the separate system there are certain conditions under which it may be advantageous. Dr. Hering summarized[1] some of them as follows:

"The separate system is suitable—

"Where rain-water does not require extensive underground removal and can be concentrated in a few channels slightly below the surface, or where it can safely be made to flow off entirely on the surface. Such conditions are found in rural districts where the population is scattered, on small or at least short drainage areas, and on steep slopes or side hills.

[1] Report to National Board of Health on European methods of Sewerage and Sewage Disposal, 1881.

"Where an existing system of old sewers, which cannot be made available for the proper conveyance of sewage, can yet be used for storm-water removal.

"Where purification is expensive, and where the river or creek is so small that even diluted sewage from storm-water overflows would be objectionable, especially when the water is to be used for domestic purposes at no great distance below the town.

"When pumping of the sewage is found too expensive to admit of the increased quantity from intercepting sewers during rains, which can occur in very low and flat districts.

"Where it is necessary to build a system of sewers for house drainage with the least cost and delay, and the underground rain-water removal, if at all necessary, can be postponed.

"The principle of separation, although often ostensibly preferred on sanitary grounds, does not necessarily give the system in this respect any decided advantage over the combined, except under certain definite conditions. Under all others, preference will depend on the cost of both construction and maintenance, which only a careful estimate, based on the local requirements, can determine."

Influence of Topography on Sewerage Plan.—The arrangement of the small and large sewers which make up a sewer system is influenced largely by the topography of the city. In a large city situated on a flat plain without any neighboring rivers or lakes into which the sewage can be discharged without elaborate treatment, the radial system may prove best. This has its most elaborate development in Berlin, where it was introduced by Hobrecht. The city is divided into a number of sectors and the sewage of each sector is carried outward by pumping to its independent disposal farm, or the trunk sewers of two or more sectors may be connected to a farm. The advantage of this system is that most of the sewers are likely to be of adequate capacity for a long period, and the large expensive sewers are reduced to their minimum length.

In most cases such an arrangement is rendered impracticable by the existence of hills, water courses and other topographical conditions. Usually, moreover, old sewers complicate the problem, for it is always desirable to utilize existing structures so far as practicable. Only in rare cases does the engineer have an opportunity to design a complete sewerage system for a large city, as was the case in New Orleans and in Baltimore.

In Baltimore, where the sewage had to be taken 5¾ miles outside the city for treatment, it was apparent that the storm water should be collected separately, for there was no objection to its discharge into the nearest water courses adapted to receiving it. The city is intersected by four streams, which discharge into branches of the Patapsco River. One of these streams receives so much foul run-off that it has been covered over; the others are open. The Patapsco and its branches are tidal arms of Chesapeake Bay. The drainage area was divided into 28 districts, and the storm-water drains in each one were planned independently of the rest, to fit the topography and arrangement of streets in the best way. These drains were kept as close to the surface as possible, in the interest of economy and

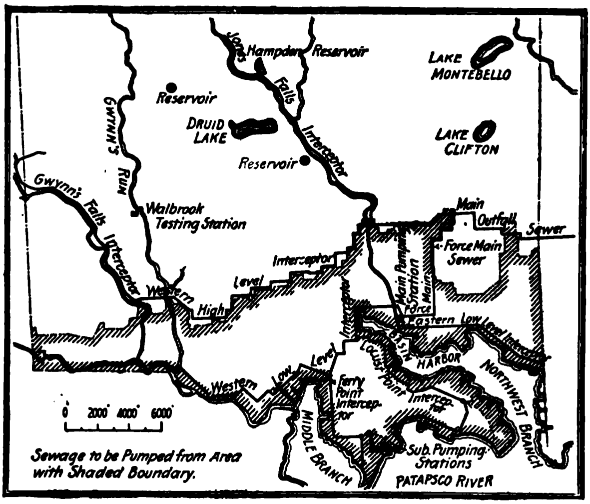


FIG. 2.—Baltimore intercepting sewer system.

in order not to force the sewers so low that it would be difficult to connect the houses with them. In one low-lying district where the drainage problem was particularly difficult, the plans called for raising the street grade and building a drain to carry the storm water into the Patapsco River instead of a nearer stream which was liable to have its surface raised considerably during floods, a condition which might cause a surcharge of the drains emptying into it.

The removal of the house sewage was a much more complicated problem. Part of it comes from districts which are high enough to enable the sewage to flow by gravity to the treatment works, but a large part has to be pumped. The dividing contour line between these two service districts was determined by two factors, the elevation at which the sewage must be discharged at the treatment works and the minimum safe grade of the outfall sewer from the city to the works. The accompanying plan of the intercepting sewers, Fig. 2, from *Engineering Record*, Dec. 5, 1908, shows where the outfall sewer reaches the eastern boundary of the city and is continued through it toward the western boundary as a high-level intercepter, receiving all the sewage that can be delivered by gravity to the disposal works. The sewage of the low-lying portions of the city is collected by four intercepting sewers, two of which contain small pumping plants to lift the sewage enough to prevent a deep position of these sewers, which is undesirable on account of the high cost of construction in deep trenches in water-bearing soil, and the difficulty of connecting the tributary sewers satisfactorily with deep-lying intercepters. All these intercepters run to a station containing five pumps, each with a nominal rating of 27,500,000 gal. a day against a total head of 72 ft. These pumps force the sewage through two lines of 42-in. cast-iron mains 4,550 ft. long into a sewer about a mile long, discharging by gravity into the main outfall sewer.

Between the arrangement of sewers in the Borough of Manhattan, discharging both storm water and house sewage through short lines into the nearby rivers, through many outlets, and the arrangement at Baltimore, with its separation of the storm water and the sewage, its high and low levels, pumping stations, long outfall sewer and elaborate sewage treatment works, there is an infinite variety of combinations practicable. In every case, however, the topography suggests the natural drainage and the street plan exercises a more or less strong modifying influence. Special attention should be paid to the low-lying districts, for it is there that the largest sewers must be built in many cases, and the difficulties of construction are the greatest. It may be found advisable to reduce such work to a minimum by constructing an intercepting sewer at a somewhat higher level and thus restrict the construction in the low-lying sections to small sewers only deep enough to serve the property of those districts.

Another influence of topography on sewerage plans, often overlooked, was stated as follows by Dr. Hering in his report of 1881 to the National Board of Health:

"In case of sudden showers on a greatly inclined surface which changes to a level below, the sewers on the latter will become unduly charged, because a greater percentage flows off from a steeper slope in a certain time. To avoid this uneven reception, the alignment should, as much as possible, be so arranged as to prevent heavy grades on the sloping surface, at the expense of light ones on the levels. In other words, the velocity should be equalized as much as possible in the two districts. This will retain the water on the slopes and increase its discharge from the flat grounds, thus corresponding more to the conditions implied by the ordinary way of calculating the capacity of sewers. It will therefore become necessary not to select the shortest line to the low ground, but, like a railroad descending a hill, a longer distance to be governed by the gradient. This does not necessarily imply a longer length of sewers for the town, because more than one sewer for a street is not required by it."

Still another decided influence of topography is shown where the configuration and surroundings of the city are such that it is advisable to employ combined sewers in all parts of the city down to the lowest contour line which will permit storm-water overflows to be used. This is the rule adopted by E. J. Fort for the new sewerage works of Brooklyn. Below this contour line, the combined sewers become storm-water drains, run at a higher level than the house sewers, so as to have a free outlet to tide water, and the house sewage of the low districts is pumped to points of disposal.

After the most favorable location of the sewers has been determined from an exclusive sewerage viewpoint, the desirability of minor changes of position to avoid needless interference with travel through busy streets should receive attention. The construction of a sewer in a narrow or crowded street costs the community a considerable sum in indirect damages and directly affects those having places of business on the streets.

Sewage Disposal and Treatment.—Having collected the sewage by a sewer system, it becomes necessary to dispose of it in some way which will not cause objectionable conditions. In connection with such work certain terms are used frequently, and accordingly their definitions, as recommended by the Committee on Sewerage and Sewage Disposal of the American Public Health Association, are given here:

"*Sewage disposal* is a generic term applied to the art of disposing of sewage by any method.

"*Sewage treatment*[1] is the process to which sewage is subjected in order to partially remove its impurities or so to change them as to render the effluent fit for final discharge.

"*Contamination* is the introduction into a water of bacteria or other substances which tend to render it unsuitable for domestic use.

[1] The term "sewage purification" should be abandoned.

"*Pollution* is the introduction into a water of substances of such character and in such quantity that they tend to render the body of water or river objectionable in appearance, or to cause it to give off objectionable odors.

"*Influent* is sewage, or partially treated sewage, flowing into any sewage treatment device.

Effluent is partially or completely treated sewage flowing out of any sewage treatment device.

"*Putrescibility* is the susceptibility of sewage, effluent, or wet sludge, to putrefaction under the conditions to which it is subjected.

"*Stability* is the capability of sewage or effluent to resist putrefaction under the conditions to which it is subjected.

"*Relative stability* is the ratio of available oxygen to the oxygen required to prevent putrefaction, expressed in per cent.

"*Suspended solids* are those which are removed from sewage or effluent by standard laboratory methods of filtration.

"*Settling solids* are those suspended matters which will subside in quiescent sewage in 2 hr.

"*Clarification* is a relative term denoting the partial removal of suspended and colloidal matter by straining, sedimentation, or by coagulation and sedimentation.

"*Colloidal matter* (conveniently defined) is the suspended matter which is so finely divided that it will not subside in 2 hr., yet will not pass through a parchment membrane in the ordinary process of dialysis.

"*Screens.*—A screen is a device containing openings of proper size to retain a part of the suspended matter of sewage.

"*A coarse screen* is one having openings in excess of 1 in. in least dimension.

"*A fine screen* is one having openings of ¼ in., or less, in least dimension.

"*A medium screen* is one having openings intermediate between a coarse screen and a fine screen.

"*A bar screen* is one composed of parallel bars or rods.

"*A mesh screen* is one composed of a fabric, usually wire.

"*A grating* consists of two sets of parallel bars in the same plane, the sets intersecting at right angles.

"*A band screen* is one consisting of an endless band or belt of wire mesh or other screening material which passes over upper and lower rollers.

"*A wing screen* is one having radial or curved vanes, usually composed of uniformly spaced bars, which rotate on a horizontal axis.

"*A drum screen* is one in the form of a cylinder or cone, consisting of perforated plates or a wire mesh which rotates on an approximately horizontal axis.

"*A disc screen* consists of a rotating circular perforated disc, with or without a concentric truncated cone of similar material.

"*A cage screen* consists of a cage, usually with sides of parallel bars or rods, so arranged that it may be lowered into the sewage and raised therefrom for cleaning.

"*Tank treatment* is the detention of sewage or sewage sludge in tanks, either quiescent or with continuous flow.

"*A grit chamber* is a chamber or enlarged channel in which the cross-section is so designed that the velocity is such that only heavy solids, such as grit and sand, are deposited, while the lighter organic solids are carried forward in suspension.

"*A sedimentation tank* is a tank for the partial removal of suspended matter either by quiescent settlement or by continuous flow at such velocity and time of retention as to allow deposition of suspended matter.

"*A Dortmund tank* is a vertical sedimentation tank, usually cylindrical above and with a conical or hopper shaped bottom, into which the sewage or partially treated sewage is introduced near the center and after rising through the tank passes out at the surface—the sludge being drawn off, without emptying the tank, before it becomes septic.

"*A hydrolytic tank* is a sedimentation tank in which by biological processes a portion of the suspended matter is converted into liquid and gaseous form.

"*A septic tank* is a horizontal, continuous-flow, one-story hydrolytic tank in which the suspended matter is retained until anaerobic decomposition is to a considerable extent effected.

"*A Travis tank* is a two-story hydrolytic tank consisting of an upper or sedimentation chamber, with steeply sloping bottom, terminating in one or more slots through which the solids may slide as deposited into the lower or sludge digestion chamber, through which a predetermined portion of the sewage is allowed to pass for the purpose of seeding and maintaining bacterial life in the sludge and carrying away decomposition products, thus inducing digestion of the sludge attended by its reduction in volume.

"*An Imhoff tank* is a two-story hydrolytic tank, consisting of an upper or sedimentation chamber, with steeply sloping bottom, terminating in one or more slots through which the solids may slide as deposited into the lower or sludge digestion chamber—these slots being trapped so as to prevent the rise of gas and solids from the lower chamber—the lower chamber being provided with vents for the escape of the gases, the tank being so constructed as to facilitate the passage of the sewage quickly through the upper chamber and prevent the flow of sewage through the digestion chamber, and intended to be so operated that the sludge may be thoroughly decomposed, rendered practically free from offensive odor and so filled with gas that it can be readily drawn off and dried.

"*Activated sludge process* is the agitation of a mixture of sewage with about 15 per cent or more of its volume of biologically active liquid sludge in the presence of ample atmospheric oxygen, for a sufficient period of time at least to coagulate a large proportion of the colloidal substances, followed by sedimentation adequate for the subsidence of the sludge flocculi; the activated sludge having been previously produced by aeration of successive portions of sewage and maintained in its active condition by adequate aeration by itself or in contact with sewage.

"*Chemical precipitation* is the addition to, and thorough mixing with, the sewage of such chemicals as will, by reaction with each other or with the ingredients of the sewage, produce a flocculent precipitate; and subsequent sedimentation.

"*Sludge* is the suspended solids of the sewage deposited in tanks or intercepted at the surface of filters, mixed with more or less water.

"*Sludge digestion* is the biological process by which organic matter in sludge is gasified, liquified, mineralized, or converted into stable organic matter.

"*Separate sludge digestion* is the digestion of sludge in tanks entirely independent from the tanks in which it is produced.

"*A sludge drying bed* is a natural or artificial layer of porous material upon which sludge is dried by drainage and evaporation.

"*Sludge concentration* is the process of reducing the volume of sludge, and increasing its proportion of solids by allowing it to stand in a suitable tank until the solids settle down, and drawing off the relatively clean water at the top.

"*Sludge drying* is the process of drying sludge by drainage and evaporation.

"*Sludge dewatering* is the process of removing a portion of the water contained in sludge by draining, pressing, centrifuging, or by other natural or mechanical processes.

"*Sludge pressing* is the process of dewatering by subjection to pressure, the solids being retained by a cloth fabric which permits the water to pass through it.

"*Sludge cake* is the mass of dewatered sludge resulting from sludge pressing.

"*Commercially dry sludge* is sludge containing not more than 10 per cent of water.

"*Scum* is a mass of sewage solids, buoyed up in part by entrained gas or grease, and which, consequently, floats at the surface of the water.

"*Screenings* constitute the material removed from sewage by screens.

"*Grit* is the heavy mineral matter deposited from sewage.

"*Sleek* is the oily film of microscopic thickness present on the surface of waters about and often extending a considerable distance from sewer outlets.

"*Sewage oxidation* is the process whereby through the agency of living organisms in the presence of air, the organic matter is converted to a more stable condition or into mineral matter.

"*Irrigation* is the process of sewage treatment in which the sewage is applied to land for the primary purpose of purifying the sewage and the secondary purposes of supplying water and fertilizer to crops.

"*Surface irrigation* is the process in which sewage is applied to and distributed over the surface of cropped ground.

"*Sub-surface irrigation* is the process in which sewage is distributed beneath the surface of the ground by means of open-jointed pipes.

"*An intermittent filter* is a natural or artificial bed of sand or other fine-grained material to which sewage is intermittently applied in doses, and which, by its capillarity, holds the sewage for a time sufficiently long, in the presence of air, to effect by biological processes a high degree of purification.

"*A contact bed* is a watertight basin filled with coarse material, such as broken stone, in contact with which the sewage is held for a time by control of the underdrains—the cycle of operation involving periods of filling, standing full, emptying and resting empty; so regulated as to secure such contact with the bacterial films adhering to the surface of the coarse material, and such aeration of the bacterial surfaces, as may be required to oxidize the sewage.

"*A slate bed* is a watertight tank filled with slabs of slate or other similar material, laid horizontally and spaced an inch or more apart vertically, equipped so that it may be filled with sewage, allowed to stand full for a definite period of time, drained and allowed to stand empty for a time, for the purpose of oxidizing the organic matter deposited upon and adhering to the slates.

"*A trickling filter*[1] is an artificial bed of coarse material, such as crushed stone or clinkers, over which the sewage is distributed as a spray from fixed nozzles or as a film from moving distributors, through which it trickles to the underdrain system, coming in contact with the bacterial films adhering to the surface of the stones, and in which such aeration of the bacterial surfaces as may be required to oxidize the sewage is afforded.

"*A dosing apparatus* is the apparatus used for regulating the application of sewage to filters or for applying the required quantity of chemicals to sewage.

"*A dosing tank* is a tank into which raw or partially treated sewage is introduced and held until the desired quantity has been accumulated, and then discharged at such a rate as is necessary for the distribution essential to the subsequent treatment.

"*A sprinkler nozzle* is a nozzle used for applying sewage in the form of a spray to trickling filters.

"*A distributor* is a movable perforated pipe, channel or waterwheel which distributes sewage upon the surface of a trickling filter. There are two types of distributors—the rotary and the traveling; the rotary moves about a central axis with delivery to a circular filter; the traveling moves back and forth the length of a rectangular filter.

"*Disinfection* is the destruction, by the agency of some chemical, of a large percentage of the bacteria in sewage or contaminated water, so as to materially reduce the danger of infection.

"*Sterilization* is the destruction, by the agency of some chemical, of all the bacteria in sewage or contaminated water, including their spores."

Why Sewage Treatment is Necessary at Times.—So far as rainfall run-off is concerned, it is only when its quantity is large that the disposal of it presents any difficulty, for there are usually opportunities for discharging unpolluted water into bodies of water or on land in the neighborhood. The main difficulty in sewage disposal is due to the substances present in sewage and in the portion of the street wash which flows away during the early part of a rain. While these deleterious organic substances are very small in amount—generally less than one-tenth of 1 per cent, the remaining 99.9 per cent of the sewage

[1] The committee is of the opinion that the term "trickling filter" is preferable to the terms "sprinkling filter" or "percolating filter" which are frequently used as synonymous terms.

being water—their successful handling constitutes the essential problem in sewage disposal.

The part of the street wash which enters the sewers and drains after the refuse on the pavements has been carried off is relatively much less polluted, and on this account it can often be discharged into waters or over land unfitted to receive seriously polluted water. Where the streams into which sewage must be discharged are small and it is very necessary to keep them clean, as is generally the case in Great Britain, the chief objection to the discharge of storm water into them is due to the suspended solids carried by the flowing water. These may form undesirable deposits on the banks and the bed of a stream and the refuse floating on the surface may give an objectionable appearance to the water. Consequently it is the general engineering rule in Great Britain that in places where the dry-weather sewage must be treated before it is discharged into a body of water, the storm water must also be treated before discharge into the same body of water, until its volume amounts to about three times the normal flow of dry-weather sewage. It is not unlikely that some American cities with separate sewers will later be faced with the necessity of installing works for removing part of the suspended matter from the storm water now discharged without treatment into neighboring rivers and lakes.

The necessity for a large part of the treatment which sewage should receive before its final disposal is due to the organic matter it contains. Under normal conditions, this changes fairly rapidly into other substances, some of them offensive, and the change is accompanied by a rapid growth of bacteria, some species of which may be objectionable in the water receiving the sewage. The purpose in treating sewage is to remove the solids which cause deposits in the rivers and ponds, as well as undesirable surface conditions, and to carry out certain changes in the organic matter, under the control of the operator of the treatment works, until the final effluent from the works has its organic matter in such a state that any further changes which it may undergo can proceed in the river or lake without detriment to the quality of the water for the purposes for which it is used.

There are many different methods of treatment, such as dilution in a large body of water, removing the solids by screens or by sedimentation in small or large tanks, allowing the sewage or the organic solid matter settling from it to remain in tanks under conditions which will cause various changes in it, changing the character of sewage by aeration, and filtering the sewage or the effluent from some previous form of treatment. In each of these methods of treatment a certain minimum fall through the tanks or filters and the outfall sewer is necessary and must be provided, naturally or artificially by pumping, and consequently the method of treatment and disposal of sewage affects not only the direction from the city which the outfall sewer or sewers must take but also the slopes or grades of such sewers.

Influence of Method of Disposal and Treatment upon General Sewerage Plan.—There are three general methods of disposal that affect the general design of a sewerage system.

The first of these disposal methods is directly into a river or other body of water on the shore of which the city lies. Probably the Borough of Manhattan, New York City, offers the best example of this among large cities, with its numerous main sewers running east and west to outlets at the Hudson and East Rivers.

The second method of disposal is to intercept the sewage and carry it to a point in the adjoining body of water where it will not cause trouble; this may not be necessary at first, but in most cases it is inevitable if the city grows as rapidly as do most American municipalities, and attention must be paid to it, particularly to the future desirability of separating the house and industrial sewage from part of the storm water. A proposed Cleveland system, shown in Fig. 3, from *Engineering News*, March 28, 1912, is an example of this intercepting plan.

The third method of disposal is by some treatment of the sewage which will materially change its character before it is discharged into a body of water. This makes it necessary to deliver the sewage to treatment works, suitable sites for which are difficult to procure in many cases, particularly where the country is well built up, not enough open land properly located is available in the city, and neighboring towns object to the plant being located within their limits. The separation of storm water from the house sewage often becomes financially advisable, so as to permit the former to be discharged by short, direct lines into the river, lake or bay nearby, and to keep down the cost of the long sewer to the disposal works, and the disposal costs as well. In the case of combined sewers, the same end is attained by making provision at one or more points for the discharge of the storm water in excess of a predetermined amount, through overflow weirs or chambers into channels or other outlets leading directly to the river or lake.

In cities situated on rivers it is customary to convey the sewage to a point below the thickly settled district, whether the sewage is to be discharged in its raw state or is to be treated by some artificial means before discharge. This plan avoids the danger of causing objectionable conditions along the water front and, perhaps, the contamination of the local water supply by the discharge of sewage into the river. Similarly, cities situated on the Great Lakes may convey their sewage in the direction of the trend of the waters to a point well beyond the populous districts. This policy is not so well established nor so effective as in the river cities, for the lacustrine current is not so marked as most river currents and it may be altered temporarily by adverse winds. Cities situated on tidal waters often build long intercepting sewers to convey the sewage to a favorable location for discharge.

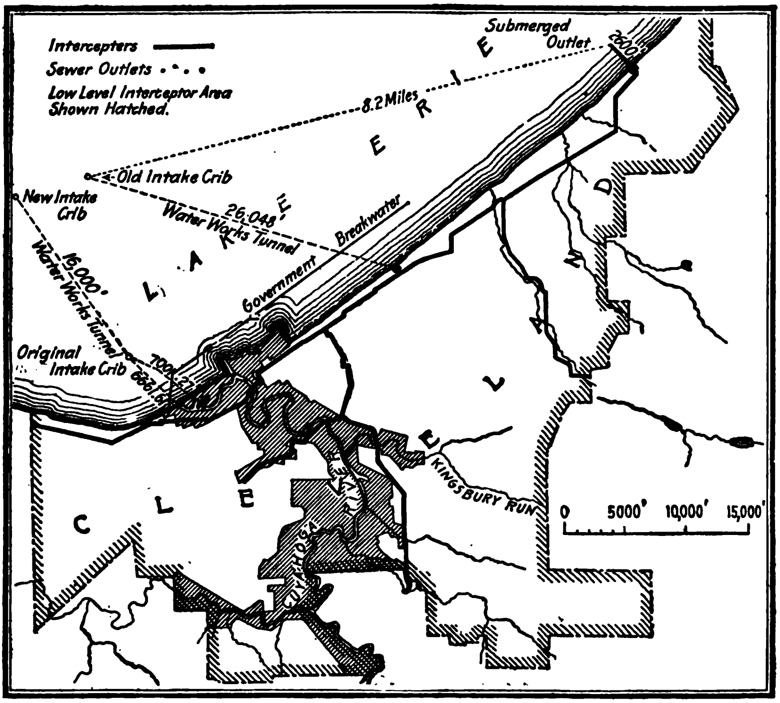


FIG. 3.—Cleveland intercepting sewer system.

Each method of treatment has certain features materially affecting if not actually determining the location at which it can be practiced. Intermittent sand filtration requires large areas of suitable sand. Rapid filtration through deep beds of coarse aggregates requires more head than other methods involve. Aeration requires a power plant to supply compressed air. Some tank treatments are accompanied by offensive odors. These conditions, explained in later chapters, make it particularly desirable to study the practicability of discharging sewage at several nearby points, after suitable treatment to fit it for the conditions at each outlet, rather than the delivery of all of the sewage through a long, expensive outfall sewer to a single treatment plant. It should be understood at the beginning of any study of sewage disposal that the purpose of treatment is to protect the water or land receiving the sewage. Changing the character of the sewage merely for the sake of making it less offensive or dangerous is a waste of money unless it is necessary, just as it is a waste of money to carry the sewage farther than the local conditions in each case require. It should also be understood that where the conditions warrant treatment by dilution, which means the discharge of sewage into large bodies of water, that method of treatment is not only the cheapest in first cost in most cases but is just as well established as a truly scientific process as the most elaborate artificial treatment by sedimentation, aeration, filtration and sterilization, needed by a city not favored by Nature with conditions admitting the use of less expensive methods.

Municipal Liability for Inadequate Sewers.—There is a quite general belief that a city is never liable for damages caused by the inadequate capacity of sewers. As a matter of fact, a city may be liable for such damages under certain conditions, and even where there is no legal liability a grave moral responsibility rests upon the engineer who designs a sewerage system. The present legal aspect of the subject places the engineer upon a strictly professional plane like that of a physician or lawyer, and it is one of the few instances in which the courts have recognized the professional nature of engineering.

It has been held that if a city has a collection of sewers and it cannot prove that this collection was built or rebuilt in accordance with a comprehensive plan prepared by an engineer presumably competent to design such works, the city is liable for any damages which may result from the inadequate capacity of the sewers.[1] This makes it necessary for the city to employ competent engineers to design its sewers and to approve the sewers in any additions incorporated within the municipal boundaries from time to time.

In some cases the courts have held that if a city employs a presumably competent engineer to design its sewerage system and the construction proceeds in conformity with his plan and subsequent modifications of it by presumably competent engineers, it is not liable for damages due to a fault in the plan. In Massachusetts, it is not clear that the sewer system must be designed by an engineer, the law being stated[2] as follows, "Certain governing principles of law are well established. A municipality is not responsible for any defect or want of efficiency in a general plan for drainage adopted by a board of public officers, even though private individuals may be exposed to great inconvenience and loss." The city is liable, however, for damages due to the known lack of repair of sewers or its failure to maintain them to the standard of efficiency of the original construction.

The damages caused by inadequate sewers are generally due to an insufficient estimate of the quantity of sewage for which provision must be made, particularly where the combined system is used. Too large capacity not only causes needless expense but develops a tendency for the sewage to flow so slowly and in such shallow streams that part of the suspended solids will settle on the inverts, forming offensive deposits. If the sewers are too small, cellars may be flooded with sewage, which may even rise from manholes and gutter inlets and flow over the surface of the ground, resulting in damage and litigation. The subject is, in some respects, the most important in the whole field of sewerage engineering, one calling for sound judgment as well as knowledge.

[1] A typical ruling of this sort was made in the case of Hart *vs.*, City of Neillsville, 104 N. W. Rep. 699, where the Wisconsin Supreme Court made the decision.

[2] Diamond *vs.* North Attleborough, 219 Mass. 590; see, also, 4 N.E. Rep. 321 (New York Court of Appeals); 61 Atl. Rep. 180 (Maine Supreme Court).

CHAPTER II

QUANTITY OF SEWAGE

In this chapter the term "sewage" will be restricted to the liquids which are ordinarily discharged by *separate sewers*, comprising (1) domestic and industrial wastes derived primarily from the public water supply, (2) industrial wastes derived primarily from privately owned water supplies, and (3) infiltration.

The main sewers for removing this sewage must be designed to meet future rather than present conditions. The ideal plan is one which will provide satisfactory sewerage facilities during such a period of years that the average annual cost of this service shall be a minimum. Smaller sewers than the economic size for the first part of this period and additional sewers for the last part might give equally satisfactory service, but usually at a higher cost. Larger sewers than the economic limit would not only cost more, but, on account of the small proportion of their capacity utilized during the early years and the consequent low velocities of flow and resulting probability of forming deposits on the flat grades, they might prove less satisfactory during the early part of their service.

In determining the period for which sewerage facilities must be provided, assumptions must be made regarding a number of future conditions, such as changes in the rate of interest which a city must pay for money borrowed to build the sewers, increase in the population, incorporation of additional territory within the city, development of industries, and changes in the habits of the residents in respect to the use of water. The fluctuations in rates of interest fall within the field of economics, but the other influences lie within the field of the engineer who must rely mainly on his own knowledge and judgment in reaching decisions, after careful study and analysis of them. The assumptions made may have a serious future result, as when the volume of sewage is so underestimated that either the main trunk sewer shall prove inadequate or the treatment plant to which it runs, be outgrown in a few years and be impossible of adequate enlargement, on account of lack of available land. In such case, the expense of intercepting part or all of the sewage and taking it to some other place of disposal may add greatly not only to the total cost of the works necessary for satisfactory service, but also to the annual cost of their operation and maintenance. Sound judgment in such matters can only come from observation and study of the actual growth and development of individual cities and the reasons therefor, somewhat along the lines conducted by the traffic departments of some railways and the economists of some large banks.

Population.—The increase in the population of a city is sometimes affected by exceptional conditions, which may be temporary, as was the case in Washington during 1917 and 1918, or permanent, as where a new industry develops phenomenal growth, like the rubber industry in Akron or the automobile industry in Detroit. Sometimes, as at Gary, Ind., practically a new city develops in a few years about a great industry. But in the average case, the past records of growth of a city and the records of the growth of similar places furnish fairly reliable guides for estimates of future development for periods of 25 to 40 years.

The various available records are used in four ways, according to the assumptions made regarding the rate with which the population increases. These four assumptions are:

1. *Assumed Uniform Percentage Rate of Growth.*—When cities are young and thriving, it often happens that their rate of growth is high for some years, but an assumption that this uniform percentage rate will persist for a long period will certainly lead to an over-estimate of the population in most cases. This is clearly shown in Fig. 4, where the line *A* indicates the increase in population which would have taken place had the growth of the cities been 31 per cent per 10 years, while the lines marked "Average of 6 Cities" and "Average of 4 Cities" show what it actually was. The assumption of a uniform percentage rate of increase is apparently most reliable in the case of old, large cities not subject to periods of great commercial or industrial activity.

2. *Assumed Curvilinear Rate of Growth.*—The information furnished by diagrams of the past growth of cities is very instructive, but an attempt to predict the future growth of a city from its past development alone, by extending the curve of that

development, is likely to give misleading results, as will be shown later. Diagrams have a useful place in the study of changes in population, but they are not a substitute for an investigation of the various influences which have affected the city's growth in the past and may affect it in the future.

3. *Assumed Arithmetical Rate of Increase.*—Occasionally it is assumed that a city will gain approximately the same number of inhabitants annually, leading to a straight-line increase of population like that shown by line *B*, Fig. 4. This assumption may be warranted only in the case of a few large cities with good

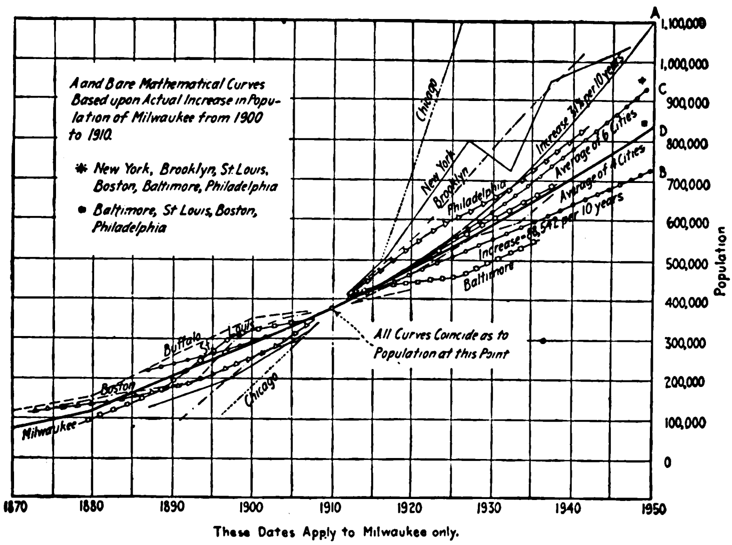


Fig. 4.—Growth of large American cities.

transportation facilities into suburban towns, which result in many persons doing business in the large city being enumerated as suburban residents. As their business life is spent in the large city, they must be classed for purposes of water supply and sewerage as inhabitants of both places, a fact often overlooked in engineering discussions of population. Commercial districts often require large sewerage capacity, yet the census taker will report a very small population in them. Small cities without industries, which are the commercial centers of a prosperous agricultural region, occasionally show a tendency toward this rate of growth, but if they become producers as well as traders, through the development of manufacturing, they are likely to grow for a time more rapidly than the assumption of arithmetical progression will indicate.

4. *Assumed Decreased Rate of Growth.*—As a general rule it is found that the larger a city becomes, the smaller will be its percentage rate of growth from year to year. Fig. 4 shows this clearly. The growth of Chicago has been abnormal and that of New York (Manhattan) and Brooklyn has been influenced by unusual conditions, so that these cities are not reliable guides

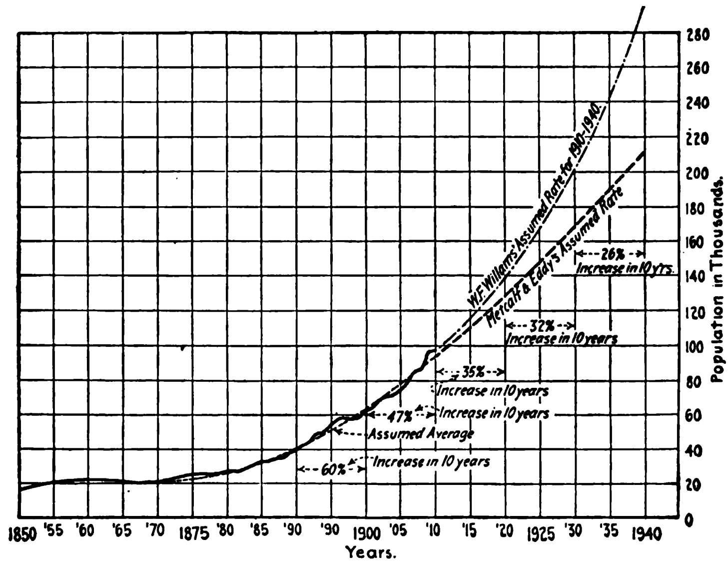


FIG. 5.—Growth of population of New Bedford.

in general investigations of changes in population. There is not only decrease in the percentage rate of growth of cities as they grow in size, but there is also a general decrease in the percentage rate of growth of the entire population of the country. As a rule, this assumption is the most reliable of the mathematical methods of estimating future populations, particularly if it is checked by basing the prediction on the experience of comparable cities which have already passed the present population of the city under consideration. This is done, as shown in Fig. 4, by arranging the lines indicating the change in population of different cities so that when they reach the present population of the city under consideration, they all pass through the same point. This method will tend to give slightly too high a percentage rate of increase, since it does not allow for the average progressive decrease throughout the country as the years go by.

It is interesting to note in connection with Fig. 5, showing the growth of population in New Bedford, that this diagram, prepared in 1911 to accompany a report by the authors, shows a forecast for the population of 1920 of about 128,000. The 1920 U. S. Census gives a population of 121,217. Thus the forecast for a nine-year period was within about 5 per cent of the actual conditions realized.

Increase in Area.—The tendency of many cities to an increase of area by the absorption of suburbs is shown by the growth of Cincinnati since it was incorporated as a city in 1819:

Year	1819	1850	1870	1890	1900	1910	1913
Area, square miles	3.00	6.16	19.05	23.73	35.27	50.26	69.85

Such enlargements of area may cause large and sudden increases in population, which if not anticipated, may result in overtaxing intercepters within the period for which they were intended to be adequate.

Increases in area may also require long extensions of main sewers and may result in greatly increased quantities of ground water made tributary to the intercepters. Where the community is served by combined sewers, there is also the probability that for considerable periods in the future, or until the population becomes quite dense, brooks will be turned into the main sewers, thus adding materially to the nominal dry-weather flow of sewage. It is also of vital importance to consider where the estimated increase in population will occur in order that the lower sections of the intercepter may be placed at elevations from which it will be possible to make extensions into new territory that may become populated within the period for which the intercepter is designed.

Density of Population.—An idea of the density of population, in persons per acre, in American cities is given by the following figures showing a range in 1910 of from 85 in Hoboken, N. J., downward:

Persons per Acre	Places
85	Hoboken
29	Baltimore
27	Boston
26	New York
25	Milwaukee
23	Newark
20	Pittsburgh
19	Cleveland, Philadelphia, Chicago, Harrisburg, Providence
18	Detroit
17	St. Louis, Buffalo, Louisville, Rochester
16	Savannah
14	San Francisco, Columbus, Albany.

The density within a city varies greatly, and it is difficult to estimate its probable future changes. A residential section of the present decade may become a commercial or manufacturing district in the next decade, or the detached homes of persons of means may be replaced by crowded tenements or commercial developments. As estimates of the future population in the different parts of a city are part of the basic data upon which sewerage works are designed, and they must be determined by the exercise of sound judgment, the importance of studying present tendencies toward growth or stagnation, and their causes, is evident.

The difference in policy of our cities, with respect to the extension of their boundaries, has been very marked. In our Eastern communities the city limits have usually been rather more restricted than in our western cities. This tendency has led to rather dense population in the Eastern cities and sparse population in Western cities of like size. Thus, for example, we find from the 1915 U. S. Census Bureau, *Bulletin* 133, "Estimates of Population," with respect to cities having a population of 250,000, more or less, the facts given in Table 1.

Such conditions must be noted. The scattered population involves greatly increased cost of the public utilities or services of various kinds and far greater difficulty of accurate forecast of the probable future direction and degree of development and of increase in population.

Proportion of Municipal Water Supply Reaching Sewers.—As the wastes which form sewage are mainly water from the water works supplying the city, the proportion of the water supply which will reach the sewers must be estimated. A considerable

TABLE 1.—POPULATION AND AREA OF CERTAIN AMERICAN CITIES IN 1915

City	Population	Land area in acres	Acres per 1,000 of population	Density of population per acre
Oakland, Cal.	190,803	31,591.0	165.5	6.0
Denver, Col.	253,161	37,028.0	146.3	6.8
St. Paul, Minn.	241,999	33,388.0	137.9	7.3
Kansas City, Mo.	289,879	37,555.8	129.6	7.7
Portland, Ore.	272,833	32,748.8	120.1	8.3
Seattle, Wash.	330,834	37,481.0	113.3	8.8
Toledo, Ohio	187,840	16,025.6	85.3	11.7
Indianapolis, Ind.	265,578	22,165.1	83.4	12.0
Columbus, Ohio	209,722	14,149.6	67.5	14.8
Louisville, Ky.	237,012	15,368.4	64.8	15.4
Rochester, N. Y.	250,747	14,876.3	59.4	16.9
Providence, R. I.	250,025	11,353.0	45.4	22.0
Jersey City, N. J.	300,133	8,320.0	27.7	36.1
Average	252,351	24,003.9	95.9	13.3

part of the water supply used by railroads, by manufacturing establishments and power plants, in street and lawn sprinkling, in extinguishing fires, and by consumers not connected with sewers, does not reach the sewers. There is also some leakage from the water mains and service pipes which does not reach the sewers. In 1911 the Milwaukee Sewage Disposal Commission estimated that of the total daily supply of 105 gal. per capita 40 gal., or 38 per cent of the supply, never reached the sewers. This 40 gal. included:

	Gallons
Steam railroads	5
Industrial uses	5
Street sprinkling	5
Lawn sprinkling	2½
Consumers not connected with sewers	7½
Leakage from water mains and services	15

In a few cities where the infiltration of ground water and the discharge of roof water into sewers is low, the quantity of sewage is actually less than the quantity of water supplied, as at Brockton, Mass., where the sewage has ranged from 56 to 73 per cent of the water supply.

Often, however, infiltration, roof water, and water used in industries and obtained from privately owned sources, make the quantity of sewage larger than that of the public water supply. This is shown by the ratio of sewage to water supply in the following places:

Locality	Water to Sewage
Massachusetts North Metropolitan Sewerage District	1 to from 1.16 to 1.29
Worcester, Mass.	1 to from 1.09 to 1.73
Quincy, Mass.	1 to from 1.05 to 1.43
Providence, R. I.	1 to from 1.14 to 1.56

With well-built sewers and with roof water excluded from the sewage, the variation from year to year in the ratio of sewage to water supply in a city is not great, unless there is a substantial change in the industrial uses of water. For this reason, the consumption of water in a city affords basic data for sewerage as well as water-supply engineering.

Water Consumption in Cities.—The consumption of water in cities is usually expressed in gallons per capita daily. A comparison of such records from different cities is likely to be very misleading because in some cities large quantities of water used industrially are obtained from privately owned supplies, as at Fall River, and in other cities, as Philadelphia and Buffalo, the industries use the municipal supply mainly. Furthermore, the care taken to reduce the waste of water through leaks in mains, services and plumbing has a decided effect on the per capita consumption. Waste is prevented mainly by metering the consumers' supplies, which tends to reduce them as shown in Fig. 6, and by inspection of the condition of the piping and plumbing. The waste and unaccounted-for water may range in practice from 20 to 40 per cent, more or less, of the total water entering the supply-pipe system. In any particular case, therefore, it is important to ascertain the present practice in preventing waste of water, the probability that more attention will be paid to such work, and the probable development of heavy industrial uses. In such studies valuable aid can be obtained from a report[1] on water consumption by Metcalf, Gifford and Sullivan.

From the statistics available two conclusions seem warranted: First, there is a tendency toward a gradual increase in the quan-

[1] *Jour.* N. E. Water Works Assoc., March, 1913.

tity of water used per capita of population. This is undoubtedly due, so far as it relates to domestic uses, to more elaborate plumbing. The number of fixtures per person, as well as the quantity of water required per fixture, has greatly increased in recent years. In the larger cities, the increased consumption may be due in part to the difficulties surrounding the management of the water departments, which are usually much greater than in the smaller cities and towns. Second, the evidence furnished by such cities as Providence, Worcester, Fall River and Lawrence, indicates that with careful management, aided, perhaps, by thorough metering, it is possible to hold down the increase in quantity of water consumed to reasonable proportions.

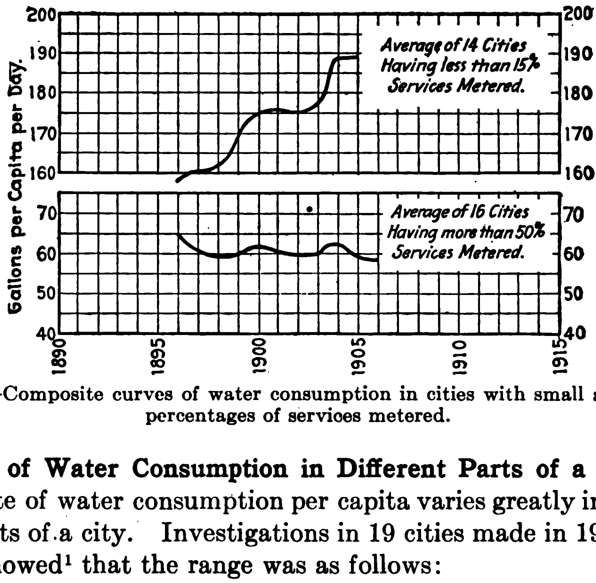


FIG. 6.—Composite curves of water consumption in cities with small and large percentages of services metered.

Rate of Water Consumption in Different Parts of a City.—The rate of water consumption per capita varies greatly in different parts of a city. Investigations in 19 cities made in 1910 and 1911 showed[1] that the range was as follows:

	Water Consumption in gal. per Day per Capita	
CLASS OF BUILDINGS	RANGE	AVERAGE
Apartment houses	24 to 135	62
First-class dwellings	15 to 75	54
Middle-class dwellings	11 to 66	34
Lowest-class dwellings	4 to 37	15

The industrial uses vary greatly, according to the nature of the manufacturing. Fuertes reported[2] in 1906 that the range

[1] *Jour.* N. E. Water Works Assoc., March, 1913.
[2] Waste of Water in New York.

in 11 cities was from 0.4 gal. per capita daily in Wellesley, Mass., to 81 gal. in Harrisburg, Pa. It was 30 gal. in Boston, 40 gal. in Cleveland and 45 gal. in Milwaukee. These figures are based on the entire population of the city, but as manufacturing is carried on only in certain districts, as a rule, the per capita consumption for manufacturing in those districts is much higher than the average for the city. In practical designing work it is very desirable to make an actual inspection of the industries and a careful estimate of the quantities of water they use from all sources. The same is true of the consumption in business districts.

Fluctuations in Water Consumption.—While it is important to know the average quantity of water consumption, it is of still greater value to have data relating to the fluctuations, as a sewer must be designed to take the sewage when flowing at its maximum rate. The maximum rate of water consumption usually occurs during summer months when water is in demand for street and lawn sprinkling and the excess is not likely to reach the sewers, or in the winter when large quantities are allowed to run to prevent freezing of pipes and fixtures, this excess usually finding its way into the sewers. Records of maximum water consumption for 67 Massachusetts cities and towns (1910) have been compiled by a Committee on Water Consumption Statistics and Records.[1] The average figures are as follows:

Average water consumption, gallons per capita per day	63
Daily average in maximum month, percentage of average for year	128
Daily average in maximum week, percentage of average for year	147
Maximum consumption in one day, percentage of average for year	198

There were, however, instances in which the maximum rates greatly exceeded these averages. For example, in Manchester and Mansfield, Mass., the maximum daily consumption was 302 and 368 per cent of the average for the year, respectively. These high rates of consumption, however, almost always occur at times when the usual proportion of the flow does not reach the sewers, as in the driest portion of the summer, or in winter when water from other sources, as for example, ground water, is likely to be at a minimum.

In addition to the fluctuations in flow already discussed, there is an important variation from hour to hour each day, as illus-

[1] *Jour.* N. E. Water Works Assoc., March, 1913.

trated by Fig. 7 and 8 taken from the same report. It will be seen from Fig. 8 that the maximum peak flow during the week

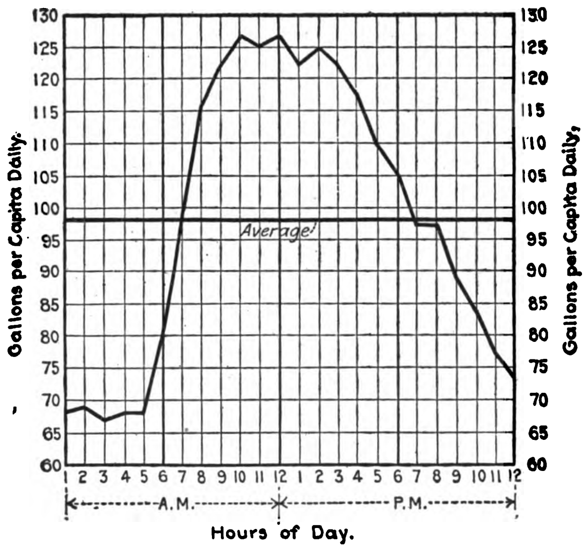


Fig. 7.—Hourly water consumption for average day in Holyoke in November, 1905.
Estimated population supplied, 51,000.

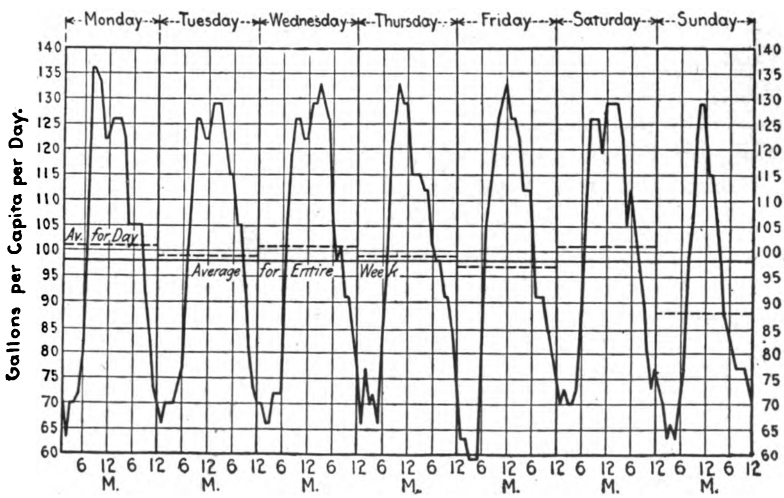


Fig. 8.—Fluctuations in water consumption in Holyoke during week ending November 17, 1905.

occurred on Monday, when the draft was about 135 per cent of the average for the day, and the minimum peak draft was on

Sunday when it was 146 per cent of the average for the day, these rates being 139 per cent, and 132 per cent, respectively, of the average rate of draft for the week. The hourly fluctuation in rate of water consumption has a decided effect upon the rate of sewage flow, as discussed later in this chapter. It is not, however, entirely responsible for the fluctuation in the rate of flow of sewage, for in some places large quantities of ground water are pumped by industrial establishments and discharged into the sewers during the working hours of the day, thus tending to increase the peak flow beyond the amount resulting from the normal fluctuation in the draft on the municipal water supply.

In the absence of more authoritative information an addition of 50 per cent over the average rate of water demand during any 24 hr., may be made as a fair allowance for the excess of the maximum demand during the 24 hr., from all sources, over the average daily demand, at the time when the flow of sewage is at its maximum. This rate, however, will vary in different places.

If this peak consumption is applied to the maximum draft for a single day, of 198 per cent of the average annual consumption, and it is assumed that the portion of the annual consumption which finds its way into the sewers averages 50 gal. per day, we have a maximum rate of contribution from the public and private water supplies of about 150 gal. per capita daily ($50 \times 1.98 \times 1.50 = 148.5$). This will serve to illustrate the theory of the yield of sewage based on water consumption, but should not be applied in design unless local conditions are found to warrant it.

Ground Water.—A portion of the rainfall ordinarily runs quickly into the storm-water drains or other drainage channels. Another portion is evaporated or is absorbed by vegetation and the remainder percolates into the ground becoming ground water. The proportion which thus percolates into the ground depends upon the character of the surface and soil formation, upon the rate and distribution of the precipitation according to seasons, and the time of its occurrence. Any reduction in permeability, as, for instance, that due to buildings, pavements, or frost, decreases the amount of ground water and increases the surface run-off correspondingly. The amount of ground water flowing from a given area may vary from a negligible amount for a highly impervious district or a district with a dense subsoil, to 25 or 30 per cent of the rainfall for a semi-pervious district with a sandy subsoil permitting rapid passage of water into it. The percolation of water through the ground from rivers or other bodies of water sometimes has considerable effect on the ground-water table, (the ground-water table is the surface of the water in the soil) which rises and falls continually.

The presence of large quantities of ground water results in leakage into the sewers and an increase in the quantity of sewage and the expense of disposing of it. This infiltration from water-bearing earth may be as low as 5,000 gal. per day per mile of sewers and may be as high as 40,000 gal. or more. During heavy rains, when there is leakage through manhole covers as well as infiltration into the sewers themselves, the quantity may exceed 100,000 gal. per day per mile of sewer. It is a very variable part of the sewage, depending upon the quality of the materials and workmanship in the sewers, the character of the maintenance, the height of the ground-water table and the extent the sewers have been injured when house sewers were connected to the common sewers.

The early sewers first built in a district usually follow the water courses in the bottoms of valleys, close to and occasionally in the beds of brooks. The water of the brooks is taken into these sewers. The discharge of the brooks grows gradually smaller after the immediate run-off from a storm has passed by, and the flow until the next storm occurs is mainly ground water draining out of the earth. As a result, these old sewers receive comparatively large quantities of ground water, while sewers built later at higher elevations in the districts will receive relatively smaller quantities of ground water. With an increase in the percentage of area in a district which is paved or built over, comes an increase in the percentage of storm water which is conducted rapidly to the drains, sewers and water courses, and a decrease in the percentage of the storm water which will percolate into the earth and tend to leak into the sewers. A sharp distinction is to be made between maximum rates of leakage or infiltration into sewer pipe systems and average rates. The former are necessary in determining required sewer capacities: the latter, in the discussion of operating problems such as the annual cost of pumping sewage. The records of measured leakage of ground water into sewers given in Tables 2, 3 and 4 are significant.

TABLE 2.—INFILTRATION OR LEAKAGE INTO SEWERS

City or town	Length of sewers, miles	Diameter of sewer, inches	Infiltration or leakage: Gallons per mile of sewer daily			Infiltration or leakage: Gallons per capita daily			Remarks
			Maximum	Average	Minimum	Maximum	Average	Minimum	
In Massachusetts	137	8 to 36		40,000					Before any connections were made to sewers.
In Massachusetts	700		80,000			100			Densely populated section—800 persons per mile of sewer. Reported on "Discharge of Sewage into Boston Harbor," State Board of Health, page 23.
Batavia, N. Y....	30	8 to 27 (25 miles of 8 in.)		23,300			175 ±		Patent gasket used with cement joints. Ground water head was 3 to 5 ft. before mortar set. Gallon per capita is based on assumption of five persons per house. (*Eng. Rec.* Nov. 9, 1912.)
Reading, Pa......				5,172					*Proc.* N. A. Cement Users, vol. vii, p. 697.
Providence, R. I.							47		*Proc.* N. A. Cement Users, vol. vii, p. 702.
Canton, Ohio....				70,000					*Proc.* N. A. Cement Users, vol. vii, p. 698.
Brockton, Mass...	16			25,000					*Proc.* N. A. Cement Users, vol. vii, p. 698.
Brockton, Mass...			117,000		4,000				4,000 gallons before connections were made; 117,000 gallons in wet weather. } *Proc.* N. A. Cement Users vol. vii, Table, I, p. 703.
Madison, Wis....					48,000				Brick sewer in wet ground; trunk sewer in dry weather. } *Proc.* N. A. Cement Users vol. vii, Table, I, p. 703.

TABLE 2.—INFILTRATION OR LEAKAGE INTO SEWERS.—(*Continued*)

New Orleans, La.										
Station "A"....	200				55,000				Year 1907 estimated dry weather flow	*Twenty-fourth Semi-annual Report* of Sewer and Water Board, p. 100.
Station "A"....	240				53,000				Year 1908 estimated dry weather flow	
Station "A"....	280				51,000				Year 1909 estimated dry weather flow	
Station "A"....	296				51,000				Year 1910 estimated dry weather flow	
Station "A"....	323				48,000				Year 1911 estimated dry weather flow	
Station "A"....	360				42,000				Year 1912 estimated dry weather flow	
Ocean Grove, N. J.	3.25	4 to 12			15,120±				Infiltration in a dry period	Vol. lxxvi, *Trans.* Am. Soc. C. E., p. 1936.
Ocean Grove, N.J.	3.25	4 to 12	43,850±						Infiltration in a wet period	
North Metropolitan Sewer District, Boston.			78,900	50,600	30,900	93.2	62.2	38.7	Leakage in wettest periods April and May for several years.	"American Sewer Practice," vol. i, p. 183.
Worcester, Mass.	130.65 miles to 133			23,126±			27		Sewer leakage on certain Sundays, Oct., 1903 to Oct., 1905. The Sunday of each month was selected which was least affected by conditions of rainfall, temperature and regulators. (H. P. E. and A. L. F.) Population in 1904 tributary to sewers, 111,700.	
Worcester, Mass.	63.9 to 89.0		43,400 (Max. for 1 month)	22,050±	15,100 (Min. for 1 month)	40.9 (Max. for 1 month)	20	13.2 (Min. for 1 month)	Average results for 7 years, 1891 to 1897 inclusive. H. P. E. and A. L. F.	

TABLE 3.—ESTIMATE OF INFILTRATION INTO SEWERS AT WORCESTER, MASS., 1890–1897

(Million gallons per day)

Month	1890	1891	1892	1893	1894	1895	1896	1897
January......	.....	1.508	1.610	1.719	1.834	1.966	2.056	2.139
February.....	.....	1.014	1.083	1.156	1.234	1.323	1.383	1.439
March.......	.....	1.275	1.361	1.453	1.550	1.663	1.738	1.808
April.........	.....	2.701	2.885	3.080	3.286	3.524	3.684	3.832
May.........	.....	1.628	1.738	1.856	1.980	2.122	2.220	2.309
June.........	1.254	1.350	1.442	1.539	1.643	1.761	1.840	1.915
July..........	1.003	1.080	1.153	1.231	1.314	1.409	1.473	1.532
August.......	0.964	1.038	1.109	1.183	1.263	1.354	1.416	1.472
September....	1.325	1.427	1.524	1.628	1.736	1.861	1.946	2.024
October......	1.334	1.435	1.533	1.637	1.747	1.873	1.958	2.037
November....	0.906	1.975	1.042	1.112	1.187	1.272	1.330	1.384
December....	1.089	1.172	1.252	1.338	1.427	1.530	1.600	1.663
Averages....	1.466	1.466	1.478	1.578	1.685	1.805	1.888	1.963
Total mileage........		63.90	68.22	72.78	78.03	81.60	84.87	88.96
Average gallon per mile per day......		22,960	21,650	21,690	21,580	22,120	22,250	22,080
Population on line of sewers..........		75,644	79,015	83,272	83,588	89,195	90,754	93,737
Gallon per capita per day...............		19.4	18.7	18.9	20.3	20.2	20.8	20.9

From 1891 to 1897, inclusive, the average leakage per day was 22,047 gal. per mile or 19.9 gal. per capita.

The quantity of leakage depends on the length of sewer, and to a certain extent on the population, which affects the number of connections to buildings and the lengths of the sewers. The authors have found that the leakage through defective joints, porous concrete and cracks is large enough in most cases to lower the ground-water table to the crown of the sewer and usually well down toward the invert, although its elevation varies greatly with the quantity of rain and snow water percolating into the ground.

TABLE 4.—LEAKAGE OF GROUND WATER INTO SEWERS

Place	Gal. per day per mile of sewers	Extent of sewers considered
Alliance, Ohio	195,000	......
Altoona, Pa.	41,000	1.2 miles
Altoona, Pa.	86,000	0.6 miles
Altoona, Pa.	264,000	0.95 miles
Brockton, Mass.	45,000	2,000 ft.
Brockton, Mass.	61,000*a*	10,400 ft.
Brockton, Mass.	178,000*b*	10,400 ft.
Canton, Ohio	26,000	11 miles
Clinton, Mass.	32,500	......
Concord, Mass.	30,000	whole system
East Orange, N. J.	22,000*c*	29 miles
East Orange, N. J.	9,000	25 miles
Framingham, Mass.	35,000	whole system
Gardner, Mass.	45,000	whole system
Joint Trunk Sewer	25,000*d*	150 miles
Madison, Wis.	48,000	......
Malden, Mass.	50,000	whole system
Marlboro, Mass.	50,000	whole system
Medfield, Mass.	25,000*e*	whole system
Metropolitan System	40,000*f*	137 miles
Natick, Mass.	80,000 to 100,000	8.58 miles
New Orleans, La.	32,000 to 60,000	......
North Brookfield, Mass.	24,000	1.41 miles
Peoria, Ill.	100,000	......
Reading, Pa.	5,000	......
Westboro, Mass.	1,072,000	3,010 ft.
Worcester, Mass.	32,000	......

a. Water in river low. *b*. Water in river high. *c*. Great precautions taken to prevent leakage, as construction was carried on in quicksand and the ground-water table was naturally 10 ft. or more above the sewer. *d*. This relates to the sewer serving parts of Newark and Elizabeth, N. J., and smaller places westward to Summit. *e*. Before house connections were made. *f*. Before any connections were made.

The Malden figures are from *Eng. News*, Aug. 27, 1903; the Concord figures from the 1900 report of the Sewer Commissioners, and the remainder from reports of the Mass. Board of Health and *Trans*. Am. Soc. C. E., vol. lxxvi, p. 1909, *et seq*.

Sewage per Day.—In Table 5 are given average statistics of the sewage of a number of Massachusetts towns and small cities having separate sewers. The gagings of the sewage from

TABLE 5.—MAXIMUM AND AVERAGE FLOWS OF SEWAGE IN MASSACHUSETTS CITIES, 1920[1]

Place	Population census of 1920	Average yearly quantity of sewage, gallons per 24 hr.			Average quantity of sewage in maximum month, gallons per 24 hr.		
		Per inhabitant	Per connection	Per mile of sewer	Per inhabitant	Per connection	Per mile of sewer
Andover	8,268	30	330	......	42	460	
Attleboro	19,731	30	510	19,000			
Brockton	66,254	42	410	31,500			
Clinton	12,979	84	690	50,800			
Concord[5]	6,461[4]	82	1,120	59,800	112	1,520	81,400
Fitchburg	41,029[3]	73	....	52,400	102	....	73,200
Framingham[5]	17,033	74	540	47,700	124	910	79,900
Franklin[5]	6,497	32	390	13,100			
Gardner	16,971	59	550	35,5 0			
Hopedale	2,777	31			57⊙		
Hudson	7,607	58	650	42,400	83⊙	930⊙	61,200⊙
Marlborough	15,028	72	500	36,100	121	840	61,000
Milford	13,471	72	740	54,200	75	770	56,300
Natick	10,907	137	1,060	......	359	2,780	
North Attleborough	9,238	84	1,200	46,800	92⊙	1,320⊙	51,200⊙
Northbridge[5]	10,174	57					
Norwood	12,627	95	1,060	67,300			
Pittsfield	41,763	84	730	57,500			
Westborough[5]	5,789	91	950	55,100	153	1,590	92,400
Worcester[2]	179,754	135	.....	131,000			

[1] From a forthcoming Annual Report of the Massachusetts Department of Health, (1920).
[2] Includes flow from 69.81 miles of combined sewers. ⊙Accuracy of these figures is questionable.
[3] Largely combined system.
[4] Approximately two-thirds of the sewers laid below ground water level.
[5] Substantial portion of population not served by sewers.

several districts in Chicago, Table 6, exceed greatly the Massachusetts records. The Chicago sewers are built on the combined system, but the gagings were made in dry weather. The large flow is due mainly to the heavy consumption of water in the city, which averaged 242 gal. per capita daily in 1910.

TABLE 6.—TYPICAL DRAINAGE AREAS AND DRY WEATHER RUN-OFFS, CHICAGO, 1910 AND 1911

(From Wisner's Report on Sewage Disposal. San. Dist. of Chicago, 1911)

Sewer outfalls	Drainage area in acres	Population estimated 1911	Dry weather run-offs, Cu. ft. per sec.	Cu. ft. per sec. per acre	Cu. ft. per sec., per sq. mile	Gal. per cap. per 24 hours.	Density of population per acre	Period covered by observation
Diversey Boulevard (W).	890	23,550	8.65	0.0097	6.22	238	26.4	Aug. 15–17, 1911, 2 days.
Randolph St. (W).	240	11,368	6.10	0.0254	16.25	348	47.4	Aug., 3 days.
Robey St. (S).....	2,500	38,728	10.1	0.0040	2.58	169	15.5	June 1–3, 1911, 2 days.
Ashland Ave., (S)..	980	44,581	23.2[1]	0.0237	15.1	338	45.5	May 18–20, 1911, 2 days.
Center Ave., (S)...	660	23,463	20.9[2]	0.0317	20.3	578	35.6	May 16–18, 1911, 2 days.
Thirty-ninth St., pumping station.	14,340	285,900	140.0[3] 100.0[4]	0.0098 0.0070	6.25 4.46	318 227	20.0	....
Ninety-second St...	98	3,666	1.84	0.0188	12.0	325	37.4	209 days. Aug. 1, 1910. Mar. 31, 1910.
Wentworth Ave., (S), (Calumet).	5,300	30,464	12.4	0.0023	1.5[5]	264	5.8	Aug. 1, 1910. July 31, 1911. 253 days.

[1] Daily variation average
8 A.M. to 8 P.M. 28.5 c.f.p.s. contains large amount of industrial waste.
8 P.M. to 8 A.M. 18.6 c.f.p.s.

[2] Daily variation average
8 A.M. to 8 P.M. 25.7 c.f.p.s. contains large amount of industrial waste.
8 P.M. to 8 A.M. 17.0 c.f.p.s.

[3] This run-off or more for 76 days in 1909.

[4] This run-off or more for 276 days in 1909.

[5] 2.4 c.f.p.s. per square mile occurred 329 days in the year.

The flow on different days of the week varies considerably, as might be expected from the daily variations in the consumption of water. It is larger on Monday and smaller on Sunday than on other days. There is also a difference between the average daily volume of sewage at different seasons of the year, due in part to variations in water consumption and in part to groundwater conditions.

The average rates of flow upon which estimates of cost of pumping and treatment may be based are much below the maximum rates, and, from the data available, appear to range in a general way between 100 and 125 gal. per capita per day for the larger cities. For small towns average rates appear to range from about 25 to 60 gal. per capita per day.

Sewage per Hour.—The flow of sewage per hour is more important to the designing engineer than the flow per day,

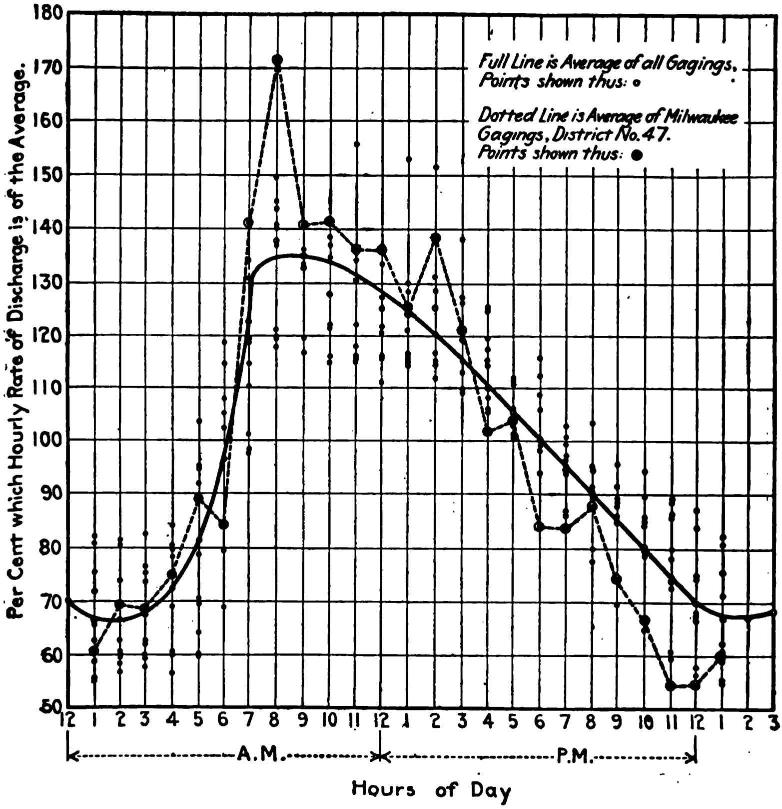


FIG. 9.—Hourly variation in flow of sewage in various cities.

The records used in preparing this diagram were from the following sources: Birmingham, England, average of two years, 1906–7; East Orange, N J., March 16–17, 1910; Gloversville, N. Y., Oct. 30, 1906, and Sept. 12, 1907; City A, of 15,000 population, typical average curve; Milwaukee, Wis., Oct. 24–28, 1920; Toronto, Ont., 1900 and 1908; Worcester, Mass., Nov. 13, 1909, and March 21–27, 1910.

because he has to provide sufficient sewer capacity to carry off the largest quantity of sewage which it is reasonable to expect. In Fig. 9 are plotted the hourly rates of flow in several cities, expressed as percentages of the daily average. The smooth curve is drawn through points obtained by averaging points taken from curves representing the hourly rates of flow in the cities named in the note below the illustration. In preparing curves of this nature it is desirable to synchronize them by making an allowance for the time required for the sewage to flow from the city to the gaging point. This time differs, and if an allowance is not made for it the data from different cities are not easily compared. The dotted line in Fig. 9 represents the flow of sewage from a large Milwaukee residential district.

It is interesting to compare the hourly fluctuation shown in Fig. 9 with the fluctuations in water consumption referred to previously in this chapter (see Fig. 7).

Sewage from Residential Districts.—The quantity of sewage from a residential district will depend upon its area, density

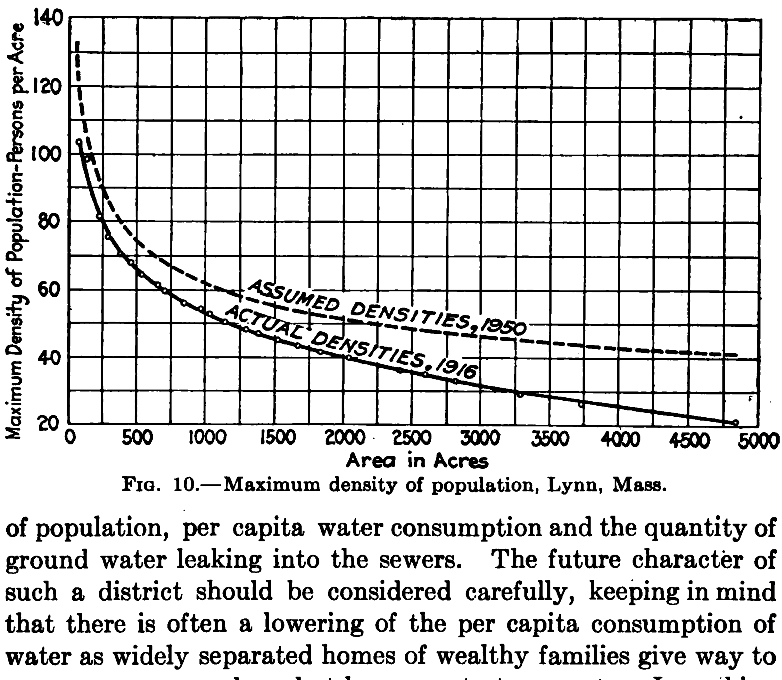


FIG. 10.—Maximum density of population, Lynn, Mass.

of population, per capita water consumption and the quantity of ground water leaking into the sewers. The future character of such a district should be considered carefully, keeping in mind that there is often a lowering of the per capita consumption of water as widely separated homes of wealthy families give way to more numerous and modest homes or to tenements. In making estimates of the quantity of sewage from such a district, it is incorrect to use the average density of population of the entire city, which rarely exceeds 25 persons per acre, because there are many residential districts where it is much higher. For example, it was found in Philadelphia in 1910, that the population per acre was 58 to 64, where the prevailing building type was pairs of two- and three-story houses, and 97 to 123 where the buildings were solid rows of houses. The sewage from the former type of district amounted to 9,800 to 14,200 gal. per acre daily, and from the latter type 10,500 to 26,300 gal.

The decrease in average density of population with increase in area is a factor of importance in the design of separate and intercepting sewers. It is impossible to predict with certainty the locations of future areas of high density. This is a further reason for liberality in the design of the laterals. Fig. 10

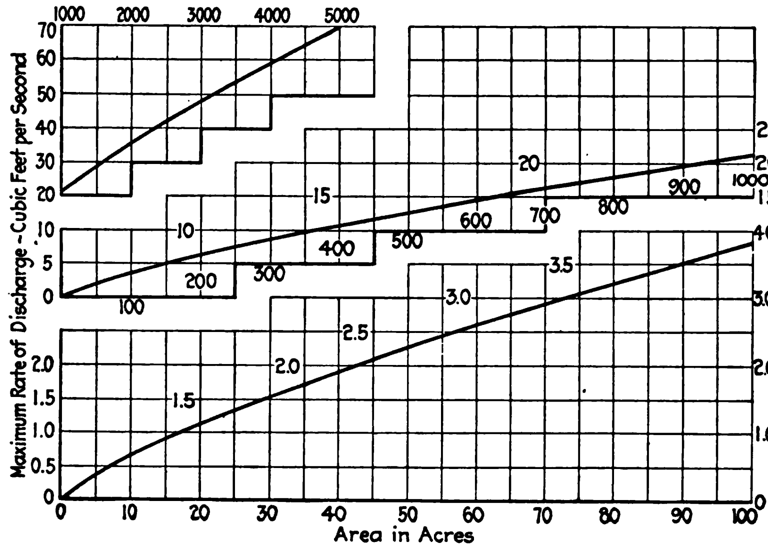


Fig. 11.—Maximum rate of discharge of residential sewage, Lynn, Mass.

shows the actual maximum densities for different areas found in Lynn, Mass., in 1916 and the densities estimated for the year 1950, during which period it was assumed that the total population would double. Data sufficiently accurate for making and checking such curves may be obtained by actual count, by the use of assessors' estimates, from the census reports where the population is sufficiently large so that it is divided into wards and precincts, or from data contained in directories, post office counts and lists of polls.

It is evident from the foregoing that the maximum rate of discharge of residential sewage per unit of area may be much higher for small areas than for larger ones. Fig. 11 shows the estimated maximum rate of discharge of sewage for different given areas based upon densities of population shown in the higher curve of Fig. 10 and the maximum per capita rate of sewage flow of 220 gal., otherwise determined for the district under consideration.

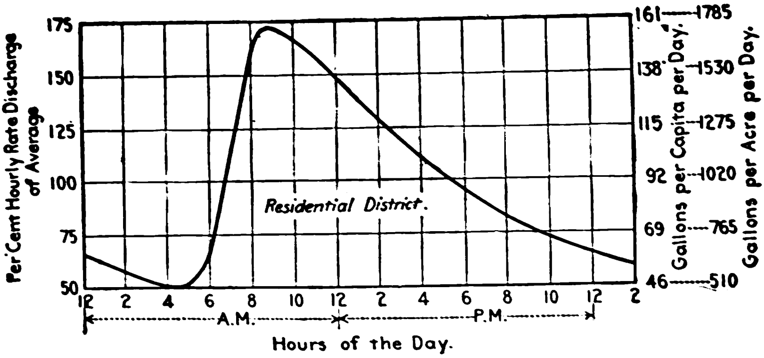
The rate per hour fluctuates greatly in the case of moderately developed medium and high-class residential districts, because of the great variation in the hourly consumption of water. Fig. 12, curve *A*, gives the average hourly rate determined from two sets of gagings of a Cincinnati district of 1,617 acres with a population of 11.1 persons per acre. This district is very largely residential, and the maximum observed hourly flow was 273 per cent of the average although the district is so large that such a wide range in the rate of flow in its main sewer is unusual. In the usual case of a district of this size, the maximum hourly rate may be expected to reach 160 to 170 per cent of the average.

Sewage from Commercial Districts.—A commercial district yields much more sewage per acre than a residential district, because of the concentration there during the day of many persons in stores, office and public buildings, hotels and often some apartment houses. Such a district is relatively small compared with the other districts in a city, but its growth should be forecasted carefully on account of the large quantity of sewage coming from it.

The allowance for used water from a mercantile district is more difficult to estimate than that from a residential district. If the estimate is made in connection with the design of intercepting sewers, pumping stations or treatment works for use with existing main district sewers, the latter should be gaged and a suitable allowance made for future development. It may also be helpful to ascertain the water supply of the district from both municipal and private sources.

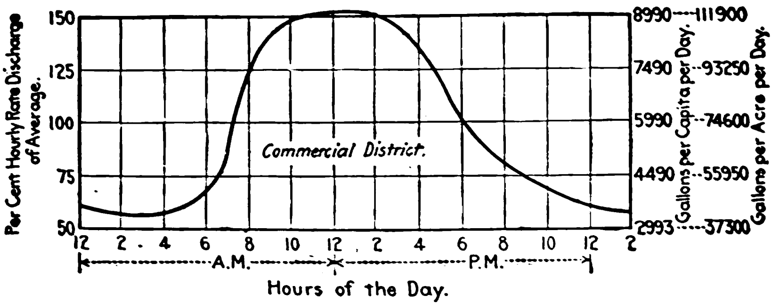
The average quantity of sewage from a 123-acre commercial district in Philadelphia was 92,800 gal. per acre per day. This district was entirely built over, the area occupied by buildings being 80 acres and that by streets 32 acres. This is a rather large quantity of sewage from a commercial district. Gagings of the flow from eight such districts in Cincinnati ranged from 14,000 to 72,000 gal.

The hourly rate of flow from districts of this character varies greatly, as is to be expected, but the high rate of discharge lasts

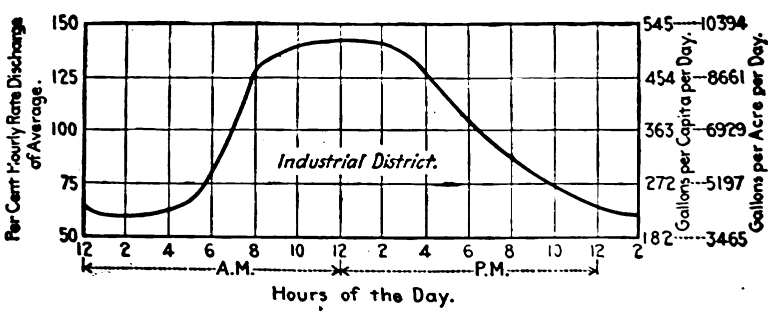


Hourly Variation in Flow of Sewage.

Curve A. Ross (Bloody) Run Sewer.



Curve B. Vine Street Sewer.



Curve C. Marshall Ave. Sewer.

FIG. 12.—Hourly variations in flow of sewage Cincinnati.

much longer than does the early morning peak in the curve of flow from residential districts. Curve *B* in Fig. 12 illustrates this, and shows the hourly rate in a district of 23.6 acres with 12.5 persons per acre, the center of business in Cincinnati. The average daily flow in this case was 72,000 gal. per acre and the maximum during the gaging period was 139,000 gal.

Sewage from Industrial Districts.—The quantity of industrial wastes not originating in the public water supply should be studied individually in each case. The following estimates, in gallons per capita daily, have been made in connection with the design of sewerage works:

Cincinnati, Ohio	50
Fitchburg, Mass., maximum	81
Louisville, Ky	57
Neponset Valley intercepter, Mass	25
Milwaukee, Wis	57
Passaic Valley intercepter, New Jersey, maximum	38
Paterson, N. J., maximum	18
Providence, R. I., maximum	42

The variation in the hourly rate of flow from a typical industrial district with some residences, in Cincinnati, is shown by curve *C*, Fig. 12. It had an area of 294 acres and a population of 19.1 persons per acre. The curve is based on 3 days' gagings during which the average rate was 6,787 gal. per acre and the maximum rate 13,485 gal. This maximum is equivalent to an hourly rate of 708 gal. per capita daily, 198 per cent of the average daily rate.

Provision for Storm Water in Intercepting Sewers.—There is a general impression that it is wise to provide in intercepting sewers for a small quantity of storm water, expressed often as being sufficient for the "first flushings" of street surfaces and sewers. This impression is based upon the assumption that there are accumulations of sewage sludge in the sewers and quantities of filth on the streets which will be immediately flushed into the intercepting sewers with the first run-off due to rain. In some sewers laid on very flat grades and where sewers have settled or have been built with depressions in them, there may be such deposits, but where sewers are laid on grades which give satisfactory velocities such deposits are believed to be exceptional. Where deposits do occur they are generally found to consist largely of sand and other heavy detritus, dropped by the current, as the velocity of flow decreased with the decrease in volume and hence depth of flow in the sewer. It is apparent, therefore, that these deposits will not be picked up again by the current until a very substantial rate of discharge is reached.

Intercepters are fed by trunk sewers serving rather large districts. Considerable time is required to flush the major part of the systems to the intercepter, during which a large flow is likely to reach them from the nearer portions. Unless considerable surplus capacity is provided, the intercepters will often be running full before the flushings from much of the tributary area can reach them. Therefore, too much stress should not be laid on their ability to care for the "first flushings," although as ordinarily designed they can accomplish something in this direction.

Assuming that the average daily flow of sewage is 100 gal. per capita and that the capacity of the intercepter is 300 gal. per capita, there will be a surplus capacity available for "first flushings" equivalent to twice the average flow of sewage, if such flushings come at a time when the flow is at the average rate. The maximum rates of flow generally occur in the spring when ground water is high and at other times there will always be some surplus capacity. Furthermore, as intercepters are built for many years, there will be a considerable excess capacity during the earlier years, although this should not usually be counted upon to care for storm water, for it is a gradually diminishing allowance accompanied by a gradually increasing need, if need there should be.

Under the foregoing conditions the sewage will be diluted to three times its normal flow. Furthermore, consideration must be given to the excess of water used in this country, giving a more dilute sewage, and to the quantity of ground water which leaks into the sewers. With these eliminated the quantity of sewage would be comparable with that obtained in Europe. Taking all these conditions into account, it is evident that the dilution approaches the standard of the Royal Commission on Sewage Disposal, which is six times the dry-weather flow.

In view of these conditions, it may be assumed that the 300-gal. daily per capita intercepter capacity, allowed in a previous paragraph, contains a sufficient allowance for storm water.

Caution.—The foregoing outline of a rational method of estimating the quantity of sewage to be provided for applies to the design of intercepters, pumping equipment, grit chambers,

etc., and to large trunk sewers and the units adopted are for maximum rates of flow when the sewers are running full.

These principles, also, constitute the basis of design for a separate system of sewers, or a system from which it is intended that storm water shall be excluded. It is important to note, however, that in practice it is not uncommon to find a considerable amount of surface run-off finding its way into manholes and numbers of storm water roof conduits connected to separate sewers, in spite of rules or ordinances prohibiting such connection. It is wise, therefore, to make some allowance in the maximum rate of sewage flow, for a certain amount of storm water.

The steps and basic data used in the design of intercepting sewers are similar to those used in the design of a separate system, an example of which follows, worked out in detail.

The Design of a Separate Sewer System.—The district shown in Fig. 13 is a portion of the Carlisle Brook District of the City of Springfield, Mass. This is a residential district, which, in 1920, was about two-thirds developed with detached houses. This district is so nearly fully developed that the probable future density of population can be estimated with a fair degree of certainty and it is not necessary to make a study and curve such as is shown on Fig. 10.

It is estimated that the future average density of population will be 65 persons per acre. The maximum rate of flow of sewage is estimated at 250 gal. per capita per day, including 30 gal. per capita per day allowance for storm water.

The subsoil of the district is sand and will permit a large portion of the rainfall to percolate into the ground.

The maximum rate of infiltration of ground water into the sewers, to be provided for, is 2,000 gal. per acre per day.

The minimum size of sewer is to be 8 in. The minimum velocity of flow in the sewer when flowing full is to be 2 ft. per second (the more desirable limit of 2.5 ft. was impracticable because of the prohibitive expense that would have resulted from its adoption).

The minimum depth of cover allowable is 7 ft.

Problem.—Design a system of separate sewers for the area included within the dash-dot line. This system is to discharge into the Carlisle Brook main sewer at the right-of-way leading from Shattuck St. about opposite Courtland St. to the main sewer.

Prepare a plan showing the location, size and slope of each sewer section between manholes in each street.

Prepare a profile of each sewer showing all connections with each lateral sewer.

On the plan show the location of manholes. The profile should show the street surface; the manholes; the proposed sewers, their sizes and slopes; and the invert elevations in figures, at each manhole.

Determine the required elevation for the Carlisle Brook main sewer at the right-of-way.

This problem may be combined with the problem for the design of the storm-water drainage system. This would, however, involve additional complications, as precaution would have to be taken that proper connections of sewers and drains could be made without interference with each other. It often happens in practice that this feature of the design of a separate system is very troublesome, particularly where a system of sewers has been installed without particular study of the desirable relative position of the pipes in the streets when the drains shall have been added.

Having determined the basic data for any problem, the different steps required to reach a rational solution are as follows, in the order given:

First.—Draw a line to represent the sewer in each street or alley to be served. Near the line place an arrow pointing in the direction in which the sewage is to flow. Except in special cases, the sewer should slope with the surface of the street. It is usually more economical to plan the system so that the sewage from any street will reach the point of disposal by the most direct and consequently the shortest route.

If the lines are properly drawn, the system will resemble a tree and its branches, all the laterals connecting with the sub-mains and these in turn with the main or trunk sewer, which leads to the point of discharge.

Second.—Locate the manholes, giving each an identification number. Manholes should generally be placed at all angles, or bends, changes in grade, at all junctions of common sewers, at the upper ends of all laterals and at intermediate points where the distance exceeds 400 ft. or such other maximum limit as may be established.

Third.—Sketch the limits of the drainage areas for each lateral, unless a single lateral will be required to accommodate an area larger than can be drained by the minimum size of sewer with the minimum slope, in which case a further subdivision must be made. Where the streets are laid out, the drainage lines may be assumed as being midway between them. If the street layout is not shown on the plan, the limits of the different areas cannot be determined as closely and the topography must serve as a guide.

Fourth.—Measure the area of the different drainage areas. For this a planimeter will give results within the accuracy required.

Fifth.—Prepare a table, see Table 7*a*, for the different steps in the computation and for each length of sewer between manholes. This is the most concise, time-saving method and keeps the results in better shape for the subsequent use of the draftsman. Use column 1 for numbering the lines of the table, for ready reference. Determine by inspection the manhole which is farthest from the point of disposal and enter its identification number in the first line of column 2, and the number corresponding to the manhole next on the line toward the trunk sewer in column 3. Enter the name of the street or alley in column 4, the length between manholes in column 5, and the area in acres to be drained by the sewer at a point just above the lower manhole in column 6. On the next line enter the same data for the next stretch of sewer and in column 7 enter the sum of the drainage areas listed in column 6. The area in column 7 is the basis for computing the required capacity of the sewer. Enter the data for each stretch of sewer in the

TABLE 7a.—METHOD OF TABULATING DATA AND COMPUTATIONS FOR A SEPARATE SEWER SYSTEM: PROJECT *A*

Number of line	Manhole number		Location, street	Length, feet	Area, acres		Sewage, million gals. daily[1]	Ground water at 2,000 gal. per acre daily	Total maximum flow, sewage and ground water		Size of sewer internal diameter in inches	Capacity, cubic feet per second	Velocity, feet per second inter	Slope decimal of a foot per foot nal	Surface elevation upper end	Invert elevation	
	From	To			Increment	Total			Million gallons daily	Cubic feet per second						Upper end	Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	2	3	Dawes.........	315	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	206.2	198.00	196.17
2	3	4	Dawes.........	315	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	206.5	196.17	194.34
3	4	5	Dawes.........	315	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	206.6	194.34	192.51
4	5	6	Dawes.........	156	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	207.1	192.51	191.60
5	6	7	Andrew........	288	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	207.4	191.60	189.93
6	7	8	Burr..........	400	.....	13.61	0.221	0.027	0.248	0.38	8	0.7	2.0	0.0058	206.1	189.93	187.61
7	8	9	Burr..........	31	5.89	19.50	0.316	0.039	0.355	0.55	8	0.7	2.0	0.0058	203.2	187.61	187.43
8	9	10	Burr..........	225	10.00	29.50	0.480	0.059	0.539	0.84	10	1.1	2.0	0.0042	203.2	187.26	186.31
9	10	11	Burr..........	225	5.68	35.18	0.572	0.070	0.642	1.00	10	1.1	2.0	0.0042	201.9	186.31	185.36
10	11	12	Oak Grove Ave.	130	12.07	47.25	0.767	0.095	0.862	1.33	12	1.55	2.0	0.0032	201.6	185.19	184.77
11	12	13	Oak Grove Ave.	82	11.80	59.05	0.960	0.118	1.078	1.67	12	1.70	2.2	0.0036	202.1	184.77	184.47
12	13	14	Shattuck.......	280	5.30	64.35	1.046	0.129	1.175	1.82	15	2.60	2.0	0.0022	202.8	184.22	183.60
13	14	15	Shattuck.......	275	19.23	83.58	1.360	0.167	1.527	2.36	15	2.60	2.0	0.0022	203.2	183.60	182.99
14	15	16	Shattuck.......	113	12.14	95.72	1.555	0.192	1.747	2.70	15	2.70	2.0	0.0026	203.6	182.99	182.70
15	16	17	Shattuck.......	245	2.75	98.47	1.600	0.197	1.797	2.78	15	2.80	2.3	0.0029	203.7	182.70	181.99
16	17	18	Shattuck.......	375	12.43	110.90	1.800	0.222	2.022	3.13	18	3.55	2.0	0.0017	202.3	181.74	181.10
17	18	19	Right of way...	130	4.22	115.12	1.871	0.231	2.101	3.25	18	3.55	2.0	0.0017	196.0	181.10	180.88
18																	
19			Cambridge St.	Lateral													
20																	
21	69	70	Cambridge.....	350	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	208.5	200.3	198.27
22	70	71	Cambridge.....	350	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	206.7	......	196.24
23	71	72	Cambridge.....	350	.....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	204.4	......	194.21

[1] 250 gal. per capita daily and 65 persons per acre.

Since the capacity of the minimum size of sewer with a minimum velocity of 2 ft. per second is 0.7 cu. ft. per second, equivalent to the maximum rate of discharge from 24.6 acres $\frac{24.6 \times (65 \times 250 + 2{,}000) \times 1.55}{1{,}000{,}000} = 0.7$, all laterals will be 8 in. diameter (the minimum) as no lateral is to drain an area exceeding 24.6 acres.

above manner, following the line down to the point of discharge including the trunk or main sewer.

Enter in column 8 the rate of discharge of sewage, which is equal to the maximum per capita rate of sewage flow multiplied by the assumed future density and this product multiplied by the area shown in column 7.

In column 9 enter the rate of ground water flow, which is equal to the rate of flow per acre to be provided for, multiplied by the area in column 7.

Column 10 gives the sum of columns 8 and 9, in million gallons per day, and column 11 is this quantity converted into cubic feet per second, the more convenient way of expressing the capacity of sewers and the form in which most diagrams and tables for determining the carrying capacity of sewers are given.

The designer is now ready to select the sizes and slopes of sewers required to carry the quantities computed above. The remaining paragraphs of this chapter may, however, be postponed advantageously until the student has studied Chapter IV.

In column 12 is the required size of sewer; column 13, the carrying capacity; and column 14 the velocity when flowing full or half full, corresponding to the slope, column 15.

In column 16 is the elevation of the ground surface at the manhole, corresponding to the identification number in column 2.

Columns 17 and 18 are the invert elevations of the upper and lower ends, respectively, of each stretch of sewer.

If the computations for the entire system are carried out in all the detail above outlined, it may not be necessary to construct a profile for the preliminary study, but it is advisable to make profiles of the main sewers and the more important laterals as a check upon the work. For construction purposes, a profile should be prepared for each sewer from data obtained by field observations and survey, showing the ground surface, the depth and location of existing cellars or basements, the proposed sewer, its slope and size in figures, and the invert elevation at each manhole, as well as the size and elevations of the sewer into which the sewer under consideration is to discharge. The scale to be used in preparing the profiles will depend upon the number of obstacles to construction, and hence the amount of detail required. In city work, scales of 20 to 40 ft. per inch horizontal, and 2 to 4 ft. per inch vertical, are commonly used.

From the tabular form of computation, depths of cut may be computed and estimates of cost prepared.

About 15 ft. depth of cut for the main line west of Cambridge St. could be saved by carrying the Dawes St. lateral northwesterly across private land from manhole No. 5 to Cambridge St., as shown in Table 7*b*.

The trunk line west of Cambridge St. will be at 1.68 ft. less depth by Project *B*, Table 7*b*, than by Project *A*, previously studied. Assuming a 3 ft. 6 in. width of trench and excavation at this 20 ft. depth to cost \$5.00 per cubic yard $(3.5 \times 1.68 \times 5) \div 27 =$ \$1.09, saving per foot for 2,080 ft., a sum equal to \$2,265. In addition there would be a saving in the cost of the sewer in Andrew St. and in Burr St. from Andrew St. to Cambridge St. where the sewer would be about 10 ft. shallower $(10 \times 3.5 \times 5) \div 27 =$ \$6.50; for 590 ft., this will be \$3,835. On the other hand the sewer in

TABLE 7b.—A POSSIBLE MODIFICATION OF TABLE 7a. PROJECT *A* TO REDUCE COST: PROJECT *B*

Number of line	Manhole number		Location, street	Length, feet	Area, acres		Sewage[1] million gallons daily	Ground water at 2,000 gallons per acre daily	Total maximum flow, sewage and ground water		Size of sewer, internal diameter in inches	Capacity, cubic feet per second	Velocity, feet per second	Slope, feet per foot	Surface elevation upper end	Invert elevation	
	From	To			Increment	Total			Million gallons daily	Cubic feet per second						Upper end	Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	2	3	Dawes.........	315	....	......	.....	.....	.....	....	8	0.7	2.0	0.0058	206.2	198.00	196.17
2	3	4	Dawes.........	315	....	......	.....	.....	.....	....	8	0.7	2.0	0.0058	206.5	......	194.34
3	4	5	Dawes.........	315	....	.....	.....	.....	.....	....	8	0.7	2.0	0.0058	206.6	......	192.51
4	5	9*a*	Right-of-way...	286	....	8±	.....	.....	.....	....	8	0.7	2.0	0.0058	207.1	......	190.85
5	9*a*	9	Cambridge.....	300	10±	18±	0.292	0.036	0.328	0.51	8	0.7	2.0	0.0058	203.0	......	189.11
6	9	10	Burr...........	225	11.5	29.50	0.480	0.059	0.539	0.84	10	1.1	2.0	0.0042	203.2	.. ...	etc.
7	10	11	Burr...........	225	5.68	35.18	0.572	0.070	0.642	1.00	etc.	etc.	etc.	etc.	201.9		
8	11	12	Oak Grove Ave.	130	12.07	47.25	etc.	etc.	etc.	etc.	..	...	...	......	etc.		
9	12	13	Oak Grove Ave.	82	11.80	59.05											
10	13	14	Shattuck.......	280	5.30	64.35											
11	14	15	Shattuck.......	275	19.23	83.58											
12	15	16	Shattuck.......	113	12.14	95.72											
13	16	17	Shattuck.......	245	2.75	98.47											
14	17	18	Shattuck.......	375	12.43	110.90											
15	18	19	Right-of-way...	130	4.22	115.12											
16																	

[1] 250 gal. per capita daily and 65 persons per acre.

Cambridge St. would be about 3.5 ft. deeper for 300 ft., $(3.5 \times 3.5 \times 5) \div 27 = \$2.27$; for 300 ft., this will be $680, and the sewer in the right-of-way would cost say $6 per foot for 286 ft., or $1,720; the cost of the right-of-way will be about $500. The net saving, then, would be about $3,200.

Summary of Saving Effected by Project B

1. Saving in trunk line changes west of Cambridge St.		$2,265
2. Saving in Andrew and Burr Streets		3,835
3. Gross saving		$6,100
4. Increase in Cambridge Street	$680	
5. Sewer in Right-of-way	1,720	
6. Cost of Right-of-way	500	$2,900
7. Resulting net saving of Project *B* over Project *A*		$3,200

Whether or not this saving is justified in spite of the disadvantage of laying the sewer partly in a private right-of-way, may depend upon other considerations than relative cost.

CHAPTER III

QUANTITY OF STORM WATER

The rate at which storm water, including melted snow and ice, flows from the surface of the ground into drains and combined sewers must be estimated before designing these conduits. The quantity of storm water is usually so much larger than the quantity of sewage for which combined sewers are designed that their size is determined primarily by the rates of rainfall and run-off and the slopes of the sewers. The amount and intensity of rainfall can be measured. The run-off or part of the rainfall which reaches the drains or combined sewers depends upon the ratio of the impervious to the relatively absorbent surface, the intensity and duration of the rainfall, the character, shape, and slope of the drainage area, etc. It is manifest that a smaller proportion of a light rainfall on a dry sandy tract will reach the sewers than in the case of a heavy rainfall on a clay tract of the same size and topography.

In sewerage engineering the rate of rainfall during periods just long enough to produce maximum run-off conditions is more important than the total daily rainfall. Such rates can only be measured practically by automatic recording rain gages, and on account of the small number of these instruments in use until recently, there are few long-time records of this character.

Automatic Rain Gages.—The gages which give the records most helpful to sewerage engineers make charts on which it is possible to determine accurately the precipitation in time intervals as short as 2 and 5 min.

In all such instruments the rainfall on a circular collecting area or opening, from 8 to 12 in. in diameter in different instruments, passes to an operating mechanism, of which there are three general types. The first type receives the water in a container supported on a weighing device which actuates recording mechanism that moves a pen pressing on a chart held by a drum revolved by clockwork. In the second type, the rain water is delivered to tilting pans, which tip when full and thus discharge their contents. Every act of tipping actuates an electrical or mechanical device which makes a record of the discharge on a chart moved by clockwork, and as the quantity of water discharged by each tipping is known, the rate of rainfall is thus determined. In the third type, the water flows into a container in which there is a float with a vertical arm carrying a pen. This pen records the height of water in the container on a chart moved by clockwork, and the container is usually of such size that the float will rise at a faster rate than the rate of rainfall. Fig. 14 illustrates a tipping-type of gage made by Julien P. Friez, Baltimore. The rain is collected in a funnel 12 in. in diameter and conducted through a tube into a bucket with two compartments, each holding the equivalent of 0.01 in. of rainfall. The bucket is supported on trunnions in such a manner that as soon as a compartment is full the bucket tips, the compartment is emptied and the other compartment brought into

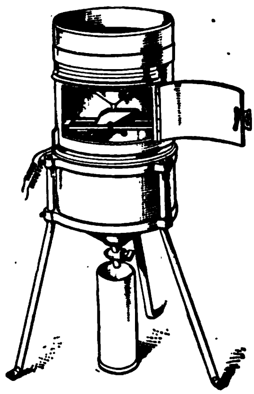


Fig. 14.—Friez tipping bucket gage.

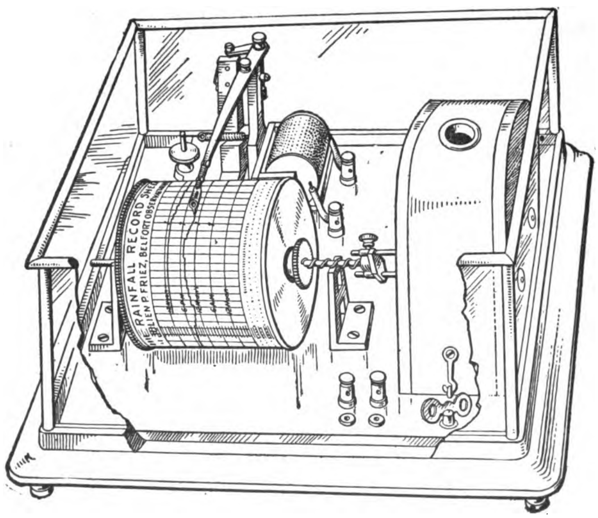
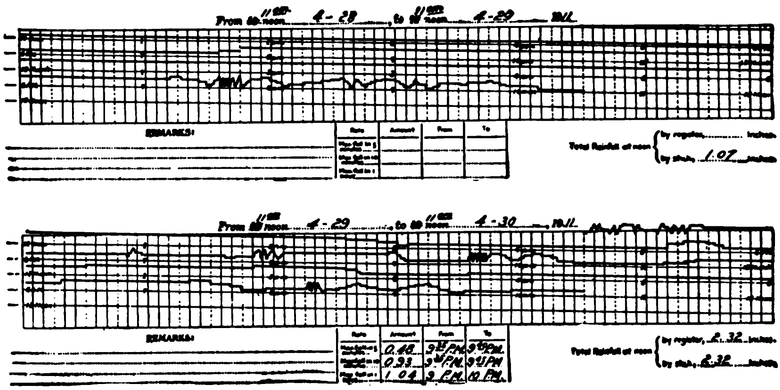


Fig. 15.—Register for Friez gage.

position for filling. Each time the bucket tips, it closes an electric circuit which causes a pen to record a step on a chart carried by a revolving cylinder, Fig. 15. A sample chart from this instrument is shown in Fig. 16. The curve traced does not represent directly the progress of the storm, the motion of the pen being reciprocating, up for 0.05 in. and down for 0.05 in.



The chart is 10¼ inches long.
FIG. 16.—Chart from Friez gage.

The time-scale of this chart is 2½ in. to an hour. The amount of rainfall is indicated, not by measurement on the chart, but by counting the number of steps, or of "flights" of 5 steps each. It is therefore possible to determine the rates of rainfall from this record with a good degree of precision. The Weather Bureau carefully investigated the accuracy of the instrument and determined that on account of the appreciable time required for the bucket to tip, the error due to the inflow of water into a compartment already full before the bucket could tip and present the empty compartment is sufficient to produce an error of about 5 per cent at times of very heavy rain.

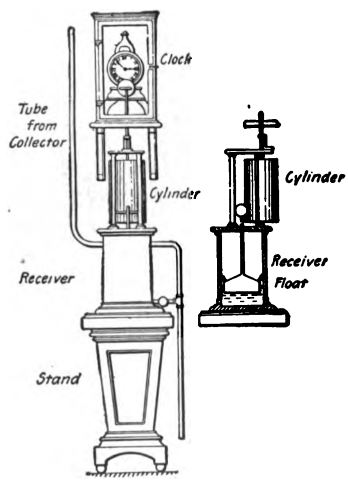
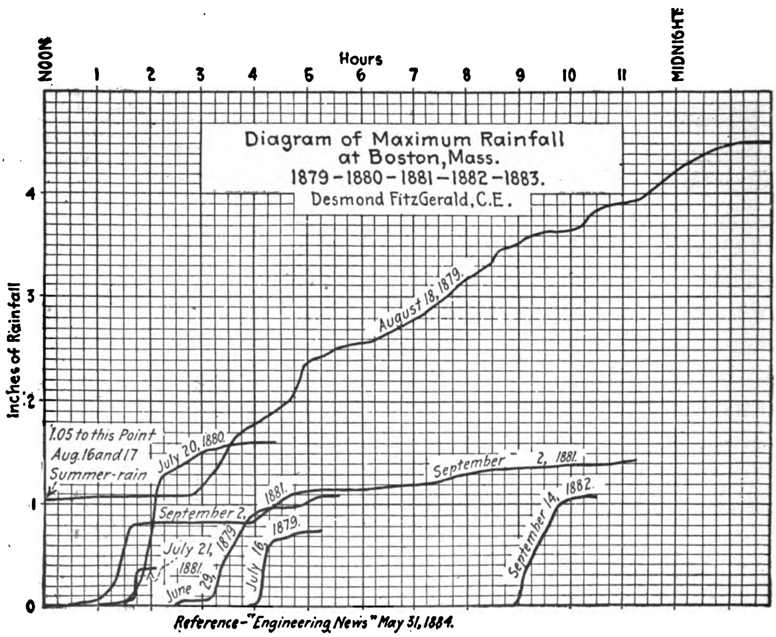


FIG. 17.—FitzGerald gage.

One of the oldest and most satisfactory gages, devised by Desmond FitzGerald, is shown in Fig. 17, and the chart made from it is of the kind shown in Fig. 18. The rain is collected in a funnel 14.85 in. in diameter, and conducted through a tube into a receiver containing a float. The diameter of the receiver is such that 1 in. of rain causes the float to rise 2 in. The float carries a pencil bearing directly upon the chart carried by a revolving cylinder. This cylinder is of such a size that a chart 24 in. long is revolved once every day so that the time-scale is 1 in. per hour. It is therefore possible to determine rates of precipitation with fair accuracy.

Gages should be set[1] at least 50 ft. from objects which may cause wind currents interfering with accurate measurement, and



The complete diagram is 24 in. long.
FIG. 18.—Chart from FitzGerald gage.

the collector ring or opening should be within 30 in. of the ground. It is often difficult to comply with the latter condition, and if the collecting ring is at a considerable elevation above ground, it will probably receive a markedly smaller quantity of water than if placed within the standard distance of the ground. If it is impossible to place the collecting ring practically at ground level, a standard non-recording rain gage should be maintained near

[1] *Circular E*, Instrument Division, U. S. Weather Bureau, on "Measurement of Precipitation," gives valuable information on this important subject.

the automatic gage and the records of the latter adjusted to agree with those of the standard gage.

Intensity of Precipitation.—In a general way the intensity of precipitation varies inversely with the duration of the downpour, or in other words, very heavy showers do not last as long as rains of lesser intensity. Intensity is expressed in terms of inches per hour; that is, if 1 in. of rain falls in 20 min., the rate or intensity of precipitation is 3 in. per hour.

By careful measurements of a gage chart the rates of precipitation for such periods as 5, 10 and 15 min. during the storm can be ascertained. When such records are available for a considerable period of years they furnish a fair indication of the intensities which may be expected on the average during a term of years approximately equal to the period of record. They may not, however, cover the extreme conditions likely to occur or they may include a great storm due to occur only once in a much longer period.

If records are available for 30 years, a curve drawn through all the points of maximum precipitation, suitably platted, will represent the maximum limit of intensity which may be expected once in 30 years on the average, subject to the qualifications just stated.

In a similar manner a curve may be drawn through the points representing the intensities which may be equalled once and exceeded once in 30 years on the average. Such a curve may be considered as representing storms having a frequency of 15 years. A sewer system having just sufficient capacity to carry away the run-off from storms of the intensities represented by a 15-year curve should prove inadequate but once in 30 years on the average. This lack of capacity, however, may occur during several storms, for the limit of rainfall intensity not causing flooding in one sewer district may be exceeded by a downpour lasting only 10 min., while such a storm would cause no flooding in another district where a 20-min. downpour at that rate would be necessary to overtax the sewer.

Curves showing intensities which may be equalled or exceeded but once in specific periods at the Chestnut Hill Reservoir (Boston) gage, based on records covering 38 years, are reproduced in Figs. 19 and 20 from a paper by one of the authors.[1] A similar probability curve which was prepared by the authors for Galves-

[1] *Jour.* Boston Soc. C. E., Feb., 1920, vol. 7, No. 2.

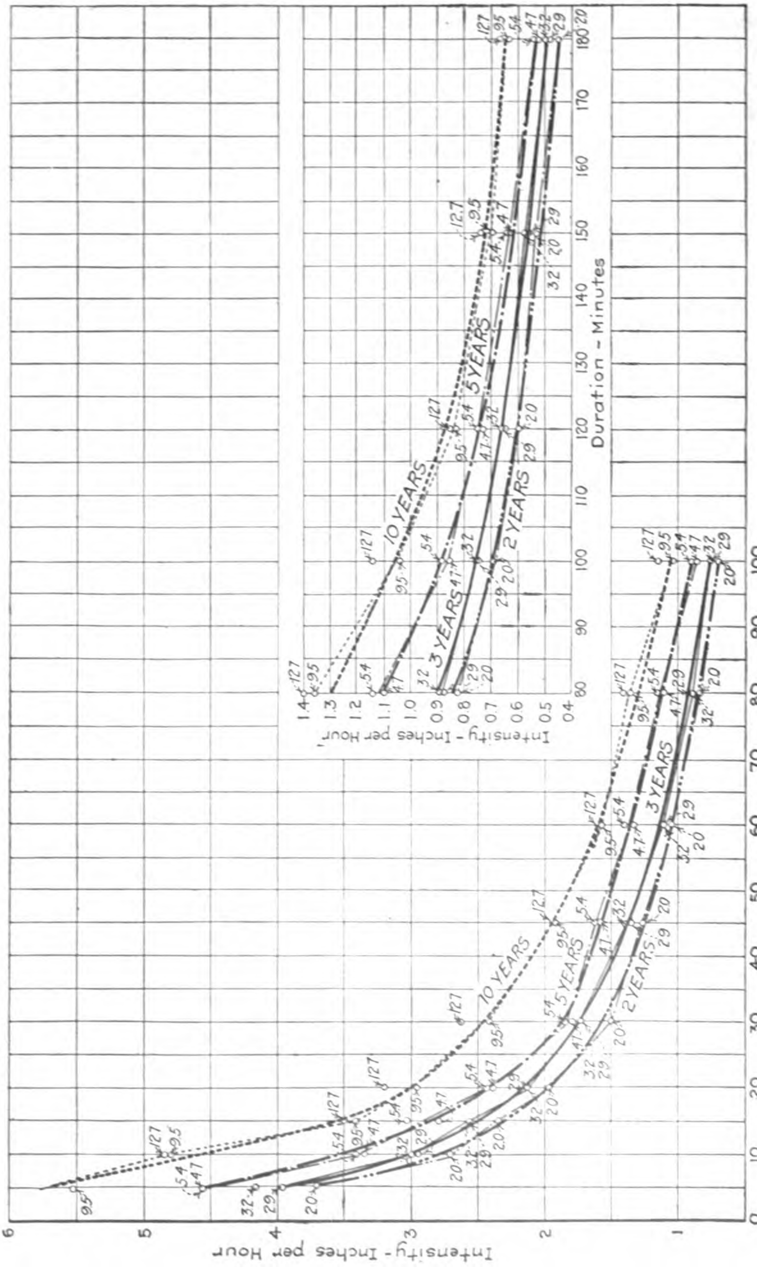


Fig. 19.—Rainfall curves for Chestnut Hill Reservoir (Boston), from 38-year record (1879–1916).

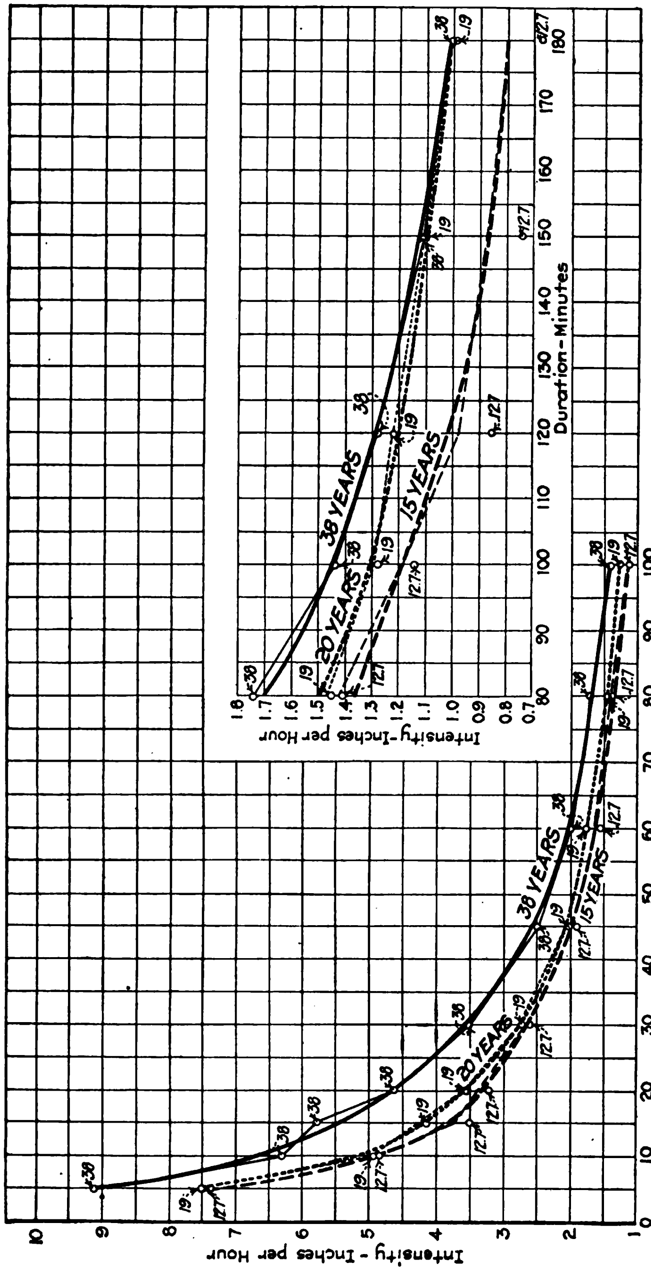


FIG. 20.—Rainfall curves for Chestnut Hill Reservoir (Boston), from 38-year record (1879–1916).

ton, Tex., based upon a 30-year record from 1890 to 1919, is shown in Fig. 21

Curves drawn through the points representing the maximum rates of precipitation are obviously irregular in form. From suitable data, however, approximate smooth curves can be drawn and many of them may be mathematically expressed. Such curves showing the relation between intensity and duration of precipitation for extreme and other storms at several places are reproduced in Figs. 22 to 26.

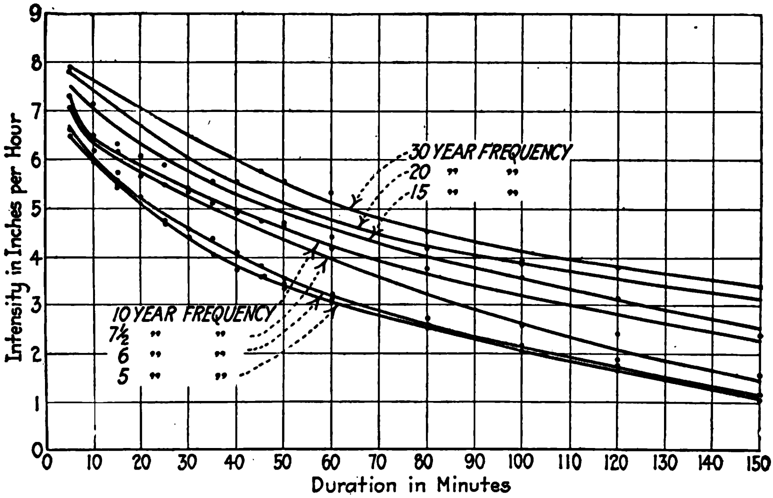


FIG. 21.—Rainfall curves for Galveston, Tex., from 30-year record (1890–1919).

The selection of the rainfall curve to be adopted as the basis for design of sewers in any community is a very important matter. While it would be desirable to provide drainage for the greatest storms of which there are local records, such a course will usually require very large expenditures which in many cases will be prohibitive, particularly if proper provision be made for increases in the proportion of impervious area likely to take place within the next 30 to 50 years in a rapidly growing community. It may be necessary, therefore, to reduce the allowances and one of the most logical ways in which this can be done is by the adoption of a rainfall curve having a lower frequency, as, for example, 15, 10, or even as low as 5 years. In making the selection, consideration should be given to the several economic conditions of the case, as the funds reasonably available for construction and

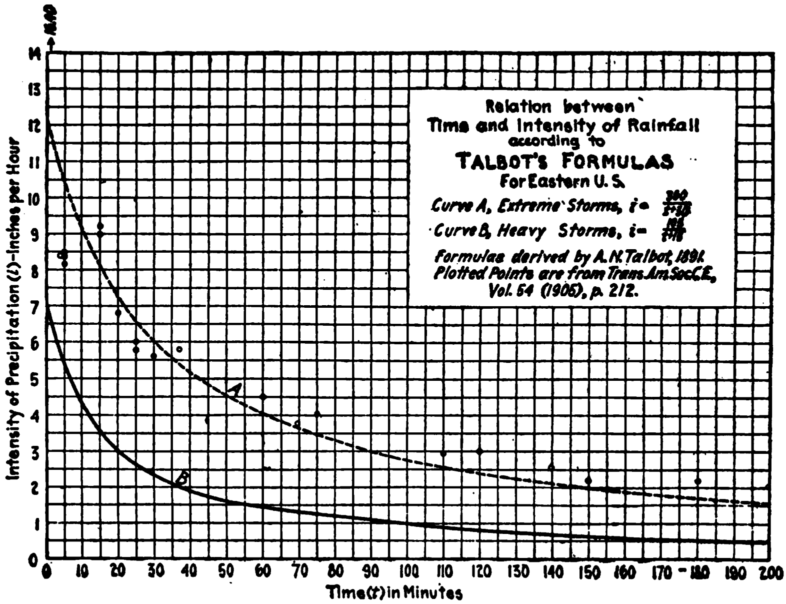


FIG. 22.—Talbot's intensity of rainfall curves.

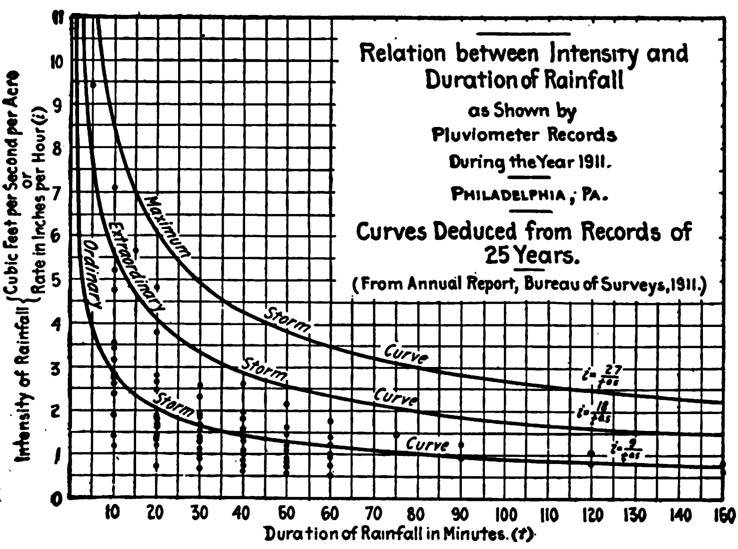


FIG. 23.—Philadelphia intensity of rainfall curves.

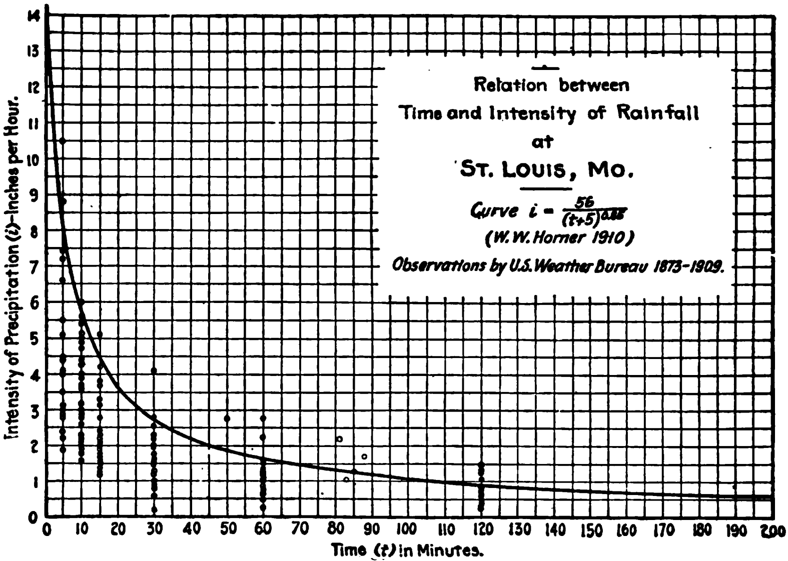


Fig. 24.—St. Louis intensity of rainfall curve. In 1916 a curve of somewhat higher intensities was adopted by the city for the design of drains.

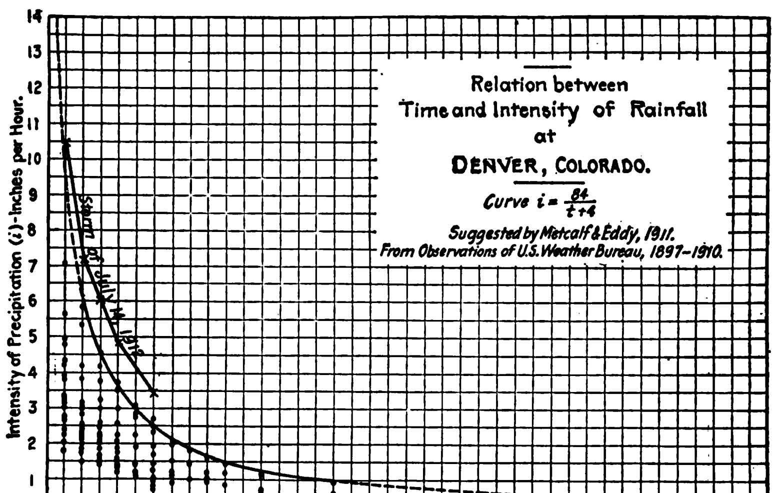


Fig. 25.—Denver intensity of rainfall curves.

the probable amount of damage and inconvenience caused by occasional flooding.

Form of Rainfall Curve.—Most equations of rainfall curves have been written in the form $i = a/(t + b)$. It usually expresses the actual observations with a fair degree of accuracy for rains of 10 min. to 2 hr. duration. Beyond these time limits, the results are generally too low. In many cases the exponential equation, $i = a/t^b$ has been found accurate. The exponent b is usually between 0.5 and 0.7. The best practice is probably to use a curve of rainfall plotted from actual records,

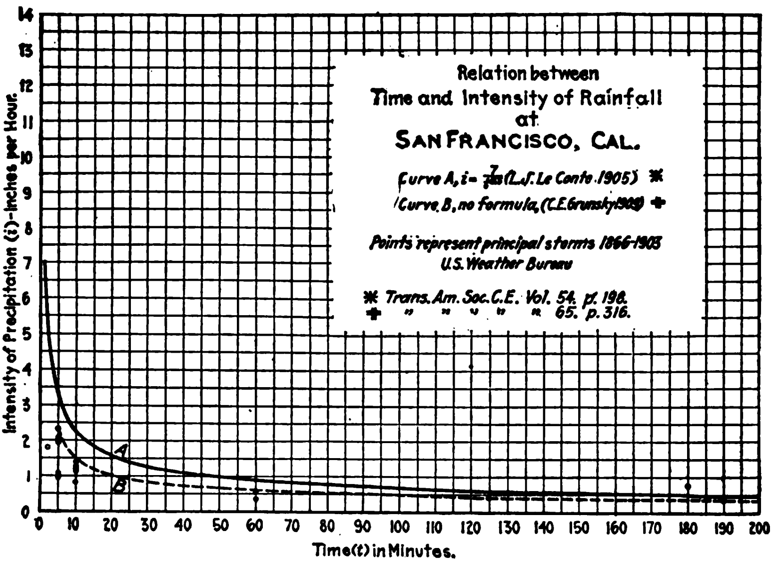


FIG. 26.—San Francisco intensity of rainfall curves.

without attempting to express it by an equation. The intensity to be expected for any given duration of time would then be taken directly from the curve, instead of being obtained by solving an equation. There is no apparent reason why the relation between the intensity and duration should follow any mathematical law. Table 8 shows how the results of different formulas differ.

Phenomenal Rainstorms.—Storms of extreme intensity, commonly called "cloud-bursts," are occasionally experienced. They are usually of such rare occurrence as to be classed as "Acts of God," for which it would not be reasonable to provide

TABLE 8.—COMPARISON OF RAINFALL CURVES

Locality	Formula	Proposed by	Conditions represented by formula	Time t in minutes: 5	10	15	20	30	45	60	90	120	180	240	300	600
				Intensity of precipitation i, inches per hour												
Eastern U. S.	$i = \frac{360}{t+30}$	Talbot	Maximum	10.29	9.00	8.00	7.20	6.00	4.80	4.00	3.00	2.40	1.72	1.33	1.09	0.57
Eastern U. S.	$i = \frac{105}{t+15}$	Talbot	Ordinary	5.25	4.20	3.50	3.00	2.33	1.75	1.40	1.00	0.78	0.54	0.41	0.33	0.17
Boston, Mass.	$i = \frac{150}{t+30}$	Dorr	Basis of design	4.28	3.75	3.33	3.00	2.50	2.00	1.67	1.25	1.00	0.71	0.56	0.45	0.24
Boston, Mass.	$i = \frac{106}{t+13}$	Kuichling	Basis of design	5.89	4.61	3.79	3.21	2.46	1.83	1.44	1.03	0.80	0.55	0.42	0.34	0.17
Boston, Mass.	$i = \frac{120}{t+20}$	Kuichling	Basis of design	4.80	4.00	3.43	3.00	2.40	1.85	1.50	1.09	0.86	0.60	0.46	0.38	0.19
Boston, Mass.	$i = 38.64 \div t^{0.687}$	Sherman	Maximum	12.80	7.95	6.01	4.94	3.74	2.82	2.31	1.75	1.44	1.09	0.90	0.75	0.48
Boston, Mass.	$i = 25.12 \div t^{0.687}$	Sherman	Basis of design (max.)	8.32	5.17	3.91	3.21	2.43	1.84	1.51	1.14	0.93	0.71	0.60	0.50	0.31
Boston, Mass.	$i = 15.5 \div t^{0.5}$	Metcalf & Eddy	Basis of design (max.)	6.93	4.90	4.00	3.47	2.83	2.31	2.00	1.63	1.41	1.16	1.00	0.90	0.63
Philadelphia, Pa.	$i = 27 \div t^{0.5}$	Bureau of Surveys	Maximum	12.07	8.54	6.99	6.04	4.94	4.02	3.49	2.85	2.46	2.01	1.74	1.56	1.10
Philadelphia, Pa.	$i = 18 \div t^{0.5}$	Bureau of Surveys	High	8.05	5.69	4.65	4.03	3.29	2.68	2.32	1.90	1.64	1.34	1.16	1.04	0.73
Philadelphia, Pa.	$i = 9 \div t^{0.5}$	Bureau of Surveys	Ordinary	4.02	2.84	2.32	2.01	1.64	1.34	1.16	0.95	0.82	0.67	0.58	0.52	0.37
Baltimore, Md.	$i = \frac{300}{t+25}$	Hendrick	Maximum	10.00	8.58	7.50	6.67	5.47	4.29	3.54	2.61	2.07	1.46	1.12	0.90	0.48
Baltimore, Md.	$i = \frac{105}{t+10}$	Hendrick	Basis of design	7.00	5.25	4.20	3.50	2.63	1.91	1.50	1.05	0.81	0.58	0.42	0.34	0.17
Savannah, Ga.	$i = \frac{191}{t+19}$	de Bruyn-Kops	Maximum	7.95	6.58	5.61	4.89	3.89	2.98	2.42	1.75	1.38	0.96	0.74	0.60	0.31
Savannah, Ga.	$i = \frac{163}{t+27}$	de Bruyn-Kops	Once in 2 years	5.10	4.41	3.88	3.47	2.86	2.27	1.88	1.40	1.11	0.79	0.61	0.50	0.26
Savannah, Ga.	$i = \frac{141}{t+27}$	de Bruyn-Kops	Once a year	4.41	3.82	3.36	3.00	2.47	1.96	1.63	1.21	0.96	0.68	0.53	0.43	0.23
Chicago, Ill.	$i = \frac{120}{t+15}$	Hill		6.00	4.80	4.00	3.42	2.67	2.00	1.60	1.14	0.89	0.62	0.51	0.38	0.20
Louisville, Ky.	$i = 14 \div t^{0.5}$	Metcalf & Eddy		6.26	4.42	3.61	3.13	2.56	2.08	1.81	1.48	1.28	1.04	0.91	0.81	0.57
New Orleans, La.	$i = 19 \div t^{0.5}$	Metcalf & Eddy		8.50	6.00	4.91	4.25	3.47	2.83	2.45	2.00	1.73	1.42	1.23	1.09	0.78
St. Louis, Mo.	$i = 56 \div (t+5)^{0.85}$	Horner		8.00	5.60	4.39	3.63	2.73	2.01	1.61	1.17	0.93	0.66	0.47	0.39	0.24
Denver, Colo.	$i = 84 \div (t+4)$	Metcalf & Eddy		9.33	6.00	4.42	3.50	2.47	1.71	1.31	0.90	0.68	0.46	0.34	0.28	0.14
San Francisco, Cal.	$i = 3.68 \div \left[\frac{2t}{t+60} + t^{0.4}\right]$	Grunsky		1.79	1.31	1.10	0.97	0.81	0.68	0.60	0.51	0.45	0.38	0.34	0.32	0.25
San Francisco, Cal.	$i = 5 \div t^{0.5}$	Metcalf & Eddy		2.24	1.58	1.29	1.12	0.91	0.75	0.65	0.53	0.46	0.37	0.32	0.29	0.20
Spokane, Wash.	$i = 23.92 \div (t+2.15) \div 0.154$	Brackenbury		3.49	2.12	1.55	1.24	0.90	0.66	0.54	0.41	0.35	0.29	0.25	0.23	0.19
Berlin, Germany	$i = \frac{25.6}{t} + 0.61$	Knauff		5.73	3.17	2.31	1.89	1.46	1.18	1.04	0.89	0.82	0.75	0.72	0.70	0.65
General formulas	$i = 32 \div t^{0.8}$	Gregory		8.86	5.07	3.67	2.92	2.10	1.52	1.21	0.87	0.69	0.50	0.40	0.33	0.19
	$i = 8 \div t^{0.5}$	Metcalf & Eddy		3.58	2.53	2.06	1.79	1.46	1.19	1.03	0.84	0.73	0.60	0.52	0.46	0.33
	$i = 10 \div t^{0.5}$	Metcalf & Eddy		4.47	3.16	2.58	2.23	1.82	1.49	1.29	1.05	0.91	0.75	0.65	0.58	0.41
	$i = 12 \div t^{0.5}$	Metcalf & Eddy		5.36	3.80	3.10	2.68	2.19	1.79	1.55	1.26	1.09	0.90	0.78	0.69	0.49
	$i = 15 \div t^{0.5}$	Metcalf & Eddy		6.71	4.75	3.88	3.36	2.74	2.24	1.94	1.58	1.37	1.12	0.97	0.87	0.61

in designing storm drains. They are not to be confused, however, with special topographical conditions, more particularly, in the mountainous country, producing records abnormally high according to usual standards but not unusual for the specific locality under consideration. These cases are rare, however.

During 1913 New York City experienced four storms, in all of which the intensity of precipitation, practically throughout the storm, was greater than that given by the equation $i = 15/t^{0.5}$. The significant facts relative to these storms and the intensities obtained by this formula are given in Table 9.

TABLE 9.—PHENOMENAL RAINFALL IN NEW YORK CITY, 1913

Date	July 10	July 28	Sept. 5	Oct. 1	$i = \frac{15}{\sqrt{t}}$
Place	100 Broadway	Central Park	Central Park	Richmond	
t-minutes	Intensity i-inches per hour				
1	....	....	....	8.40	15.00
2	....	....	....	8.10	10.60
4	....	....	....	6.45	7.50
5	9.88	6.12	7.20	....	6.72
7	....	....	....	6.24	5.68
10	7.56	5.76	6.90	5.64	4.75
15	6.52	4.80	6.36	5.16	3.88
19	....	....	....	5.05	3.45
30	4.18	2.96	5.24	....	2.74
37	....	....	....	4.84	2.47
49	....	....	....	4.75	2.15
59	....	....	....	4.44	1.95
60	2.30	2.73	3.31	....	1.94
85	....	....	....	3.80	1.63
106	....	....	....	3.37	1.46
120	1.28	1.56	1.85	....	1.37
123	....	....	....	3.06	1.35

Rational Method of Estimating Run-off.—Any method of estimating run-off requires the engineer to rely upon his judgment. No two engineers working independently are likely to reach precisely the same conclusions regarding the extent of the improvement of a drainage district within the economic period for which drains should be built now, the rate of rainfall for which drainage should be provided, and the rate at which storm water will reach the sewers.

The rational method of estimating run-off has come into favor because it enables the engineer to apply judgment directly to specific allowances which are subject to analysis, measurement and estimate. It requires him to exercise judgment logically, after an analysis of the local conditions. The rational method is based on the direct relation between the rainfall and run-off expressed by $Q = CiA$, in which Q = the run-off in cubic feet per second from a given area; C = a coefficient representing the ratio of run-off to rainfall, generally called "run-off coefficient;" i = the intensity of rainfall in cubic feet per second per acre (or nearly enough, its approximate equivalent, the rate of rainfall in inches per hour); A = the drainage area in acres. The engineer can measure A, but he must determine the proper values of the other factors. The value of i to be used depends not only on the curves of the intensity of rainfall which fit the local conditions and the assumed period of recurrence, but also on the "time of concentration" it takes water to flow from the most distant point in the district to the nearest sewer inlet and thence through the sewers to the point in the sewerage system for which the maximum quantity of flow is to be estimated, as a basis for design of the sewer section or size. It is the greatest uniform intensity of rainfall during this time of concentration for which provision must be made, not the intensity for a shorter period. The value of C must be estimated from a study of the soil, slope and character of the surface, and a consideration of probable future development.

Time of Concentration.—Time of concentration is made up of "inlet time," the period consumed by water in flowing from the most distant point in the drainage area to the sewer, and "time of flow" in the sewer. The latter is readily estimated by hydraulic computations explained in Chapter IV. Inlet time is frequently the most important factor in determining the probable run-off, particularly in small districts or in fairly large districts with steep slopes. In cities it is seldom less than 3 or more than 20 min. Horner's investigations[1] led him to conclude that along improved streets with grades of 0.5 to 5 per cent, the water from streets, sidewalks and roofs will reach the sewer in 2 to 5 min., but the velocity over grass plots is so low that even in heavy rains water will take 10 to 20 min. in flowing 100 ft.

The sewers are designed to meet the most serious conditions which are anticipated during the economic period of service for which they are planned. They are assumed in the designing work to run full, with the velocity of flow practically a maximum, and the run-off from the roofs and streets and the flow in gutters

[1] *Eng. News*, Sept. 29, 1910.

at maximum rates, giving a minimum time of concentration which is a constant for a given sewer district in a particular state of development.

When gagings of the storm-water flow are made, the conditions are usually different from those assumed in designing, and it is the actual time of concentration which fixes the period of rainfall with which the resulting flow must be compared. During a moderate storm, the sewer may be but partly filled and the velocity of flow may, therefore, be considerably less than the maximum. Moreover, unless rain has previously been falling for some time, the filling of depressions and the accumulation of sufficient head to cause flow over rough or nearly flat surfaces will require an appreciable amount of time. The actual time of concentration will, therefore, exceed the minimum in all cases except those which produce the same or a greater rate of run-off than that for which the sewer was designed.

Run-off Factor.—The run-off factor depends upon a large number of elements and is not a constant for a given area, even during a single storm. It is seldom unity, even when the entire surface is covered with roofs and pavements, for some evaporation takes place, even during a storm, nominally impervious surfaces absorb some moisture, and irregularities of the surface tend to hold back some of the water. The run-off factor gradually increases for some time after the beginning of a rain until the soil has become thoroughly saturated, the impervious surfaces thoroughly wetted and the depressions filled. After that time the factor remains substantially constant for a given area. It therefore makes considerable difference in the amount of run-off whether the critical precipitation comes near the beginning of a storm or after rain has been falling for some time. Prof. A. J. Henry of the U. S. Weather Bureau prepared[1] Table 10 which is helpful in this connection by showing the percentage of cases of downpour in Washington, Savannah and St. Louis, in which the maximum rate of precipitation occurred within various periods after the beginning of the storm. If a warm rain falls when the surface is covered with ice, the run-off factor may even exceed unity.

It is evident that an exact determination of the run-off factor for conditions which will exist in the future is not possible.

[1] *Bulletin D*, "Rainfall in the United States" U. S. Weather Bureau, *Jour.* West. Soc. Engrs., April, 1899.

The engineer's problem is to forecast to the best of his ability the changes which will take place in the district and to estimate the rate of precipitation for which he must provide.

When rainfall records from which to construct a curve of the rate of precipitation in the city where the sewers are to be built are not available, the formula $i = 12 \div t^{0.5}$ will give intensities unlikely to be exceeded more often than once in 10 years in New England and New York. The formula $i = 15/t^{0.5}$ will give the probable maximum rate of precipitation during a period of about 15 years and Table 11 has been computed by its use. In this table the periods of time are not necessarily measured from the beginning of a storm or even from the beginning of the downpour.

TABLE 10.—PERCENTAGE OF CASES IN WHICH THE MAXIMUM INTENSITY OF PRECIPITATION OCCURRED WITHIN VARIOUS PERIODS FROM THE BEGINNING OF THE STORM

Minutes after beginning of storm	Per cent of cases in which maximum intensity occurred within period at		
	Washington	Savannah	St. Louis
5	17	10	31
10	38	31	61
15	59	52	69
20	64	65	74
25	72	72	76
30	81	82	78
35	86	87	80
40	91	88	88
45	93	92	93
50	94	97	98
60	100	100	100

TABLE 11.—QUANTITY IN INCHES OF RAIN FALLING IN THE SPECIFIED PERIODS OF TIME AT THE RATES INDICATED BY CURVE OF INTENSITIES, $i = 15/t^{0.5}$

Time, minutes	Rate of precipitation, inches per hour	Accumulated depth of precipitation, inches
5	6.71	0.56
10	4.75	0.79
15	3.88	0.97
20	3.36	1.12
30	2.75	1.38
45	2.24	1.68
60	1.94	1.94
90	1.58	2.37
120	1.37	2.74

In making an allowance for the effect of the rate of precipitation on the run-off factor, it must be kept in mind that in most cases the maximum rate of precipitation does not occur over a large area, and it may be proper in planning a large drainage system to use a lower rate of precipitation than should be used for a smaller area.

In a district where the duration of rainfalls is short, the run-off coefficient may not be so high as where the rainfalls are protracted, so that the depressions in the surface become filled with water, the earth becomes saturated and the conditions are such that a larger proportion of the rainfall runs off than is the case with shorter rainfalls.

The duration of storms has another effect on run-off, for if the duration of a storm causing flood conditions is less than the time required for water to flow from the most distant point on the drainage area to the point for which computations are made or at which gagings are made, then the maximum discharge will come when less than the entire drainage area is contributing water. This condition need rarely be considered in preparing designs, and then only for very large drainage areas, but it must be considered in studying gagings of the storm flow in sewers and drains.

There is some retardation in run-off due to the time required to fill gutters and sewers and build up sufficient head to carry off the water in the drains as fast as it falls upon the tributary area. In designing sewers, it is usually best to take no account of this storage capacity, but it must be considered in studying and comparing gagings of storm flow in sewers with rainfall records.

Values of Run-off Factor.—There will inevitably be a difference in the influence on the run-off factor which different engineers attribute to each of the conditions mentioned in the last section. These differences are shown in Table 12.

Other authorities do not attempt to make close estimates of the different kinds of surface in an urban district, but content themselves with average values of the proportional run-off, as follows:[1]

For the most densely built-up portion of the district.	0.70 to 0.90
For the adjoining well built-up portions............	0.50 to 0.70
For the residential portions with detached houses...	0.25 to 0.50
For the suburban portions, with few buildings......	0.10 to 0.25

[1] See also Horner's assumptions for St. Louis, Mo., shown in Table 13 following. The authors have used similar figures modified to meet special conditions and have developed in some cases a zone system.

TABLE 12.—RANGE IN ESTIMATES OF RUN-OFF FROM DIFFERENT CLASSES OF SURFACE IN PROPORTION TO THE RAINFALL INTENSITY

From Bryant and Kuichling's Report on the Adequacy of the Present Sewerage System of the Back Bay District of Boston, etc., 1909

For water-tight roof surfaces	0.70 to 0.95
For asphalt pavements in good order	0.85 to 0.90
For stone, brick and wooden block pavements with tightly cemented joints	0.75 to 0.85
For same with open or uncemented joints	0.50 to 0.70
For inferior block pavements with open joints	0.40 to 0.50
For macadamized roadways	0.25 to 0.60
For gravel roadways and walks	0.15 to 0.30
For unpaved surfaces, railroad yards and vacant lots.	0.10 to 0.30
For parks, gardens, lawns and meadows, depending on surface slope and character of subsoil	0.05 to 0.25

In general, in the absence of suitable information from which to estimate directly the run-off factor for a given area under conditions assumed to exist at the end of the "economic period of design," this factor may be most satisfactorily approximated by estimating the "equivalent percentage of totally impervious area," as it is sometimes called. Thus, if it is assumed that in the future 15 per cent of the area of the district will be covered by roofs for which the coefficient would be 0.95, 30 per cent by pavements with coefficient 0.90, 40 per cent by lawns with coefficient 0.15, 15 per cent by gardens with coefficient 0.10, the resulting coefficient for the entire district will be 0.4875, or, in round numbers, 0.50.

Relative Adequacy of Laterals and Mains.—Much more damage will generally result from the flooding of a main sewer than from the inadequacy of one submain or lateral; yet the damage from overtaxing the capacity of a large number of submains may be much more serious than that from overtaxing the main sewer, particularly if the latter is provided with storm relief overflows. Moreover, it is simpler and usually less expensive to reinforce a main sewer than to rebuild many small laterals. The additional cost of constructing the latter of ample size when first built will generally be inconsiderable, while the additional cost of a main sewer large enough to care for the run-off from the most severe storms many years hence, after the ground area has been made much more impervious by pavements, sidewalks and many more buildings, may be prohibitive. It is generally advisable, therefore, to build lateral sewers large enough for the ultimate requirements, giving the mains and submains a sufficient capacity for the probable run-off during some years, with the expectation that relief sewers will eventually be required to care adequately for the entire run-off from the district.

SOME EXAMPLES OF THE USE OF THE RATIONAL METHOD

The procedure[1] followed in using the rational method of design for storm-water drains at St. Louis is as follows:

The problem was to design a drainage system in a residence district in a new subdivision. An average half city block there measures 172½ ft. from the center of the street to the center of the alley and 860 ft. between the centers of cross-streets, giving a total area of 3.38 acres, and its population is about 40 per acre. The impervious portion of this half block will be:

Streets, 20,000 sq ft., 13.7 per cent of whole area.
Alleys, 6,500 sq. ft., 4.5 per cent of whole area.
Sidewalks, 6,500 sq. ft., 4.5 per cent of whole area.
House roofs, 27,500 sq. ft., 18.9 per cent of whole area.
Shed roofs, 4,000 sq. ft., 2.7 per cent of whole area.
Yard walks, 2,000 sq. ft., 1.4 per cent of whole area.
Total impervious area, 1.35 acres, 45.7 per cent of whole area.

The rate of precipitation for which drains were designed in St. Louis in 1910 is given in Fig. 24. More recently a curve giving higher intensities has been used. For a rain of 10 min. duration it is assumed that 60 per cent of the water falling on impervious surfaces and 20 per cent of that falling on lawns and hard ground will run off. For rains of longer duration, it is assumed that these percentages will increase as shown in Table 13. These

TABLE 13.—ASSUMED PERCENTAGES OF RUN-OFF FOR ILLUSTRATION OF RATIONAL METHOD

Duration t in minutes	Per cent run-off from		Coefficient C
	Impervious portion	Pervious portion	
10	60	20	0.40
15	70	30	0.50
20	80	35	0.55
30	85	40	0.60
60	95	50	0.70
120	95	60	0.75

percentages of run-off are based on the assumption that the critical rainfall of the assumed duration occurs at the beginning of a storm, before the surface has been thoroughly saturated. This is not always true, particularly for the shorter periods, and a somewhat safer basis for many cases would be to make no reduction in the coefficient for the shorter times of concen-

[1] A more detailed explanation of this procedure is given in "American Sewerage Practice," Vol. 1, p. 275–287, amplified from an article by W. W. Horner in *Eng. News*, Sept. 29, 1910.

tration. From these data the run-off curve, Fig. 27, is plotted to give the value of Ci for any time of concentration. This curve is only applicable to the conditions in this particular residential district.

The run-off curve is constructed by taking the values of C as found in Table 13 and multiplying each by the value of i for the corresponding duration as given in Fig. 24. The results are plotted as ordinates in cubic feet per second per acre to the corresponding durations in minutes as abscissas. For example, the coefficient of run-off, C, for 30 min. duration is 0.60 and the intensity of precipitation, i, is 2.75. Then the rate of run-off in cubic feet per second per acre is $Ci = 0.60 \times 2.75 = 1.65$. Records of the future may make possible the introduction of a factor to cover also the effect upon the intensity of rainfall and rate of run-off of the relative extent of the area. Available data are too meagre to permit this now.

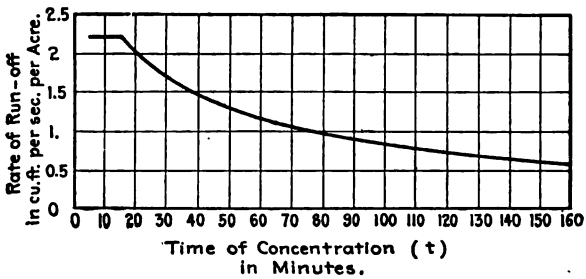
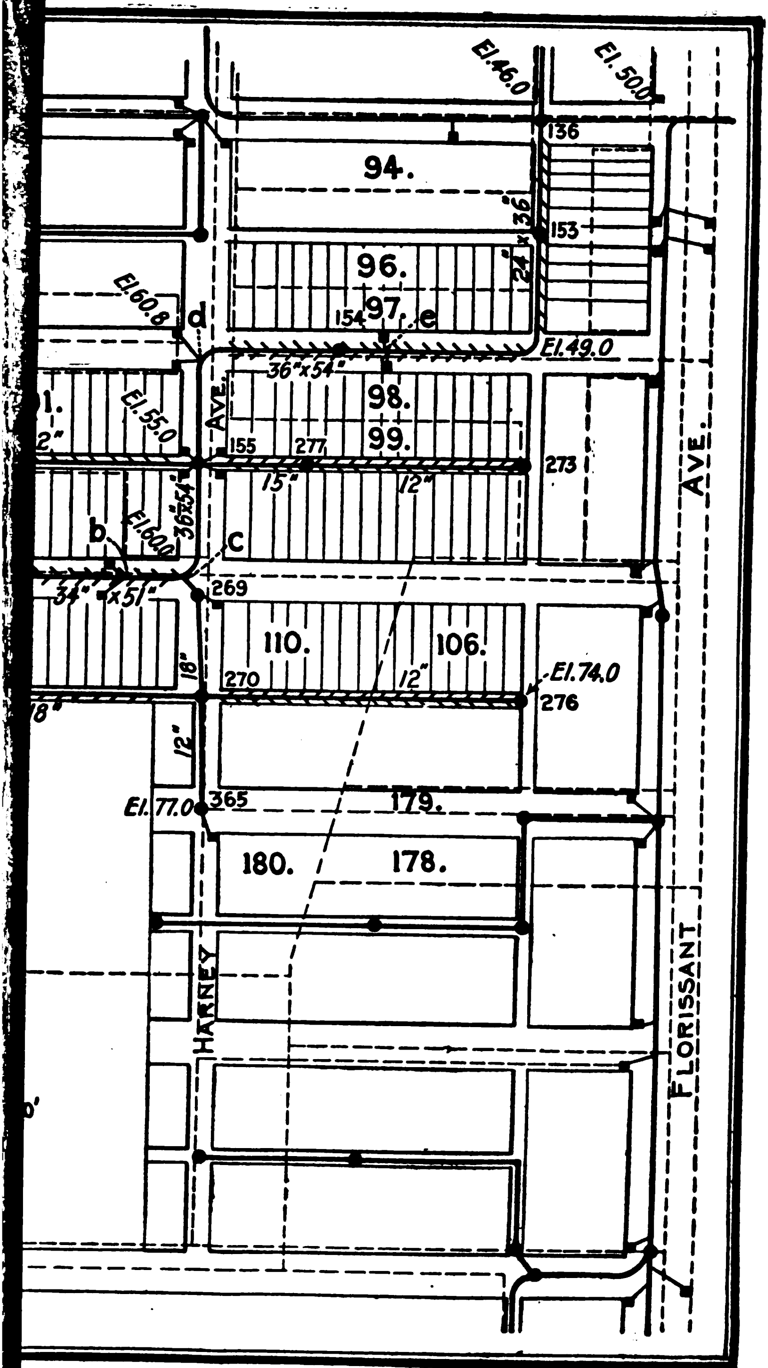


FIG. 27.—Rate of run-off for St. Louis residence districts.

The remaining data for the computations must be taken from an accurate plat of the district, similar to Fig. 28. On this are entered the elevations of the proposed or established street and alley grades and, if no contour map has been made, the existing surface elevations should also be shown. The storm water inlets along the gutters of the streets and alleys are then located on the plat on the higher side of all street intersections and at all low points between streets, with no interval greater than 600 or 700 ft. between inlets. After the location of the inlets has been fixed on the map, sewers to reach them are laid out, attention being paid to the sewerage of all the private lots in the district. The most economical layout usually follows the natural surface slopes in the shortest line toward the outlet of the district and concentrates the storm flow as rapidly as possible. Sometimes several preliminary layouts should be made and compared to ascertain which is the cheapest.

It is now comparatively easy to calculate the area tributary to the sewer at any point. The designer must form a mental picture of the district as it will be at the end of the period of economic construction, with the grading and paving done and buildings erected. This concept is necessary in order to locate the minor ridge lines dividing the small areas draining into streets from those draining into alleys, and to fix the areas tributary to each inlet.

The final step in preparing the data is to fix in a preliminary way the grades of the sewers. The start in this work of approximating grades is made at the lower end, where the elevation is fixed approximately by



(Facing Page 74)

outside conditions. Then in the second trial, beginning at the upper end, the final grades can be established at the same time the sizes are determined.

A simple problem from the authors' work will be used to illustrate the various steps taken in the computations.

Hypothesis.—The district shown in Fig. 29 is the same as that used in the problem outlined in the previous Chapter on "Separate Sewers, *viz.;* a portion of the Carlisle Brook drainage area in the City of Springfield, Mass. This district has been selected as a basis of study because it is part of an actual problem and is of interest on that account. Some modifications, however, have been made to simplify the conditions. The location of the proposed main drain (the so-called "Carlisle Brook drain") is shown on the plan and the invert elevation is given at the point where the proposed branch drain is to be connected and for which provision has been made in the design of the main drain.

The general scheme of drainage of this portion of the city provides for the deepening and improving of the channel of Carlisle Broook, so that for a few years it will serve to carry away the storm water run-off from this drainage area. For this reason the branch drain must be constructed at a sufficient elevation to enable it to discharge into the improved brook channel and also be capable of extension to the Carlisle Brook drain on the further side of the brook, when that drain shall have been constructed. The required minimum elevation of the invert of the branch drain is, therefore, given at the proposed point of discharge into the Carlisle Brook drain.

A careful study of local conditions, including the present and probable future development of the district, indicates that a coefficient of imperviousness of 40 per cent is a reasonable allowance, and that it is reasonable to assume a coefficient of run-off of $C = 0.35$.

The time of entrance, that is, the time required for the water to flow from the most distant point of the drainage area to the upper end of each drain, has been assumed to be 20 min.

The rate of rainfall is to be taken from the assumed curve of intensity of precipitation represented by the formula $i = 20.4/t^{0.61}$ where i is the intensity of precipitation in inches per hour and t is the time in minutes. The formula indicates the rate of rainfall which may be expected to be equalled or exceeded once in every 5 years. It resulted from a careful analysis of the rainfall records. While it was recognized that drains designed on this basis might be overtaxed on the average once in 5 years, in view of other financial obligations facing it, the city was not thought to be justified in going to the extent of providing for storms of greater intensity, which would have involved greater cost. During the earlier years of the life of the drains they will be able to care for much greater rates of flow than later, because the assumed coefficient of run-off is based upon future rather than present conditions. A progressive increase in impervious surface and rate of run-off is caused by the gradual substitution of roofs and paved areas for woods, grasslands, etc., usually referred to as unimproved areas, as the city grows. In the future when the district is more densely built up and when the city has available funds, relief drains can be constructed, to serve those areas where flooding has become sufficiently serious to warrant the expenditure.

Fig. 29 shows the streets with surface contours, also the drainage area, within the dash and dot lines.. The limits of this area are influenced not only by the surface contours but also markedly by the areas served by existing combined sewers and drains not shown on the plan.

The drains are to be designed in general with the crown at a depth of at least 5 ft. below the surface of the street.

The minimum size of drain is to be 12 in.

The assumed minimum allowable velocity is 3 ft. per second when flowing full.

For determining the capacity of drains, use a value of $n = 0.015$ in the Kutter formula for sizes of 24 in. and under and $n = 0.013$ for sizes over 24 in.; it being assumed that the smaller sizes will be of vitrified pipe and

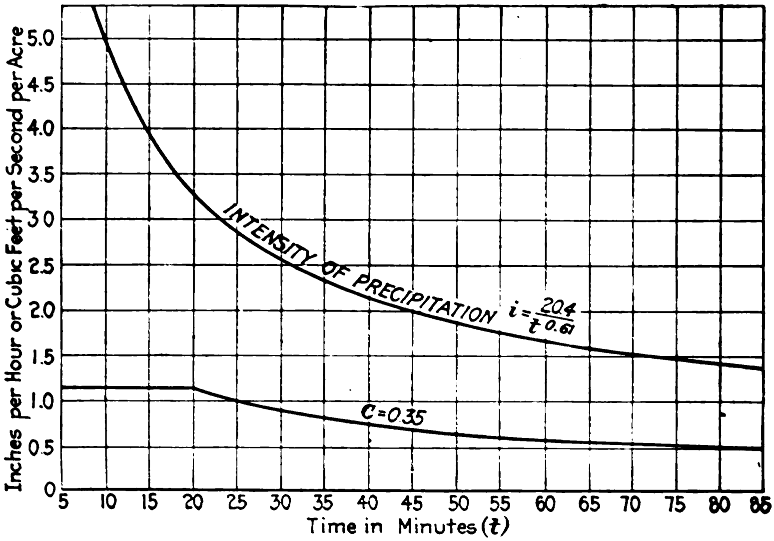


Fig. 30.—Rainfall and run-off curve (problem in combined sewer design).

the larger sizes of concrete with smooth interior surfaces. If other materials are to be used, the value of n should be fixed accordingly.

Solution of Problem.—Having determined the rainfall intensity curve, the probable future percentage of impervious surface and the corresponding run-off coefficient (see "Hypothesis"), a run-off curve is constructed as previously described from which the values of Ci can be easily read (Fig. 30).

The following steps must now be taken in order to reach a rational solution.

First.—Draw a line to represent the drain in each street or alley to be drained. Place an arrow near each drain to show the direction of flow in it. The drains should, in general, slope with the street surface. However, it will usually prove to be more economical to so lay out the system that the water will reach the main drain by the most direct route. Also it will, in general, prove best to concentrate the flow from small areas as quickly as possible into one drain.

In some localities with a large percentage of pervious surface, the roof water is allowed to discharge upon the ground, thence flowing to the gutter inlets and the drains. In some such instances drains are provided only to the last gutter inlet, rather than to a point opposite the last house lot, thereby effecting some saving in cost. This practice is not recommended, however, as it does not give equal privilege to all property and is, therefore, undesirable. In the district under consideration it is intended to provide drainage facilities for all property within the area designated.

Second.—Locate the manholes tentatively, giving to each an identification number. There should be a manhole at each bend or angle, at all junctions of drains and at all points of change in size or slope, and at intermediate points where the distance exceeds 400 ft., except where a good velocity will be available during practically all conditions of flow, in which case the interval may be increased to 700 or 800 ft. In any circumstance, sufficient manholes should be built to allow ready access for inspection and cleaning; later when the profiles are drawn and the final slopes fixed, it may be found desirable to change the location of some manholes in order to have the drains at the most advantageous depth, particularly where the slope of the street surface is not substantially uniform. Other considerations, such as obstacles under the street, may require the installation of additional manholes, due to change in alignment or special forms of construction involved in junctions or connections with other drains.

Third.—Sketch the limits of the tributary drainage areas at each manhole. The assumed character of future development and the topography will determine the proper limits.

Fourth.—Measure each individual area by planimeter or other method which will give equally satisfactory results.

Fifth.—Prepare a table in which to record the data and steps in the computations for each section of drain between manholes (see Table 14).

Column 1 identifies the lines in the table.

Columns 2 and 3 contain the identification numbers of the manholes.

Column 4, the street name in which the drain is located;

Column 5, the length in feet of the drain between manholes.

Column 6, the area increment tributary to the section of drain under consideration.

Column 7, the summation of the area increments given in column 6, and is the cumulative area from which the run-off in the drain is computed.

Column 8, the time in minutes elapsed to the upper end of the section of drain. It is equal to the inlet time or time-of-entrance plus the time required for water to flow in the drains from the upper end to the point under consideration.

Column 9, the time increment or time required for passage of water through each individual stretch of drain. It is equal to the length divided by the velocity in feet per minute and cannot be computed until after the size and slope of the drain have been determined.

Column 10, the rate of run-off in cubic feet per second per acre. It is taken from the run-off curve, Fig. 30, for the duration corresponding to the elapsed time given in column 8.

Column 11, the maximum rate of run-off for the section of drain. It is the product of the total tributary area, column 7, by the rate of run-off, column 10.

Column 12, the slope of drain, feet of fall per foot of length.

Column 13, the velocity in feet per second for the drain when flowing full.

Column 14, the internal diameter of the drain, in inches.

Column 15, the capacity in cubic feet per second corresponding to the slope given in column 12. It should at least equal the maximum rate of run-off shown in column 11. It may exceed it because of the limitation as to minimum velocity, minimum sewer diameter adopted, etc.

Column 16, the elevation of ground surface at the upper end of the section.

Column 17, the fall in feet between manholes. It is equal to the product of the length shown in column 5 and the slope, in column 12.

Columns 18 and 19, the invert elevations of the drain at the upper and lower manholes respectively.

As the velocities are low in this drain no special provision has been made for velocity head. The head required is negligible in amount and is in a measure allowed for in the design, since the required capacity of each section of drain is computed on the basis of the total area tributary at its lower end, though at its upper end the flow is less than this, and consequently there will not be required quite so steep a slope as that for which the section is designed.

TABLE 14.—METHOD OF TABULATION OF DATA AND COM-

Line	Manhole		Location, street	Length, feet	Area in acres		Time elapsed in minutes		Run-off Ci from curve cubic feet per second per acre
	From	To			Increment	Total	To upper end of section	In section	
		3	4	5	6	7	8	9 = col. 5 ÷ (col. 13 × 60)	10
1	1	2	Dawes	300	2.28	2.28	20.0	1.7	1.15
2	2	3	Dawes	300	2.40	4.68	21.7	1.7	1.10
3	3	4	Dawes	300	2.15	6.83	23.4	1.7	1.05
4	4	5	Dawes	165	1.54	8.37	25.1	0.9	1.00
5	5	6	Andrew...... .	325	2.16	10.53	26.0	1.6	0.99
6	6	7	Burr...........	400	3.11	13.64	27.6	2.1	0.94
7	7	8	Burr	35	6.02	19.66	29.7	0.2	0.90
8	8	9	Burr....	230	10.14	29.80	29.9	1.1	0.90
9	9	10	Burr....... ...	240	5.65	35.45	31.0	1.0	0.88
10	10	11	Oak Grove Ave.	110	11.91	47.36	32.0	0.5	0 86
11	11	12	Oak Grove Ave.	95	11.02	58.38	32.5	0.4	0.85
12	12	13	Shattuck..... ..	295	5.60	63.98	32.9	1.1	0.84
13	13	14	Shattuck..... ..	260	19.73	83.71	34.0	1.0	0.82
14	14	15	Shattuck.......	145	11.30	95.01	35.0	0.6	0.81
15	15	16	Shattuck.......	225	2.88	97.89	35.6	0.8	0.80
16	16	17	Shattuck.......	380	13.07	110.96	36.4	1.3	0.79
17	17	18	Right-of-way...	165	4.16	115.12	37.7	...	0.77

The student may fill in the data of columns 2, 3, 4, 5, 6, 7 and 16, also the first line of columns 8, 10 and 11, from the principles and data given heretofore in this chapter.

Before proceeding further with the complete solution the student must study Chapter IV on "Hydraulics of Sewers."

For report or preliminary purposes, a profile of the main drain should be prepared showing the street surface, the drain, the manholes and the invert elevations at each manhole given in figures, also the names of streets and avenues where they intersect the line of the drain. It is advisable also to give the "station" of each manhole or the distance in feet between manholes.

Each lateral should be investigated to make certain that its elevation is such that when the main drain is running full, the carrying capacity of the lateral should not be reduced.

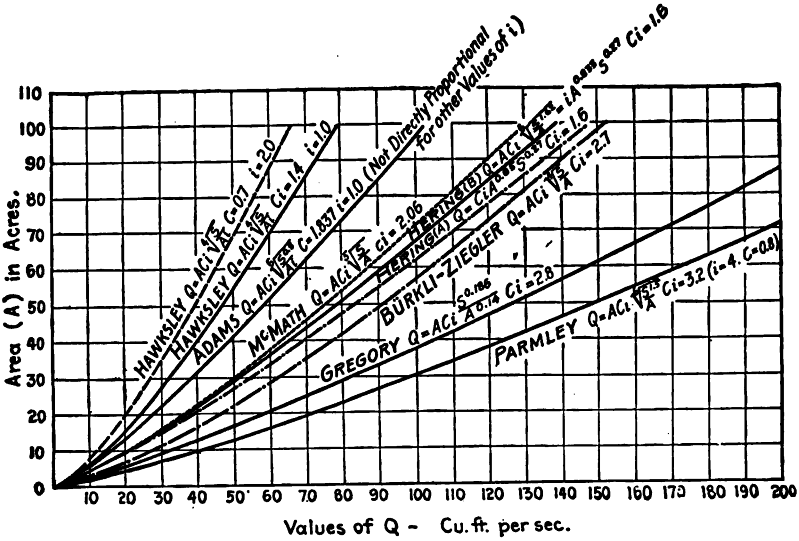
Each lateral is designed with reference to the requirements of the "hypotheses" and economy of its construction. Therefore, its design must be reviewed subsequently, in connection with that of the main drain, to make certain that it will, in fact, have the free discharge into the main drain presupposed in its design and contemplated in the "hypotheses" under conditions of maximum flow. If the assumed conditions of flow in the lateral cannot be realized under the designs, it will be necessary either to redesign the lateral or drop the main drain to an elevation which shall permit this realization.

PUTATIONS FOR STORM DRAINS AND COMBINED SEWERS

Maximum rate of run-off, cubic feet per second	Slope of drain, decimal of a foot per foot	Velocity, feet per second	Size internal diameter in inches	Capacity, cubic feet per second	Elevation of ground surface at upper end	Fall in feet	Elevation of invert at	
							Upper end	Lower end
11 = (cols. 7 × 10	12	13	14	15	16	17 = (cols. 5 × 12)	18	19
2.6	0.0049	3.0	15	3.7	206.2	1.47	199.85	198.38
5.1	0.0037	3.0	18	5.3	206.6	1.11	198.13	197.02
7.2	0.0028	3.0	22	7.9	206.6	0.84	196.69	195.85
8.4	0.0031	3.2	22	8.4	207.1	0.51	195.85	195.34
10.4	0.0029	3.3	24	10.4	207.4	0.94	195.17	194.23
12.8	0.0017	3.2	27	12.8	206.1	0.68	193.98	193.30
17.7	0.0018	3.6	30	17.7	203.2	0.06	193.05	192.99
26.8	0.0011	3.3	39	27.0	203.2	0.24	192.24	192.00
31.2	0.0014	3.8	39	31.3	201.9	0.34	192.00	191.66
40.7	0.0011	3.7	45	40.7	201.6	0.12	191.16	191.04
49.6	0.0012	4.0	48	49.6	202.1	0.11	190.79	190.68
53.8	0.0014	4.4	48	55.0	202.8	0.41	190.68	190.27
68.6	0.0012	4.4	54	70.0	203.2	0.31	189.77	189.46
77.0	0.0011	4.3	57	78.0	203.6	0.16	189.21	189.05
78.3	0.0012	4.5	57	78.5	203.7	0.26	189.05	188.79
87.6	0.0014	5.0	57	88.0	202.3	0.54	188.79	188.25
88.6	0.0015	5.0	57	89.0	199.0	0.24	188.25	188.01
					196.0 lower end			

In Table 14 is indicated a concise and desirable method of recording the data and elements of design of the main drain and laterals. The computations are not complete but they are carried through the line of the main drain to indicate the procedure. The difficulties encountered in harmonizing the laterals with the main drain will appear to the student, as the solution develops.

A characteristic variation in practical design is indicated in Problem 2, through the solution of which there may be developed by the student the approximate difference in cost resulting from two different assumptions as to direction of flow in a short section of one of the laterals.



Q = Cu. ft. per sec. Reaching Sewers.
A = Drainage Area in Acres.
C = Constant.
i = Rainfall in cu. ft. per sec. per Acre Practically = Inch per Hour.
S = Slope (Feet per 1000)

FIG. 31.—Comparison of run-off formulas for slope, S, of 10 ft. in 1,000 ft., and small areas.

Problem 2.—With the same hypotheses as to minimum depth, size, velocity, rainfall, run-off, etc., given for the preceding problem, design the laterals in Edgewood, Sycamore, Acorn and Center St., and in College Ave. between Acorn and Shattuck Sts., with the flow in the direction shown in Fig. 29. Prepare a profile of each drain showing the street surface, the manholes and the drain, and mark in figures the elevation of the invert at each manhole. Assuming a width of trench 2 ft. greater than the inside diameter of the drain, compute the quantity of pavement and earth to be removed and replaced.

With the following unit prices, estimate the cost of construction of the above drains and of the sections in Oak Grove Ave. between Center and Shattuck Sts. and in Shattuck St. between Oak Grove and College Ave. Earth excavation, 0 ft.–8 ft. deep at $2.00 per cubic yard.

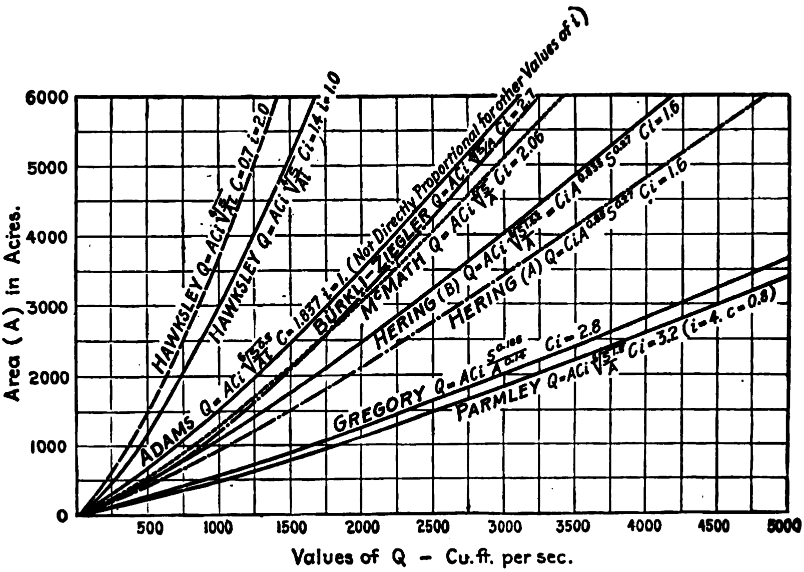
Earth excavation below 8 ft. deep at $4.00 per cubic yard.

Drains (including cost of laying) at $1.50 per linear foot for each foot of diameter.

Manholes $10 per foot of depth, assuming the depth to be to the invert of the drain for sizes under 48 in. in diameter and to springing-line for sizes 48 in. and over in diameter.

Pavement (including removal and renewal) at $2 per square yard.

Note that these assumptions have been condensed for convenience and to save time in solution. No added principles would be illustrated in more detailed figures.

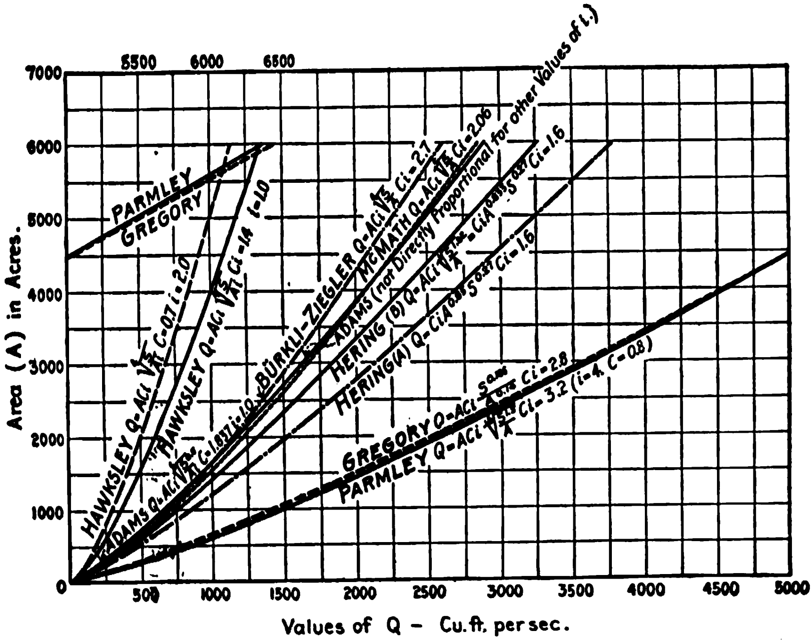


Q = Cu.ft. per sec. Reaching Sewers.
A = Drainage Area in Acres.
C = Constant.
i = Rainfall in cu.ft. per sec. per Acre Practically = Inch per Hour.
S = Slope (Feet per 1000)

FIG. 32.—Comparison of run-off formulas for slope, S, of 10 ft. in 1,000 ft. and large areas.

Now design the drain in Acorn and Center Sts., in College Ave. between Acorn and Shattuck Sts., in Oak Grove Ave. between Center and Shattuck Sts., and in Shattuck St. between Oak Grove and College Aves., with the flow in Acorn St. between Sycamore St., and College Ave. running toward College Ave. so that the College Ave. drain shall receive the water from both Sycamore and Edgewood Sts. Prepare profiles and estimates of cost to determine which is the more economical layout on the above unit prices.

Run-off Formulas.—All of the rainfall does not run off the surface into the drains and sewers, and various methods of estimating the actual run-off have been adopted. When British engineers began building underground drains, they did not even consider the rainfall, but from their knowledge of the size of the gutter or ditch which carried off the storm water from a given area, they estimated the size of the sewer needed for a given area. Later they adopted various formulas in which there were factors making some allowance for the effect on the rate of run-off of the size or shape or slope of the drainage area, and occasionally containing a factor dependent on the intensity of the rainfall. The best-known of these empirical formulas, reduced to uniform notation and with the introduction of a term expressing rate of rainfall (which was not originally used in all of them) are given in the following tabulation and in Figs. 31 to 33 inclusive.



Q = *Cu. ft per sec. Reaching Sewers.*
A = *Drainage Area in Acres.*
C = *Constant.*
i = *Rainfall in cu. ft. per sec. per Acre Practically = Inch per Hour.*
S = *Slope (Feet per 1000)*

FIG. 33.—Comparison of run-off formulas for slope, S, of 4 ft. in 1,000 ft.

Hawksley[1] (London, 1857):

$$Q = ACi\sqrt[4]{(S/Ai)},$$

in which $C = 0.7$ and $i = 1.0$, so that

$$Q = 3.946\, A\sqrt[4]{(S/A)}, \text{ since } s = S/1{,}000.$$

[1] This is a modification of the original Hawksley formula which is

$$\log d = \frac{3 \log A + \log N + 6.8}{10}$$

in which d = diameter in inches of a circular sewer intended to care for the run-off from a rainfall of one inch per hour, one half of which reaches the sewer during the hour; A = the area drained in acres; and N = the distance in which the sewer falls per foot.

Bürkli/Ziegler (Zurich, 1880):

$$Q = ACi\sqrt[4]{(S/A)},$$

in which $C = 0.7$ to 0.9 and $i = 1$ to 3.

Adams (Brooklyn, 1880):

$$Q = ACi\sqrt[12]{(S/A^2i^2)},$$

in which $C = 1.837$ and $i = 1$.

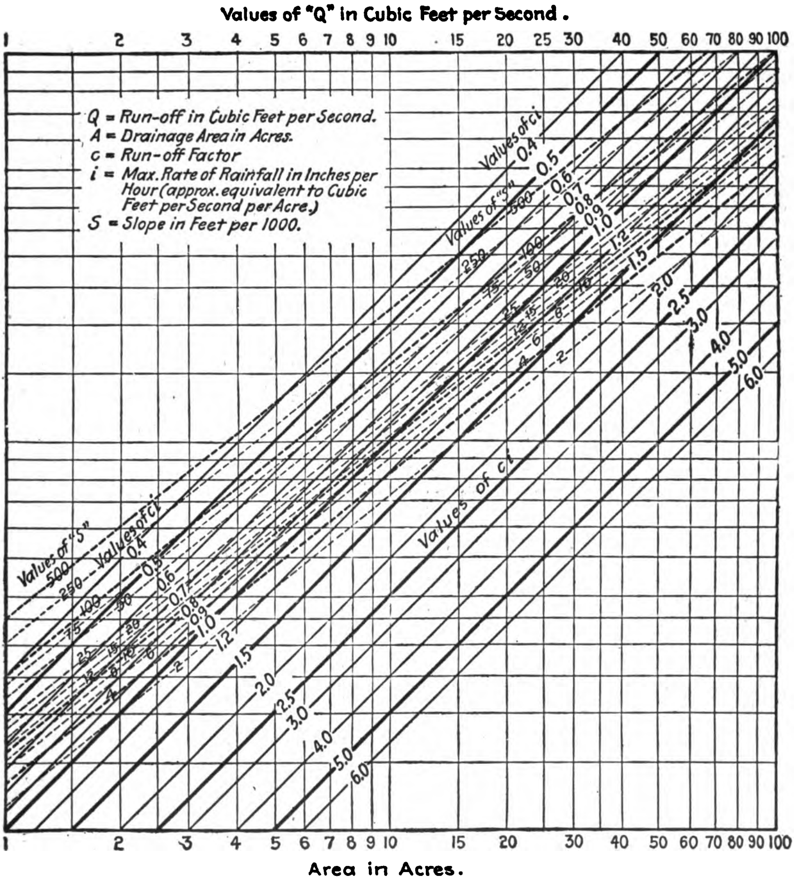


Fig. 34.—Run-off from sewered areas of 1 to 100 acres, by McMath's formula.

McMath (St. Louis, 1887):

$$Q = ACi\sqrt[5]{(S/A)},$$

in which $C = 0.75$ and $i = 2.75$.

Hering (New York, 1889):

$$Q = CiA^{0.85}S^{0.27}$$

or

$$Q = ACi\sqrt[6]{(S^{1.62}/A)} = CiA^{0.833}S^{0.27}.$$

in which Ci varies from 1.02 to 1.64. These two formulas give results differing approximately 15 per cent.

Parmley (Cleveland, 1898): $Q = ACi\sqrt[6]{(S^{1.5}/A)}$, in which C is between 0 and 1, and $i = 4$.

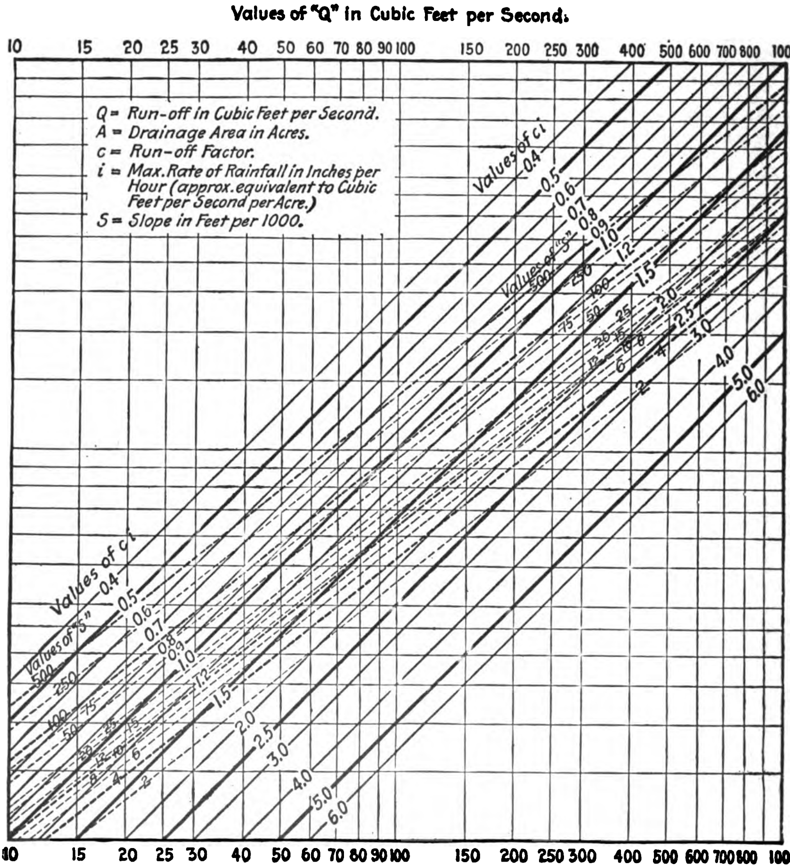


FIG. 35.—Run-off from sewered areas of 10 to 1,000 acres, by McMath's formula.

Gregory (New York, 1907): $Q = ACiS^{0.186}/A^{0.14}$, in which $Ci = 2.8$ for impervious surfaces.

In these formulas:

Q = the maximum discharge fromt he drainage area in cubic feet per second.

i = the maximum rate of rainfall in inches per hour.

A = the extent of the drainage area in acres.

S = the average slope of ground surface, in feet per thousand.

While the use of these formulas is not recommended when sufficient information is available for estimating run-off by the rational method previously described, there are cases when their use may be warranted. That of McMath is probably most favorably known, and it has been widely used, often, no doubt, without study of its applicability. It can be employed easily

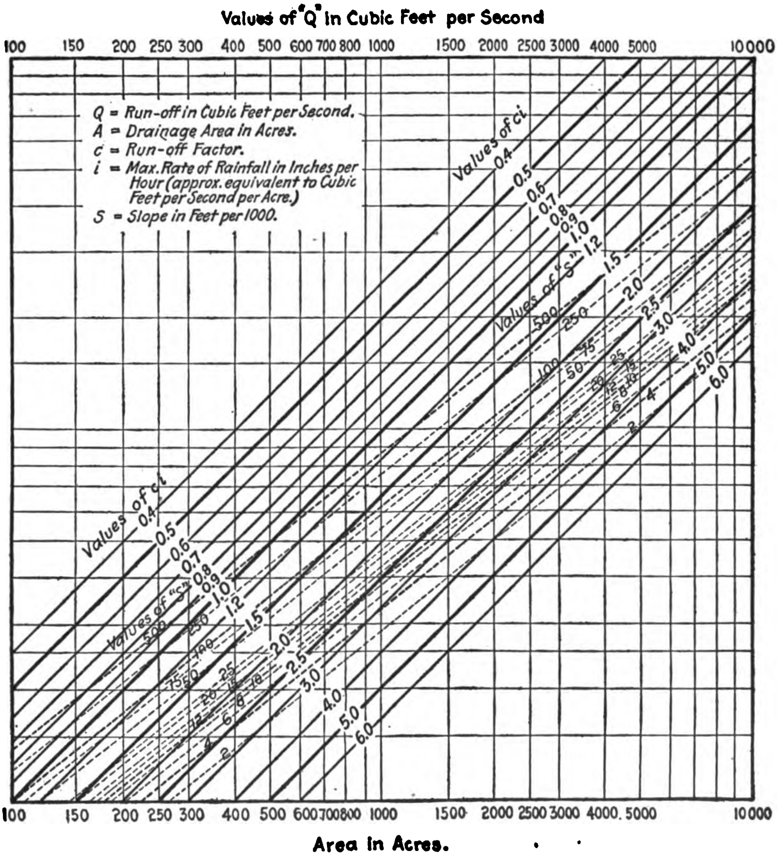


FIG. 36.—Run-off from sewered areas of 100 to 10,000 acres, by McMath's formula.

by means of Figs. 34, 35 and 36, with as much precision as it really possesses. The values of *ci* to be used with these diagrams are given in Table 15. The values of *c* are based on the percentage of the total area of the district which is covered by roofs and pavements and on the character of the soil of the remaining parts of the district.

TABLE 15. —VALUES OF ci FOR USE WITH FIGS. 34, 35, AND 36

c	i					
	2.25	2.50	2.75	3.00	3.50	4.00
0.3	0.68	0.75	0.83	0.90	1.05	1.20
0.4	0.90	1.00	1.10	1.20	1.40	1.60
0.5	1.13	1.25	1.38	1.50	1.75	2.00
0.6	1.35	1.50	1.65	1.80	2.10	2.40
0.7	1.58	1.75	1.93	2.10	2.45	2.80
0.75	1.69	1.88	2.06	2.25	2.63	3.00
0.8	1.80	2.00	2.20	2.40	2.80	3.20
0.9	2.03	2.25	2.48	2.70	3.15	3.60

These relations[1] are;

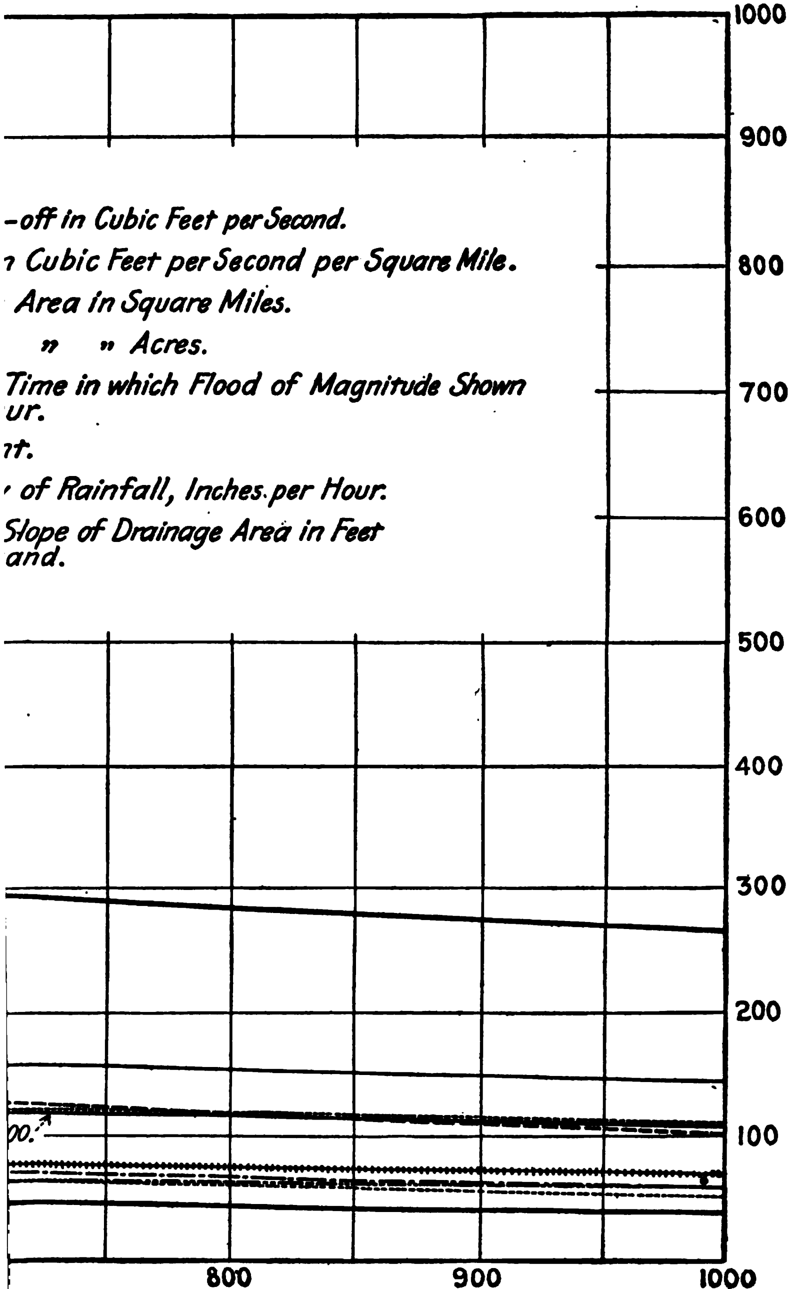
Impervious area, per cent									
Sandy soil	0	5	10	16	25	37	53	73	100
Clayey soil	....	....	....	5	15	28	46	70	100
Value of c	0.10	0.14	0.18	0.23	0.30	0.40	0.50	0.70	0.90

In using these diagrams, start with the given area at the bottom of the diagram and follow a vertical line to its intersection with the slope line; then follow a horizontal line to its intersection with the ci line; from this point follow a vertical line to the scale of quantities at the top of the diagram. For example, if impervious surfaces cover 5 per cent of a sandy area, use $c = 14$: or, in 5 per cent of a clayey area, $c = 0.23$. The value of ci may be obtained by the use of Table 15, or by multiplying c by the value of i selected for the particular case.

Flood Flows from Large Areas.—There are also formulas for estimating the flood flows from large drainage areas, which may include both steep and gentle slopes, impervious and pervious areas, wooded and cultivated land, so that portions yielding their run-off rapidly are offset by others where the yield is retarded. On such large areas the precipitation of the greatest density is often limited to a part of the drainage basin and the average density over the entire basin is considerably lower than the maximum. The maximum rate of run-off from a small basin will, therefore, be greater than from a large one. Formulas used for estimating the flood flows from these large drainage areas are plotted in Fig. 37.

Sewer Gaging.—It is rarely practicable to gage the flow of storm water in sewers by weirs, current meters and other devices employed in open channels. Generally the depth of the sewage

[1] "American Civil Engineers' Pocket Book," (Second Edition, pp. 969–970).



(Facing Page 86)

is observed at several places and from such data and the slope and an estimate of the roughness of the sewers between these places the discharge is computed by the methods explained in Chapter IV. Storms may occur at many times when observers cannot be stationed in the sewers to measure the depth of the storm water, so automatic recording gages are very desirable. At least two are necessary to determine the slope of the water surface, which is frequently very different from the slope of the

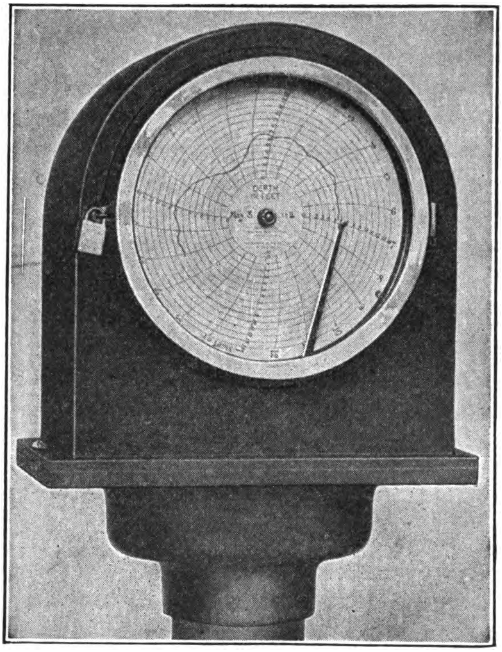


Fig. 38.—Water-level recorder (Builders Iron Foundry).

sewer. In addition to the depth gages, it is desirable to have a number of maximum-flow gages, which show the greatest depth of sewage at the point of installation since the last observation, but give no further information.[1]

Gages.—The float gage used in recording the water level in a sewer is actuated by a float in a pipe or other suitable guide in which the sewage stands at the same height as in the sewer. A cord, chain, tape or rod runs from the float to the recording

[1] There is a great need of more gagings of the storm-water flow in sewers, checked against precipitation records so as to furnish definite information about run-off factors. Records of some such gagings are given in "American Sewerage Practice" vol. i, pp. 316–327.

apparatus, of which there are several types. All are driven by clockwork and the clock movement should be regulated to keep correct time and be synchronized with the clocks of all other gages furnishing records to be studied jointly.

An apparatus of this type made by the Builders Iron Foundry is shown in Fig. 38. A cord from the float moves an arm carrying a pen in front of a circular chart rotated by clockwork. The pen moves in a circular arc, and consequently the time-scale is unduly small when the pen is near the center of the chart. The instrument is enclosed in a cast-iron box mounted on a hollow standard through which the float cord passes. It is made in

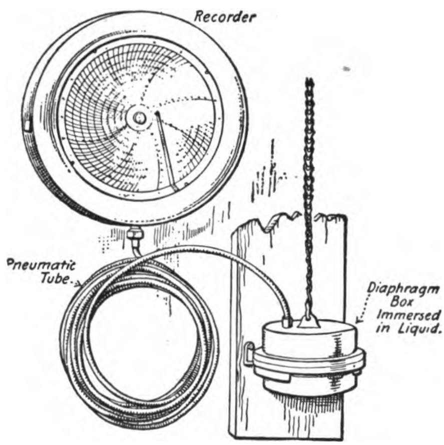


Fig. 39.—Diaphragm pressure gage.

two sizes, with 8-in. and 12-in. dials. The scale of heights as recorded on the chart will depend upon the range to be covered and the size of the chart.

The pneumatic type of pressure gage has a diaphragm box or pressure chamber which is placed in the liquid. The rise and fall of the surface of the liquid varies the pressure within the box, and these variations are transmitted through a small air pipe to the recording apparatus. Fig. 39 shows an instrument of this type made by the Industrial Instrument Company and The Bristol Company. It is made for either 8-in. or 12-in. charts.

In another form of pneumatic pressure gage, the Sanborn water-level recorder, made by the Sanborn Company, of Boston, a "compensator" functioning like a diving bell is placed in the sewage. It is a 1½-in. tube, varying in length from a few inches for small sewers to 3 ft. for 20-ft. sewers, slanting at an angle of 45 deg. in the direction of flow and extending to within a few inches of the invert. The inlet is at the bottom. As the surface of the sewage rises and falls, the pressure of the air in the compensator changes and these variations are transmitted through a small air pipe to a recording device using a circular chart.

A type of maximum flow gage devised at Cincinnati under the direction of H. S. Morse is shown in Fig. 40. It consists of a

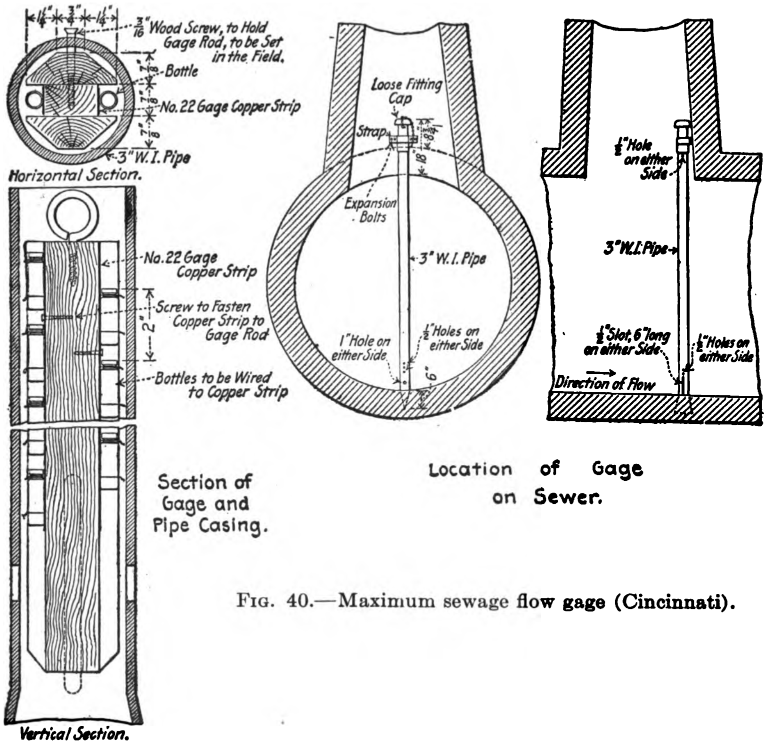


FIG. 40.—Maximum sewage flow gage (Cincinnati).

wooden staff held firmly in place inside a vertical steel pipe. Attached to the staff are bottles arranged so their mouths are 1 in. apart vertically. There are perforations in the steel pipe through which sewage enters and rises alongside the staff, so that the highest bottle filled with sewage shows that the sewer has been filled to at least that height. After a storm, the staff is lifted out of the pipe, the maximum flow elevation ascertained from the highest filled bottle, the staff is reversed to empty the bottles and then set in place in the pipe ready for service again. This gage has proved satisfactory under ordinary conditions but not with velocities exceeding 8 ft. per second.

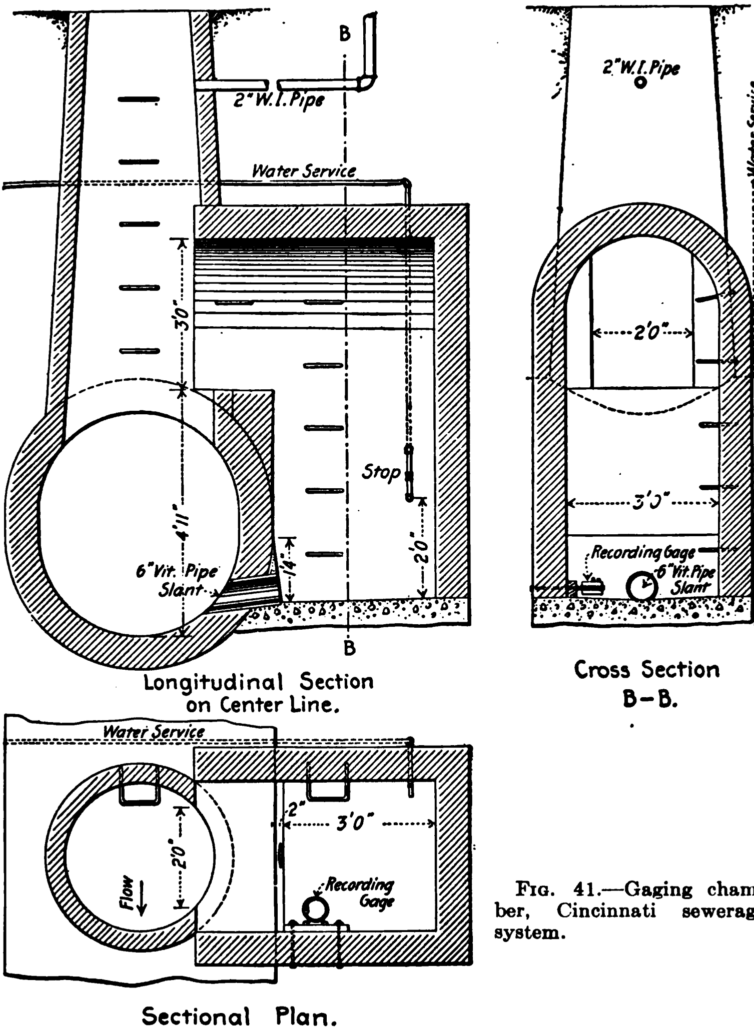


FIG. 41.—Gaging chamber, Cincinnati sewerage system.

Setting Sewage Gages.—The gage must be placed so that the condition of the sewer for a considerable distance upstream and a less distance downstream permits the discharge to be computed from the depth. The cross-section and slope must be uniform, there must be no curves, inlets or obstructions to disturb the flow, the velocity of flow should not be great, and there should be no doubt about the coefficient of roughness to use in the formula for computing the flow. In order to be sure of the results, it is necessary to have gages at each end of such a stretch of sewer to determine the slope of the water surface.

The gaging apparatus should be installed in a separate chamber or recess in a manhole so as to be protected and also easily accessible. The chamber should be connected with the sewer so that the elevation of the sewage will be the same in it and in the sewer. It is desirable to have a small flow of clean water through the gaging chamber so as to have the liquid about the apparatus or floats free from matter that might clog or derange them and become offensive through decomposition. Fig. 41 shows such a gaging chamber built at Cincinnati for holding a pneumatic-type pressure gage. The air pipe runs to an iron box at the curb to hold the recording apparatus, or the recorder can be set within a house. If a float-actuated gage is used, there is not so much range of choice for the location of the recording apparatus, but it should not be placed in the chamber if this can be avoided, because the moisture there affects the delicate metal parts of the mechanism and the paper of the chart.

CHAPTER IV

HYDRAULICS OF SEWERS

In applying the principles of hydraulics and the lessons taught by observing the flow of sewage in sewers to the design of a sewerage system, it is necessary to keep in mind the importance of maintaining a velocity sufficient to prevent the settling of solids on the invert and sides of the sewer yet not so great that the materials of which the invert is composed will be eroded by the hard suspended matter carried by the sewage. The necessity of minimizing the expense of construction in comparatively flat communities, where natural slope of the ground is lacking, compels the limiting of the grade assumed to the lowest practicable amount which will insure self-cleansing and not involve abnormal maintenance charges. Furthermore conditions may be such that particular types of cross-section will be specially suited for meeting either construction or hydraulic problems and that the adoption of the customary circular or egg-shaped sections would entail needless expense. The use of such special sections involves an examination of their hydraulic properties when the sewage flowing through them is at various depths, and the determination of the proper elevation at which branch sewers should discharge into these special sections sometimes presents problems which cause much study.

Quantities of Sewage to be Provided for.—In Chapters II and III the methods of estimating the quantities of sewage and storm water for which provision must be made were explained. There will always be some uncertainty concerning the accuracy of these estimates, just as there is uncertainty in the closeness with which the behavior of a riveted joint or a pin connection in a truss approaches the assumptions made in the computations of the stresses which it must carry. A separate sewer is subject to more or less infiltration of ground water. A wholly unforeseen construction of a group of buildings housing many persons may increase the quantity of sewage from a block along some street, far beyond the estimate. Consequently, just as the structural steel designer uses a factor of safety in designing details, some sewerage engineers use a factor of safety in designing a sewerage system. The difference in cost between small (pipe) sewers of different diameters is not a large percentage of their entire cost in place in the ground, and consequently lateral sewers are sometimes figured as running half full when carrying the maximum quantity of sewage which it is assumed will reach them. Some engineers continue this policy until pipes as large as 18 in. in diameter have been reached; in computing the working capacity of this size they assume that the maximum depth of the sewage will be seven-tenths of the vertical diameter and use seven-tenths of the diameter as the elevation of the hydraulic gradient. The authors prefer to make all allowances for unusual increments of flow or for factor of safety by additions to the estimates of the quantity of sewage or storm water to be provided for, and to choose a sewer section which will carry this maximum quantity when flowing full and at the same time maintain self-cleansing velocities under ordinary conditions of flow.

Velocity of Flow.—The mean velocity of flow is used in computations relating to the size of sewers. The experiments of Williams, Hubbell and Fenkell[1] have shown that in the case of large cast-iron water mains the mean velocity is from 0.8 to 0.85 of the center velocity and occurs at about three-fourths of the pipe radius from the center of the pipe.

The velocity of flow depends upon the head or slope and the resistance to flow of the wetted portion of the interior of the sewer.

The head or slope in sewerage design always refers to the position of the hydraulic grade line, which is the line assumed by the top surface of the flowing water when free to rise vertically. In the case of sewers flowing partly full, it is the surface of the sewage; in the case of sewers flowing under pressure, that is so full that there is an upward pressure on the crown of the sewer, it is the elevation to which the sewage would rise were vertical pipes, open to the atmosphere at the top, inserted in the sewer. Sewers which are discharging under pressure are often termed "surcharged," and this condition may be due to more sewage and storm water reaching the surcharged section than it is able to carry except under pressure, or it may be due to the section of the sewer forming an inverted siphon, or it may be due to the section discharging into a body of water which submerges the crown

[1] *Trans. Am. Soc. C. E., April,* 1902.

during part or all of the time, as is the case in a subaqueous outfall sewer.

In a large proportion of the sewers which the engineer is called upon to design, the distinction between the slope of the invert and the hydraulic gradient is unimportant, and it is generally disregarded in designing separate sewers and in estimating the dry-weather discharge of combined sewers. But it must be considered in estimating the discharge of combined sewers while carrying storm water, and in all cases it should be kept in mind that while for construction purposes the grade of the sewer is the invert grade, it is the hydraulic gradient when questions of velocity and discharge must be solved. Serious trouble has been caused in the operation of the sewer systems of some cities by a failure to use the hydraulic gradient in computing the capacities of the larger sewers, particularly those affected during portions of the day by the entrance of tidal water into their lower ends.

The invert of a long sewer is generally a concave curve with the steeper part at the upper end. If it is desired to have the hydraulic grade lie parallel with the invert and at the same time have the sewer run full, it follows that a part of the sewer must be under pressure during severe storms, as shown in Fig. 42, the amount of the pressure being determined by the position of the hydraulic gradient. If it is desired to avoid this, the computations must be made with the excess head added to the invert slope, which will result in some sections of the sewer running only partly full, or the invert must be dropped from time to time, Fig. 43, or the cross-section must be widened. Dropping the invert involves a loss of total available fall, but it can be arranged to give better details than with the continuous invert as the drops in grade are located at the inlets of the larger branches or submains, as shown in Fig. 44. Such a detail avoids a reduction in velocity in both the branch and trunk sewer, which is particularly desirable with small depths of sewage when solids are most likely to settle on the invert. It is practicable to avoid checking the velocity in the branch by giving the latter a suitable elevation above the invert of the trunk sewer into which it discharges, but this arrangement does not help the unfavorable condition in the trunk sewer.

A special condition arises in combined sewers where there is a relief outlet. When a large quantity of storm water is flowing and the outlet is in operation, Fig. 45, there is an increase in the hydraulic gradient for some distance above the outlet. Moreover, in the part of the sewer affected by this change in the hydraulic gradient, the entering branches are also similarly affected and there is a corresponding general increase in the velocity.

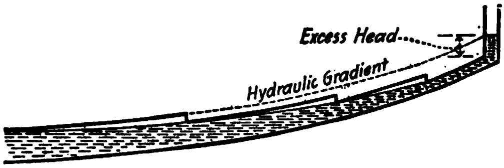


FIG. 42.

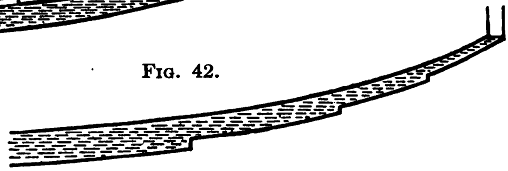


FIG. 43.

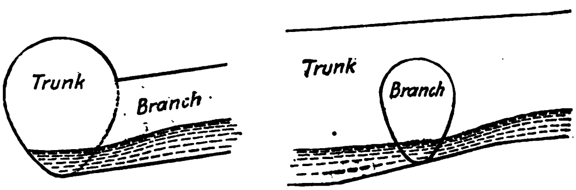


FIG. 44.

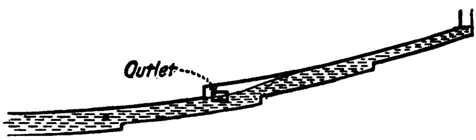


FIG. 45.

Transporting Power of Water.—The transporting power of a stream of water depends fundamentally upon the velocity of flow, particularly near the bottom. It is therefore important to take into account the varying velocities in any vertical stream and not merely the maximum or mean velocities. Other conditions which have an effect are the specific gravity and size of the substances to be transported, the depth of the water and the corresponding pressure upon the particles.

Experiments have shown that the effect of the specific gravity on susceptibility to the velocity of water is as recorded in Table 16. With a material of definite uniform specific gravity, the size of the particle determines the velocity which will permit it to settle or which will dislodge it and carry it along. The Metropolitan Sewerage Commission of New York assumed in its report of 1910 that the velocities given in Table 17 were necessary to move solid particles.

In general, it is found that a mean velocity of 1 ft. per second, or thereabouts, is sufficient to prevent serious deposition of sewage solids upon tidal flats, if the solids are reasonably comminuted. It is not enough, however, to prevent deposition of mineral matter such as sand and gravel.

Table 16.—Effects of Specific Gravity on Susceptibility to Velocity of Water

Nature of bodies	Specific gravity	Velocity in feet per second required to transport
Coal	1.26	1.25 to 1.50
Coal	1.33	1.50 to 1.75
Brickbat	2.00	1.75 to 2.00
Piece of chalk	2.05	
Oolite stone	2.17	
Brickbat	2.12	2.00 to 2.25
Piece of granite	2.66	
Brickbat	2.18	2.25 to 2.50
Piece of chalk	2.17	
Piece of flint	2.66	2.50 to 2.75
Piece of limestone	3.00	

Table 17.—Currents Necessary to Move Solids
(Metropolitan Sewerage Commission, New York)

Kind of material	Velocity required to move on bottom	
	Feet per second	Miles per hour
Fine clay and silt	0.25	about 1/6
Fine sand	0.50	about 1/3
Pebbles half inch in diameter	1.0	about 2/3
Pebbles 1 in. in diameter	2.0	about 1 1/3

Minimum Velocities.—The sewage should flow at all times with sufficient velocity to prevent the permanent settlement of solid matter in the sewer. Theoretically, the transporting capacity of flowing water varies as the sixth power of the velocity, and, therefore, there is a certain velocity which is just able to carry along certain classes of suspended matter but which, if checked very little, will drop these substances. It is manifestly unwise to approach too closely to this velocity, for a slight increase in the roughness of the interior of the sewer or the presence at any point of deposits of heavy substances might cause enough decrease in the velocity to cause the undesirable settling of the suspended matter. Moreover it is important to note that the form and adhesive nature of some of the suspended matter are such that if this matter is once deposited it has a tendency to remain on the invert, even when the velocity of flow increases again to a rate ample to keep the material in suspension had it not been deposited.

It is the velocity near the bottom of the sewer which is significant in studying the transporting power of flowing water, and very little is known about such velocities. It has been settled by observation, however, that a mean velocity of 2½ ft. per second will ordinarily prevent deposits in combined sewers and 2 ft. per second will generally prevent their occurrence in separate sewers under favorable conditions. These are minimum figures. It is very desirable to have a velocity of at least 3 ft. per second wherever practicable. At least 3 ft. velocity should be obtained in inverted siphons, access to which, for cleaning, is always difficult. Slopes giving velocities as low as 1½ ft. per second have been used successfully in some special cases in sewers, but the latter must be built and their interior surface finished with great care in order to achieve successful working conditions. Repeated removal of sludge and hard materials from sewers is expensive work, and if such deposits are not cleaned out they cause troublesome conditions gradually increasing in their annoying character; it is very desirable, therefore, to use grades which will give self-cleansing velocities in all cases, even in those where the resulting increase in cost of construction due to steeper slopes will involve fixed charges greater than the added cost of maintaining the sewers if laid on flatter slopes, for if such maintenance work is frequently neglected during the period in which there is a substantial deposit, the sewer cannot perform its functions properly and in emergency may fail to carry the sewage and storm water tributary to it, resulting in damage to property.

Examination of Sewer Design with Reference to Minimum-flow Conditions.—It is necessary, after designing a sewer, particularly a trunk or intercepting sewer, for a given service in the future, to consider the actual conditions of operation likely to arise under dry weather or minimum flow during the first few years after its construction, in order to make certain that the velocities will not be so low, for significant periods of time, as to cause serious deposits in the sewer, the removal of which would involve unwarranted cost. The construction of a sewer to serve for a long period would be unwarranted if this cost of cleaning should exceed the cost of building a smaller sewer in the first instance, to serve for a shorter period of time and until the anticipated growth had developed in some degree, and of then building a second sewer to take care of the additional sewage flow resulting from the added growth. While the latter plan would involve greater first cost of construction, enough might be saved in fixed charges and in the cost of operation, in the early years of the use of the sewer, to more than cover this increased cost.

It is desirable that the sewer sections and slopes should be so designed that the velocity of flow will increase progressively, or at least be maintained, in passing from the inlets to the outlet of the sewer, so that solids washed into the sewer and picked up and transported by the flowing stream may be carried through and out of the sewer, and not be dropped at some point owing to a decrease in velocity. It is, however, seldom possible to fully attain this condition, due to topographical conformation.

TABLE 18.—MINIMUM GRADES IN SEPARATE SEWERS; FOR 2-FT. VELOCITIES

Diameter, inches	Minimum fall in feet per 100 ft.
4	1.2
6	0.6
8	0.4
10	0.29
12	0.22
15	0.15
18	0.12
20	0.10
24	0.08

In general, the minimum grades given in Table 18 for small pipe sewers in the separate system have been found safe though steeper grades are always desirable. These grades are the least ordinarily permitted by the New Jersey State Board of Health. In its 1913 regulations governing the submission of designs, it stated:

"The sewers should have a capacity when flowing half full sufficient to carry twice the future average flow 25 years hence, plus a sufficient allowance for ground-water infiltration. When grades lower than those given are used, an explanation and reasons for the use of such grades should be included in the engineer's report."

Maximum Velocities.—The erosive action of suspended matter depends not only on the velocity with which it is carried along the invert of a sewer but also on its nature. As it is this erosive action which is the most important factor in determining the safe maximum velocities of sewage, the character of the suspended matter must be considered when designing combined sewers where velocities will probably be high. For instance, velocities which caused serious erosion of the inverts of brick sewers at Worcester, Mass., caused no appreciable wear at Louisville, Ky. The difference is due to the fact that the street detritus entering the combined sewers at Worcester is made up largely of particles of hard quartz while that at Louisville is largely clay and particles of limestone, having little effect upon rather soft brick much less resistant than that used in the Worcester sewers.

Vitrified clay pipe resists abrasion well and when it is used there is rarely any occasion to set a maximum velocity which must not be exceeded by the sewage passing through it. Sewer brick differs so greatly in quality that it is not possible to lay down any general rule for permissible maximum velocities where it is used. It is not advisable to use it for inverts where the velocity is over 8 ft. per second and where the street grit washed into the sewers contains much silica. If the street grit is soft, like that at Louisville or St. Louis, there will probably be little erosion of sewer brick with sewage flowing at velocities of 10 or 12 ft. per second. Where concrete sewers are constructed it is desirable to have the inverts given a hard finish, preferably a granolithic surfacing if velocities exceed 5 ft., and if the velocities exceed 8 ft. it is prudent to line the sewers with vitrified paving brick.

Maximum velocities need rarely be considered except in combined sewers or storm-water drains where the grades are such that the flow of sewage may become swift. The maximum velocity should always be ascertained for such sewers in order to be certain that the designer of the cross-sectional details knows that he must provide against erosion. The greatest objection to high velocities in small vitrified pipe sewers is that with reduction in the volume of sewage flowing and consequent decrease in its depth, they are likely to leave stranded on the inverts, where they may become so firmly lodged that the next rush of sewage will not detach them, large floating substances which at times enter all sanitary sewerage systems. Theoretically rags, old brushes, pieces of wood, corncobs and such things should not be allowed to enter separate sewers, but they are thrown into the house fixtures and may be left on the invert of a small pipe sewer in which the sewage flows intermittently in swift flushes, as is likely to be the case where the grades are steep. It is for this reason that in some European cities maximum grades of 1 : 15 and 1 : 20 are prescribed for 6-in. house connections and small lateral sewers respectively.

Another feature of steep grades and their attendant high velocities which must not be overlooked is the possibility that the discharge of these steep sewers will concentrate sewage so quickly at the upper ends of the sewers receiving their discharges that the latter will become surcharged. For this reason it is desirable to design sewers in such a way that the velocities of flow in them will be approximately equalized, or increase but slightly progressively to the outfall. This can be done sometimes by giving a sewer running down a hillside a longer length than is necessary simply to deliver the sewage and by using this increased length to obtain a moderate grade, just as is done in railway location under similar conditions. Another expedient used in small sewers is to employ a drop manhole, while in large sewers a flight sewer, described in the next chapter, is sometimes employed.

Formulas for Velocity.—There are two formulas in general use for calculating the velocity, one developed by Allen Hazen and Gardner S. Williams and the other by E. Ganguillet and W. R. Kutter, commonly called by the name of the latter.

The Hazen and Williams formula agrees closely with the results of gagings of pipes and can be employed with facility by means of a special slide rule graduated for the solution of problems by it. While the formula has been employed most often in calculating the discharge of pipes under pressure, it may also be used in computing the velocity in sewers by employing suitable coefficients. The formula is:

$$v = cr^{0.63}s^{0.54}0.001^{-0.04}$$

in which v = velocity, in feet per second

c = coefficient of roughness

r = hydraulic mean radius in feet

s = slope in feet per foot.

The coefficients, c, employed with this formula are:

140 for new cast-iron pipe when very straight and smooth;

130 for new cast-iron pipe under ordinary conditions;

100 for old cast-iron pipe under ordinary conditions; this value to be used for ordinary computations anticipating future conditions;

110 for new riveted steel pipe;

95 for steel pipe under future conditions;

140 for new lead, brass, tin or glass pipe with very smooth surface,

130 to 120 ditto, when old;

120 for smooth wooden pipe or wooden stave pipe;

140 for masonry conduits of concrete or plaster with very smooth surfaces and when clean;

130 ditto, after a moderate time when slime-covered;

120 ditto, under ordinary conditions;

110 for cement-lined pipe (Metcalf and Eddy);

100 for brick sewers in good condition;

110 for vitrified pipe sewers in good condition (Metcalf & Eddy).

The older, Kutter, formula is most generally employed in sewerage designing. It is:

$$v = \left(\frac{41.66 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.66 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}} \right) \sqrt{RS}$$

in which v = the mean velocity of flow, in feet per second,

R = the mean hydraulic radius, in feet, determined by dividing the cross-sectional area in square feet of the flowing water by the length in feet of the wetted portion of the perimeter of the sewer,

S = the slope of the water surface per unit of length, and

n = the coefficient of roughness of the wetted perimeter.

The authors recommend the use of the following values of n, the coefficient of roughness for sewer pipes, conduits and channels under reasonably good operating conditions:

	n
For vitrified pipe sewers	0.015
For concrete sewers of large section and best work laid on slopes giving velocities of 3 ft. per second or more	0.012
For concrete sewers under good ordinary conditions of work	0.013
For brick sewers lined with vitrified or reasonably smooth hard burned brick and laid with great care, with close joints	0.014
For brick sewers under ordinary conditions	0.015
For brick sewers laid on flat grades and rough work	0.017 to 0.020

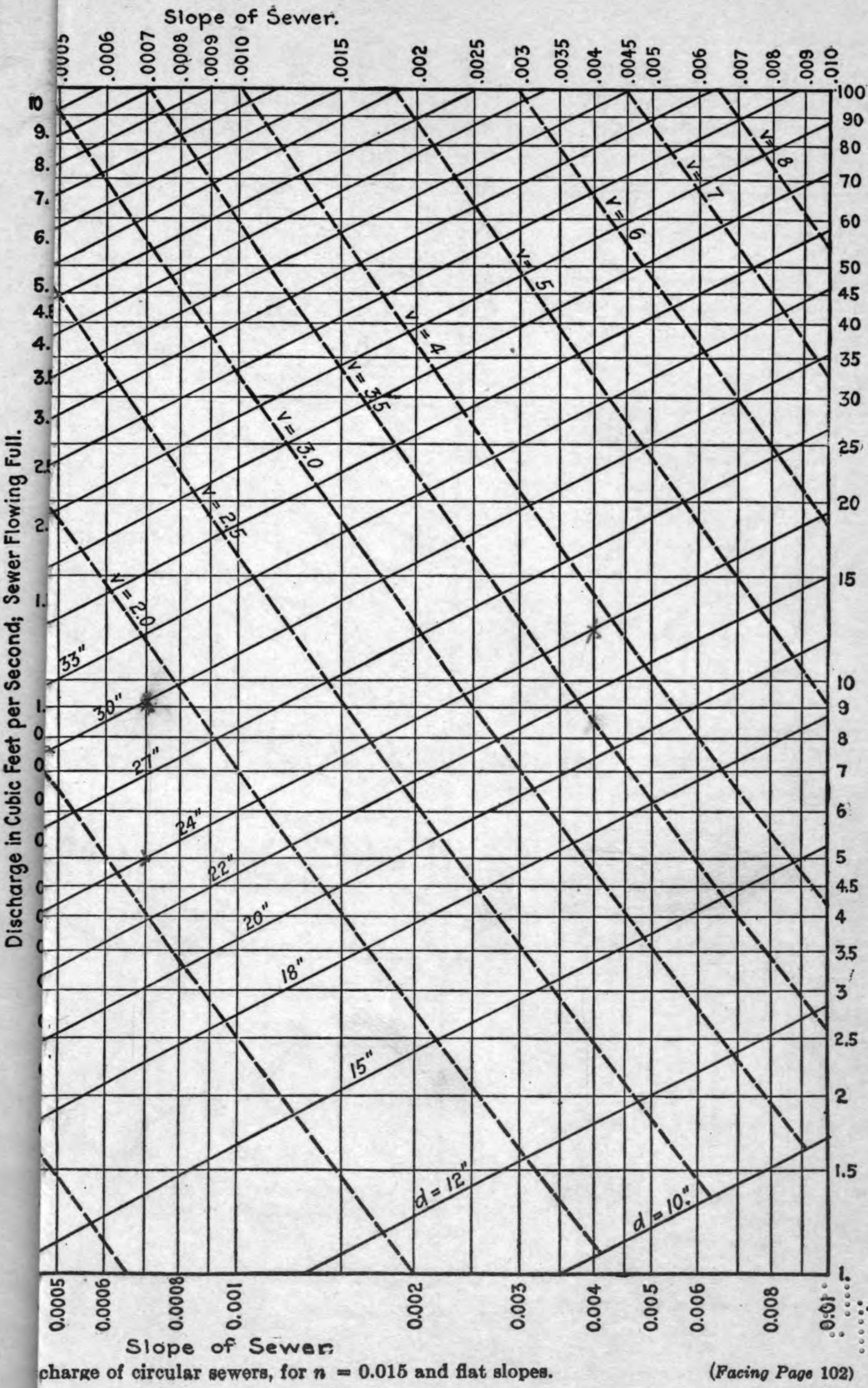
Although many engineers employ $n = 0.013$ for vitrified pipe sewers, the authors favor $n = 0.015$ where the grades permit, in view of the possibility of rough pipe and poor pipe-laying, which will increase the frictional resistance. If $n = 0.013$ is assumed, great care must be taken in specifying and accepting materials, to make certain that the character of construction required is obtained, and the sewers must be kept reasonably clean.

For ordinary work the Kutter and the Hazen-Williams formulas agree closely enough to permit the use of either. That is, the difference in sizes of sewers based on the two formulas is usually within the range of commercial pipe sizes, as shown in Table 19. For especially large and important work, special

TABLE 19.—COMPARISON OF SEWER SIZES IN INCHES RESULTING FROM THE USE OF THE KUTTER AND HAZEN-WILLIAMS FORMULAS

Quantity of sewage		Slope of sewer per 1,000					
		0.1		1.0		10.0	
Cubic feet per second	Million gallon daily	Kutter	Hazen and Williams	Kutter	Hazen and Williams	Kutter	Hazen and Williams
2	1.29	24	24	15	15	10	10
10	6.46	42	42	24	27	18	18
50	32.30	77	78	50	49	33	30
2	1.29	25	25	16	16	10	10
10	6.46	45	45	30	30	20	18
50	32.30	81	84	55	52	33	33

NOTE: In the computations for the first three lines of figures n was taken as 0.013 and c as 120; for the last three lines n was taken as 0.015 and c as 100.



charge of circular sewers, for n = 0.015 and flat slopes.

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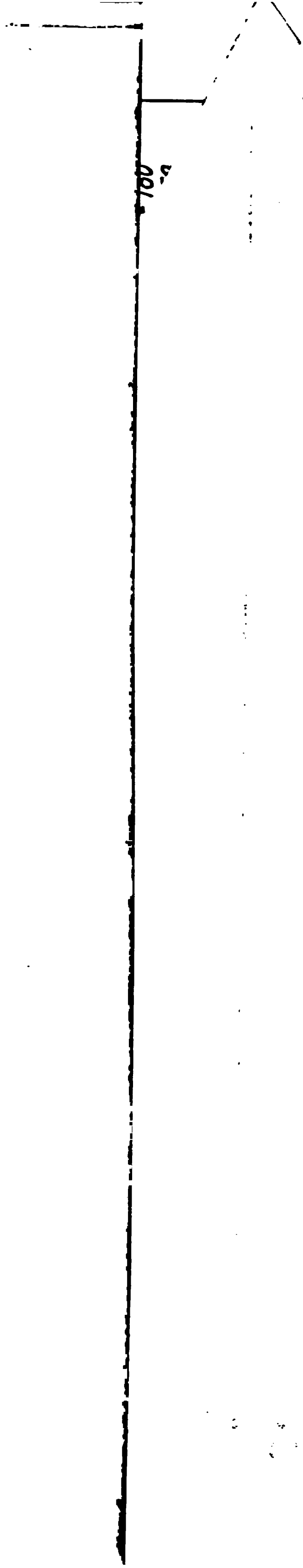
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studies are warranted and less reliance should be placed upon coefficients and the arbitrary selection of sizes from the diagrams ordinarily used.

When n is taken as 0.013 in Kutter's formula, c in the Hazen-Williams formula may be taken as 120; when n is taken as 0.015, c may be taken as 100.

Kutter's formula is generally used with the aid of diagrams giving not only the velocity but also the discharge. Figs. 46, 47 and 48 are examples of such diagrams prepared by the authors and Fig. 49 is one of an admirable set of hydraulic

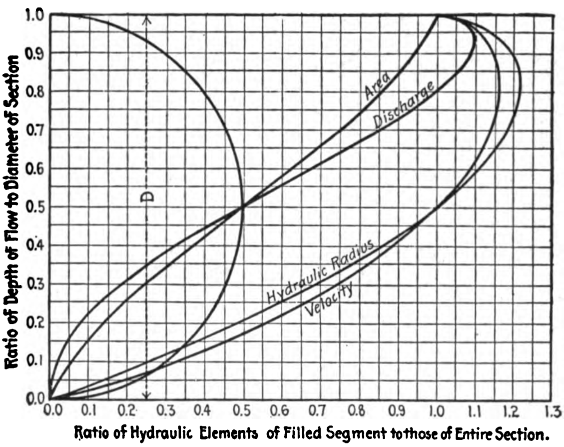

FIG. 50.—Hydraulic elements of circular section.
$n = 0.015$; $s = 0.005$; $D = 1$ ft. Area $= 0.785D^2$; Wetted Perimeter $= 3.1416D$; Hydraulic Radius $= 0.250D$.

diagrams prepared by Prof. John H. Gregory.[1] The discharge is equal to the product of the mean velocity by the cross-section of the flowing stream.

In making the preliminary studies of a system of sewers, tables of the discharge of sewers laid on a grade of 1 per cent (such as shown in Tables 20 and 21) coupled with the use of a slide rule, keeping in mind the principle that velocities and discharges vary about as the square roots of the grades, have been used in the past, particularly in Europe, and may be devised in the field in emergency. The modern method, and the one followed by most engineers, however, is to make use of

[1] Other examples of the Gregory diagrams are given in "American Sewerage Practice," vol. i, p. 94, and in the published set of the diagrams, which may be obtained from Prof. Gregory, The John Hopkins Univ., Homewood, Baltimore, Md.

diagrams, like Figs. 46 to 57, from which the final results may be read directly.

The Limitations of Kutter's Formula.—Being essentially

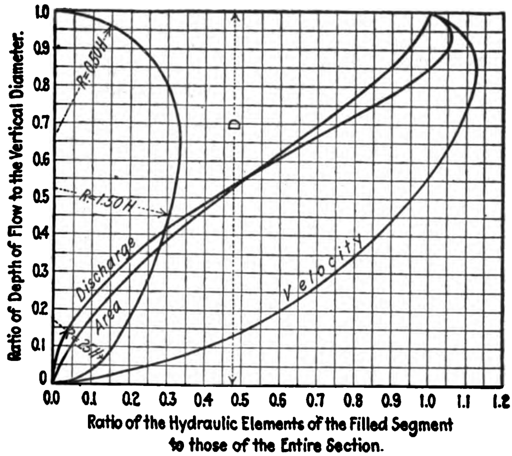

FIG. 51.—Hydraulic elements of egg-shaped section.
$n = 0.015$; $s = 1/1{,}600$; $H = 4$ ft.; $D = 6$ ft. $= 1.254$ diam. equiv. circle; $H = 0.836$ diam. equiv. circle; $A = 1.1485\,H^2 = 0.5105D^2$; $R = 0.2897H = 0.1931D$.

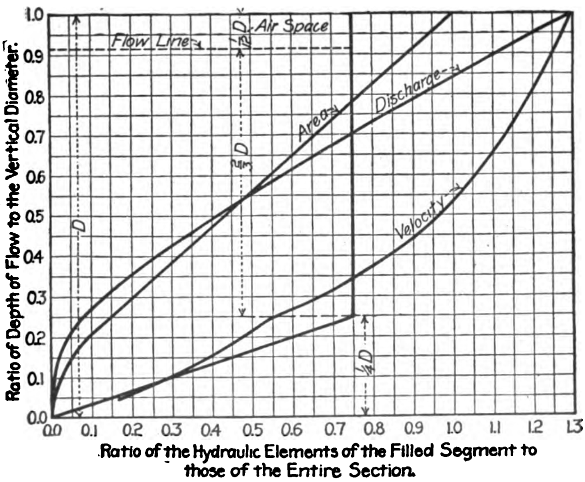

FIG. 52.—Hydraulic elements of rectangular section.
$n = 0.013$; $s = 0.001$; $D = 6$ ft.

empirical and based upon actual gagings, it is important to remember the limits within which observations have been made and further to remember that while velocity varies approximately s (*H*) of the Diameter (*D*, Inches) Filled

	36-in.		42-in.		48-in.		60-in.	
	V	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*
15	3.46	2.04	3.85	3.09	4.22	4.42	4.90	8.02
4	5.39	6.13	6.00	9.27	6.57	13.66	7.61	24.1
8	6.93	12.96	7.71	19.6	8.44	18.00	9.81	50.9
5	8.48	22.7	9.43	34.3	10.32	39.1	11.99	89.2
2	9.63	34.04	10.71	51.50	11.73	73.70	13.62	133.75
	10.40	45.6	11.56	69.0	12.67	98.7	14.71	179.2
	10.98	57.2	12.21	96.4	13.37	123.8	15.52	224.5
	11.17	66.7	12.42	100.9	13.61	144.4	15.80	262.2
	11.07	73.1	12.32	110.6	13.50	158.2	15.67	287.0
5	9.63	68.09	10.71	103.00	11.73	147.40	13.62	267.5

with Different Proportions (*H*) of the Height

	60/40-in.		63/42-in.		66/44-in.		69/46-in.	
	V	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*
	4.7	2.9	4.8	3.1	5.0	3.7	5.1	4.2
	6.9	10.1	7.2	11.6	7.4	13.1	7.6	14.8
	8.5	21.8	8.8	24.8	9.1	28.1	9.4	31.6
9	8.98	28.35	9.29	32.32	9.60	36.65	9.91	41.37
	9.7	39.1	10.0	44.7	10.3	50.6	10.6	57.0
	10.8	60.9	11.2	69.5	11.5	78.7	11.9	88.5
	11.9	84.1	12.3	96.0	12.17	108.8	13.1	122.6
9	12.06	101.27	12.46	115.34	12.86	130.70	13.25	147.21
	12.3	108.8	12.7	124.1	13.1	140.6	13.5	158.3
	12.6	133.5	13.1	152.2	13.5	172.5	13.9	194.1
	12.6	152.4	13.1	173.6	13.5	196.6	13.9	221.6
	11.37	145.16	11.76	165.45	12.14	187.45	12.51	211.08

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as the square root of the head under velocities corresponding to the ordinary conditions of flow, it varies more nearly directly as the head under extremely low velocities. Within the ordinary

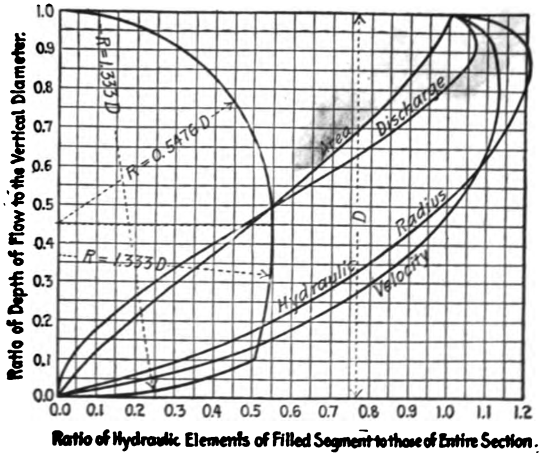


Fig. 53.—Hydraulic elements of horseshoe section, Wachusett type.
$n = 0.013$; $s = 0.003$; $D = 7$ ft.; Horizontal diameter, $H = 7$ ft. 8 in.; Area $= 44.74$ sq. ft. $= 0.913D^2$; Wetted perimeter $= 24.26$ ft. $= 3.466D$; Hydraulic radius $= 1.841 = 0.263D$.

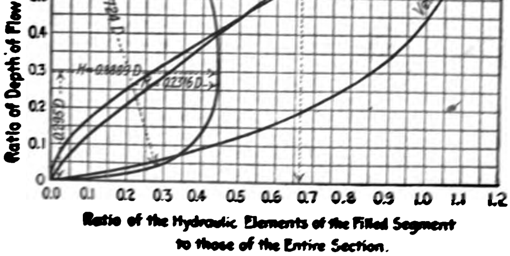


Fig. 54.—Hydraulic elements of catenary section.
$n = 0.015$; $s = 1/3{,}000$; $D = 7.44$; Vertical diameter $= D = 1.063$ diam. equiv. circle; Area $= 0.70277D^2$; $R = 0.23172D$.

velocity limits of from 1 to 6 ft., the formula finds its most trustworthy application. It is fairly reliable up to 10 ft. per second velocity.

For special cases which may be outside the range of the formula, such as 20 ft. per second or higher velocity, the engineer should consult the original data.

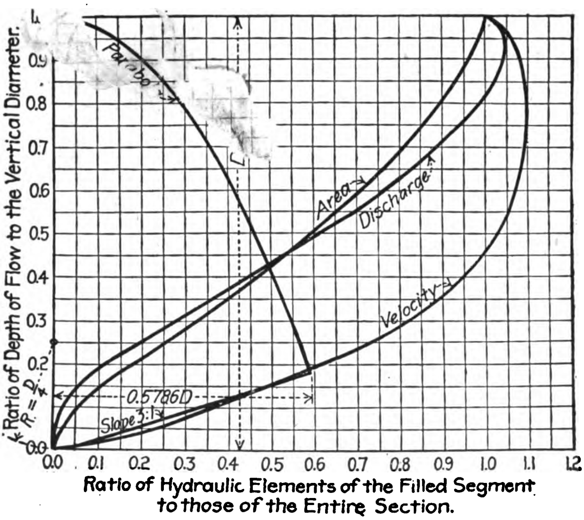


Fig. 55.—Hydraulic elements of parabolic or delta section.
$n = 0.013$; $s = 0.001$; $D = 7$ ft. 4 in.; Area $= 0.744D^2$; Hydraulic radius $= 0.2245D$.

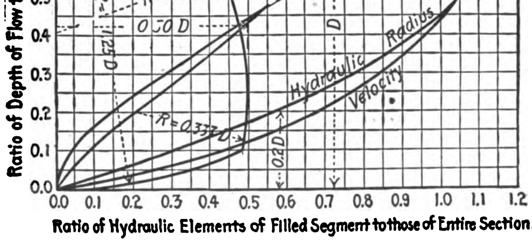


Fig. 56.—Hydraulic elements of Louisville semi-elliptic section.
$n = 0.013$; $s = 0.0003$; $D = 7\frac{1}{2}$ ft.; Area $= 0.785D^2$; Wetted perimeter $= 3.26D$; Hydraulic radius $= 0.242D$.

Hughes and Safford have summed up[1] the application of this formula in an excellent manner as follows:

[1] Hydraulics, p. 343.

"That, for hydraulic radii greater than 10 ft., or velocities higher than 10 ft. per second, or slopes flatter than 1 in 10,000, the formula should be used with great caution. For hydraulic radii greater than 20 ft., or velocities higher than 20 ft. per second, but little confidence can be placed in the results.

"That, considering the variable accuracy of the data on which the formula is based, results should not be expected to be consistently accurate within less than about 5 per cent.

"That, for any slope steeper than 0.001 the values of C computed for S = 0.001 may be used with errors less than the probable error in the ordinary use of Kutter's formula.

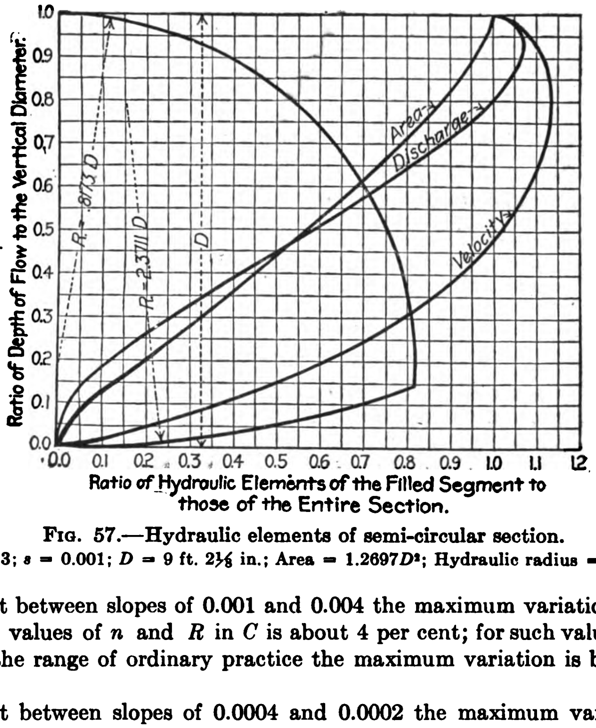


FIG. 57.—Hydraulic elements of semi-circular section.
n = 0.013; s = 0.001; D = 9 ft. 2⅛ in.; Area = 1.2697D^2; Hydraulic radius = 0.2946D.

"That between slopes of 0.001 and 0.004 the maximum variation at the extreme values of n and R in C is about 4 per cent; for such values as fall within the range of ordinary practice the maximum variation is but 2 per cent.

"That between slopes of 0.0004 and 0.0002 the maximum variation is about 5 per cent, but for such values as fall within the range of ordinary practice the maximum is less than 3 per cent.

"That for higher values of S the divergence in the values of C increases; but the occasions when slopes flatter than 0.0004 are to be considered in design are not common, and when they do occur they are usually for structures of such high character that they warrant special study and some basis in addition to a general empirical coefficient. And considering that a degree of precision of 0.001 is rarely exceeded in leveling for ordinary construction work, and that in picking out the value of n, a variation of 0.001 for small values of n and R may change the value of C as much as 17 per cent, and for moderate values as much as 5 to 8 per cent, it should be obvious that hair-splitting calculations with the Kutter formula are a needless waste of time, producing merely mechanical accuracy instead of a high degree of precision."

Flow in Partly Filled Sections.—In many of the problems arising in sewerage design it is necessary to know the velocity and discharge when a sewer is partly filled. This information is obtained in actual practice from diagrams like those of Figs. 50 to 57 inclusive. They are usually prepared by plotting the cross-section carefully and then measuring with a planimeter the area of the stream when the section is filled to various depths. These areas divided by the wetted perimeter, also carefully measured, give the mean hydraulic radius. A slope and a coefficient of roughness, n, are then assumed and the velocities and discharges for the different depths are computed and plotted in terms of the velocity and discharge when flowing full. The curves thus obtained show the relative velocities and discharges at different depths as compared with those of the full section. These curves, although strictly correct only for the data given, are sufficiently close for other sizes, slopes and friction factors to be of general use, and as a rule the difference may be neglected.

Attention is particularly called to the egg-shaped section, Fig. 51. This was introduced about 1846 by John Phillips in order to obtain adequate velocities in combined sewers when the depth of sewage in them was small. When the quantity of sewage is not more than two-tenths of the total capacity of the sewer, the velocity in an egg-shaped sewer will be somewhat greater than in a circular sewer of the same total capacity. The depth of the sewage will also be greater and there is a resulting better flotation for solids. These differences are not of great magnitude, however.

Discharge and Velocity at Different Depths.—The proportionate velocity and discharge at different depths are shown in Figs. 50 to 57 inclusive. For a 12-in. sewer, the capacity and velocity when running full are 2 cu. ft. per second and 2.5 ft. per second respectively, the slope being 0.005 and n being 0.015.

When the depth of flow is 0.1 ft., the discharge will be but 0.06 cu. ft. per second or 3 per cent of its capacity when running full. At that depth, however, the velocity of flow will be 33 per cent of that of the full sewer, or 0.84 ft. per second. At a depth of 0.2 ft., the discharge will still be small, but the velocity will have increased substantially, to 1.38 ft., 55 per cent of the velocity when the sewer is full. In other words, the velocity increases much more rapidly with increasing depth than does the discharge, as shown in detail in Table 22.

TABLE 22.—ACTUAL AND PROPORTIONATE DISCHARGES AND VELOCITIES FOR A 12-IN. SEWER ON A SLOPE OF 0.005 WITH n TAKEN AS 0.015

Proportionate depth	Actual		Proportionate	
	Capacity, cubic feet per second	Velocity, feet per second	Capacity	Velocity
0.1	0.04	0.84	0.02	0.33
0.2	0.16	1.38	0.08	0.55
0.3	0.36	1.85	0.18	0.74
0.4	0.64	2.23	0.32	0.89
0.5	1.00	2.50	0.50	1.00
0.6	1.34	2.70	0.67	1.08
0.7	1.70	2.85	0.85	1.14
0.8[1]	2.00	2.90	1.00	1.16
0.9	2.16	2.85	1.08	1.12
0.93[2]	2.18	2.77	1.09	1.12
1.0	2.00	2.50	1.00	1.00

[1] Point of maximum velocity.
[2] Point of maximum discharge or capacity.

This proportionately greater increase in the velocity than in the discharge is an important condition of which the designing engineer should take advantage. He should also keep in mind that a circular sewer will have the same velocity when running half full as when full, that the maximum velocity is generally reached when the sewer is between 0.8 and 0.9 full, and that the maximum capacity is generally reached when the sewer is between 0.9 and 0.95 full. The velocity ordinarily used as a basis of design is that of the sewer when half full or full. The maximum velocity, however, may have an important influence in washing out deposits which form at times of very low flow.

Selection of Sewer Sections.—The problem of the design of masonry sewers is not solved with the determination of the required carrying capacity, but includes a number of other features which may be of considerable importance.

The most economical shape for the waterway cross-section can only be selected after careful consideration of the special conditions imposed and the relative merits of one type as against another to meet these special conditions. While the circular cross-section has been used for a large number of the masonry sewers constructed in this country, Figs. 58, 59 and 60, there has been an increasing use of other forms such as the horseshoe, Fig. 61, semi-elliptical, Fig. 62, and rectangular sections, Fig. 52. In the older combined sewerage systems constructed previous to 1890, and built for the most part of brick for sizes above 24 in. in diameter, the egg-shaped cross-section, Figs. 58 and 63, was frequently used, but since that time the extended use of separate systems has caused it to decrease in popularity. The old Massachusetts North Metropolitan System was a departure

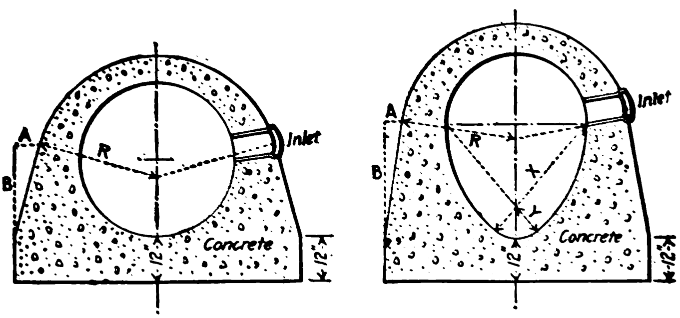


FIG. 58.—Standard plain concrete section (Borough of the Bronx, New York).

from the practice of the time in that it included such types as the Gothic, catenary, Fig. 64, and basket-handle sections.[1]

The general adoption of concrete for masonry sewers has brought about a more extended preference for the flatter types of inverts on account of their being more easily constructed than the inverts of circular or egg-shaped sections.

Aside from the hydraulic properties, such considerations as the method of construction, character of foundation, available space and stability may be instrumental in determining the best type of sewer section to adopt for a given case.

The selection of the proper thickness of masonry for a given size of sewer, unless determined in the light of experience with

[1] Chapter XII, vol. i, "American Sewerage Practice" contains a large number of illustrations and descriptive notes of different types of sewer sections, some of which are shown in Figs. 65 and 66.

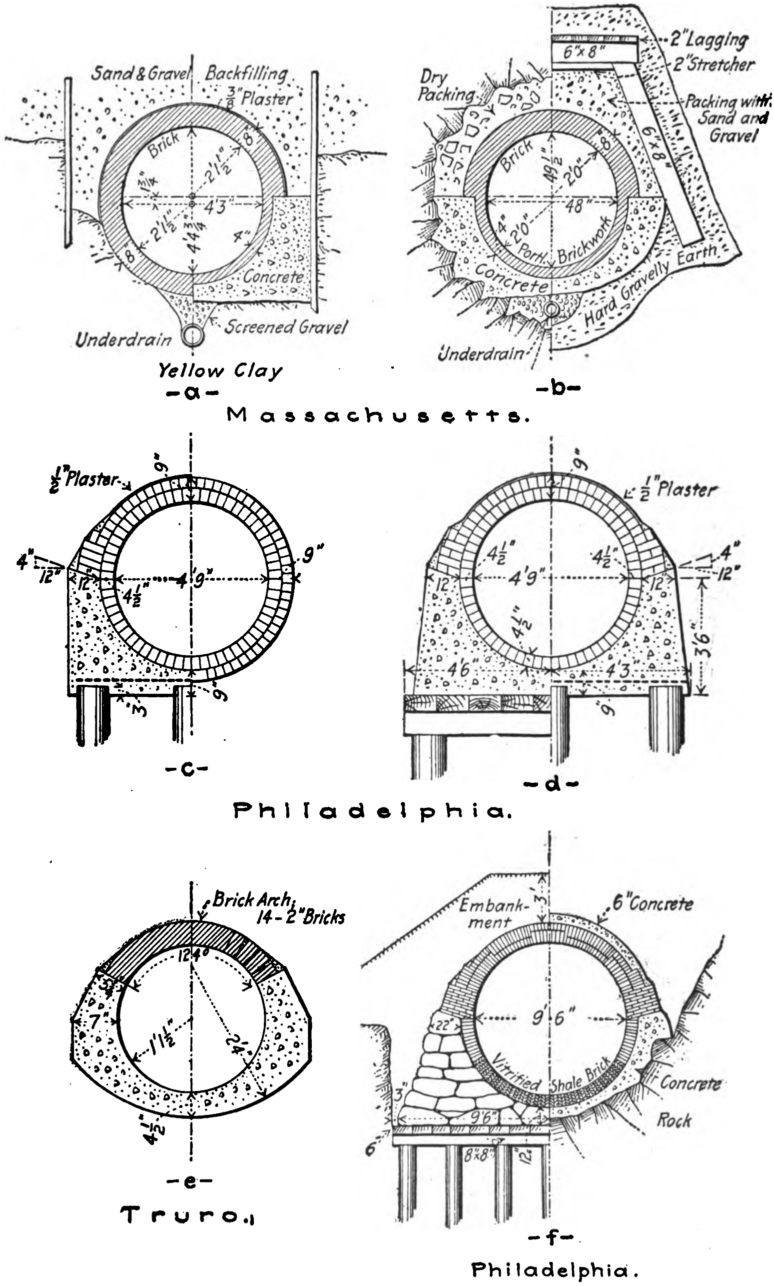


FIG. 59.—Typical circular sections with brick arches.

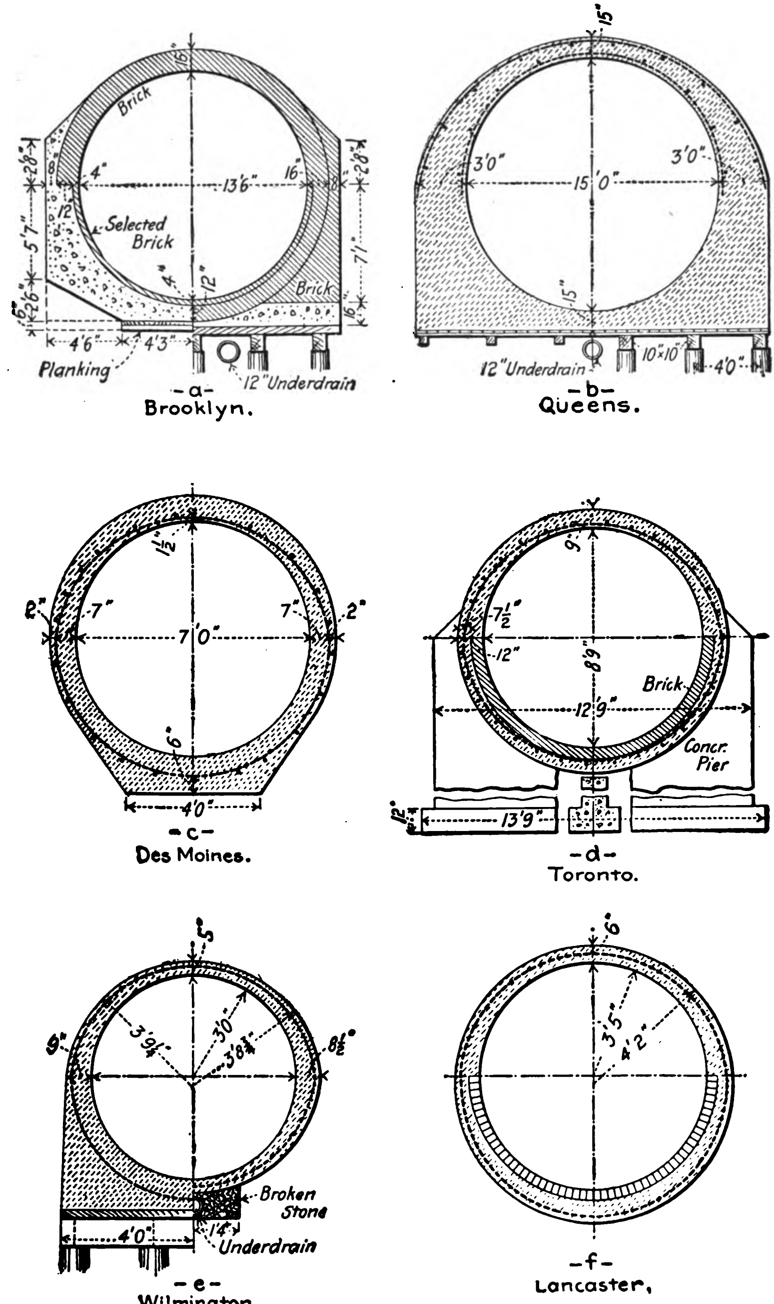
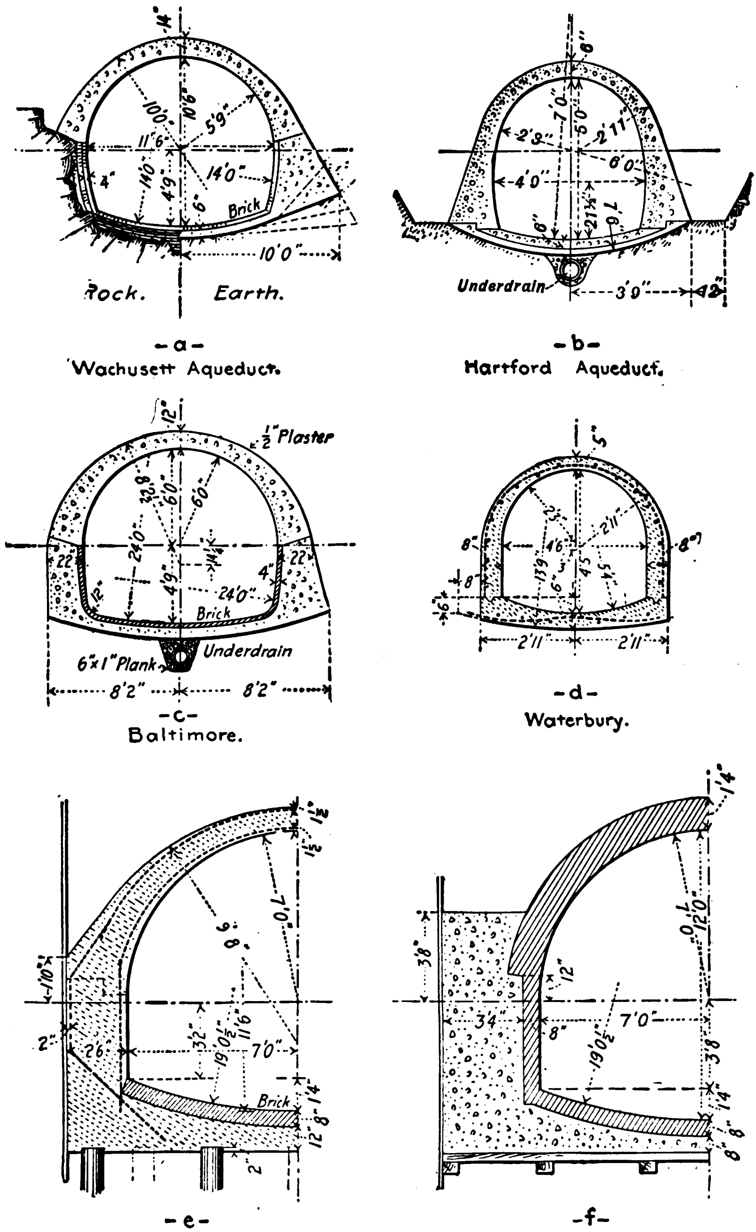


Fig. 60.—Typical circular sections.

Horse shoe: 1. good hydraulic properties 2. Resist. to ext. l.
3. facility in construction 4. convenience in maint.



Boston

FIG. 61.—Typical horseshoe sections

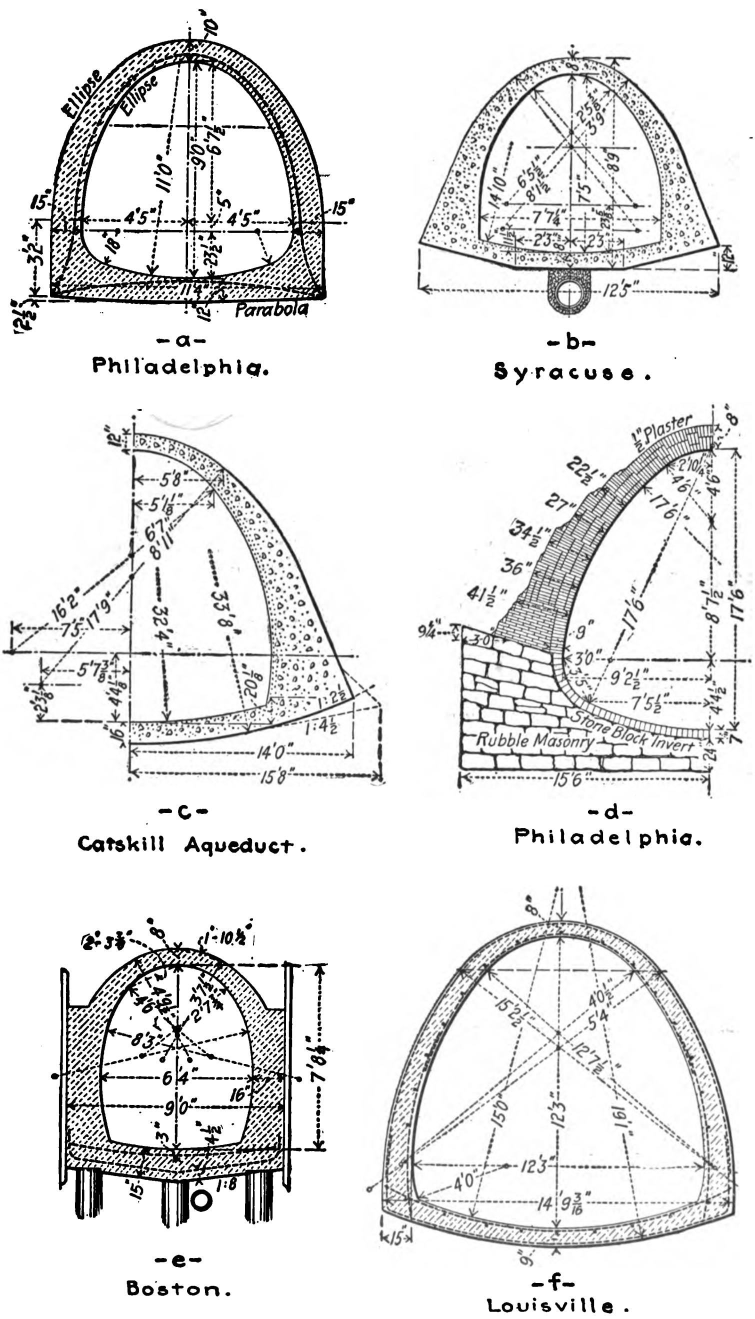


FIG. 62.—Typical semi-elliptical sections.

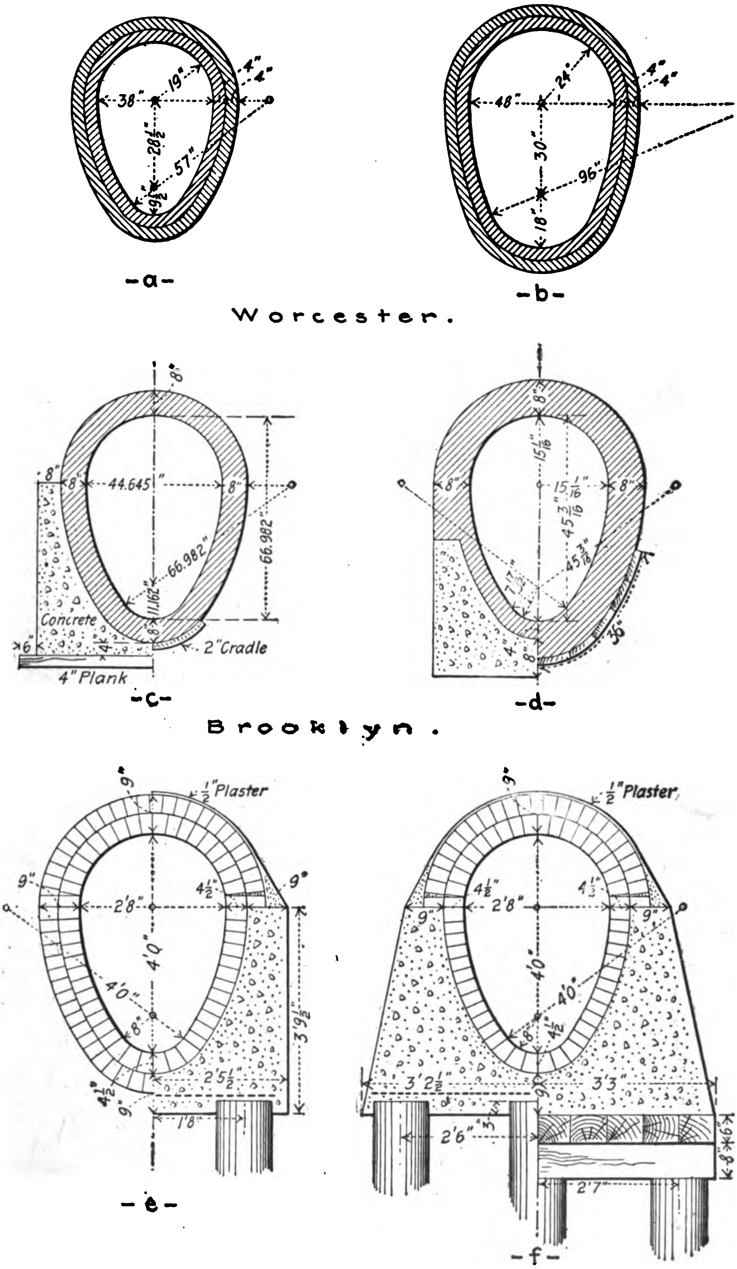


FIG. 63.—Typical egg-shaped sections.

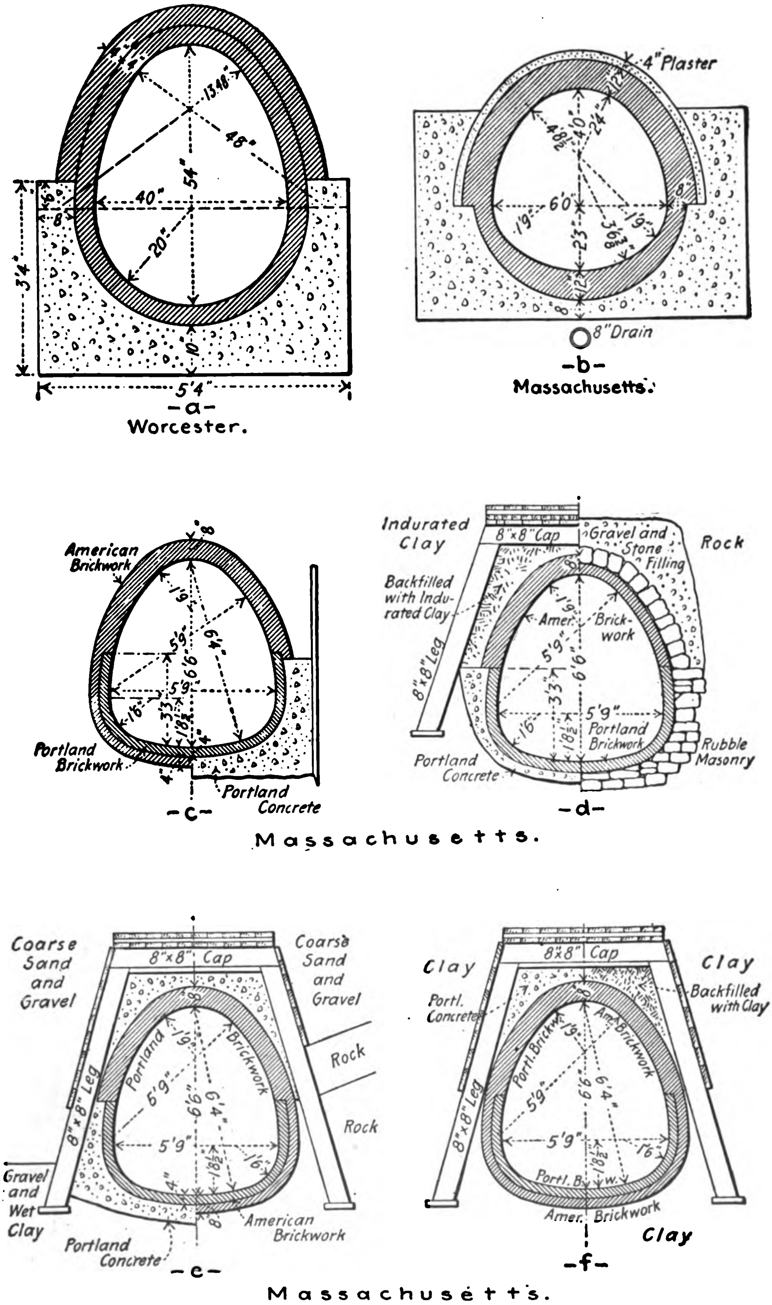


FIG. 64.—Typical inverted egg-shaped and catenary sections.

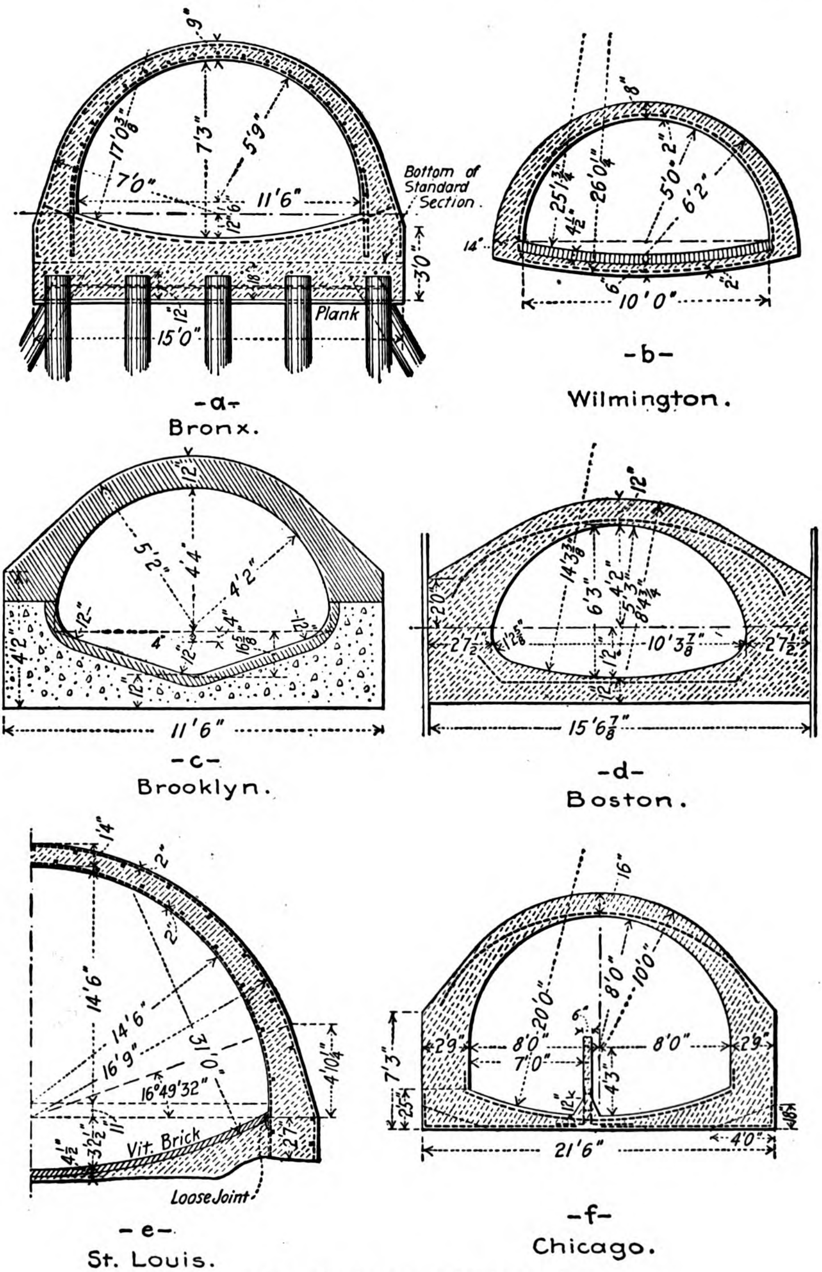


FIG. 65.—Typical semi-circular sections.

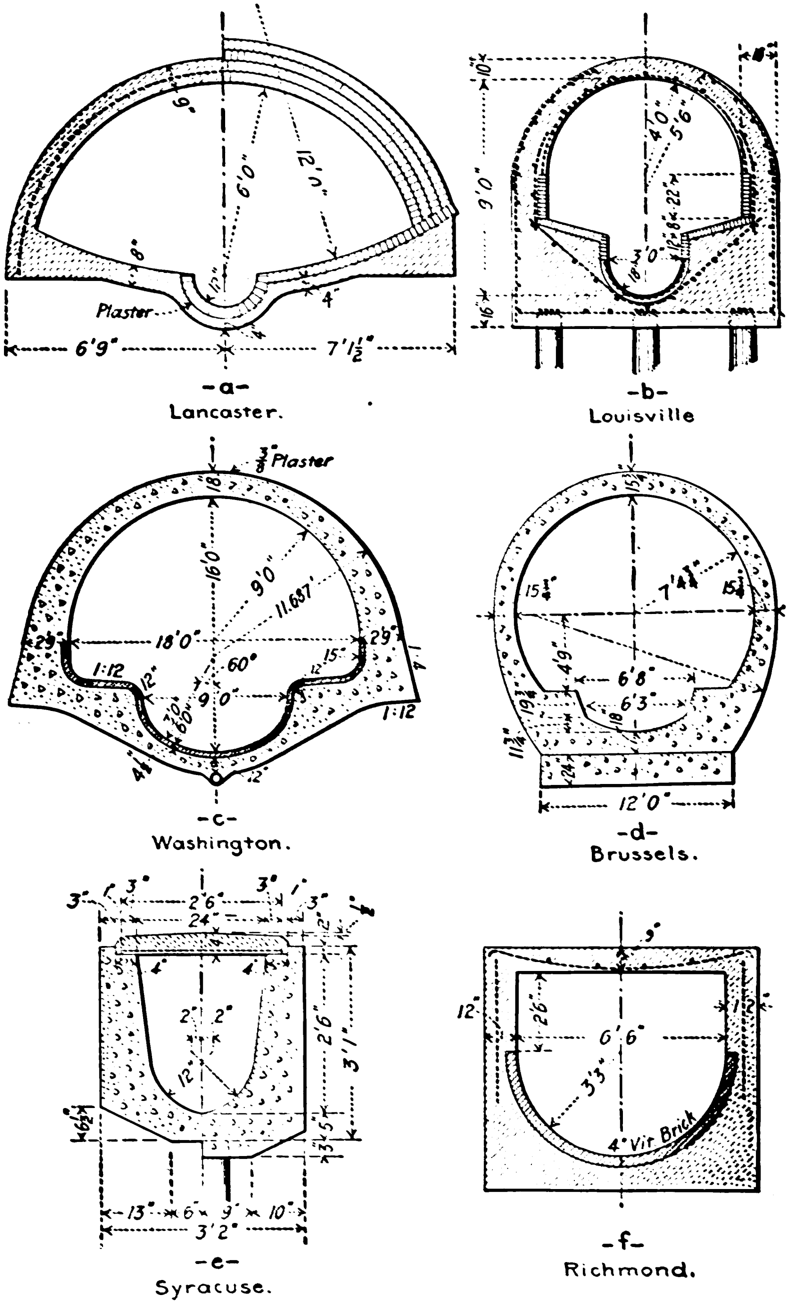


FIG. 66.—Cunette and U-shaped sewer sections.

similar structures, should be the result of a careful consideration of the forces to be encountered and an analysis of the stresses as determined by the best available methods. This applies particularly to the larger sewers, 6 ft. in diameter and over.

A study of existing sewers is one of the best guides to safe construction although not necessarily the most economical construction. Empirical formulas founded on experience have some value, but should not be depended upon without an adequate analytical check.

The proper selection of the materials of construction involves not only a comparison of the cost of one material with that of another but also a consideration of the relative wearing qualities of the materials. This is specially true of the materials used for the lining of the invert.

In some localities the erosion of sewer inverts has been a serious problem responsible for the failure of the entire structure. To resist this wear, a lining of vitrified brick has been found satisfactory.

Sewers are subjected to the action of external forces due to surface loads transmitted through the backfill and to the pressure of the back filling material itself. Surface loads may be divided into live and dead loads. The former includes such loads as locomotives and other railroad rolling stock, road rollers and heavy vehicles; the latter includes loads from piles of lumber, brick, coal and other materials commonly stored in commercial and manufacturing districts.

With the advent of reinforced concrete has come a greater need for the careful analysis of the masonry section for large sewers. With sewers constructed of brick or plain concrete, the sewer arch if properly designed is subjected only to compressive stresses and depends largely for its stability on the ability of the side walls or abutments to resist the arch thrust. With reinforced concrete, however, the structure as a whole from invert to crown can be designed to resist heavy bending moments and act as a monolith.

The so-called "elastic theory" presents the most rational and practicable means for the analysis of sewer sections. The method of analysis under this theory as described by Turneaure and Maurer in "Principles of Reinforced Concrete Construction" is one of the simplest and best, but for an analysis of the structure as a whole, particularly where the sewer is to be built in compressible soil, the method developed by Prof. A. W. French for the authors is preferable.[1]

Although the previously mentioned aids in design are of the greatest assistance, there must be behind them all sound judgment coming from experience if the best results are to be obtained.

Compensation for Curvature.—Changes in direction of small sewers are made by curves in the bottom of the manholes. These curves are usually quite sharp and to prevent any checking of the velocity of flow through them many engineers give an arbitrarily determined drop of ½ in. between the beginning and end of such a channel, no matter what may be its length.

Changes in direction of large sewers must always be made by curves, and it is desirable that the radius of curvature of the axis of the sewer should not be less than 5½ times the interior width of the sewer. This is not always practicable, however, and the authors have been compelled to use a radius as short as 16 ft. for quite large sewers. Some compensation must be made for the loss of head due to increased friction on curves, and several engineers estimate the additional loss as $0.5v^2/2g$, where v is the mean velocity and g is the acceleration of gravity. Other designers arbitrarily allow a certain amount of fall between the beginning and end of the curve, the amount being governed by the conditions of each case.

Specific information regarding loss of head due to curvature, is meager and thus far a reliable formula for general application has not been evolved. It is recognized, however, that the losses in curved stretches of channels, conduits or pipes exceed those in straight stretches. The Markmann formula[2] has been used to some extent in the past, but is less used today. A method of compensating for curvature used by the authors is to assume a higher coefficient of roughness, n, to apply in the length of the curve, than that assumed for the straight section.

E. G. Hopson of the U. S. Reclamation Service has reported[3] significant data on observed losses due to curves in open channels. The observations were made with velocities of 7 ft. per second only, which is unfortunate, as the results do not furnish a basis for estimating the variation in resistance with change in velocity. Analysis of the observations made indicates the reasonableness of providing for the additional loss in head due to curvature by

[1] "American Sewerage Practice," vol. i, p. 488.
[2] *Eng. News*, Sept. 29, 1910.
[3] "Gagings in the Concrete Conduit of the Umatilla Project," *Eng. Rec.*, Oct. 21, 1911.

variation of the coefficient of roughness in Kutter's or other flow formulas.

In Table 23*a* are shown the results of analysis of data, reported[1] by others, upon the apparent effect of curvature upon the value of n in Kutter's formula.

The analysis indicates that the loss of head due to curvature increases with velocity and with decrease in length of the radius of the curve. The range of increase in the value of n appears to have been from one-tenth to one-third, with velocities varying from about 2 to 20 ft. per second. The corresponding amount of increase in n was from about 0.002 to 0.004.

The engineers of the Miami Conservancy District[2] concluded that

> "The value of 0.025 which is applicable for a smooth uniform straight channel in gravel, free from vegetation or other obstruction, may be increased 10 per cent or more by curvature in alignment."

In one instance in designing a channel in which there were many curves and reversals of curvature, the radius of curvature often being of necessity very short, and in which a velocity of about 10 ft. per second was involved, the authors used in their computations a value of n in the Kutter formula 0.005 greater than the value used for a straight channel, giving the following values in different materials:

Channel	Value of n	
	On tangents	On curves
1. Concrete lining	0.013	0.018
2. Smooth, firm gravel	0.025	0.030
3. Well-built rubble	0.017	0.022

These values of n were applied in this case by determining a weighted average and using it throughout the length of a section of combined curves and tangents, instead of constantly changing from one value of n to another as each curve or tangent was

[1] Data for the first analysis were obtained from *Eng. Rec.*, Oct. 21, 1911, and for the following analyses from *Bulletin* 194 of the U. S. Department of Agriculture.

[2] "Technical Reports," Part IV.

TABLE 23a.—APPARENT EFFECT OF CURVATURE UPON THE VALUE OF n IN KUTTER'S FORMULA; CALCULATED FROM SOME EXPERIMENTAL DATA

Channel (all concrete lined)	Value of n				Radii of curves, feet	Velocity of flow, feet per second
	On tangents alone	On curves alone (computed)	Per cent of increase	Addition for curvature		
1. Umatilla Conduit	0.0135 to 0.0137	0.0176 to 0.0184	30 to 34	0.0049 to 0.0039	50	7
		0.0162 t 0.0169	20 to 23	0.0025 to 0.0034	100	7
		0.0160 to 0.0173	18 to 26	0.0023 to 0.0038	250	7
2. Sulphur Creek Wasteway	0.0108	0.0140	30	0.0032	2,865	20
3. Ridenbaugh Canal	0.0121	0.0145	20	0.0024	955	3.7
4. North Canal, Central Oregon Irrigation Company	0.0177	0.0202	14	0.0025	410 and 383	3.0
	0.0176	0.0205	16	0.0029		2.9
	0.0192	0.0222	16–	0.0030		2.1

encountered. Thus, if the length of a certain section of concrete channel was composed of 25 per cent curves and 75 per cent tangent, a value of $n = (0.25 \times 0.018) + (0.75 \times 0.013) = 0.014$, was used in that section. It is to be hoped that at some time in the near future some experimenter may investigate and measure such losses in order to determine a more rational and accurate basis for safe allowance for them.

Velocity Head.—The absorption of head in the creation or increase of velocity and the possibility of its partial or complete recovery by proper design, are hydraulic factors which are often negligible under low velocities but are of serious importance if the velocities are relatively high. Fall must be provided not only for losses due to friction, bends, etc., but also for "velocity head," so called. This head, expressed by the term $v^2/2g$ increases very rapidly, by the square of the velocity, being 0.25 ft. for 4 ft. per second velocity, 1 ft. for 8 ft. velocity and 4 ft. for 16 ft. velocity. At velocities normally found in sewers in flat territory, the velocity heads may be inconsiderable, but this hydraulic element should be considered in design, and rejected only if relatively unimportant in the problem under analysis, as failure to provide for it has frequently led to serious loss of capacity, unexpected choking and overflow of sewers, and other inconvenience.

Some cases which require special attention in this particular may be cited. A large sewer with slow velocity changing for any reason to a smaller sewer with high velocity; grit chambers, where sewage moving slowly through the enlarged chambers must increase its velocity to flow away through an outlet passage or conduit of smaller dimensions; effluent conduits or channels leading away from sedimentation tanks; restrictions in conduits, such as gates or contractions necessary to pass other structures; in short, any cases where velocity must be built up to initiate flow in a channel or conduit.

Recovery of Velocity Head.—When velocity is reduced by enlargement of a channel or conduit a considerable proportion of the change in kinetic energy may be recovered as potential energy, manifested by a rise in water surface, provided the change in section is smooth and gradual. The U. S. Reclamation Service has observed the extent of this recovery in numerous instances, see Table 23*b*, where irrigation canals have been contracted in section and then enlarged again further downstream.

The principle of this recovery of velocity head is well illustrated in an analogous problem, that of water flowing under pressure in the Venturi meter. In it the outlet cone is gradually expanded so as to recover practically the entire difference in velocity head between throat and outlet.

TABLE 23b.—RECOVERY OF VELOCITY HEAD IN TRANSITION SECTIONS FROM EXPERIMENTS BY U. S. RECLAMATION SERVICE

Location and name	Velocity, feet per second		Percentage of velocity head recovered	Discharge, cubic feet per second
	Up-stream end	Down-stream end		
North Platte Project	5.79	1.64	54	150
	5.87	1.70	25	105
Hope Creek Flume	5.67	1.65	28	99
Uncompahgre Valley Project, Happy Canyon Flume[1]	6.47	2.85	100 nearly	361
Tieton Main Canal Tunnel Outlets:				
1. Trail Creek	12.12	8.06	84	271
2. Columnar	11.42	7.98	60	261
3. Tieton	11.58	7.60	68	286
4. North Fork	10.25	2.62	100	286
Okanogan Project:				
Experiment Number IV	5.39	2.61	50	51
Experiment Number VI	3.31	2.46	62	

[1] Transition unusually favorable for recovery.

Problems

The following problems may be solved to test the students' grasp of the fundamental principles of this chapter before proceeding to the application of these principles in problems such as those worked out in detail in Chapters II and III.

1. A sewer 800 ft. long with a fall of 2.4 ft. must discharge 10 cu. ft. per second when flowing full. What size of circular vitrified-pipe sewer is required? What will be the velocity of flow? What coefficient n should be used in the Kutter formula?

2. In the sewer of Problem 1, a minimum rate of flow of 0.75 cu. ft. per second is expected. Find the depth and velocity under these conditions.

3. Re-design the same sewer using an egg-shaped section flowing full and compare the depth and velocity at minimum flow with the results of Problem 2.

4. In an existing sewer system, a 96-in. concrete sewer is found to have been laid on a slope of 0.00085 and its condition is found to warrant the use of $n = 0.013$. What is its capacity and the corresponding velocity of discharge in it when flowing full-depth? When flowing 0.65 depth? When flowing 0 3 depth?

5. The sewer of Problems 1 and 2 discharges into a concrete trunk sewer (n = 0.013) 48 in. in diameter on a slope of 0.002 and which carries a minimum flow of 9 cu. ft. per second above the junction. For topographical reasons it is impracticable to build the crowns of the two sewers coincident in order to avoid surcharging the smaller sewer when the larger is flowing full; it is, however, desired to avoid backing up in the smaller sewer at times of minimum flow in both sewers. At what elevation, above the invert of the trunk sewer, must the invert of the lateral be connected?

6. By the method involving the use of tables for slope 1:100, find the size of circular sewer (with n = 0.013) required to carry 32 cu. ft. per second on a slope of 0.0025 when flowing full. Find also the size of egg-shaped sewer to carry the same quantity when flowing 0.75 full.

7. A natural stream is to be replaced by a rubble-lined channel (n = 0.017) for a length of 2,750 ft., including 800 ft. of curves. Assuming that consideration of expected velocities and the radii of the curves leads to the decision to compensate for the extra slope required on curves by the addition of 0.004 to the value of n on the curves, determine a balanced value of n to use throughout the entire length of the proposed channel.

8. A sewer discharging at 3 ft. per second, when full, comes to an abrupt pitch such that the new velocity will be 9 ft. per second when full in a sewer of properly reduced diameter or section. What is the required drop in the hydraulic gradient to give this increased velocity?

9. Show by a sketch the relative position of the two sewer sections to meet the conditions of Problem 8 without permitting surcharge of either sewer section.

10. A series of grit chambers in which the velocity is 1 ft. per second terminate in an outlet channel in which the velocity is 6 ft. per second. The flow from the chambers must pass through gates which reduce the cross-section of flow at the point of passage to 60 per cent of that available in the grit chambers. Estimate the total loss of head, including the loss of head through the gates and that necessary to produce the increase in velocity in the outlet channel but neglecting the friction loss.

11. A series of grit chambers in which the velocity is maintained at about 1 ft. per second converge into a single outlet channel. For what velocity shall the outlet channel be designed so that the loss of head (drop in water surface) involved in increasing the velocity shall not exceed 0.5 ft., out of which an allowance of 0.15 ft. must be made for the losses through gates, etc.

12. A conduit flowing at a velocity of 7 ft. per second passes through a transition section into a larger section in which the velocity is 4 ft. per second. The water surface in the larger section is found to be 0.3 ft. above the hydraulic gradient, as determined by the normal slopes required for friction alone. What is the percentage of recovery of velocity head in the transition from the smaller to the larger sections?

The student should now return to the problems at the end of Chapters II and III and follow through the method of solution there illustrated by which the sewer sizes corresponding to the basic data relating to quantities, etc., previously worked out, were determined.

CHAPTER V

SEWER APPURTENANCES AND SPECIAL STRUCTURES

A variety of accessory structures were defined in Chapter I which will be described in detail in the present chapter. These structures have never been standardized and each engineer designs them to fit the local conditions, relying mainly upon judgment based upon his previous experience and a knowledge of what has proved satisfactory elsewhere.

STREET INLETS AND CATCH BASINS

Where storm water is removed underground, street inlets are provided to take it from the gutters to the sewers. Catch basins are employed where this street wash contains so much refuse of various kinds that it is better to give it a chance to settle in an easily cleaned basin rather than to allow everything to flow without check into the sewer.

Street inlets should be located so that they will keep the gutters in such condition, even during a heavy rainstorm, that teams may drive close to the curb and pedestrians cross the street with minimum inconvenience. Officials responsible for the maintenance of gutters have found that inlets increase the difficulty of keeping the gutters in repair, because the wheels of vehicles wear away the pavement about the inlets more rapidly than where there is no break in the smooth surfaces of the street and curb. Consequently they often oppose the installation of sufficient inlets to remove the street wash, particularly on steep grades, and oppose the lowering of the gutter grades at inlets so as to leave slight depressions in the gutters to assist in diverting water into the inlets.

The best guide in locating inlets is knowledge of the quantity of water which actually flows along gutters in a given city. In the absence of such knowledge, it is customary to assume that inlets should never be more than 300 to 350 ft. apart where gutters must be kept free from more than a small quantity of water, and in no case more than 700 ft. apart. An inlet must always be placed where two grades join to form a valley. The gutters should be so constructed, the cross-section of the street so planned, and the inlets so placed that storm water will not flow across the pavement to reach an inlet. Although it is customary in many places to locate inlets at the angle of street corners, this is a poor place for them if the travel on the street is heavy, because the wheels of trucks rounding the corner close to the curb are

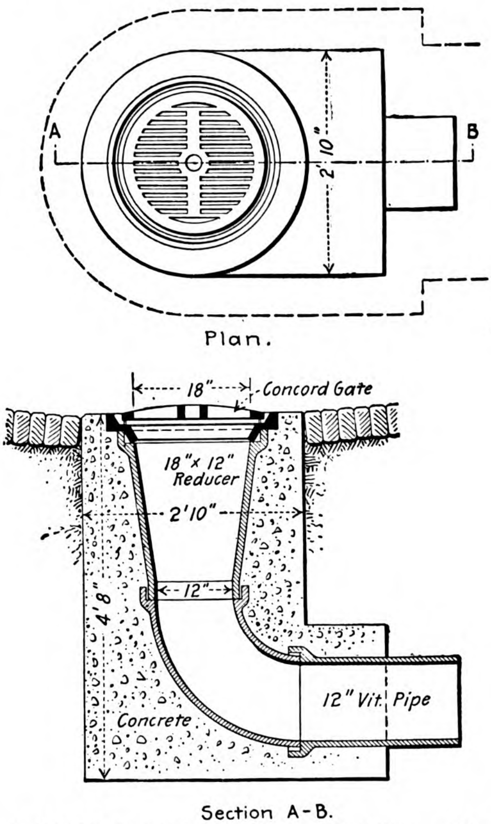


FIG. 67.—Standard inlet for park and light-traffic drives, with Concord grate (Metcalf and Eddy).

particularly hard on both pavement and inlet castings in such a position. An inlet at each side of the corner, just before the cross walk is reached, is a preferable arrangement.

Fig. 67 shows a type of inlet which the authors have found very satisfactory in parks and streets with light travel. Durability is afforded by the substantial concrete block, and other advantages are relatively low cost, a large grate area and ease of construction. The authors' experience with inlets having risers of straight pipe is that the gratings do not have adequate openings. Therefore a flaring mouthpiece is inserted in the end of the pipe to give a larger opening.

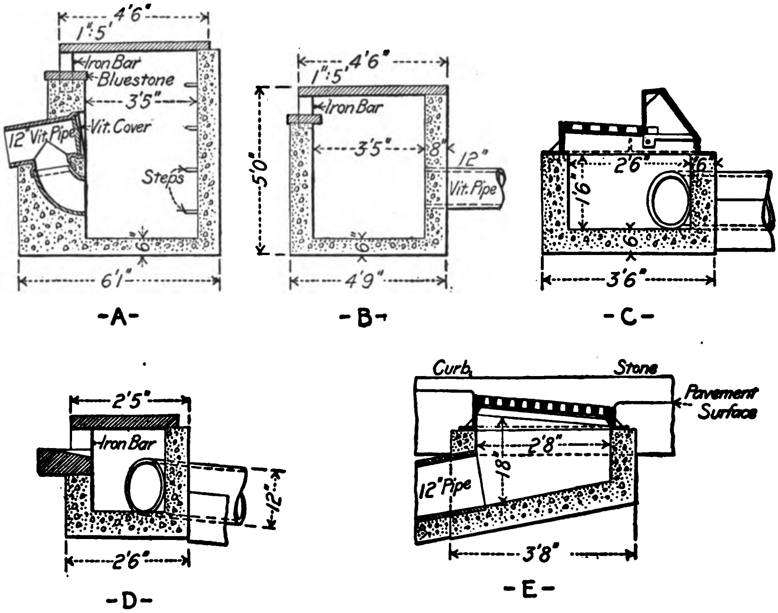


FIG. 68.—Standard street inlets, Borough of the Bronx.

Fig. 68 illustrates a number of types of street inlets adopted as standard in 1913 in the Borough of the Bronx, New York. Their main dimensions are:

Type	A	B	C	D	E
Inlet opening, inches..	7 by 32	7 by 32	....	5 by 36	
Box depth, inches.....	66	54	18	20	
Box section, inches...	41 by 42	41 by 32	30 circular	36 by 18	14 by 32

Sticks, waste paper and leaves cause most of the clogging of the openings of the inlets, and some engineers accordingly use openings which present hardly any obstacle to the entrance of these three classes of refuse. It is questionable, however, whether sticks should be allowed to enter an inlet owing to the danger of their becoming fixed in the pipe leading to the sewer.

As a general rule, street inlets are better adapted for busy streets with good pavements that are kept clean, particularly where there are no steep grades nor any topographical conditions tending to concentrate the storm run-off at a few points, than they are for streets furnishing large quantities of refuse rarely removed by street cleaning and liable to have the run-off concentrated at a few points to which many of the storms are certain to wash a large part of the street litter of every sort.

Catch Basins.—The catch basin was formerly considered an absolutely essential part of any American combined sewerage or drainage system. The velocity of the sewage in many sewers had proved insufficient to prevent the deposit of sludge within the sewers, and it was more expensive to remove this sludge from sewers than from catch basins. Since that time sewers laid on self-cleansing grades have become standard practice and streets are better paved and better cleaned, so that less refuse is flushed off by the street wash and consequently catch basins are now much less favored by engineers. The system of cleaning streets by heavy flushes of water, washing the refuse into catch basins, should not be adopted until an investigation has been made to ascertain which of the two general systems of cleaning is more economical in a given case, sweeping up the refuse and removing it before it gets into the catch basins, or flushing it into the basins and later removing it from them. If the latter plan is adopted the street cleaning department should be charged with a part of the cost of cleaning the catch basins.

While there is not the imperative need for catch basins at frequent intervals which was formerly believed to exist, they have their uses where large quantities of grit will be washed to the gutters and, if it reaches the sewers, will obstruct them. If they are used, they must be kept clean. This work is expensive and often neglected. In some localities, where water remains in them for some time during the summer, they become breeding places for mosquitoes. The practical objections to them were summarized as follows in 1914 in a report by the Metropolitan Sewerage Commission of New York:

"Theoretically desirable, catch basins are, in reality, among the most useless devices employed for the removal of solid material from sewage. They are generally ineffective because they are not cleaned with sufficient frequency to enable them to serve as traps. It seems impracticable to keep them clean. To maintain catch basins in serviceable condition requires much hand work and this is costly. The work is usually carried on to the annoyance of pedestrians and householders. Some sewerage systems are without catch basins and their elimination, as a general procedure, is much to be desired."

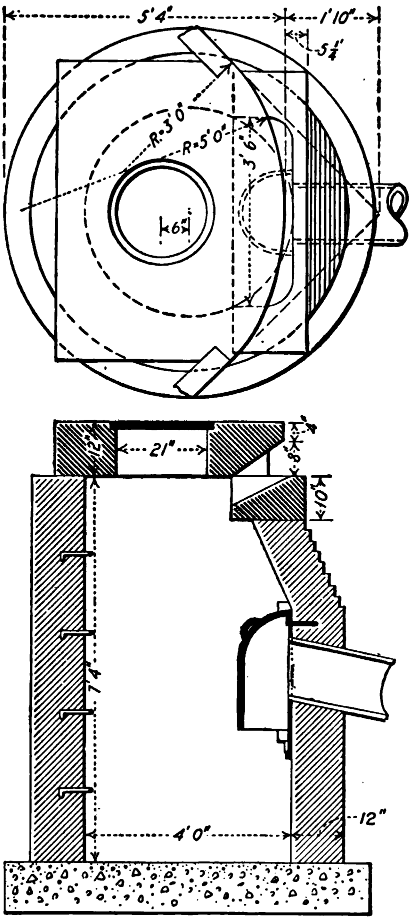


FIG. 69.—Standard catch-basin, Borough of Manhattan.

At the time this report was made, the standard catch basin in the Boroughs of Manhattan and the Bronx, New York, was that illustrated in Fig. 69. The drawing shows an inlet at the angle of a street corner, but the same construction of the basin was standard with straight inlets used between street corners. The hood which formed the trap was hung from a plate set in the masonry. The hood could be lifted off this plate and its purpose was to prevent the entrance of floating refuse into the pipe leading to the sewer.

A type of basin which long found favor, particularly in New England, is illustrated in Fig. 70. This type is put out of service automatically when it becomes filled, and its special feature is its trap. As sediment collects, it reduces the space available for water above its top and below the water line established by the lip of the trap. Eventually there will be very little water capacity, and in summer, in prolonged dry weather, the water will evaporate to such an extent that odors may arise from the catch basin. If no odors arise and the cleaning gang does not reach the basin in its regular routine, the sediment will gradually collect until it overflows the edge of the trap, blocking it. When this occurs the first heavy storm will give undeniable evidence of the necessity of cleaning. In this way the trap serves a useful purpose by preventing the escape into the sewer of large quantities of silt which might form deposits. Another advantage of this basin, due to its trap, is that the water which accumulates in it can be bailed by the cleaning gang into the trap and thus delivered directly into the sewer, instead of being lifted to the top and thrown over the street. The great disadvantage of the trap is its liability to freeze in cold weather, although it should not be forgotten that the air inside the sewers, which will come up to the sewer inlet, will be somewhat warmer than the outdoor atmosphere, and the sheltered position of the trap also has some effect in reducing the danger of this nature. Where basins are connected to storm drains there will be much greater danger of the freezing of traps. Like all attempts to use traps on catch basins or inlets, the permanence of the water seal is very questionable. It will evaporate during prolonged dry weather and it is idle to expect that a sewer department will keep all traps filled by means of a hose during such weather.

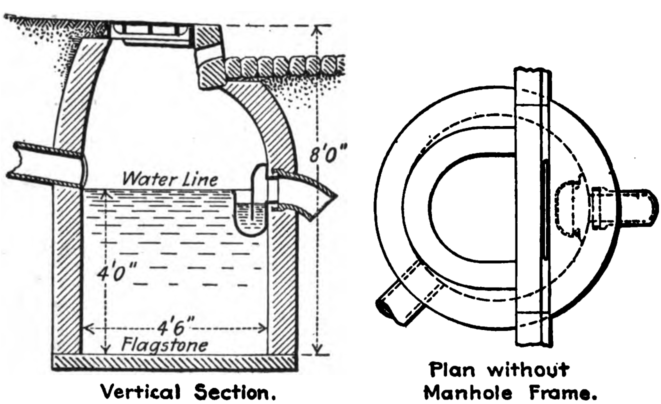


FIG. 70.—Standard catch-basin, Providence.

Attempts have been made in some cities to use traps formed of vitrified pipe elbows, but this practice is inadvisable because of the liability of damage to the elbows when the basins are cleaned. Cleaning is rough work done as quickly as possible and everything within a catch basin must be designed to withstand hard usage.

Castings for Inlets and Catch Basins.—It is difficult to obtain from the practicing engineer any definite statement of his reasons for preferring one type of casting to another. The Concord grate, shown in Fig. 71, has been used for many years in New England where the gutters are not likely to have heavy wagons passing over them. Of all the commercial types, it probably affords the easiest means for the storm water to enter the inlet, provided it does not become clogged with leaves, which experience shows have a marked tendency to accumulate. The North Berwick catch-basin head is of much the same type except that it is heavier and there is no entrance for the water around

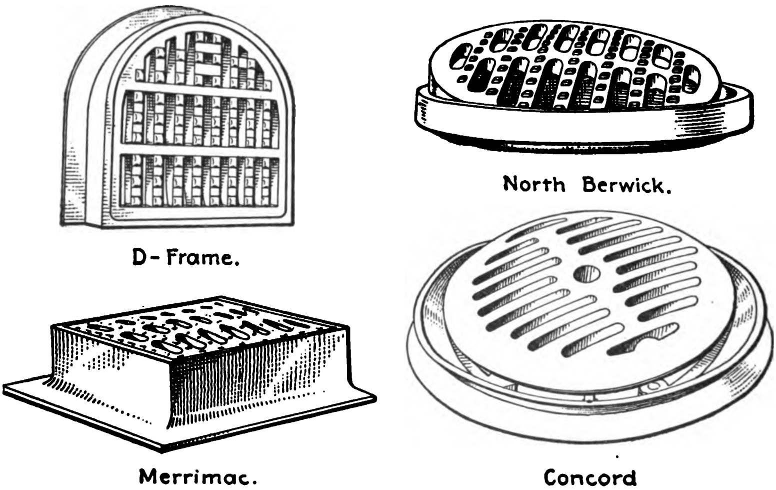


FIG. 71.—Types of commercial catch-basin covers.

the rim. The D-pattern frame and grate were long used in Boston, but about 1910 a rectangular frame was adopted. There seems to be a general tendency toward the use of these rectangular frames, of the general type shown by the picture of the Merrimac frame, Fig. 71. They have two decided advantages over types having curves. The first is that it is practicable to keep the pavement of the gutter in better condition with a square than a curved casting for it to rest against. The second advantage is that the grate can be made as strong as desired without much difficulty and still have a large area available for the passage of storm water into the inlet. The Borough of the Bronx adopted in 1913 a cast-iron inlet head shown in Fig. 72, which has a curb opening as well as a gutter drain. Whatever type is adopted should afford an opportunity for securely bedding the frame upon the masonry of the inlet or catch basin, for otherwise it will be-

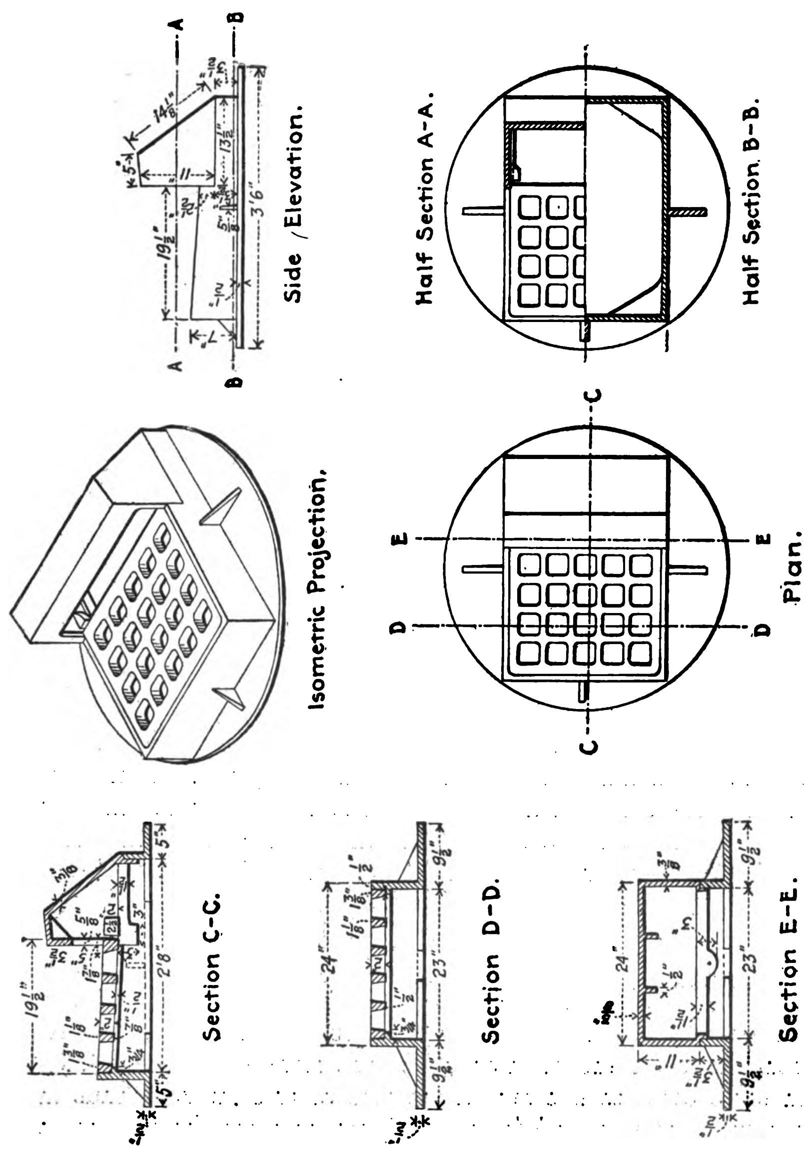


FIG. 72.—Inlet head, Borough of the Bronx.

come loosened speedily and in rocking under passing vehicles it will destroy the pavement about it.

The material from which the frames and covers are made is rarely specified. Even where it is specified it is unlikely that it is tested or inspected at the foundry. Certain foundries are known to furnish good castings and when they sublet such work to other foundries they hold up the quality of the product in order to protect their own reputation. There is a danger in very loose specifications and lack of proper inspection, particularly in times of business depression. A foundry in a territory not ordinarily serving the city may conclude that it can manufacture poor castings which will be just good enough to be accepted under the existing conditions and it can afford to send out such poor castings because it will probably never desire to do business with the city again, owing to the freight rates against it. This danger can be avoided by requiring the castings to meet the standard specifications for gray-iron castings of the American Society for Testing Materials and adding a clause requiring them to be painted with an asphaltum, coal-tar or graphite paint or other covering desired by the engineer.

MANHOLES, DROP MANHOLES AND WELLHOLES

Manholes did not come into general use on sewer systems until experience in cleaning sewers without them proved they were necessary. The chief opposition to them was based on the belief that they would permit the escape of sewer air and that this air was poisonous, an opinion not surprising when one reads about the evil odors from some old drains. In the early days of sewerage, when a sewer became so badly clogged that it must be cleaned, it was customary to dig down to the sewer, break through its walls, remove the obstruction and then rebuild the walls. Nowadays with the better design and greater care to get self-cleansing velocities in the sewage flow there is much less trouble and nuisance of this sort.

Manholes have commonly been placed approximately 300 ft. apart for convenience in entry and cleaning of the sewers, but the frames and covers cause serious inconvenience and expense in repair and maintenance of the highways, which has led in some cases to substantial increase of this interval; thus in a number of cases in Worcester, Eddy increased the spacing to intervals as large as 1,000 to 1,200 ft. without subsequent disadvantage. It appears likely that the substantial increase in cost of manholes, now nearly treble the pre-war cost, will lead to further examples of this sort. Manholes should, however, be placed at all intersections of the sewer for convenience in access and cleaning, regardless of the interval.

Fig. 73 illustrates a standard form of manhole for pipe sewers. It is generally considered desirable to have the walls of the channel rise nearly to the crown of the sewer and to have a berm on each side of the channel and sloping slightly toward it. In the Newark manhole the berms are a little higher than is often considered necessary.

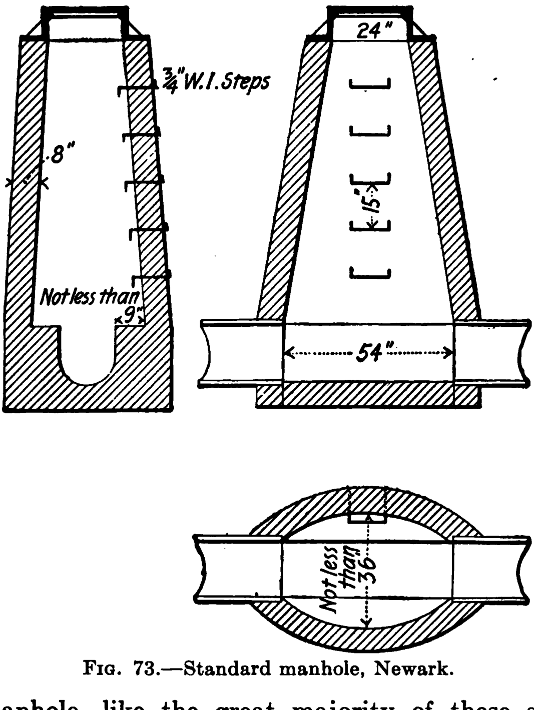


FIG. 73.—Standard manhole, Newark.

This manhole, like the great majority of these structures, is built of brick, although under some conditions concrete may possibly be used to advantage, particularly where a number are to be built so that standard forms can be utilized, or where the manholes are very deep and require considerable masonry. The expense of procuring forms and the delay which their preparation frequently entails, the difficulty of placing them and of placing the steps in the concrete, and the small quantity of concrete which is used, generally make it less expensive to employ brick in ordinary manhole construction.

Where the sewer is much larger than the diameter of the manhole, the outside of the latter is usually made tangent to one side of the sewer, for otherwise it will be difficult to enter the sewer and a special ladder will be required to reach the invert. Occasionally, on very large sewers, the manholes are built entirely apart from the sewer proper and have a passage leading into it, as shown in Fig. 74.

The manholes of small sewers are usually made about 4 ft. in internal diameter when of circular cross-section, or about 3 by 4 ft. when oval. The same size is usually maintained for all sewers except when special conditions may require manholes of larger

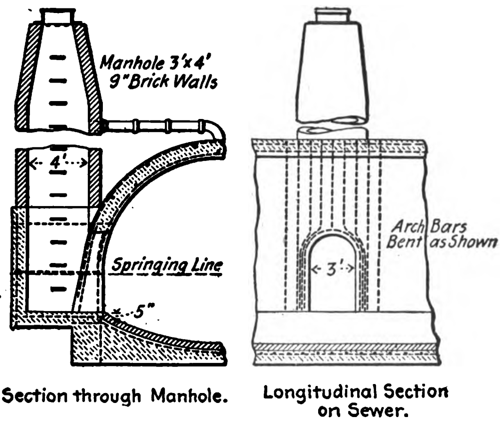


FIG. 74.—Manhole on large St. Louis sewer.

size, as where gaging devices must be used at the bottom of the manhole or it is desired to have a considerable storage capacity in the manhole chamber to enable it to hold enough sewage for flushing a long line of pipe on a flat grade. Brick manholes are usually built of 8-in. brickwork down to a depth of 12 to 20 ft., although for many years the manholes on Cincinnati sewers, and possibly those in some other cities, have been built of a single ring of brick. Below the depth stated, 12-in. brickwork is used, as a rule. The sides are carried up vertically to within 3 or 4 ft. of the top and the upper part is corbelled in or laid in the form of a dome or reverse curve, Fig. 76.

The four manhole bottoms shown in Fig. 75 illustrate somewhat different types of design. The Memphis and Seattle bottoms have flat lower surfaces while the Concord and Syracuse bottoms have lower surfaces curved to correspond with the channels through them. Which type is better adapted for the soil conditions at any site can be ascertained only by examination;

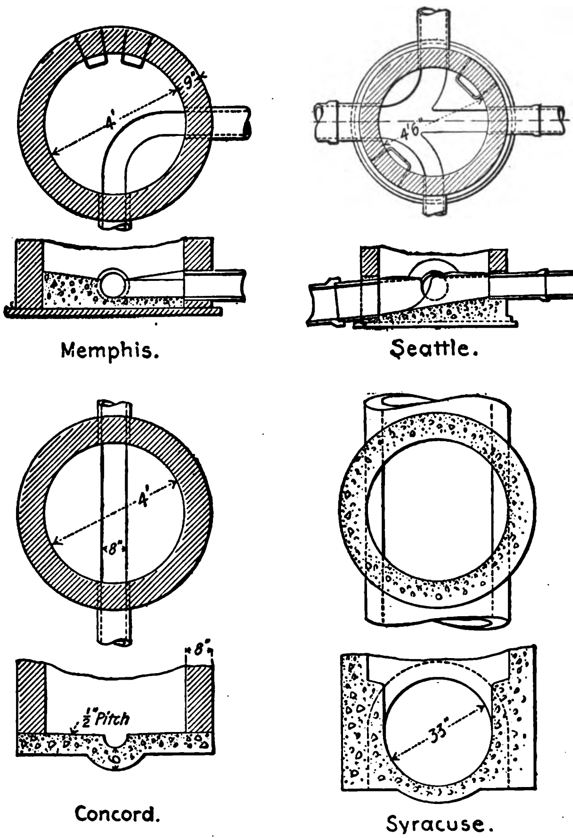


FIG. 75.—Types of manhole inverts.

the saving in material in the second type may be counterbalanced by a higher unit cost. While the base of each manhole illustrated

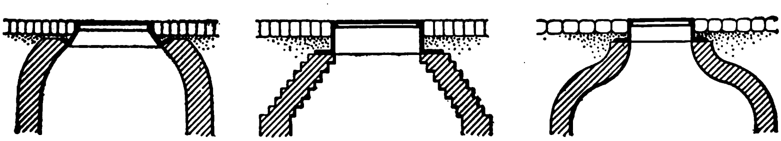


FIG. 76.—Types of manhole tops.

was constructed of concrete, as a matter of fact a good sewer mason can lay up brickwork to form practically any channel that may be desired, and can carry the work on very expeditiously if he is so minded.

Two types of concrete manholes have been used in Syracuse on concrete intercepting sewers. In the first type the manhole has a reinforced shell 6 in. thick, running up from the sewer to within 5 ft. of the ground surface, where a funnel-shaped top

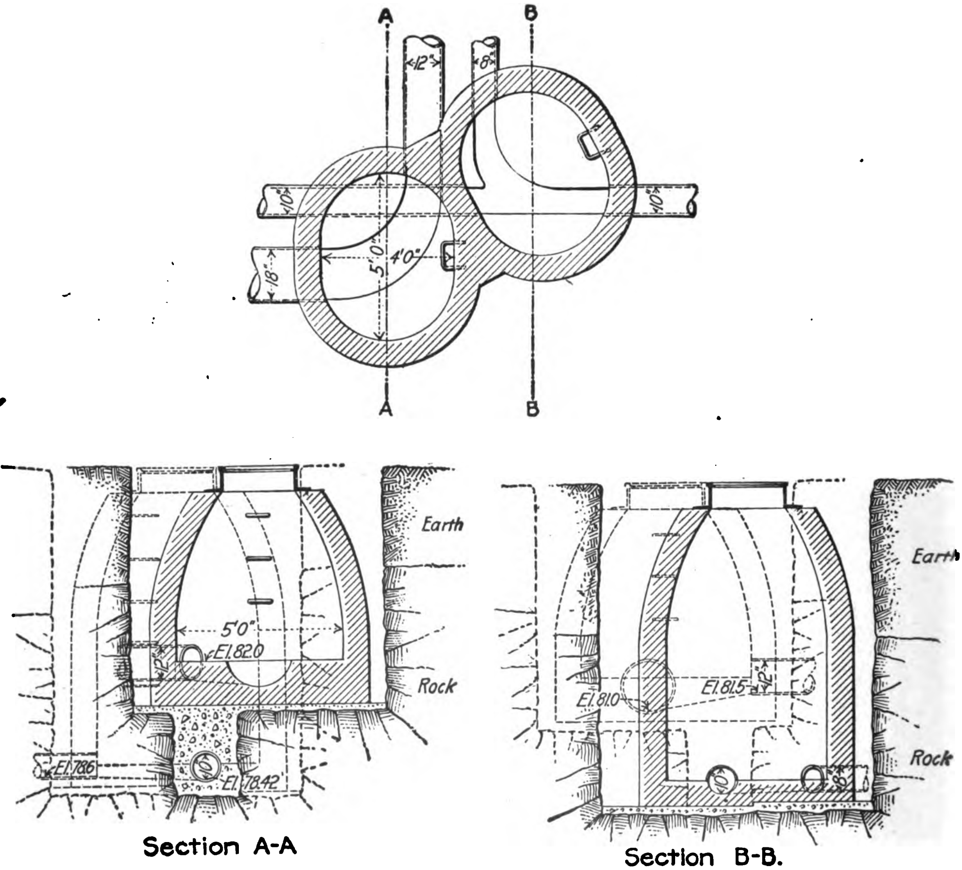


FIG. 77.—Double manhole for separate system.

begins to corbel in. The reinforcement is of ½-in. rods 12 in. apart when horizontal and 18 in. when vertical. The other type of concrete manhole is formed of pre-cast reinforced concrete pipe set on end. The sections are 4 ft. in diameter and 4 ft. long.

Double manholes are sometimes used where sewers and drains are so located as to make them convenient. The structure shown in Fig. 77 was used by the authors for such a purpose on the separate sewer system of Hopedale, Mass. Each chamber is

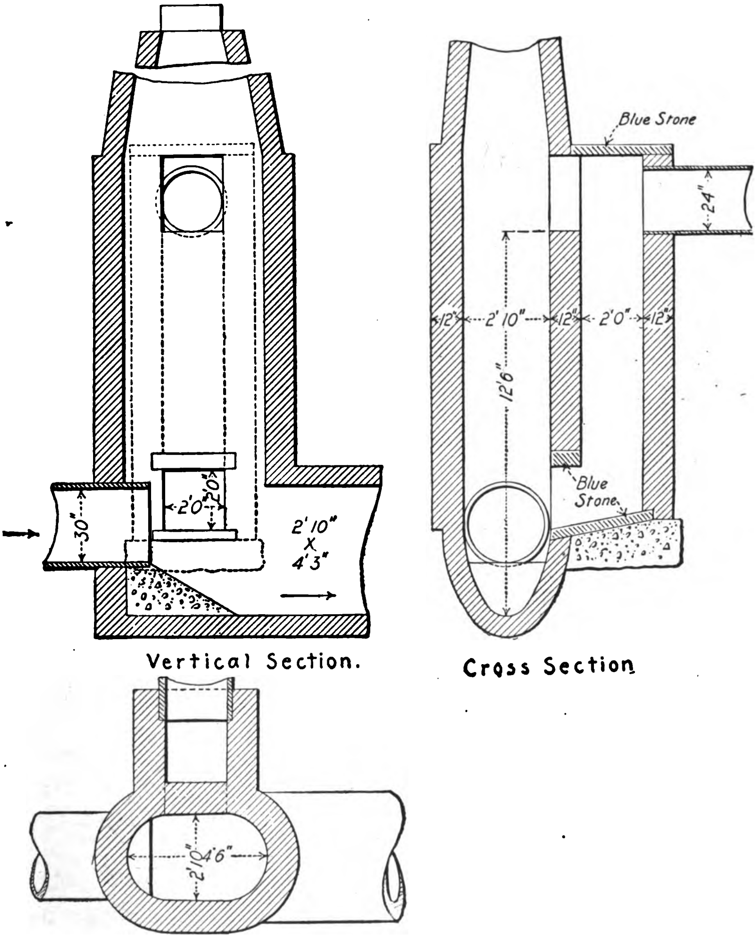


FIG. 78.—Drop manhole, Newark, N. J.

5 by 4 ft. in plan and the dome has a depth of 4 ft. The walls are 9 in. thick.

Where underdrains are employed it is sometimes desired to afford access to them, and in such cases various expedients are

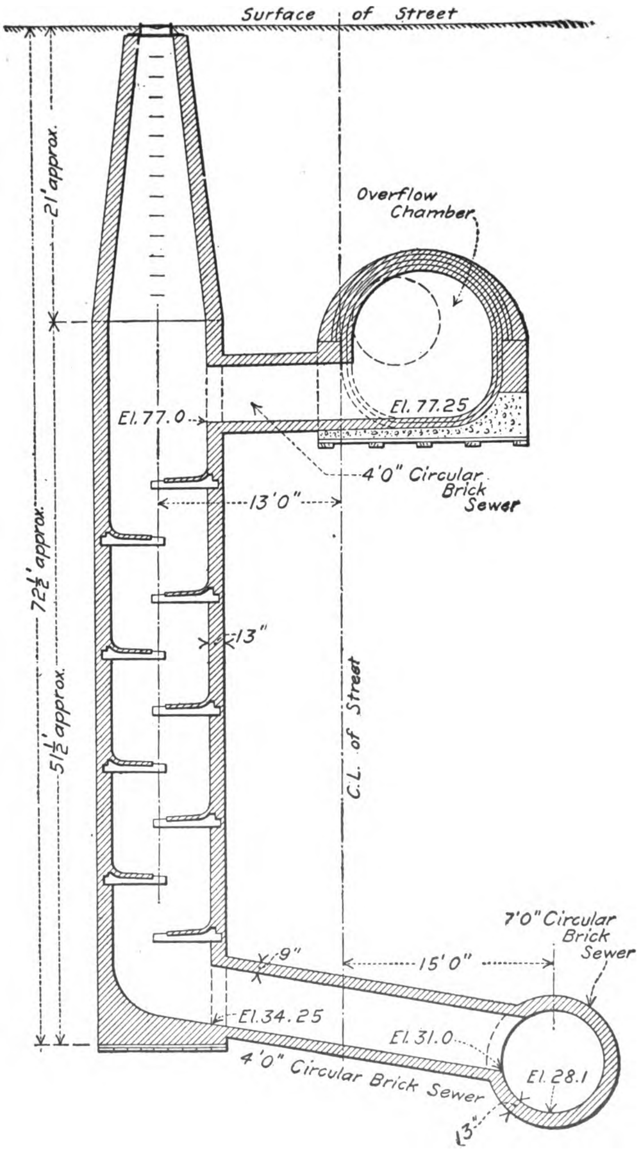


FIG. 79.—Wellhole, Morgan Run sewer, Cleveland.

employed. The most usual one is to divert the underdrain a short distance to one side of the sewer, where it passes under the manhole, and to bring up a riser from the underdrain to the floor of the manhole.

Drop Manholes.—There are many types of drop manholes, of which one of the simplest is shown in Fig. 78. This particular structure is at the head of an oval sewer 51 in. high, into which two circular sewers discharge at different elevations.

Where underdrains are used below sewers which are connected by drop manholes, the underdrain of the high sewer is generally

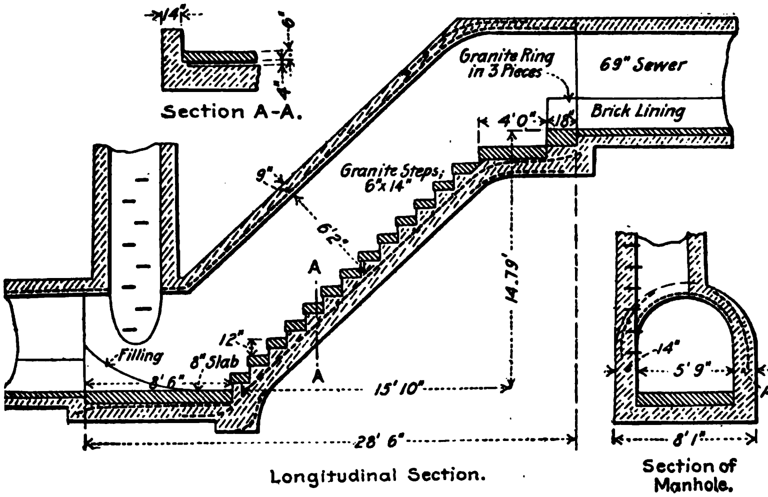


FIG. 80.—Flight sewer, Baltimore.

connected by a pipe bedded in concrete with the underdrain of the lower sewer.

Wellholes.—Where the sewage drops through a considerable distance in the manhole itself, the structure is usually termed a wellhole. Fig. 79 shows such a structure built in Cleveland, where wellholes have been used freely. In some cities, notably Minneapolis, the depth of the wellholes is sometimes over 100 ft. and special attention is paid to lining all parts subject to erosion with granite, granolithic block or other material which will resist the attrition of the sewage. Wellholes are rarely used except where the lower sewer is built by tunneling.

Flight Sewers.—While a drop manhole or wellhole affords a means of changing grade sharply, other devices have been used

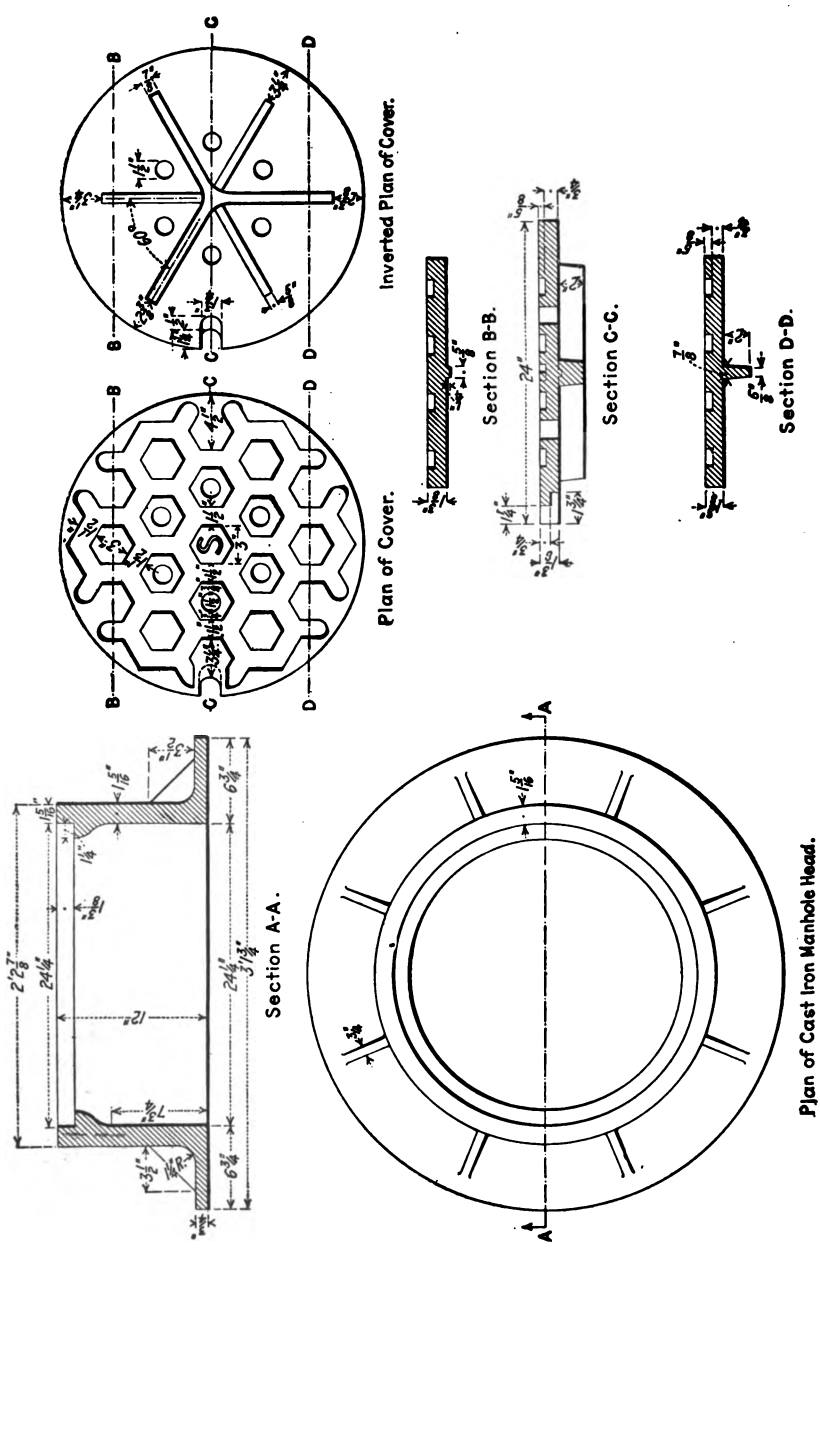


FIG. 81.—Standard manhole head and cover, Borough of Manhattan.

to accomplish the same or a similar purpose. The flight sewer, which gets its name from its resemblance to a flight of stairs, is occasionally used in such situations. It has a steep grade, but steps in the invert tend to check the velocity of the current; the resistance they offer probably diminishes as the depth of the sewage increases and if the descent is long great care should be taken to obtain massive, durable construction and freedom from obstruction to flow at the bottom of the flight. An example of such structures is shown in Fig. 80.

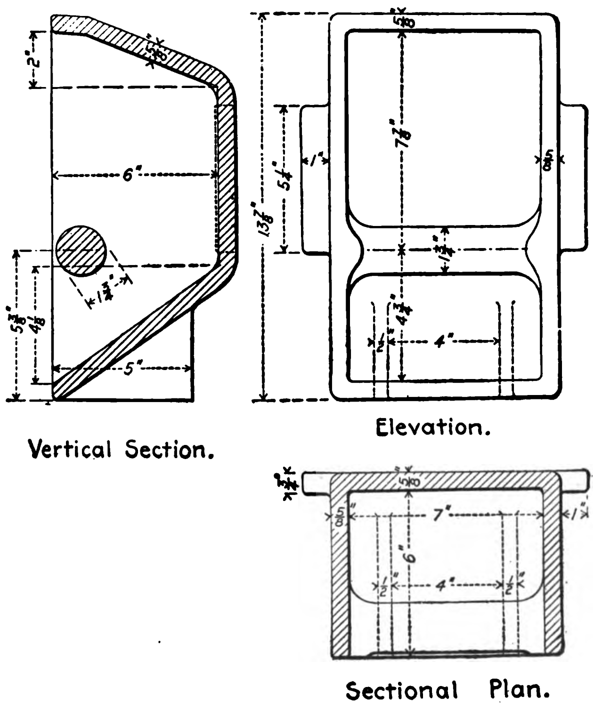


FIG. 82.—Cast-iron box step, Boston.

Manhole Castings.—Experience indicates that the outside face of a manhole frame should be vertical from top to bottom and without projections, for a blank surface apparently enables the pavement resting against it to show a little better resistance to wear than where there are projections at the top of the frame or the latter has a broken surface. Circular frames are universally used as they are better adapted to the service than rectangular frames with which there is a tendency for the formation of ruts in the pavement along the sides parallel to the direction of the traffic. The practice of making the covers rather deep, with a pocket in the top in which asphalt or wood block is placed, has given way to the use of cast-iron covers with the surface broken by a shallow pattern of some sort which will prevent horses from slipping and yet cause no jar when wheels pass over the cover. The standard design in the Borough of Manhattan, New York, shown in Fig. 81, consists of a frame weighing 475 lb. and a 135-lb. cover. The latter is raised by inserting the end of a pick or bar in the recess, *C*. The cover has six ventilating holes.

The use of ventilating covers is considered necessary by some engineers when a system of sewers is first put into service and there are but few house connections with it to afford ventilation. After the system has been in use for some time, there seems to be a general tendency to use closed covers on a considerable portion of the manholes and employ the ventilating covers only where the need for them is evident. The openings in the covers are often closed by oak plugs, but the authors have found that the best way is to have a blacksmith plug them with rivets. Attempts to fill them with cement or an asphaltic mixture are not successful for any length of time, as a rule. In some cases the manhole covers are provided with a groove for a slide below the perforated portion; when the cover is to be closed the slide is inserted in place, closing the bottom of the holes, which are then filled with grout or some equivalent material.

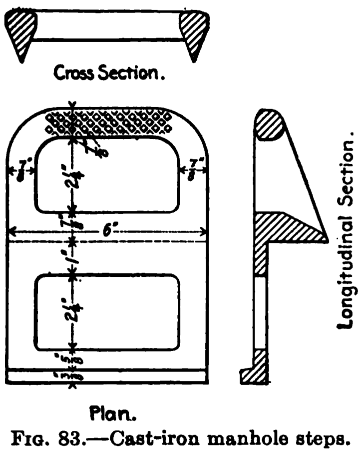


FIG. 83.—Cast-iron manhole steps.

Steps.—In shallow manholes steps were generally provided, until about 1910, by leaving bricks projecting at points about 15 in. apart vertically. They are slippery when wet and liable to become broken, so forgings came into use, as shown in the Newark manhole, Fig. 73. These forgings are usually placed from 12 to 18 in. apart vertically and somewhat staggered. Fig. 82 shows a cast-iron step used in Boston where the manhole must be kept free from projections from the wall. It is one of the best devices in use. Fig. 83 shows a cast-iron step used in the same city where there is no objection to the projection of the step from the wall. When wrought-iron steps are used they should be fashioned out of not less than ¾-in. round iron. If made of steel they corrode and waste in section rapidly.

JUNCTIONS

Where large sewers come together with a horizontal angle between their axes less than about 30 deg., a special structure called a junction is required. For many years these junctions were of the type shown in Fig. 84 and were sometimes called "bell-mouths" or "trumpet arches" on account of their shape. Where, at the springing lines, the outside surfaces of the arches of the two sewers come together, no further attempt is made to have the upper part of the two sewers independent, but a large arch is thrown across the two. A manhole is usually built just in front of the brick wall which closes the large end of the structure.

These bell mouths are usually built of brickwork but sometimes concrete has been used. So much skilled labor is required in constructing them of either material that flat-topped junctions, Fig. 85, have come into use. The particular junction illustrated is rather more expensive than most of this type because it was constructed on a large existing brick sewer which it was desired to disturb as little as possible.

In designing junctions it is desirable to have the two streams of sewage join in such a way that there shall be the minimum checking of the current in either sewer, to prevent consequent deposit of grit and other suspended matter. This condition requires the two streams of sewage to have the same surface elevation as they approach each other and the same velocity, which is not easy to bring about when one sewer is a large trunk sewer and the other is a much smaller sewer. If the invert of the smaller sewer is placed at an elevation which will make the surface elevation of both streams identical during dry weather, the two streams will probably have very different elevations when considerable storm water is flowing away. Consequently a study of conditions at the junction when the two sewers carry various quantities of sewage may show that, to prevent sewage being backed up in the smaller sewer during periods of large discharge, it will be desirable to increase the grade of the smaller sewer somewhat for a short distance from the junction.

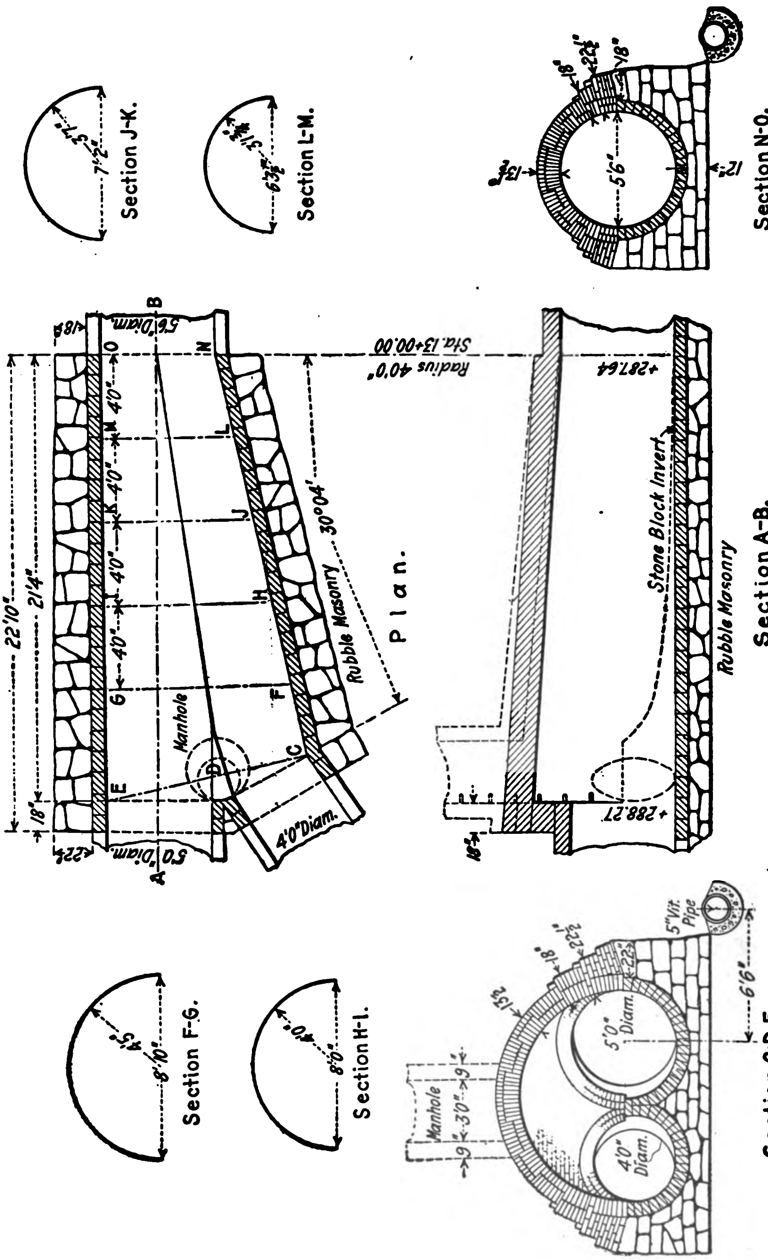


FIG. 84.—Bellmouth junction, Philadelphia.

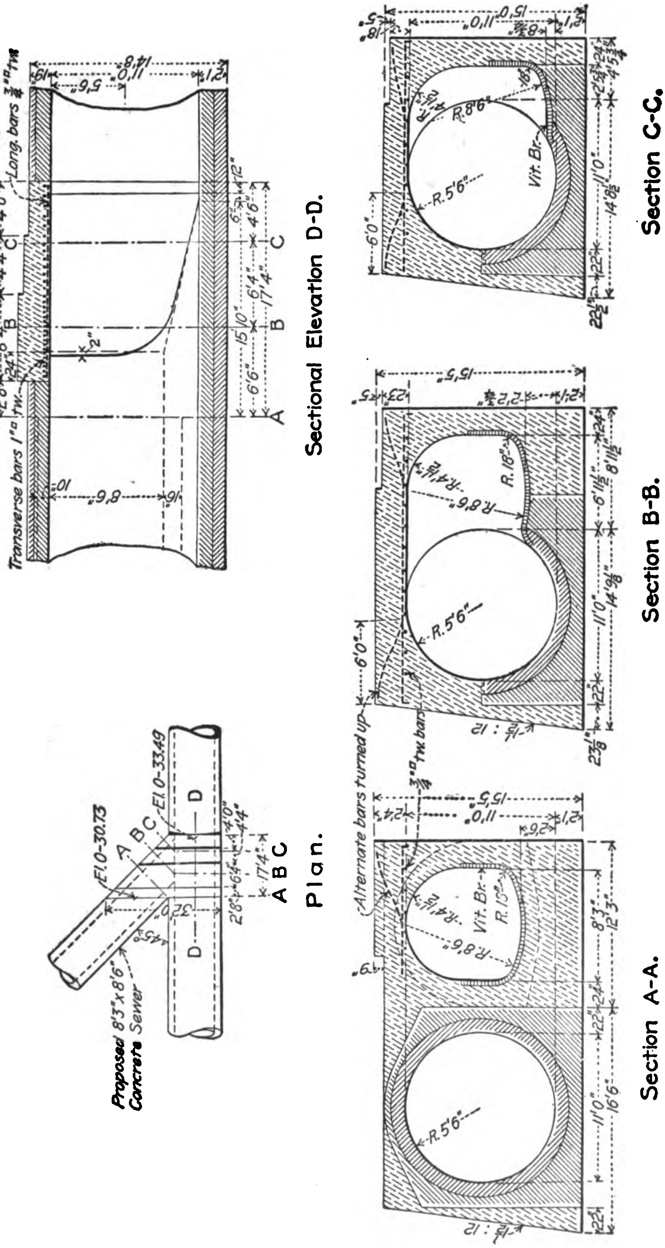


FIG. 85.—Flat-top junction, Pittsburgh.

In large cities the junctions are not always such simple affairs as those shown in Figs. 84 and 85. In Fig. 86 a complicated junction in Philadelphia is illustrated. Here there is a brick sewer 9 ft. in diameter crossing a brick sewer 8 ft. 3 in. in diameter, and the problem was to put in junction chambers and separate sewers in such a way that the course of the larger sewer, beyond this intersection, would serve as a relief for the storm water from the smaller sewer, and that the dry-weather sewage in the latter would flow into the channel which would also carry the dry-weather sewage from the former. This was accomplished by four junction chambers and two 30-in. cast-iron pipe sewers, as shown in the illustration. A very large proportion of the section of the 9-ft. sewer will be utilized before there is any discharge from it into the overflow sewer, while in the case of the 8¼-ft. sewer everything that is not dry-weather sewage will be immediately discharged into the overflow outlet.

INVERTED SIPHONS

An admirable report upon "Inverted Siphons for Sewers" was prepared by a committee of, and presented to, the Sanitary Section of the Boston Society of Civil Engineers in 1921, from which, as well as from "American Sewerage Practice" the discussion herein has been prepared.

General Features of Inverted Siphon.—Any dip or sag introduced into a sewer to pass under structures encountered, such as conduits or subways, or under a stream or a valley, is termed an inverted siphon. While the sewage both above and below the inverted siphon normally but partly fills the sewer, it completely fills the siphon. The siphon is illustrated on a small scale by the running trap on the main house-drain, which formerly was generally used but which is now more often omitted in the interest of better ventilation.

Size and Velocity.—Practical considerations, such as the increased danger of stoppage in small pipes, tend to fix the minimum diameters for inverted siphons about as for ordinary sewers, 6 or 8 in. in the separate system and say, 12 in. in the combined system, although there are rare examples of the smaller sizes. As obstructions are much more difficult to remove from an inverted siphon than from a sewer, especial care should be taken to prevent their formation. As high velocity as possible should be maintained in the siphon, say 2 to 3 ft. per second for

domestic sewage and 4 to 5 ft. per second for storm sewage, although existing conditions frequently make it practically impossible to ensure the development of such high velocities. In such cases correspondingly greater reliance must be placed on artificial flushing and other means of cleaning the siphon. In some cases catch basins or grit chambers have been built just above the siphons, but these are troublesome to clean and the material removed from them is usually very offensive. Siphons should be flushed frequently and their operation inspected regularly, in order to assure prompt removal of obstructions before they form a serious barrier. A serious experience following the clogging of the Dean St. and Fourth Ave. sewer in Brooklyn in 1911 gives point to this.[1]

"The siphon consisted of two 48-in. pipes for the storm flow and one 18-in. pipe for the dry-weather flow, and replaced a portion of the 72-in. circular sewer in Dean St. Four days were consumed in pumping out the sewage and gaining access to the siphon. When it was possible to enter the siphon chamber it was found that the entrance to all three pipes was completely obstructed with a mound of debris consisting of a mattress, a lot of lumber of various sizes, a long-handled shovel, a piece of bluestone curbing, a wooden barrel, several iron pails, umbrella frames, together with rags, wire, paper and other refuse."

Means of Flushing.—The flushing is accomplished in various ways, depending upon the available facilities and surrounding conditions. It may be done by temporarily speeding up the pumps, if there is a pumping station on the sewer line; or by receiving the sewage flow, at the head of the inverted siphon, in flush tanks which automatically discharge when full; or by providing a permanent flushing gate in the manhole at the head of the inverted siphon or else grooves for the temporary insertion of stop planks when desired; or by opening the blow-off at the low point of the pressure pipe of the inverted siphon; or by admitting clean water at the head of the inverted siphon from a permanent connection with an accessible stream, a street water main or through a hose line from a neighboring hydrant; or by cleaning by hand by the use of jointed rods with suitable scrapers or other tools, after draining the siphons.

Manholes or Clean-out Chambers.—Manholes or clean-out chambers should be provided at each end of a siphon, to give

[1] *Municipal Eng. Jour.*, 1917, paper by J. L. Hunt.

access for rodding, pumping and, in the case of pipes of large size, for entrance. There is objection to the introduction of intermediate manholes on an inverted siphon in such a manner that the sewage will be free to rise in them, since grease and other scum tends to fill up the shafts in them, with a solid plug. They are advantageous, however, if the sewage be confined within the siphon as it passes through the manhole, affording access or means of ridding the siphon of deposit, through a gated connection or similar device.

Materials of Construction.—Since an inverted siphon is subjected at all points of its cross-section to an inner pressure, the walls will be in tension, although the amount of tension may be modified if the exterior of the inverted siphon is subjected to external water pressure or to the pressure of earth. On account of these tensile stresses, inverted siphons are usually constructed of steel, iron, reinforced concrete or wood-stave pipe heavily banded, though vitrified pipe has sometimes been used successfully under a small head or if the pipe be encased in concrete.

Size of Pipe for Inverted Siphons.—The computation of the sizes of pipe for inverted siphons is made in the same way as that of sewers and water mains. Their diameter depends upon the grade and the maximum quantity of water to be carried. The latter depends in the case of inverted siphons under rivers, upon the presence or absence of a storm overflow before the inverted siphon is reached, and on the degree of dilution of the sewage before the overflow outlet comes into service. The head or drop in the hydraulic gradient actually required at any time, for the existing flow, will be the difference in level in the free water surface at the two ends of the siphon. It will equal the sum of the friction head determined by the quantity of sewage to be carried and by the character and condition of the interior surface of the siphon, its actual length measured along its axis, the head lost at bends, changes in cross-section, and the loss in velocity head involved by the variation in section. Marked variation in velocity has an important effect upon the velocity head and the total head required. It is to be remembered that the losses will be relatively small for low velocities, other things being equal, but that they increase roughly as the square of the velocity. For a clean 12-in. siphon 50 ft. long and a velocity of 3 ft. per second, a total loss of 6 in. would probably be an outside figure; for a velocity of 2 ft. per second 3 in.; but for 6 ft. per second 24 in.; the friction loss alone being not over one-third of the above values.

Practical experience has shown that where the siphons are built of unduly large size they soon silt up to a point where the reduced section will increase the velocity of flow through it to a point sufficient to maintain the section. Such a condition was observed by A. L. Shaw in a 36-in. siphon under the Brandywine River in Wilmington, Delaware, in which the equivalent reduced diameter of the pipe for the observed quantity and slope was estimated at 17 in. and 24 in. under two different conditions of flow, the former corresponding to an estimated velocity of 2.7 ft. per second, the latter to 4 ft. per second. To overcome this difficulty, experience in this country has dictated the advantage of using several or multiple pipe lines, instead of one pipe line, for the siphon, arranged in such manner as to throw additional pipe lines progressively into action with increase in discharge of the sewage. See solution of siphon problem at the end of this section.

English authors have emphasized the importance of ventilating long inverted siphons, asserting that otherwise the flow may be interfered with by accumulations of air or gas, but American sewerage practice does not seem to have developed such difficulties.

Care must be taken that inverted siphons built on or under river beds, have sufficient material to prevent their flotation.

Essential Principles in Design.—The essential principles in design have been well set forth in a paper by S. B. Bleich,[1] summarized as follows:

1. Complete and effective separation of house sewage and industrial waste from the storm run-off at all times.[2]
2. Simplicity of construction.
3. As slight and easy changes of direction of flow as are practicable at the entrance and exit legs.
4. Easy curve where necessary.
5. Uniform section throughout entire length of siphon pipes.
6. Elimination of all features tending to obstruct flow.
7. Omission of all moving parts and mechanical devices.
8. Provision for easy access to all pipes, without impairing any of the essential features.
9. Entrance and discharge openings to have sufficient area within the available height not to cause any backwater; and change of section or direction not to be too abrupt.

[1] *Municipal Eng. Jour.*, 1917, Paper 110.

[2] This first statement is not a principle, nor is it justified as broadly put.

SOLUTION OF A SIPHON PROBLEM

A problem encountered in the authors' practice involved the design of an inverted siphon of several pipes, to replace an existing single-pipe siphon which had given trouble from sedimentation due to low velocities; its solution is given as an illustration. The basic data are as follows:

Length of siphon	440 ft.
Available fall (invert to invert)	3.2 ft.
Maximum depression of siphon	9 ft.
Gravity sewers connected by siphon:	
Diameter	30 in.
Material	Concrete
Slope	0.0037
Typical rates of flow:	
Minimum	4 cu. ft. per second
Maximum dry weather	13 cu. ft. per second
Ultimate maximum	capacity of gravity sewer

The capacity of the 30-in. concrete gravity sewer on slope 0.0037 (n = 0.013) is about 25 cu. ft. per second, and the velocity flowing full 5.1 ft. per second.

Available fall	3.2 ft.
Assumed loss at inlet	0.4 (see later discussion)
Available for friction in siphon	2.8 ft.

$\frac{2.8}{440} = 0.0064$ approximate available slope.

Local conditions indicate economy in the use of vitrified pipe. The surrounding concrete (primarily for anchorage) can be easily reinforced for the internal pressure due to the 9-ft. depression of the siphon. Standard commercial vitrified-pipe sizes will therefore be selected.

Two conditions must, if possible, be satisfied; the velocity in the pipes selected should be sufficient to insure scouring (3 ft. per second, if possible;) and the available hydraulic slope of 0.0064 must not be exceeded.

Judgment suggests the selection of pipe sizes which will be particularly adapted to the minimum flow, the maximum dry weather flow and the ultimate maximum flow. Three siphon pipes are found to meet these requirements, with regulation at the inlet so that the minimum flow will be confined to one pipe; the maximum dry-weather flow to two pipes; and all three have a capacity equivalent to that of the gravity sewer in which the siphon is to be inserted. The computations follow:

For minimum flow, 4 cu. ft. per second, from Fig. 47, (n = 0.015 for vitrified pipe) 15-in. pipe requires s = 0.006; velocity, 3.3 ft. per second. Capacity at s = 0.0064, 4.2 cu. ft. per second (beyond which second pipe begins to operate).

For maximum dry-weather flow	13.0 cu. ft. per second
Capacity of 15-in. pipe	4.2
Required capacity of second pipe	8.8 cu. ft. per second
From Fig. 47, 20-in. pipe, $s = 0.0057$; velocity, 4.1 ft. per second.	
Capacity of 20-in. pipe at $s = 0.0064$	9.3 cu. ft. per second.
Capacity of 15-in. pipe	4.2
Combined 15- and 20-in.	13.5 cu. ft. per second (beyond which third pipe begins to operate).
For ultimate maximum,	
Capacity of 30-in. sewer	25.0 cu. ft. per second.
Combined capacity, 15- and 20-in. siphons	13.5
Required capacity, of third pipe	11.5 cu.ft.per second
From Fig. 47, 22-in. pipe requires $s = 0.0058$, velocity 4.3 ft. per second	
Capacity of 22-in. pipe at $s = 0.0064$	12.0 cu. ft. per second.
Combined capacity of 15- and 20-in. pipes	13.5
Total available capacity	25.5 cu. ft. per second (25 required).

Design of Inlet Chamber.—A type of inlet chamber, Fig. 87, suitable in this instance has the invert of the pipe which carries the low flows continuous with that of the 30-in. gravity sewer; one side of the sewer is cut down to an elevation which permits overflow to the second siphon when the capacity of the first is exceeded; and the other side of the sewer is cut down to a higher elevation, which allows the flow in excess of the capacity of the first two pipes to reach the third.

Ascertain, by the method of Chapter IV, the depth of flow in the 30-in. gravity sewer when the 15-in. siphon shall have reached the limit of its capacity, (4.3 cu. ft. per second), and when the combined capacity of the 15- and 20-in. siphons (13.5 cu. ft. per second) is flowing. These depths determine the elevations of the tops of the walls which permit overflow to the 20- and 22-in. siphons and are found to be about 8 and 16 in. respectively.

Under maximum conditions for either of the two larger pipes, the overflow walls will be submerged both upstream and down. They cannot, therefore, be considered as weirs but more as obstructions causing certain loss of head in passing the desired quantities of sewage over each. The flow over these walls is at right angles to that in the approaching sewer, so that this loss may be taken as the head required to produce the necessary velocity across the top of the wall, assuming the energy of velocity of approach to be lost in the change of direction.

A maximum of 9.3 cu. ft. per second must pass across the wall to the 20-in. siphon. The depth on the wall will then be at least 8 in. or the difference in elevation between the two walls, the higher wall having been figured to overflow just as the capacity of the first two pipes was exceeded.

The higher wall must pass at least 11.5 cu. ft. per second, and the depth available is the distance from the top of the wall to the crown of the 30-in. sewer, or 14 in.

Assume the length of walls as 7 ft.; the inlet losses may be approximated as follows:

15-in. pipe; negligible, since invert is continuous with that of 30-in. sewer, and transition can be made to take advantage of the velocity of approach which, even at low flows in the gravity sewer, is equal to or greater than the siphon velocity. (Velocity of approach may be determined by method of Chapter IV for the partially filled sewer.)

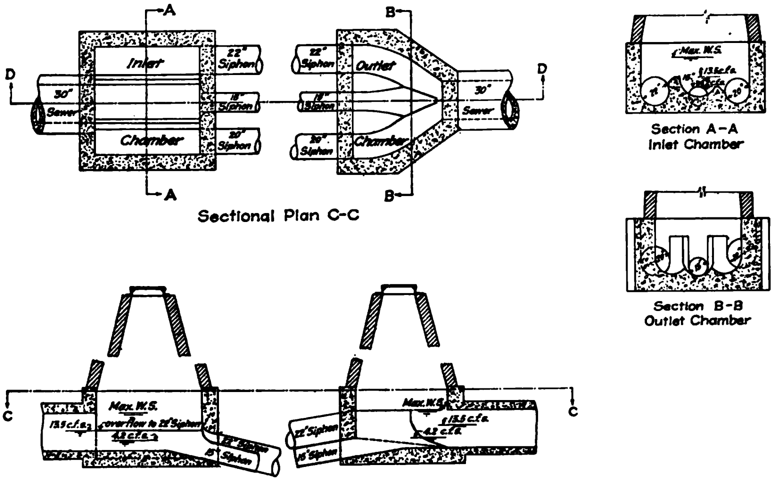


FIG. 87.—Inverted siphon inlet and outlet chambers (problem in inverted siphon design).

20-in. pipe; velocity over wall, 9.3/0.67 × 7 = 2.0 ft. per second; corresponding head, 0.06 ft. After passing over wall, velocity of 4.3 ft. per second in 20-in. pipe must be built up in new direction, which gives a loss of head of 0.29 ft.

Total 0.35 ft. (less than assumed inlet allowance of 0.4).

22-in. pipe; velocity over wall 11.5/1.17 × 7 = 1.4; corresponding head, 0.03 ft. Velocity in 22-in. pipe, 4.3 ft. per second; head, 0.29; total, 0.32 ft. Allowance of 0.4 ft. for inlet losses appears ample, and no revision of the computation of available slope is required.

Design of Outlet Chamber.—At the lower end of the siphon, the junction of the three siphon pipes with the 30-in. gravity sewer should be so designed as to reduce the opportunity for eddies to carry sediment back into those pipes which are not at the moment operating, but which are full of standing water. This is especially important in the case of the 22-in. pipe, which will not be in operation except at unusual rates of flow. It may be accomplished by maintaining the three pipes, or the corresponding channels within the junction chamber, as nearly up to the point of intersection as possible, to avoid pooling and reduction of velocity in the chamber. As a further precaution, the outlet of the 22-in. pipe (least frequently required) may be raised so that the invert of its channel has a sharp forward pitch toward the intersection. The crown of the pipe must not, however, be raised above that of the 30-in. sewer, or it will lie above the hydraulic gradient.

BRIDGES

Highway bridges are not often used to support sewers. If they are so employed, the grade of a sewer must generally first rise and then fall in passing over the structure, converting the sewer into a true siphon, which is objectionable to most designers of sewer systems. In some cases, however, it is desirable to

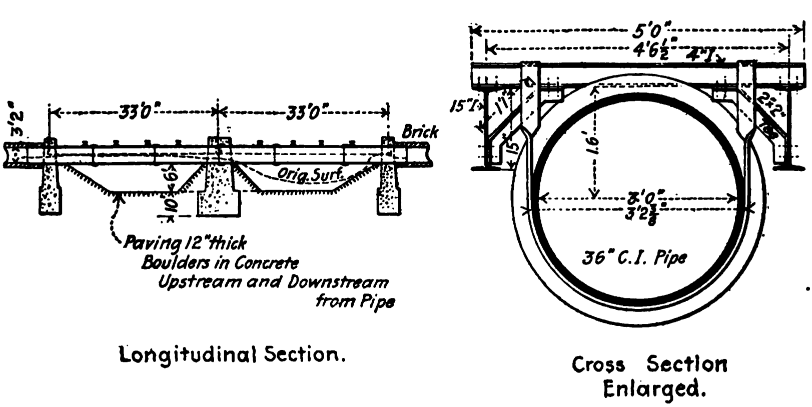


FIG. 88.—Sewer bridge, joint outlet sewer, New Jersey.

carry a sewer on a special bridge without any change in grade, as shown in Fig. 88, illustrating a structure designed by Alexander Potter. Occasionally reinforced-concrete beams are used rather than steel beams for such structures. In any case, care must be taken that the piers and abutments of the bridge do not unduly restrict the waterway of the stream crossed. Most engineers prefer to use inverted siphons rather than bridges for stream crossings, if the head necessary for the successful operation of the siphon is available.

FLUSHING DEVICES

The primary object of flushing devices is to permit sewers to be laid on flat grades which, while giving adequate velocity to assure the desired capacities at the depths assumed in the computations, are not enough to give at other depths velocities which will carry off at all times all suspended matter. The problem of flushing, strictly speaking, is usually merely one of keeping small sewers clean from their dead ends to the points where the flow of sewage is great enough to accomplish this without assistance from the water mains. The quantity of water used for this purpose in one flush is rarely over 350 gal. Occasionally the problem is one of furnishing a large volume of water to clean a main sewer or an inverted siphon.

Flushing from Brooks.—In some cities, where a large sewer is laid on flat grades near a brook or river, a flushing manhole is built, with a connection to the stream through which the sewer can be flushed with brook water. This has proved useful during dry weather when very little sewage is flowing and the low velocity permits solid matter to settle on the invert.

Flushing Manholes.—The flushing of small sewers is usually carried on either by hand or with the help of automatic apparatus. As a general proposition, all automatic flush tanks require some maintenance, and their cost is therefore dependent, in a measure, upon the time spent in inspecting and repairing them. The cost of this time, plus the interest and depreciation on the investment in the apparatus, plus the cost of the water used by the apparatus, must be offset against the cost of labor and water where hand flushing is practiced, for the difference in the cost of the manholes used in the two cases is negligible. The quantity of water to be used for flushing and the frequency of flushing depend not only upon the grade of the sewer to be kept clean but also upon the possibility of dirt finding its way into the sewer.

Hand flushing is generally done by means of a hose from the nearest fire hydrant, inserted into the manhole at the end or summit of the sewer to be cleaned. Flushing manholes are also used to some extent. In this case a 1- or 1½-in. service pipe from the nearest water main is run into the manhole and the entrance to the sewer can be closed with a flap or tripping valve. The same end is accomplished in some places where valves have not been installed, by plugging the end of the sewer with a disc consisting of sheet rubber faced with canvas and held firmly between boards about ½ in. smaller than the diameter of the sewer. When the tank is filled with water, this plug is drawn out, starting the flush.

Automatic Flush Tanks.—The flushing done with automatic apparatus is generally more frequent than where hand flushing is practiced, the usual rule being to discharge the flush tank once every 24 hr. The water usually enters the tanks through special orifices, of which a variety are manufactured by the makers of automatic siphons, so that any desired rate of flow under any street main pressure can be obtained by screwing the proper

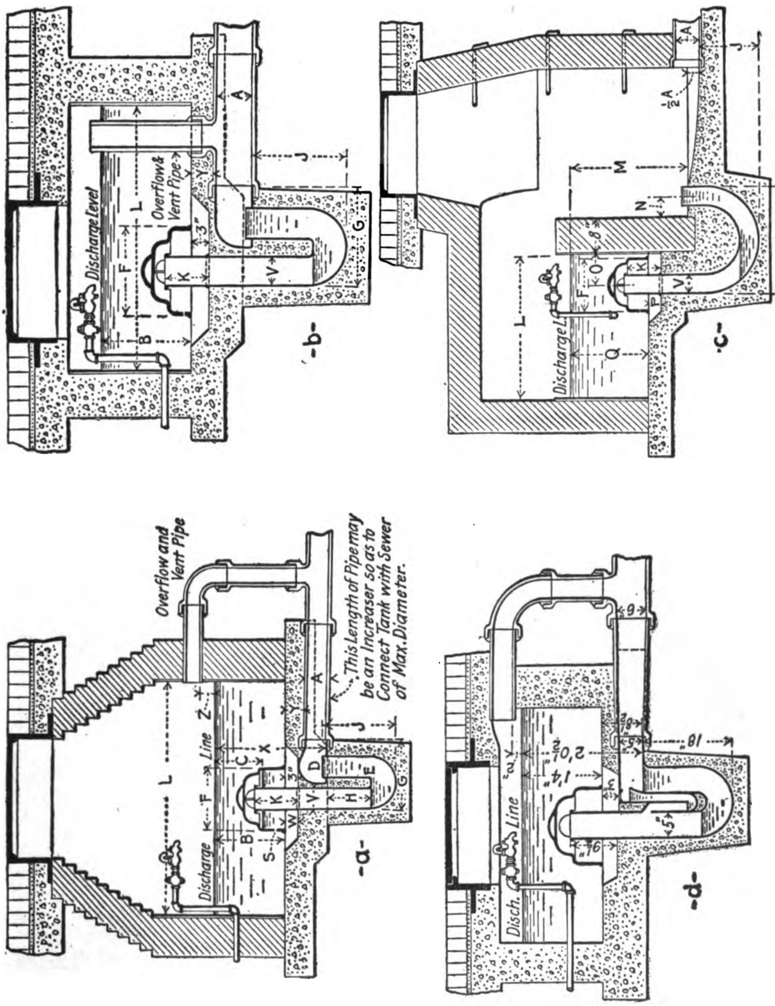


Fig. 89.—Different types of flush-tanks (Miller).

orifice or jet into the end of the service pipe. As a rule these jets are also accompanied by a mud drum or screening device and a blowoff cock, provided to keep the jet clear.

The operation of a siphon of the simplest type is as follows: In Fig. 89*a*, the siphon is shown just ready to discharge. There are two volumes of water separated by the compressed air in the long leg, *V*, of the trap. As the pressure on every part of this confined mass of air must be equal to the hydrostatic pressure, and as there are but two surfaces where the water is in contact with the compressed air, it follows that the depth of water, *C*, in the tank must be the same as the depth, *H*, in the trap. When the depth, *C*, is increased, the water flows over the raised lip of the trap at *D*, this discharge allowing a little air to escape below the bend at *E*. The air pressure being released in this way, water passes up with a rush within the bell and into the long leg of the trap.

The elevation of the lip of the short leg at *D* above the bottom of the outlet is an important detail, as upon it the first sudden discharge of the trap seems to depend. In the older types of flushing apparatus this first strong flush was accomplished by using an auxiliary siphon at the bottom of the trap casting—a

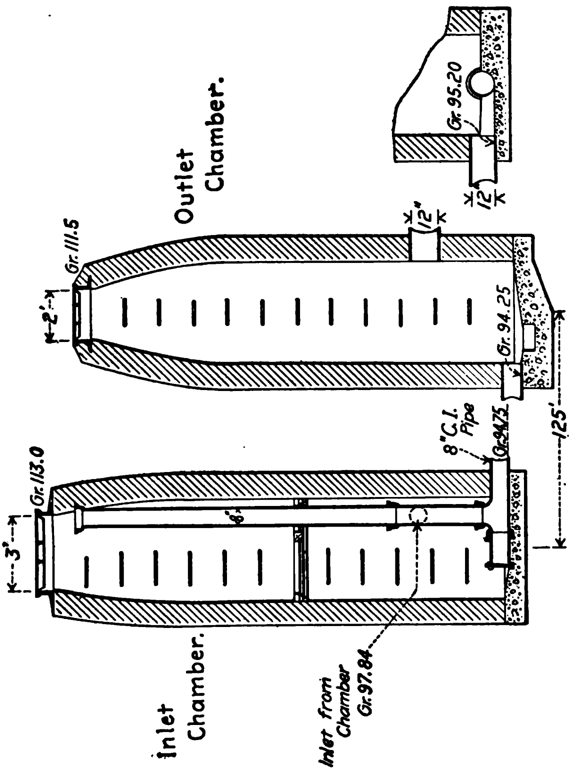
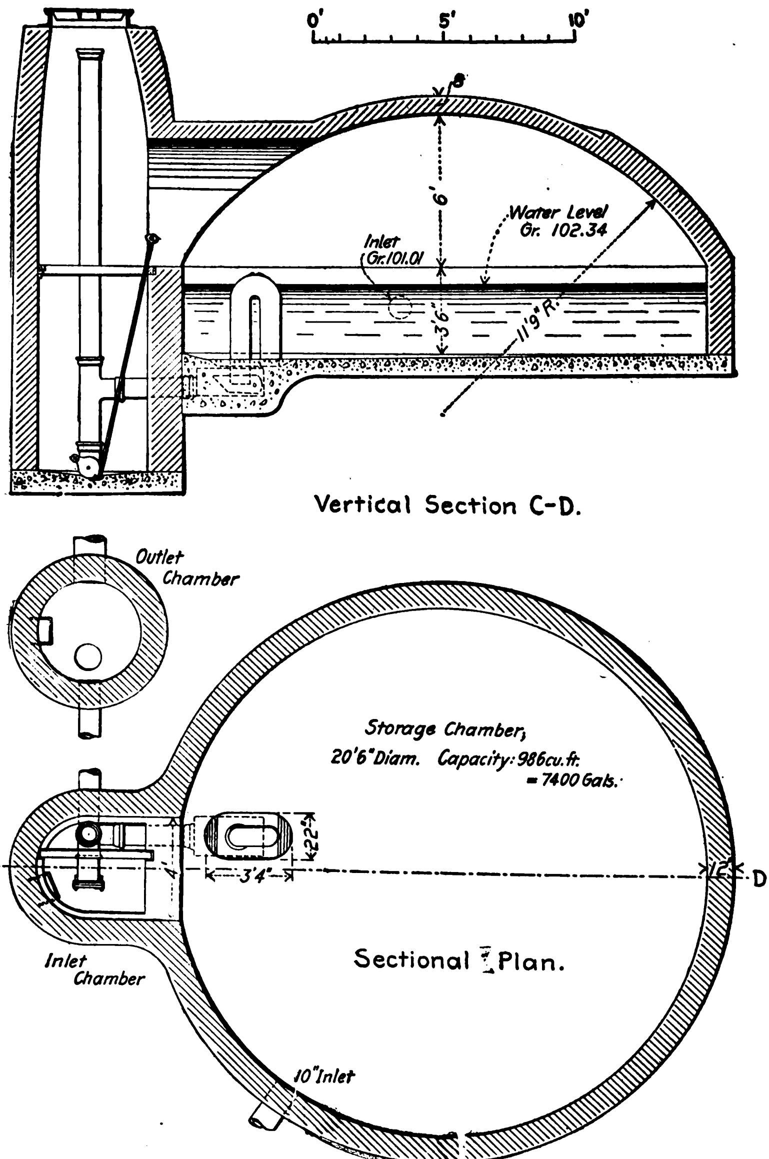


FIG. 90.—Flushing chamber,

detail retained in the Rhoads-Miller siphon, Fig. 89*d*, for use where shallow construction is imperative.

When the water has been drawn down in the tank until its surface is below the sniff hole *S*, air rushes into the bell and stops

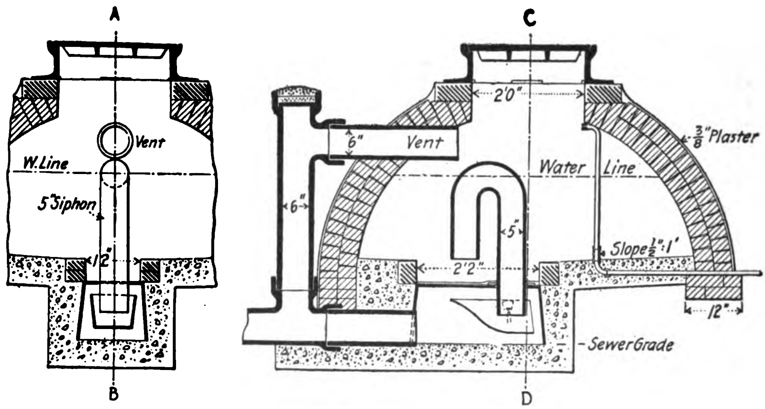


Concord inverted siphon.

the siphonic action there. In consequence the water in the two legs of the trap at once forms a seal there, and the apparatus is ready for discharge when the tank is filled again.

The dimensions of the Miller tank, required by designers, are given in Tables 24, 25 and 26. The diameter of the tank is the minimum which is generally considered desirable for siphons of the size listed. The discharge is the average given by the makers for that size and setting of siphon.

The setting shown in Fig. 89*a* does not afford access to the sewer, so the late Andrew Rosewater devised the special design shown in Fig. 89*c* to overcome this defect. The manhole at the



Section C-D. Section A-B.

FIG. 91.—Van Vranken flush-tank.

dead end of the sewer is provided with a flush tank and siphon, and while this is more expensive than the standard type, it not only affords an opportunity to insert a cleaning rod into the end of the sewer but also, it is stated, gives a higher rate of discharge.

Fig. 90 illustrates a large flushing chamber at the head of an inverted siphon at Concord, Mass. The purpose of the chamber is to accumulate the sewage until a large quantity has been stored and to then discharge it rapidly, so as to keep the inverted siphon under the river clean. The chamber discharges from 15 to 20 times in 24 hr.

With this flushing chamber the Van Vranken siphon, Fig. 91, is used. The sewer to be flushed ends in a well in the floor of the tank, which has a watertight metal cover. A 5-in. siphon has its long leg carried through the plate. The bottom of the leg is

TABLE 24.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, STANDARD SETTING, FIG. 89a

Sewer diam., in. *A*	Discharge rate, cu. ft. per sec.	Bell diam., in. *F*	Trap width, in. *G*	Trap depth, in. *J*	Trap rise, in. *K*	Tank diam., ft. *L*	Depth in., from discharge line to: Floor *B*	Depth in., from discharge line to: Invert *X*	Floor depth, in. *Y*	Rise to overflow. in. *Z*	Trap diam., in. *V*
4 to 6	0.35	13¼	14	14¼	8¾	3	14	22¼	4	3	4
6 to 8	0.73	16⅜	18¼	22¾	9½	3	23	34	5	2	5
8 to 10	1.06	20½	20⅜	29⅝	11	4	30	44	6	2	6
12 to 15	2.12	25½	27½	36½	13½	4	35	51¼	6¼	2	8

TABLE 25.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, SHALLOW SETTING, FIG. 89b

A, in.	Discharge, c.f.s.	*F*, in.	*G*, in.	*J*, in.	*K*, in.	*L*, ft.	*B*, in.	*X*, in.	*Y*, in.	*Z*, in.	*V*, in.
6 to 8	0.55	16⅜	18¼	17¼	7¾	3	15	26	5	2	5
8 to 10	0.90	20½	20	19¼	8	4	18	31	5	2	6

TABLE 26.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, SPECIAL SETTINGS, FIG. 89c

A, in.	Discharge, c.f.s.	*F*, in.	*G*, in.	*J*, in.	*K*, in.	*L*, ft.	*M*, in.	*N*, in.	*O*, in.	*P*, in.	*Q*, in.	*V*, in.
4–6	0.65	13¼	26⅛	13¼	9	3	29	5	8	3	19	4
6–8	1.02	16⅜	30⅛	17¾	9½	3	33½	6	10	3	23	5
8–10	1.49	20½	33⅛	24¾	11	4	43	6	12	4	31	6
12–15	2.97	25½	39¼	28⅜	13½	5	49½	7	15	5	35	8

trapped in a tilting tray, which is so balanced that when nearly full its center of gravity is brought forward and it tilts rapidly, allowing a part of its contents to flow out. This suddenly changes the air pressure in the siphon and starts the apparatus in action.

REGULATING DEVICES

The function of a sewage flow regulator is to prevent the surcharge of an intercepting sewer, by automatically closing a gate upon the main sewer connection, thus cutting off the sewage and forcing it to flow to another outlet.

A storm overflow is designed to allow the excess sewage above a definite quantity to escape from the sewer in which it is flowing, through an opening.

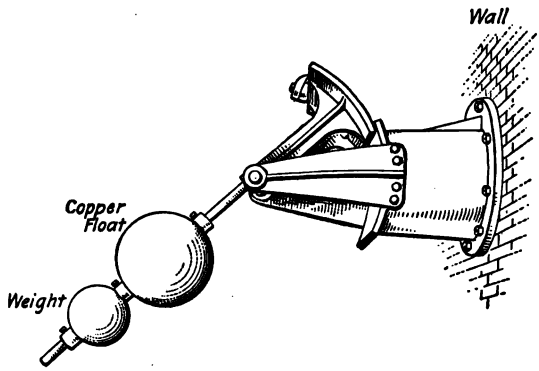


FIG. 92.—Coffin regulator.

Whenever an outlet is in a body of water subject to considerable fluctuations in elevation and it is necessary to prevent this water from entering the sewer, a backwater or tide gate is employed.

The purposes of the regulator and the storm overflow are substantially the same, namely, to allow the ordinary flow of sewage to be delivered to a distant point of discharge, and at the same time to cause the excess storm flow, which is much less foul, to pass into a nearer watercourse. Sometimes regulators are used in combination with storm overflows to safeguard an intercepting sewer by cutting off entirely the sewage entering the intercepter when the latter is filled to a certain elevation. The overflow allows the escape of the excess storm flow; the regulator may finally cause the entire flow in the main sewer, both sewage and storm water, to pass the overflow and be discharged into a nearby water course.

Regulators.—The simplest type of regulator is shown in Fig. 92. It has a cast-iron body which is bolted to the end of the main sewer and projects into a chamber in which the sewage will rise to the same height as in the intercepter with which it is connected. The steel shaft carries an adjustable

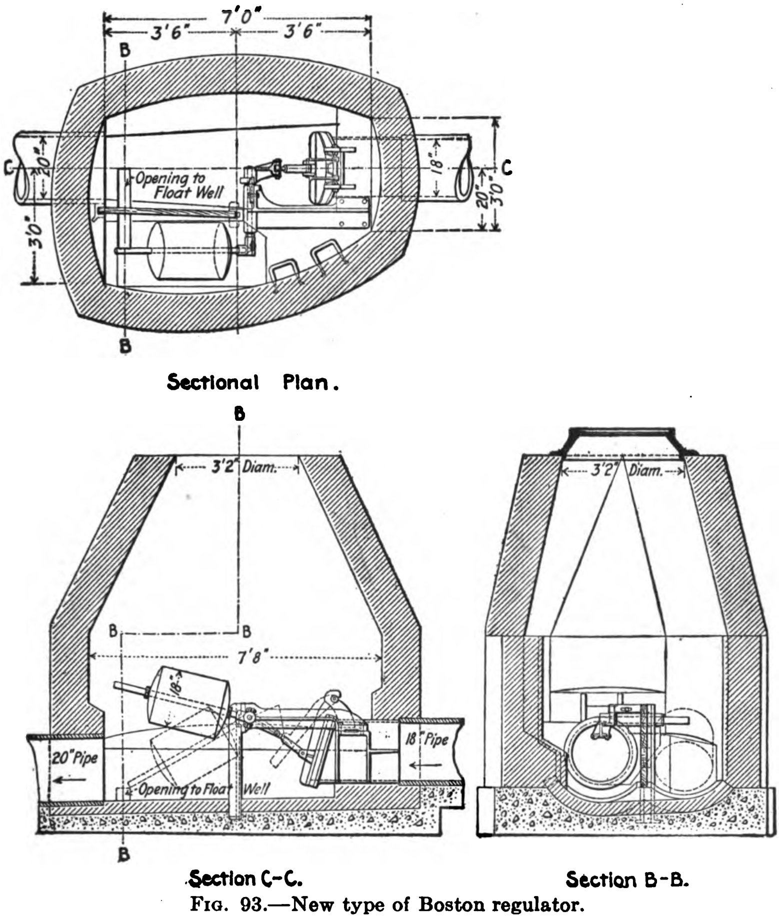
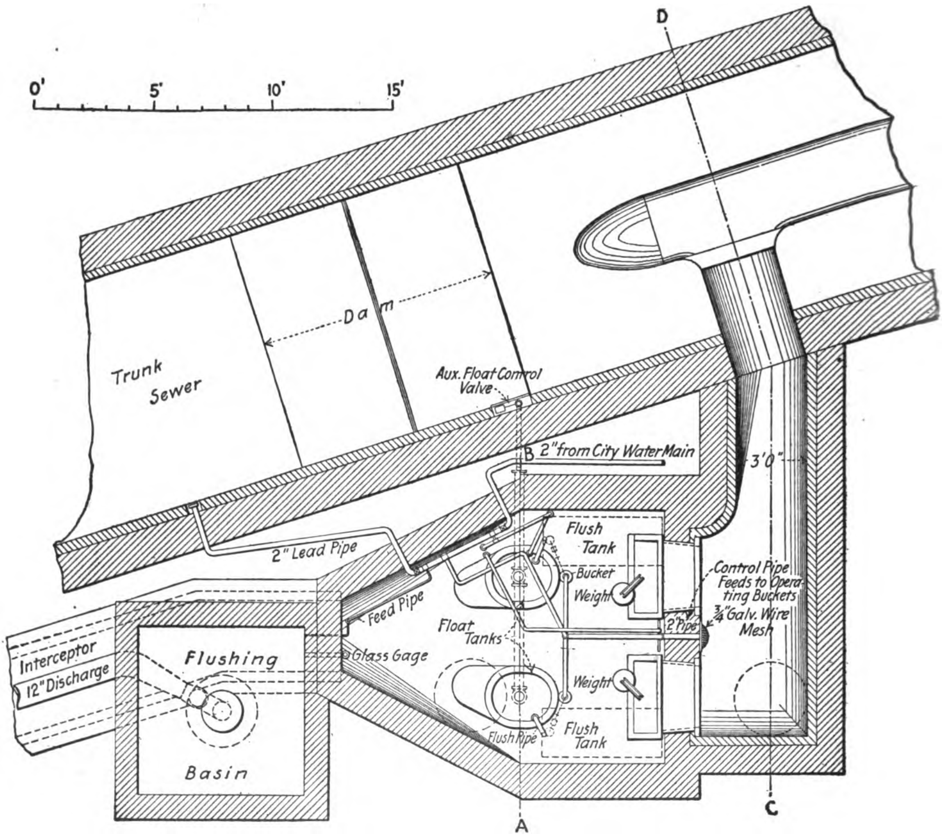


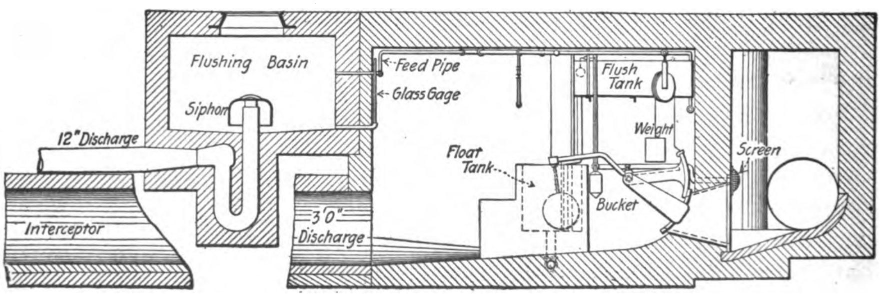
FIG. 93.—New type of Boston regulator.

copper float and a weight by which the action of the device can be varied somewhat. This regulator is also employed as a backwater valve to prevent sewage backing into sub-main sewers from trunk or main sewers that become surcharged.

Probably the best-known type of regulator is shown in Fig. 93; this was developed for the connections between the Boston

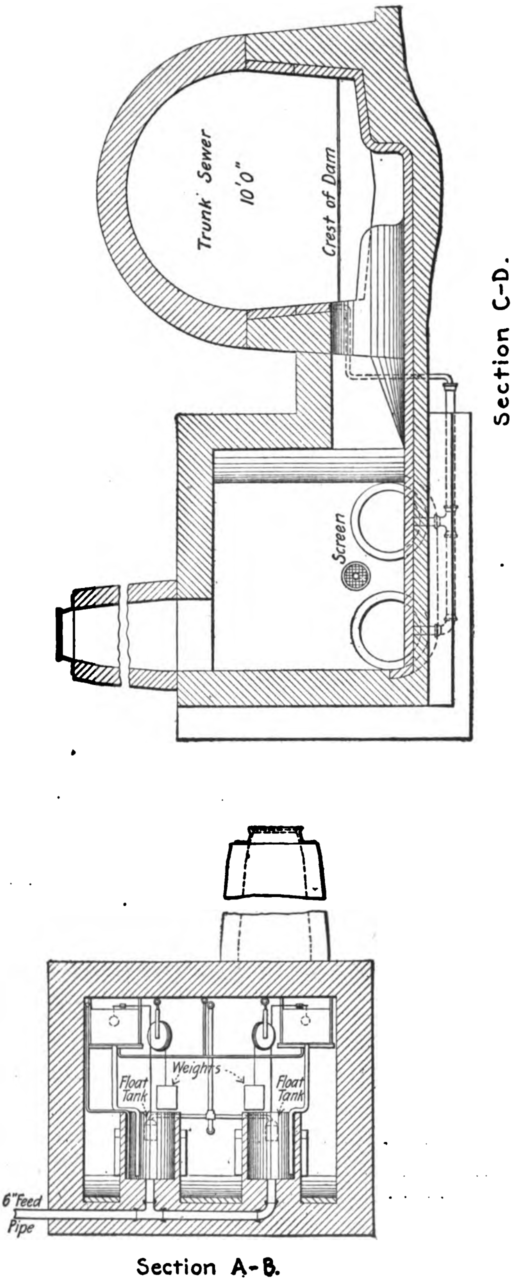


Sectional Plan.



Longitudinal Section.

Fig. 94.—Regulating chamber on



Piney Creek sewer, Washington.

combined sewers and the intercepting sewers. The orifice in the main sewer is designed of sufficient capacity to allow the desired quantity of sewage to pass through it. In some cases it is necessary to provide a low dam in the main sewer immediately below the orifice to assist in diverting the sewage. The regulating gate seats against the orifice or nozzle. This gate is hinged and connected by a toggle joint with a lever which has a large float on its other arm. This float is in a well having a connection with the intercepting sewer, so that the rise and fall of the sewage in the intercepter lifts and drops the float. The apparatus can be adjusted so that the orifice will be closed when the float rises to any predetermined elevation.

A type of regulator is used in Washington, D. C., in which the floats are operated by clean water from the city mains, admitted to the float chambers through valves controlled by the rise and fall of sewage in the main sewer. Asa E. Phillips, for 30 years superintendent of sewers, stated in 1913 that the most elaborate installation, shown in Fig. 94, had then worked with absolute regularity for 2 years. It was then so well balanced that it delivered sewage from the trunk sewer into the 3-ft. intercepter so long as the latter was not filled. As soon as the full capacity of the intercepter was utilized, the regulator cut off the flow to the intercepter, and as soon as the latter was able to receive more sewage, the regulator started the flow again. The following description of the operation of this regulator is condensed from the account in "American Sewerage Practice."

The cunette section of the trunk sewer extends below the 3-ft. connection to the regulator to divert from the intercepter large heavy substances, such as cobble stones, which excessive storms wash into this sewer. Where the tongue of the cunette ends, a low dam was provided to hold the hydraulic gradient at such an elevation that the intercepter will run full before any sewage is discharged through the prolongation of the trunk sewer into a nearby brook.

The movable gates that close the two orifices through which sewage passes on its way to the intercepter are on one end of levers carrying on the other end large copper floats, each in a float tank. By filling or emptying these tanks, the floats actuate the levers and close or open the gates. Each float tank is filled through a 2-in. pipe from a flush tank over the float tank; the flush tank is kept full by a ball cock on a service pipe from a city water main. The discharge from the flush tank to the float tank is controlled by a valve operated by the weight of a funnel pail when the latter is filled with sewage delivered by a 2-in. lead pipe from the main sewer below the dam. When the sewage rises above the inlet of this lead pipe, the funnel pails are filled, which discharges the flush tanks into the float tanks and causes the floats to close the gates.

When the elevation of the sewage in the main sewer falls below the inlet of the lead pipe feeding the funnel pails, the latter slowly empty and their weight is no longer enough to hold open the discharge valves of the flush tanks. The float tanks are then drained through small outlet holes, and as the level of the water in them falls the floats sink and open the two main gates, again throwing the intercepter into service.

Storm Overflows.—These structures usually consist of a weir in the side of the sewer, over which the excess flow in the sewer is discharged into a connection with a nearby body of water. Fig. 95 illustrates a storm overflow in Cleveland which W. C. Parmley has described[1] as follows:

"Since it was not desirable to allow the velocity in the main sewer above the overflow chamber to be reduced below about 2½ ft. per second, it was necessary to make a drop of at least 1½ ft. in passing from the invert of the 14-ft. 9-in. sewer to the invert of the 5-ft. sewer.[2] With this drop, the minimum velocity in the main sewer will be about 2½ ft. per second when 60 cu. ft. per second are flowing. For a less quantity than 60 cu. ft. per second, there will be an acceleration in the velocity above the junction for any small volumes of flow, and for no quantity less than 60 cu. ft. per second will the effect of backwater reduce the velocity to less than 2½ ft. per second. For volumes greater than 60 cu. ft. per second, the sill of the overflow must be long enough[3] to take out all but 60 cu. ft. per second, which will remain in the sewer to be carried off by the intercepter. For the maximum discharge of 2,500 cu. ft. per second for a 14-ft. 9-in. sewer there will be no backwater effect. Hence the 14-ft. 9-in. sewer will flow unobstructed when nine-tenths full.

"The elevation of the upstream end of the weir, therefore, was placed 2⁷⁄₁₀ ft. above the invert of the 14-ft. 9-in. sewer, and is carried to an elevation of 4½ ft. above the invert of the 5-ft. intercepter after the invert of the intercepter has been fixed at a proper elevation. The grade of the crest of the weir is 0.3 ft. per 100 ft. The form of the cross-section of the

[1] *Trans*, Am. Soc. C. E., vol. lv., p. 341.

[2] Depth of flow in the 5-ft. sewer would be about 4½ ft.; in the 14-ft. 9-in. sewer about 2⁷⁄₁₀ ft. The difference, 1⅘ ft., would be the drop in the invert if the water surface were level.

[3] The length of the weir was made about 95 ft.

dry-weather channel at the upper end begins as the segment of the 14-ft. 9-in. sewer, and, in passing downstream to the 5-ft. intercepter, gradually changes to the section of the 5-ft. sewer with the crest of the overflow sill nine-tenths of the diameter above the invert.

"In order to avoid any backwater effect from the storm-water overflow, it is necessary that the weir should never act as a submerged weir. That is to say, the surface of the storm water in the overflow channel must

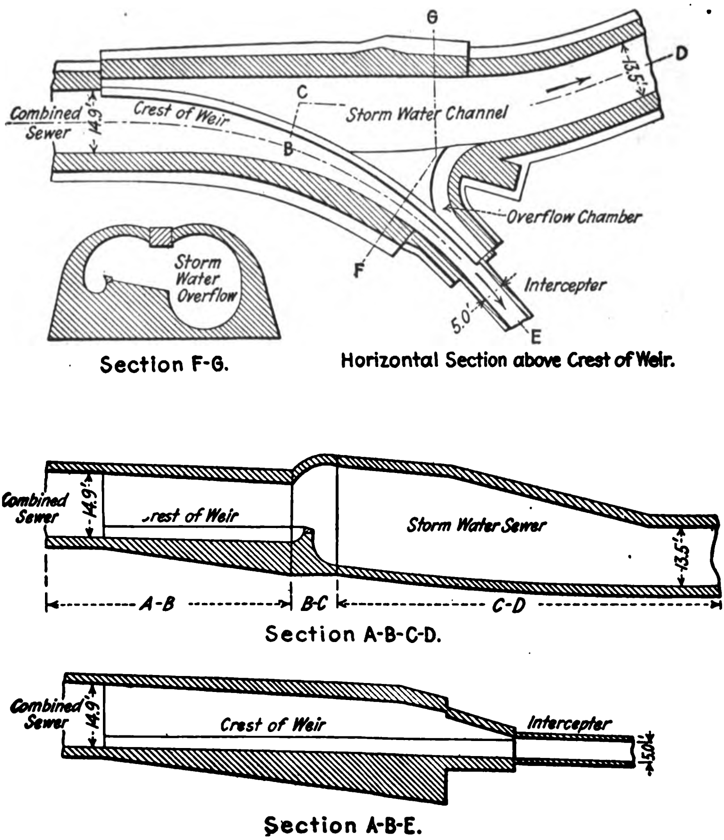


FIG. 95.—Weir for storm-water overflow, Cleveland.

always be lower than the crest of the weir. The storm-water branch below the overflow chamber was given a drop of about 12 ft. below the level of the sill, and was carried down the valley on a grade of 0.50 ft. per 100 ft. The overflow branch, therefore, was made 13 ft. 6 in. in diameter."

Tide Gates.—Tide gates are now generally made of a leaf or shutter of planks held together firmly and hinged at the top and resting against an inclined seat when closed. Formerly large tide gates were sometimes built of two leaves hinged at the sides, resting against inclined seats and kept closed when the tidal pressure exceeded that of the sewage by hitching to the back of each leaf a "bridle chain" hanging loosely from an eye in the roof of the chamber. On the lowest part of this chain, as it hung from the roof and the gate, a heavy weight was attached. This bridle chain with its weight helped to close the gates when the tide rose. Experience showed that these double gates required frequent adjustments to keep them in working condition and that floating substances accumulated on the bridle chains and interfered with their operation.

The type of tide gate and chamber now used in Boston is shown in Fig. 96. The seat is a heavy wooden frame with which the leaf makes a tight joint by means of a rubber gasket slightly recessed along each edge, so that the nails holding it to the wood do not project and prevent the proper compression of the rubber when the gate is subject to back pressure. In the case of smaller sewers, a cast-iron frame is inserted in the masonry to form a seat for the gate. Small sewers are sometimes provided with tide gates of the form shown in Fig. 92.

OUTLETS

Where the water into which sewage is discharged is quiet, the outlet is usually submerged to a considerable depth in order to disperse the sewage thoroughly through a large volume of water before any of it can rise to the surface. For example, the 66-in. steel outfall sewer of Rochester, N. Y., laid in 1913 in Lake Ontario, terminates in a timber crib 7,000 ft. from the shore. This crib is 46 ft. square and 24 ft. high, weighted with stone and surrounded with riprap extending 10 ft. up the sides.[1] The outlet is 10 ft. above the bottom of the lake, which is about 50 ft. deep at this place.

If the sewage is discharged into a stream flowing rapidly at all times, the outlet need not be submerged provided the sewage passes into the stream at a point where it is certain to be carried away and dispersed rapidly. In the case of outlets in tidal waters, the fact that it is generally impossible to place them so high that they will not be sealed at high tide, results automatically in checking the discharge of sewage during the portion of the tidal flow when it is likely to be swept back along the shore, and

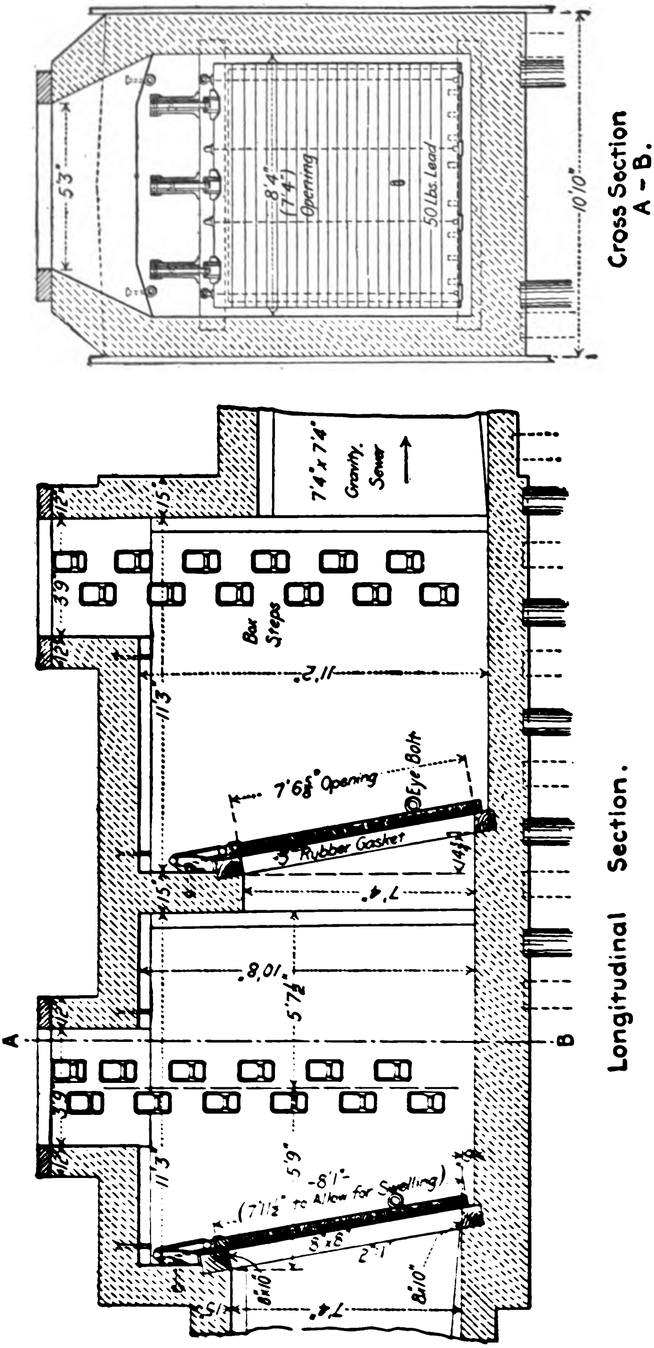


FIG. 96.—Tide-gate chamber, Boston, with Dodd gates (patented).

accelerates the discharge when the tide is going out and the hydraulic grade of the outfall is, therefore, steadily increased.

A different outlet is sometimes built for combined sewers than for separate sewers, because domestic sewage must be discharged with much greater precaution to prevent offense than storm water. Fig. 97 illustrates a duplex outlet built at Minneapolis to meet these conditions. The outlet is on the banks of the Mississippi River, where the difference between the elevations of flood and dry-weather stages is great. Two 15-in. cast-iron pipes run out below the paved apron of the storm-water outlet and discharge the dry-weather sewage 5 ft. below the low-water level of the river. The invert of the storm-water outlet is 9 in. below the high-water level in the river, so that the sewer will have a free discharge at all times.

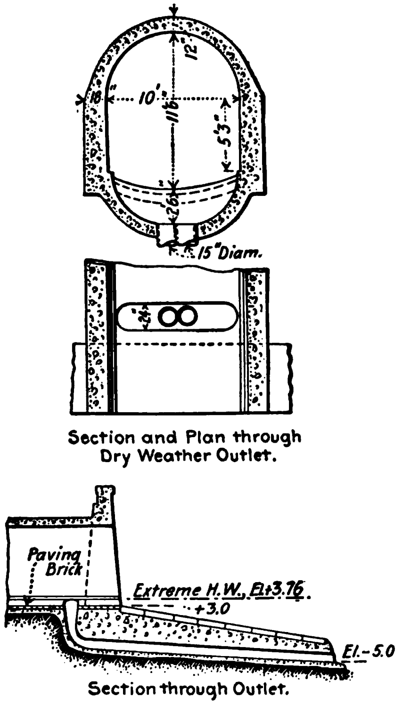


FIG. 97.—Dry-weather outlet, Minneapolis.

PUMPING SEWAGE

In the design of sewerage works, it may be necessary to resort to pumping where the sewage or storm water is collected at a point so low that discharge by gravity is impossible, as at Washington; to reach a desirable purification site, as at Baltimore; to lift the sewage from areas too low to drain into the main sewer by gravity; or to force water into streams, as at Milwaukee, or tidal inlets receiving sewage, which would become offensive unless flushed in this way.

Whether the sewage shall be lifted at one or more points is usually settled by comparing the fixed and operating expenses of different plans. The operating expense of raising all sewage at one point is less than that of doing it at two or more points. On the other hand, if all sewers are made to drain by gravity to one place, their cost may be greatly increased on account of the deep cuts and large cross-sections necessary to obtain satisfactory velocities of flow. Various projects must often be considered, both with and without pumping, and the extra cost necessary to drain to one point, together with the cost and the capitalized annual charges for operation and depreciation of the pumping station, must be compared with similar charges for a project with two or more stations.[1] Conditions may even arise where, if the 24-hr. flow can be handled by working the station at its most economical rate for 8 hr., the reduction in labor charges accomplished in this way will warrant the construction of reservoirs to store the sewage when the pumps are not running. The trunk sewers of combined systems sometimes have such large capacity that they afford considerable storage during dry weather, but the danger of deposit due to the reduction of velocity in pooling or in the development of this storage, and the possible resulting increase in cost of cleaning the sewer, is to be remembered.

Storage and Screening.—Where storage basins are used only to improve the load on the pumps and the quantity of dry-weather sewage is large enough to keep a fairly economical pump running under a good load, the tendency of engineers is to rely very little upon storage and to provide enough pumping units, thrown into service one after the other, so that there is only one unit running under an uneconomical load at one time. This is accomplished in small stations by automatic controllers, which start the first pump when the sewage reaches a certain height, another when it reaches a higher elevation and a third when it reaches a still higher elevation. The very short time when the second and third pumps are in operation makes their performance of little importance financially, reliability and low cost being the most essential items to be considered in such installations.

Except in small plants, provision is usually made for screening, and sometimes for sedimentation, of the sewage before it reaches the pumps. There is no uniformity of opinion among engineers regarding the size of screens, either as to the size of bars or the width of the openings between them. Where sewage comes from an industrial district which contributes large quantities of waste and rags to the sewage, some engineers believe that it is less expensive in the end to install a relatively large automatically

[1] The method of making such a financial comparison is explained in "American Sewerage Practice," vol. i, pp. 646 and 647.

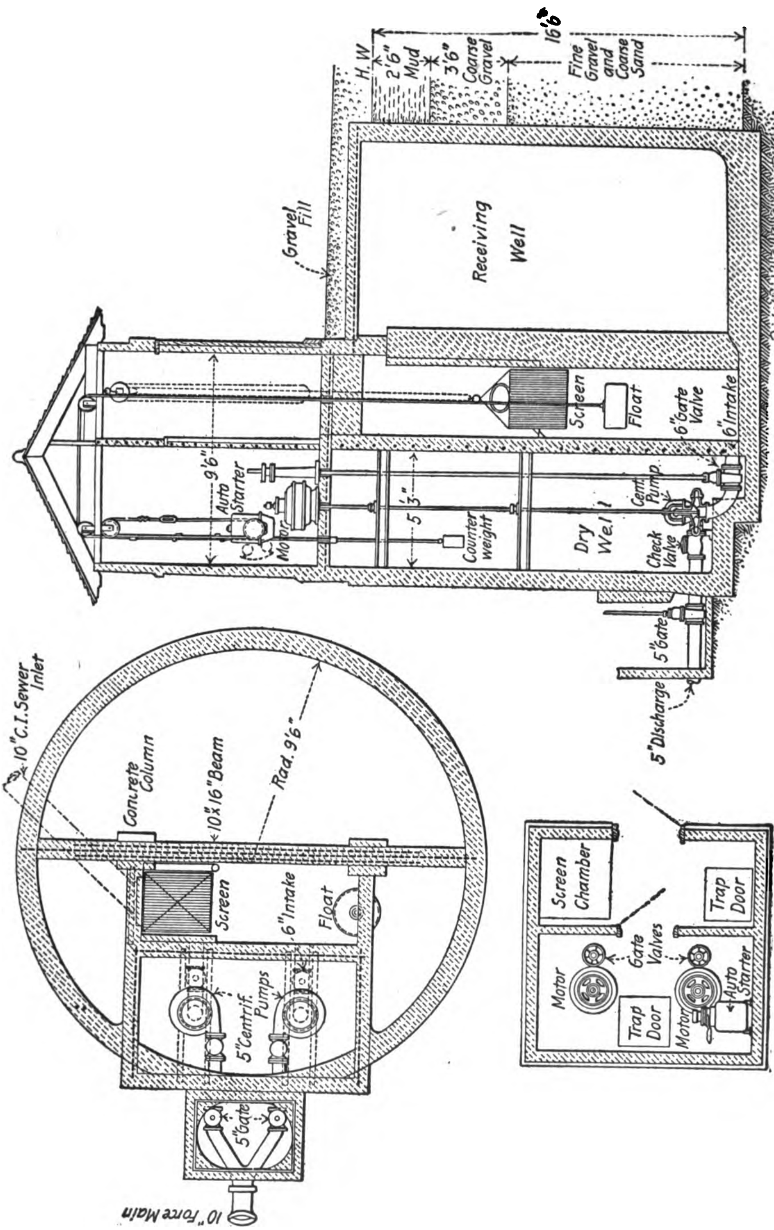


FIG. 98.—Sewage pumping station, Waltham, Mass.

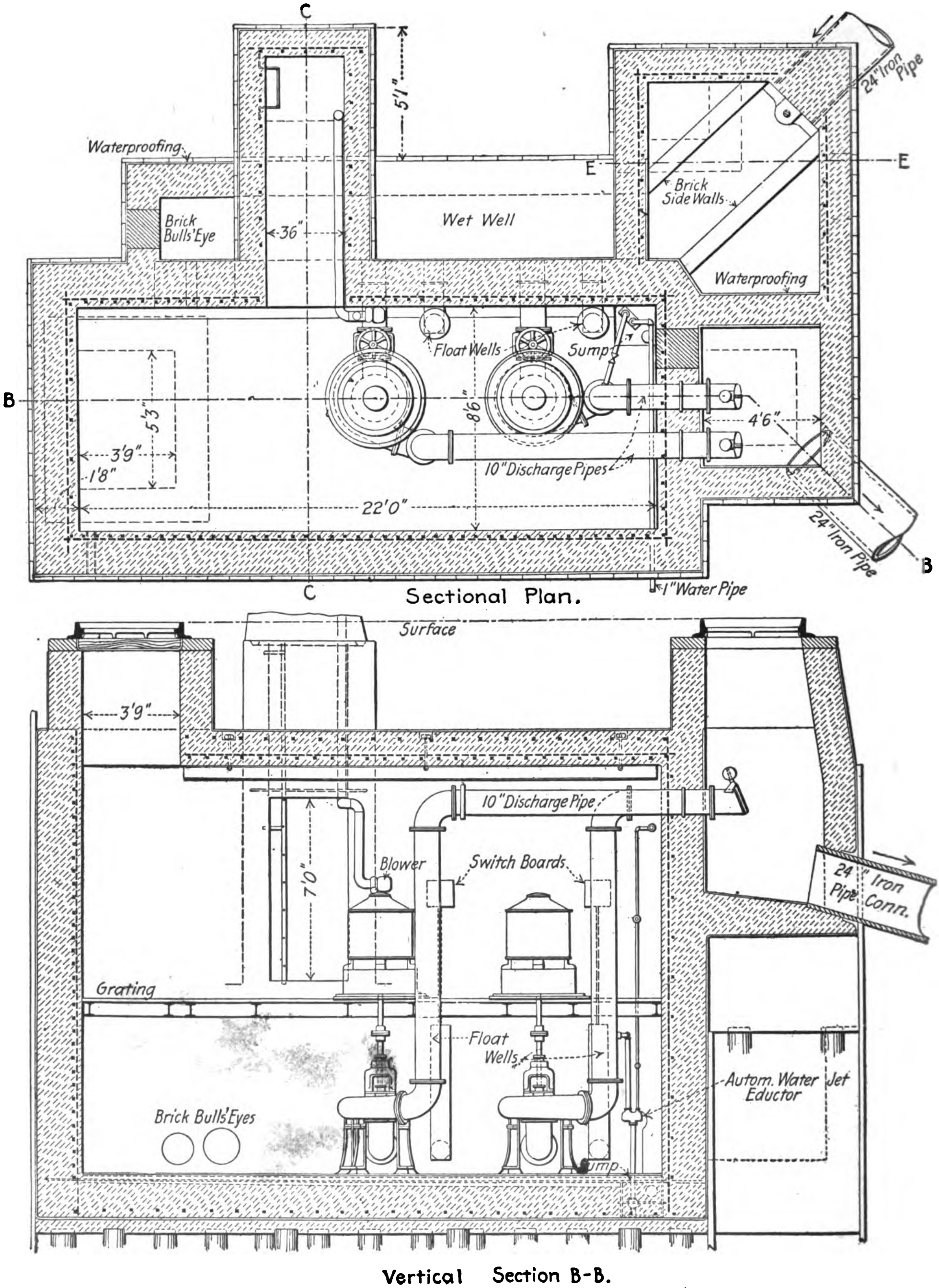
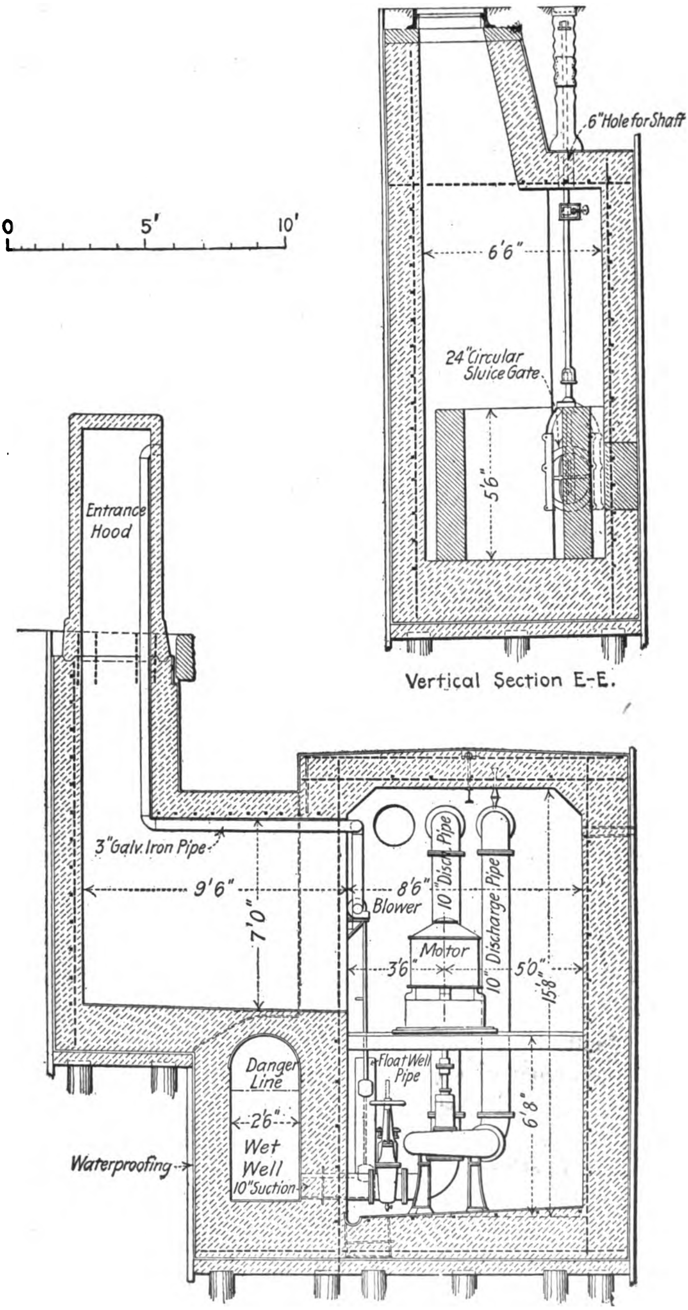


FIG. 99.—Summer Street



Vertical Section E-E.

Vertical Section C-C.

pumping station, Boston.

controlled pump which can successfully handle unscreened sewage than to use a smaller pump which makes screening a necessity, for the screening involves large labor charges to keep the screens clean. Experience seems to indicate that an 8-in. pump is the smallest that can handle such unscreened sewage.

Waltham Pumping Station.—This station, built in 1907 from the plans of Bertram Brewer, is an example of a plant where storage is provided. It is shown in Fig. 98. The automatic starting and controlling devices for the alternating-current motors operating the centrifugal pumps are actuated by a float in the receiving well. The control rope was necessarily long and its length was materially affected by changes in temperature. When too loose or too tight it would not operate the controller, but the difficulty was overcome by placing a heavy spring in the rope.

Summer Street Station, Boston.—The automatically controlled pumping station on Summer Street, Boston, Fig. 99, is an example of the type without any storage wells. The difficulty experienced at Waltham from the changes in length of the flexible cord connecting the float and controller are overcome in the Boston station by using a vertical rod to connect the float, which rises and falls in a well consisting of an 8-in. cast-iron pipe, with the controller placed directly above the well. The well pipe is closed at the top by a bolted cover and the rod rises and falls in a bronze bushing in a hub cast in the center of this cover. A few inches above the float a heavy rubber ring is attached to the rod. When the float rises to the top of its range, this ring is pressed against the cover and the joint is made tight so that no sewage can escape.

This station, designed by C. H. Dodd under the general direction of E. S. Dorr, is underground with the exception of a narrow concrete entrance hood rising above the sidewalk just inside the curb line. The sewage enters through a 24-in. pipe terminating in a sluice gate. There is no screen, because the designer considered that a 10-in. pump would pass anything likely to reach it. Provision has been made, however, for freeing the impeller of the pump from rags, which can be removed through handholes without dismantling the pump. The sewage is passed through a channel formed by brick side walls running diagonally across the gate chamber, and then enters a wet well 8½ ft. long, 6 ft. high and 2 ft. wide, with which three pumps can be connected, although but two were installed at first. The wet well is also connected to the float wells and to a sump from which the sewage and drainage can be raised by a water-jet ejector. The air in the pump room is drawn out by a blower which forces it up to the entrance hood, where it escapes through a grating in the top of the iron door by which the entrance shaft is closed.

CHAPTER VI

PRELIMINARY INVESTIGATIONS, SURVEYING, INSPECTION AND EXCAVATION

It is desirable to make comprehensive preliminary investigations of the district to be sewered, not only to obtain the data needed by the designing engineer and contractor but also to place on record authentic information as to the local conditions prior to the construction of the system which may be affected by it. Such information may be helpful in meeting future claims for damages. Contractors are justified in making lower bids when they are supplied with complete information about the conditions they will encounter than when they must guess at many of them. All facts ascertained, whether advantageous or not, must be made public, for if any are suppressed, claims for extras made by the contractor are likely to be allowed by the courts.

At the outset of the engineering work an attempt should be made to obtain all maps which furnish information about the district. City engineers, town and county surveyors, assessment boards, land title and insurance companies and public utilities often have such maps and will permit copies of them to be made. In large sewerage undertakings, the maps prepared by the U. S. Geological Survey and the Post Office Department may be useful and occasionally the Bureau of Soils of the U. S. Department of Agriculture is able to supply helpful maps. All errors discovered in any of the maps should be marked on them so that later there can be no question about the errors existing in case uncorrected copies are used as evidence in claims against the city.

Field Work.—Unless satisfactory maps are available, surveys must be made. The degree of precision required will depend on the conditions of the problem; in large, important work it is sometimes advisable to establish a preliminary triangulation system. The surveys should furnish the precise location of street and property lines, street railways, public parks and buildings, ponds, streams, large ditches and other features and structures which may influence, or be influenced by, the sewerage system.

An accurate, permanent and complete system of bench levels should be established throughout the area to be covered by the proposed sewerage system. In heavy work a bench mark should be established on each block of all streets in which sewers are to be laid, and at frequent intervals where topographic details are to be obtained subsequently. Profiles should then be run on all existing streets and alleys, and if the existing and "established" grades are different, notes about the latter should be obtained. This work should be extended to cover the district within which sewers may be needed during the next 30 to 50 years. Topographic notes should be obtained for the plotting of a map with contours at intervals of 1, 2½, 5 or 10 ft., according to the configuration of the ground. The elevations of the beds of streams, ditches, canals and culverts should be ascertained and the maximum and minimum flow in them determined as accurately as practicable.

While the surveys are in progress, full notes of all existing structures should be obtained. The elevations of the sills of buildings and the approximate depths of their cellars should be ascertained. The character, age and condition of the pavements of streets in which sewers will be laid should be recorded, and the areas of all roofs, sidewalks and paved private alleys and drives should be obtained. All outlets of public and private sewers and drains and the discharge from each should be noted. Existing sewers should be investigated sufficiently to check the accuracy of existing records of them, and if accurate records are lacking a survey of the sewers should be made. This is done by measuring the elevation of the invert at each manhole and lamphole. The condition of the sewers can be ascertained by illuminating their interior with the aid of two mirrors about 5 in. in diameter. One is held at the top of a manhole so as to reflect sunlight to a mirror held by an inspector at the bottom of the manhole, who directs the light into the sewer to be examined. All available information regarding the location of water and gas mains and electric conduits should be obtained, and where these underground structures are numerous it may be necessary to dig pits in the streets to ascertain where they are.

All local rainfall and run-off data should be collected. If the sewage is to be discharged into a river, analyses should be made of samples obtained above, within and below the city during different stages, particularly the low stage. If the sewage is to be discharged into a pond, lake or tidal waters, the condition of the water should be ascertained by analysis. All information which builders and contractors can supply regarding ground water should be recorded, and in the case of low-lying land it may be desirable to sink pits to find out what the ground-water conditions are.

The character of the soil in which the sewers must be constructed should be ascertained in order to enable the cost to be estimated with fair accuracy. For investigations of shallow depths, a sounding rod is often used. This is a steel rod or a pipe with a steel point and driving head. By placing the ear close to the rod as it is driven, it is possible, after a little practice, to determine when the point passes into and through clay, sand, gravel and other earthy materials. When the rod strikes rock it rings. A post auger is often used to obtain samples of earth from shallow depths. For greater depths, well-sinking outfits are used, particularly in earth, and core drills are sometimes employed where it is desirable to obtain true samples of all underlying strata, or in other words, a vertical section of the material to be traversed.

Complete information concerning the local wages of unskilled and skilled labor and the cost of construction materials and supplies should be ascertained. Freight rates should be looked up and also the rental charges for teams, trucks and equipment. This information is essential for the preparation of reliable estimates.

Office Work.—Work on maps and profiles should be carried along as close as practicable to the field work, so that studies preliminary to design may be started before the field work is finished. As a rule, maps on a scale of 1 in. to 200 ft. are large enough to permit all structures to be shown in adequate detail, but where there are many sub-surface structures to be shown a scale of 1 in. to 100 ft. may be necessary for clearness. The maps will usually require more than one sheet, and in such cases a key map should be drawn showing the way the main maps fit together. The maps should show the location of all street railways, railroads, pipes, conduits, manholes, gate boxes, catch basins, and the names of streets, parks, public buildings, railroads and water courses. The magnetic or true north, or both, should be indicated. After these maps have been drawn, others should be prepared without any of the details so that the sewers can be plotted on them without any confusion. It is very desirable to have the sewerage map on a single sheet, as the relations of the different parts of the system are thus shown most clearly.

A profile of each street in which a sewer is proposed should be plotted. A vertical scale of 1 in. to 4 ft. and a horizontal scale of 1 in. to 40 ft. are commonly used. On these profiles should be plotted all information obtained from borings and soundings and the location of all pipes and conduits intersecting the proposed sewer trench. The names of all cross streets, the kind of pavements, the elevations of pipes and conduits running along the street, and the depth of all deep adjacent cellars should be shown on these profiles.

When the construction is finished, record maps and profiles should be drawn to show clearly and locate accurately every detail of the system, so that there may be no uncertainty in the future concerning the works as actually built.

Reference Marks.—The laying out of trench work must be so done that when construction has begun there will be a minimum interference or delay in the work by the surveying operations. This makes it desirable to have all permanent reference marks where they will not be disturbed by the construction operations yet easily accessible when needed. Their selection will be governed somewhat by the decision of the engineer regarding the position of the line he runs, whether on the center line of the sewer or offset on one side far enough to be undisturbed by the work on the trench. In selecting an offset line, care must be taken to place it where material from the trench will not reduce its usefulness substantially. Where surveys are run through streets, the line is marked every 50 ft. by a spike, generally driven through a small piece of canvas which serves as a marker; in open country the usual stake is used.

Where an instrumentman is employed on the work constantly the line is sometimes marked only at the manholes, and the reference marks in such cases may be selected in several ways. The most rapid work is generally possible if two pairs of such marks at approximate right angles to each other can be selected, so that by stretching a cord between each pair of marks the center point is given. It is not uncommon, however, for the conditions to require setting up the transit over one mark in a pair, and locating the center point on a cord stretched between the other pair of marks. An entirely different method is to locate each center point on the sewer line from two or more nails driven into trees, posts or fences at points clearly described in the notes. If the ring of the steel tape is hung on each nail when the tie lines are measured, one man can give the center line from them without assistance.

The bench marks should be established at frequent intervals and well referenced, for strict compliance with established grades is necessary in sewer construction and leveling can be done more rapidly and accurately where it is unnecessary to set up many turning points.

Transferring the Line into the Trench.—Where the trench work is governed by a survey line along the axis of the sewer, this

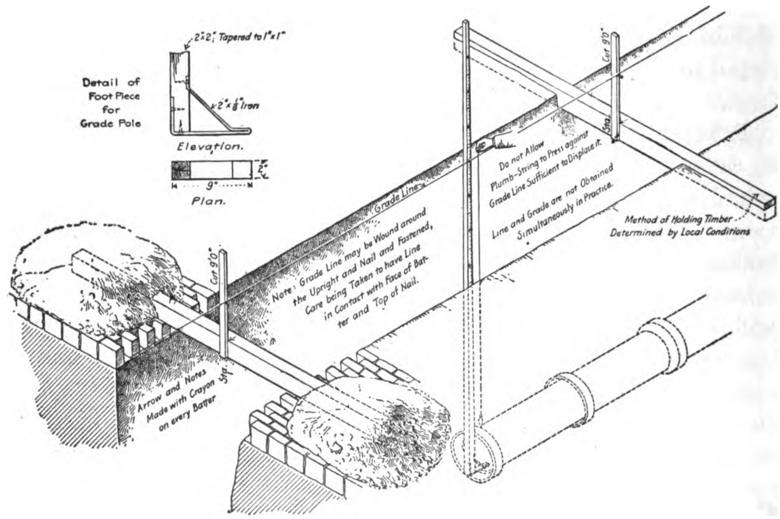


FIG. 100.—Giving line and grade for pipe sewers.

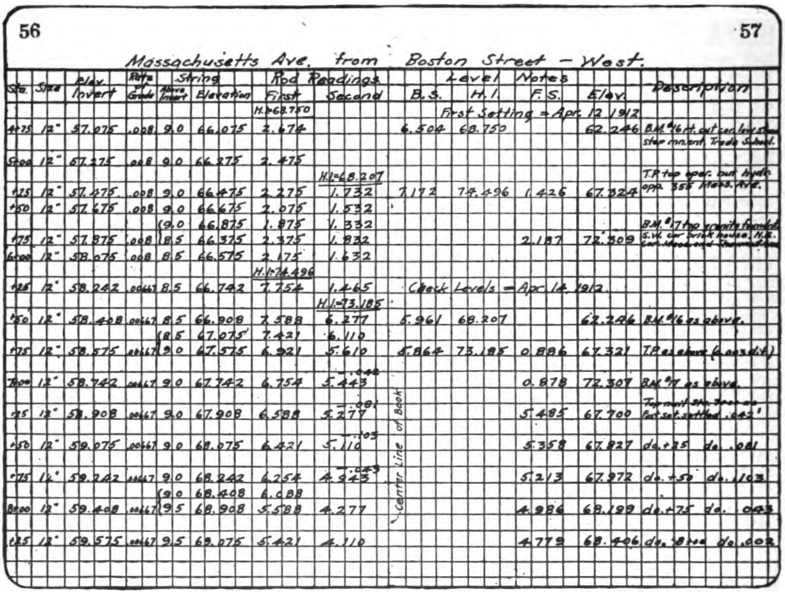
line is marked on batter boards, Fig. 100, usually set at 25-ft. intervals. If the sewer is not in a paved street the batter board is a 2 by 8-in. plank, 7 or 8 ft. longer than the top width of the trench, set on edge across the trench with its top 2 or 3 in. above the surface of the ground. Earth is tamped about each end until the board is held firmly. Where the street is paved and it is undesirable to disturb the pavement more than is absolutely necessary, a 4 by 6-in. or 6 by 6-in. timber may be used as a batter board by weighting the ends with rock or earth until they are immovable.

After a number of batter boards have been set in place a batten is set vertically on the same side of each board in such a way that one side of each batten is directly over the center line of the sewer. The same side of every batten must be used over the center line, because the alignment of these battens is checked frequently by sighting along them as the work progresses, in order to be sure that none has been disturbed, and this check would be impossible unless the same side of every batten was used over the sewer axis. The battens are made of 3 by 1-in. stock dressed on all sides and about 3 ft. long. A transitman locates a nail on the center line and near the same edge of each batter board. The batten is then nailed on the board at the place marked by the nail and plumbed by a carpenter's level or a plumb string, and thereafter the station is marked upon it.

Grade is marked on the center-line faces of the battens by finishing nails, which have no heads and are therefore best suited for the purpose. The nails are driven at a convenient distance (but a whole number offset), above the invert of the sewer, at elevations given by a level party; the nails are left projecting about ½ in. If a stout cord is fastened to a batten, drawn over the nail, stretched to the next batten, wrapped around it just above the nail, and this procedure continued as far along the trench as desired, the cord will give a line parallel with the center line of the sewer and a uniform distance above the invert. Furthermore this line will be so readily seen that it can be checked rapidly by sighting along it by the eye, without an instrument.

Grade is transferred from the cord to the invert by a graduated pole with a foot piece shown in Fig. 101. This foot piece is placed on the invert of the pipe and the pole held plumb. The pipe is then raised or lowered until it is the right distance below the cord. A plumb line is then held lightly against the cord and the pipe shifted sideways until its crown is directly below the point of the plumb bob, and the elevation determination is repeated if necessary.

Notes.—All records made in staking out trenches should be made in standard notebooks, kept thoroughly indexed during the progress of the work. Construction should never be delayed by imperfect or inadequately recorded work of surveying parties. Fig. 101 gives a form of notes which has proved satisfactory. The notes should give the elevation of the invert for each station where grades are given, the slope of the sewer, its size and all level notes by means of which such grades were determined. All secondary readings used in checking up grades should also be recorded. It is of the utmost importance that all work should be checked, preferably by a second independent determination rather than a mere checking of figures when such an independent method can be developed. Thus, for instance, in figuring grades, the elevations of successive stations, or uniform intervals thereof, are found by progressive additions or subtractions. It is important, however, that the elevation of the last station of the



56 57

Massachusetts Ave. from Boston Street — West.

Sta.	Size	Elev. Invert	Rate of Grade	String: Above Invert	String: Elevation	Rod Readings: First	Rod Readings: Second	Level Notes: B.S.	H.I.	F.S.	Elev.	Description
						H.I.=68.750		First Setting = Apr. 12 1912				
4+75	12"	57.075	.008	9.0	66.075	2.674		6.504	68.750		62.246	B.M. #16 rt. cut cor. low st. step mnmt. Trade School.
5+00	12"	57.275	.008	9.0	66.275	2.475						
							H.I.=68.207					T.P. top oper. nut hydt.
+25	12"	57.475	.008	9.0	66.475	2.275	1.732	7.172	74.496	1.426	67.324	opp. 355 Mass. Ave.
+50	12"	57.675	.008	9.0	66.675	2.075	1.532					
				(9.0	66.875	1.875	1.332					B.M. #17 top granite foundat.
+75	12"	57.875	.008	8.5	66.375	2.375	1.832			2.187	72.309	S.W. cor. brick house, N.E. [illegible]
6+00	12"	58.075	.008	8.5	66.575	2.175	1.632					
						H.I.=74.496						
+25	12"	58.242	.00667	8.5	66.742	7.754	1.465	Check Levels = Apr. 14 1912				
							H.I.=73.185					
+50	12"	58.408	.00667	8.5	66.908	7.588	6.277	5.961	68.207		62.246	B.M. #16 as above.
				(8.5	67.075	7.421	6.110					
+75	12"	58.575	.00667	9.0	67.575	6.921	5.610	5.864	73.185	0.886	67.321	T.P. as above (0.003 diff.)
7+00	12"	58.742	.00667	9.0	67.742	6.754	−.044 5.443			0.878	72.307	B.M. #17 as above.
+25	12"	58.908	.00667	9.0	67.908	6.588	−.081 5.277			5.485	67.700	Top nail Sta. 7+00 as Post set, settled .042'
+50	12"	59.075	.00667	9.0	68.075	6.421	−.103 5.110			5.358	67.827	do. +25 do. .081
+75	12"	59.242	.00667	9.0	68.242	6.254	−.043 4.943			5.213	67.972	do. +50 do. .103
				(9.0	68.408	6.088						
8+00	12"	59.408	.00667	9.5	68.908	5.588	4.277			4.986	68.199	do. +75 do. .043
+25	12"	59.575	.00667	9.5	69.075	5.421	4.110			4.779	68.406	do. 8+00 do. .002

Center Line of Book

FIG. 101.—Form of notes for staking out sewer grades; kept on facing pages of note-book.

Each of the two pages of this note-book is 4¾ in. wide and 7 in. high. The ruling on the left-hand pages is blue with every fourth vertical line red, while that on the right-hand pages is blue except for a vertical red line down the center. The lines are 3⁄16 in. apart vertically and horizontally. The date of the work and the initials of the instrumentman, rodman and note-taker should be placed on the top of one of the pages.

group figured should be independently determined by the addition to, or subtraction from, the elevation of the first station of the group, of the difference in elevation between these two extreme stations determined by the intervening rate of grade, multiplied by the distance, as an independent check on all of the work.

In running any levels, incident to setting batter boards for example, the last set up should be checked on a bench mark.

This bench mark should always be a different one from that used at the beginning in order that every group of levels may be tied to at least two bench marks. This method serves not only as a check on the work but also to disclose injury, whether accidental or malicious, to any bench mark. The cut found by leveling should be roughly checked by comparison with the cut shown upon the profile. On account of the permanent value of records of sub-surface conditions, particular attention should be paid to recording the characteristics of the soils, ledges, ground water, pipes, culverts and other underground structures encountered. It is well, also, in recording the date, to enter the weather conditions when the survey is made.

Quantity Surveys, Location of Specials, etc.—The measurements which must be made for quantity surveys will be governed by the method of payment stated in the contract. If the payment is made on the basis of the feet of trench, it is unnecessary to make such precise measurements as when the payment is computed on the unit-price basis. In the latter case, a profile along the center line must be made, showing the ground surface and the profile of the top of all ledge encountered. The finished bottom of the trench may also be recorded if desired, though this is not essential, the essential thing being the necessary removal of the ledge and the replacement of suitable foundation material. Care must be taken to make the records of all conditions affecting quantities complete in order to avoid unnecessary controversy over payments. It is customary in some engineering offices to prescribe certain limits to which the quantity of excavation shall be paid for, although the contractor may excavate beyond those limits if necessary and, if the structure can be built in a narrower excavation, he is not required to take out material to the full width shown by the "lines of excavation" on the drawings. If the contractor is paid by unit prices, however, it is advantageous that the engineer's notes should show exactly what he did, irrespective of contract agreements regarding "lines of excavation," for it is only by having accurate records of what was accomplished that the actual facts can be reported in case of controversy, and such information has real practical value to the engineer of limited experience. Difficulties often arise on account of insufficient measurements and records, but very rarely on account of too many.

In the case of works covering some area, such as pumping stations or disposal works, the surface to be excavated should be cross-sectioned, as should all areas of ledge encountered as the work progresses. All features calling for special payment under the contract should be measured in advance of the work, such as the area of pavement to be removed, the area and quantity of top soil to be removed and kept for future use, and the length of pipe taken up for relaying.

The length of each size of pipe laid in each section of the work and the character and location of all branches and other specials must be recorded. The reporting of the branches by inspectors is often unsatisfactory. It is sometimes the practice, when the engineer is not present all the time the work is in progress, for a pole to be placed at each branch, with its top extending to the surface of the street, so as to enable the engineer to prepare such reference marks or records as he considers best when he next visits the work. Other engineers require their inspectors to locate the branches from marks on the sheeting of the trench or by ties to stakes driven on one side of the trench. On account of the many mistakes which arise when branches are located by persons unfamiliar with the importance of the work, it is preferable to leave them uncovered until the engineer can take the necessary measurements. One method of location is by determination of the "station" of the branch or other fitting, by measuring the distance from the preceding batter board. This should be independently checked by the measured distance from the next batter board, and the sum of the two measured distances should be compared with the distance between the two batters. The dimensions of manholes, catch basins, flushing chambers and other structures should be measured as soon as they are completed. The length, size, location and construction of underdrains should be measured and recorded. The dimensions of all sheeting left in place and its location should also be recorded.

Structural Surveying.—The surveying done in connection with the construction of disposal works, pumping stations and like parts of a sewerage system is performed like the surveying work in building operations. The object is to transfer lines from carefully established bases and grades from permanent bench marks, and to do this work in such a way as to avoid delaying the workmen at any time. The nature of the construction operations carried on is such that the working gangs can proceed most rapidly if they are provided with means of checking many of the lines and grades approximately without calling upon a surveyor. Such means are furnished by batter boards, generally placed at the main corners and at other places where they will be most serviceable. A batter board is a horizontal board held in place, outside the building line far enough to be undisturbed by the work, on posts set vertically and firmly in the ground. On the top edge of this board one or more nails are driven directly on the building line to which the work is being conducted; at first in many cases, this line will be that of the footings, and later it will be that of the wall. As a board will often carry a number of nails, each should be marked to indicate the line on which it lies. If a cord is stretched tightly between the nails on two batter boards on opposite sides of a building it will give the line of a footing or wall. The top edges of the batter boards should, if possible, be a whole number of feet above the bottom grade, for fractional dimensions in working from batter boards often cause mistakes. During the progress of the excavation the foreman can be provided with a rod having a crosspiece nailed to it at a distance from one end equal to the elevation of the batter board above bottom grade. By sighting across the batter boards at the top of this crosspiece while the rod is held vertically it is possible to check closely the depth of excavation at any point and thus avoid taking out more material than is necessary.

In carrying out such surveying care must be taken to provide one or more base lines which will not be disturbed. These lines should have transit points accurately established on them so that the main points on the batter boards can be checked from time to time. In some soils which are much affected by the action of frost and large fluctuations of temperature, the main points on the base lines will require unusually long stakes or lengths of substantial iron pipe, for ordinary stakes will not remain in place. Similarly the bench marks must be selected so that their permanence is assured and the surveyor can use them readily as the work proceeds. Care should be taken to have sufficient bench marks to make turning points unnecessary for most of the work, and to permit ready checking of the elevation of the batter boards at frequent intervals as the work progresses.

As the work progresses lines and grades should be given by the surveyor on all forms before concrete is poured and the correct location of all parts of the structure should be tested by him before the work on them proceeds. It is only in this way that costly errors can be avoided.

INSPECTION

The object of inspection is to see that the engineer's plans and specifications are carried out faithfully. Inspectors are assigned to the resident engineer for this purpose and one of their duties is to safeguard all his reference and bench marks, batter boards and the like, and to check them at frequent intervals as well as they can, so as to detect at once any disturbance of these points which will introduce errors into the work. The inspectors should also see that all transfers of line or grade from the engineer's references by the contractor's forces are made accurately. Workmanship must be inspected constantly in order to ascertain that pipe inverts are laid smoothly at the proper grade, the joints made as required, the surplus jointing material removed from the pipe, water kept away from the joints until they have set, concrete is mixed properly and the forms are located correctly and built to give a satisfactory surface on the finished work. The eye must be trained to notice instantly any deviation from courses, lines, grades, or standards of work.

Excavation.—Carelessness in excavation may cause the settlement of adjoining structures, with consequent financial losses. The inspector should, therefore, watch out for unnecessary excavation, insufficient bracing of the trench and failure to drive the sheeting sufficiently ahead of the excavation to prevent materials from flowing into the trench. When it seems possible that the excavation may cause settlement or other damage to adjacent buildings, elevations of controlling points, such as steps, corners, window sills and the like, should be taken before the work nearby has begun and at later intervals as the work progresses. Where the structures are important or damage to them will result in large financial losses it is desirable to have photographs of them taken from time to time, the negatives being dated and signed by the person making them. When the excavation approaches grade in the trenches the inspector should take precautions to prevent the excavation of material from below grade except when it is of such a character that it must be replaced by better material.

The backfilling of an excavation must be watched carefully to see that there is no danger of settlement of the sewer or other structure. Where underdrains are laid it is particularly important to see that the specifications are carried out faithfully and all refilling below subgrade is done thoroughly. As the backfilling progresses the inspector should prevent the dumping of material from a greater height than 5 ft. on pipe or masonry. Where the fall of the material is greater than 5 ft., it should drop first on a timber grillage or other device to take up the shock.

Sheeting which has been driven below the springing line of a sewer should not be removed ordinarily, for it is very difficult to fill the voids left by such withdrawal. This is true, in even greater degree, of sheeting driven below the bottom of the trench. When sheeting is withdrawn the work should be done preferably slowly, a little at a time, and the voids thus left should be filled with the aid of a water jet. A piece of $\frac{3}{4}$-in. pipe is attached to a garden hose, forced down through the backfilling and moved about while water flows through it. The exact method of work will be determined largely by the character of the ground traversed.

Concrete Materials.—The inspector is not usually required to test cement, but he should be watchful of all deliveries to make sure that only cement which has passed the tests is used in the work. Where cement is obtained from a dealer's warehouse this will often require tagging acceptable cement and placing the tagged bags aside for the exclusive use of the sewerage work. Where the cement is delivered to a contractor's storehouse in carload lots, tagging is unnecessary, but no lot of cement which has not passed the tests should be stored with approved deliveries.

The degree and nature of the inspection of sand, gravel, crushed stone and brick will depend upon the local conditions and specifications for such materials. The inspector must be alert to detect at once any change in the quality of these materials, even if the change is not prejudicial to the work, and to watch for the presence of loam and dirt in aggregates intended for concrete.

The deliveries of reinforcing steel should be checked to make sure that they conform with the bill of materials and the cross-sections of the rods should be measured. Absolute adherence to the usual specification that rods shall be free from rust and mill scale is practically difficult of attainment and engineers are reluctant to compel a contractor to assume the substantial expense which strict compliance with the requirement entails, in view of the protective effect of the mortar on the steel. Serious rusting or pitting is ground for rejection of the steel, but the inspector should get clear instructions from the resident engineer as to what "serious" shall signify on the work in hand, and what may constitute a reasonable interpretation of the specifications relative to this subject.

Cast-iron Pipe.—This should be inspected as it is unloaded from the car and again before it is lowered into the trench. Special attention should be paid to the detection of cracks in the spigot. The inside and outside are rubbed with a heavy cotton mitten or bunch of dusty waste, which forces enough dust into a crack to make it visible. The presence of a crack can also be detected by the ring of the metal when it is struck with a wooden mallet, while suspended in the unloading slings or on the rolls.

Vitrified Pipe.—If vitrified pipe is furnished by a manufacturer with a reputation for good mill inspection it is generally considered unnecessary to inspect the pipe at the mill, but on account of the liability of being cracked in transportation the pipe should be inspected as it is unloaded from the cars and again before it is placed in the trench. The inspection comprises ringing the pipe with a light hammer and examining it for warping, deformation, defects in the bell and spigot, and poor interior condition. A light tap of the hammer on a pipe set on end will indicate the presence of a crack so small as to be invisible until the surface has been wiped with chalk and the pipe again tapped lightly with the hammer, which will cause the crack to become filled with chalk and visible.

EXCAVATION

The excavation is usually the part of sewer construction most important in determining costs. To handle it to advantage requires good management and judgment on the part of the man responsible for its execution. The materials used in sewer construction generally cost about as much as the labor, but the contractor makes little profit on supplying them. The chief profit is made by handling the work so well that labor charges are kept to a minimum. The time element is also important, for rapid work during the summer when costs can be kept low may avoid a large unnecessary expense if the work is prolonged into the winter. Charges for enginemen, firemen, tagmen and machinery are among the expenses which mount up rapidly when progress is slow. It is necessary to allow for the delays due to unfavorable weather, particularly rain and frost, over which the sewer builder has no control, and on large undertakings contractors now often arrange with the local office of the U. S. Weather Bureau for early notice of probable unfavorable conditions.

Classification.—There is little agreement among sewerage engineers as to the classification of excavated materials for purposes of payment. The prevailing opinion among Eastern engineers, whose work is with materials in which the rock is usually hard and fairly sharply distinguishable, is that a classification into rock and earth should be made. Among the engineers of the Pacific Slope it appears customary to use no classification. The authors use the following classification for rock:

"Rock, wherever used as the name of an excavated material, shall mean boulders exceeding ½ cu. yd. in volume or solid ledge rock which, in the opinion of the engineer, requires for its removal drilling and blasting, or wedging, or sledging or barring. No soft or disintegrated rock which can be removed with a pick; no loose, shaken or previously blasted rock or broken stone in rock fillings or elsewhere, nor rocks exterior to the maximum limits of measurement allowed, which may have been previously loosened in excavating for water pipes or other purposes and which, by reason of such loosening, may fall into the trench, will be measured or allowed as rock."

Opening the Trench.—Where the paving materials are to be used again they should be placed in neat piles outside the probable limits of the excavation. The crushed stone from macadam pavements is often piled on the opposite side of the trench from that used for storing the remainder of the excavated material. Where the excavation is deep or likely to be open long the material to be replaced is so likely to become lost that, if of good quality, it is sometimes hauled for storage on vacant lots or on one side of several cross streets.

Frost enters compacted ground to a greater depth than where it is in its natural condition. Where the frost sinks to a depth of 3 ft. in a field or wood lot it may penetrate 5 or 6 ft. in a neighboring well-traveled road or street. Opening a trench in frozen ground by picking and barring is so slow that the surface is first thawed by building fires on it, usually at night so the ground will be ready for work the next morning. Sometimes the surface can be steamed, using a portable boiler and a number of wooden boxes 12 to 15 ft. long, about 10 in. high and approximately as wide as the trench. They are laid with the open side downward and holes are bored in the top every 12 in. longitudinally and at 2-ft. intervals across the box. Each hole has a wooden plug to prevent the escape of steam. After the boxes are in place they are banked with earth. The steaming is done at night, one man being enough to look after the boiler and thaw the ground. He does this with a ¾-in. gas pipe about 6 ft. long provided with an iron crossbar for a handle. To the end of this pipe is attached a steam hose connected with the boiler. The operator pushes the pipe down vertically through one of the holes in the top of the box, and as the steam thaws the ground, he gradually works the iron pipe down until it has penetrated the entire frozen stratum. The pipe is then pulled up, the hole plugged and the operation repeated at the next hole. In this way a man can thaw out an area 4 to 6 ft. wide and 48 ft. long in one night if the frost is not over 4 ft. deep.

Hand Excavation.—On small work it is desirable to allot a given length of trench to each laborer or pair of laborers, or on larger work to a squad of laborers, so as to furnish each man or squad with approximately the same amount of work for a half day, or for a full day in very deep trenches. This gives the foreman an opportunity to distinguish between the good and poor men, and the men themselves are put on a competitive basis.

It is desirable to throw all material, not rock, excavated from the trench, on one side of it. That from the first 6 ft. of depth should be cast as far to the side as it can be thrown, so as to leave room nearer the edge of the trench for that from lower depths. Unless the material is plastic or sticky, it is usually possible for the men to throw it from the trench until a depth of about 8 ft. is reached. Then platforms are constructed about 6 ft. from the surface, supported on the cross braces if the trench is sheeted or on temporary braces otherwise. Such a platform is made long enough to receive material from an excavator at either end, and one staging man can keep the platform clear. Where a trench is wide the platform is sometimes made narrow so that material can be thrown on it at any point along its length, but in narrow trenches the platform extends across the entire trench. When the trench exceeds 14 ft. in depth another set of platforms must be used, placed about 12 ft. below the surface. The staging man on this platform casts the material to the higher one. Square-pointed shovels should be used on the platforms and round-pointed shovels for work in the bottom of the trench. After a depth of about 5 ft. is reached it is usually necessary to provide one man on the bank for each two in the bottom of the trench, to overcast the material thrown out so that none of it shall remain closer than 2 ft. to the edge of the trench. This space is necessary to allow the workmen to walk safely and handle materials, and to reduce the risk to the men in the bottom of the trench from stones rolling back and falling into the trench.

Trenching Machinery.—There are two general classes of trench machinery, one for excavating and the other solely for handling excavated material. The former is usually termed ditcher or trencher and the latter trench machine.

The ditcher first came into general use where the soil and earth were particularly favorable for such equipment and close sheeting and heavy bracing were unnecessary. Later, however, they were used to advantage in shale and hardpan. Pipe lines across trenches interfere with their use, but in some cases it may be more economical to run the risk of occasionally tearing out such pipes with the machine and replace them later than to forego its use. The impression seems to be that as a rule such machines do not give a substantial saving unless the trench is at least 6 ft. deep or labor expensive. The increased cost of labor since the war, has materially enlarged the advantageous limits of use of such machinery.

The trench machine makes comparatively little difference in the amount of labor employed in picking and shoveling in the bottom of the trench. To arrive at the comparative cost of all-hand excavation and excavation with such a machine, it is only necessary to compare the cost of staging and overcasting the excavated material and shoveling it back into the trench, with the cost of operating a machine, including labor, fuel, repairs, rental and all incidentals. When these two charges balance, it may be wise to adopt some kind of trench machine as it is much more convenient than hand work, does not usually obstruct a street so much, reduces the time required to perform the work, and decreases the number of men required.

In studying the practicability of machinery in sewer trenching, careful consideration should always be given to the probable quantity of work to be done. Formerly it was not uncommon to find too much plant, with resulting unnecessary costs, but the effect of the World War on the wages of labor has modified the conditions that existed prior to 1915. In any case, it will probably be inadvisable to use a machine on a short piece of work, unless it is already at hand, for the cost of unloading and erecting it, taking it down when the work is done and reloading it on the car, together with freight charges, is so great that the machine may prove a source of expense instead of economy.

The ditchers or trenchers are usually operated by engines mounted upon wheels. The digging is done by buckets carried by an endless chain running over sprocket wheels on the end of an arm suspended from the rear of the machine and is adjusted

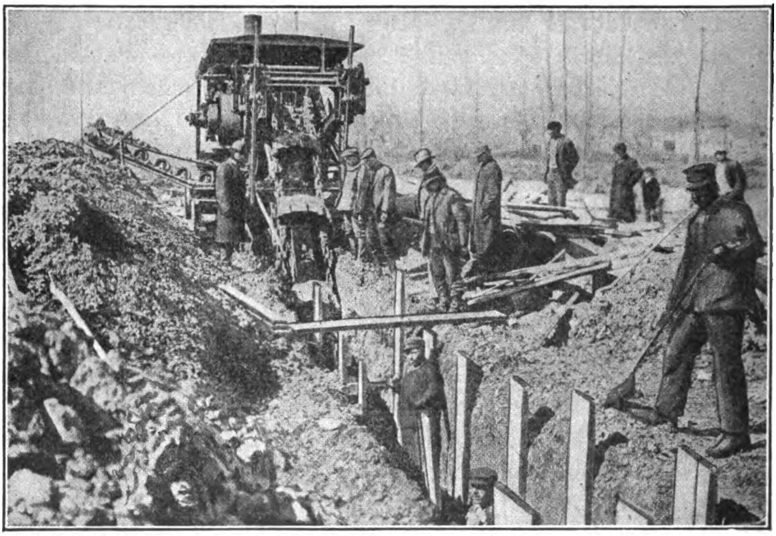


FIG. 102.—Austin trencher excavating a trench 36 in. wide and 15 ft. deep.

to permit the excavation of the trench to a precise grade regardless of the inequalities of the surface over which it passes. The width of the trench excavated is governed by the width of the buckets selected, each machine being supplied with buckets of several widths though some increase in width can be obtained by barring off additional material. The material excavated by the buckets may be dropped by an endless belt operating crossways of the ditch and discharging the excavated material in a windrow on either side of the trench or into wagons to be hauled to the rear of the work for the backfilling or to be wasted. The Austin trencher is illustrated by Fig. 102. Another somewhat similar type of trencher, the Buckeye traction ditcher, has the buckets attached to a large breast wheel instead of an endless chain, Fig. 103. The trench shown in the illustration was 32 in. wide and 12 ft. deep. Steam shovels, although not particularly well adapted to ordinary sewer construction, have been used with more or less success upon some large work. For this purpose, it is usually desirable to provide an unusually high crane or a long dipper arm, or both, although a shovel of standard dimensions may be used for first cuts in the trenches.

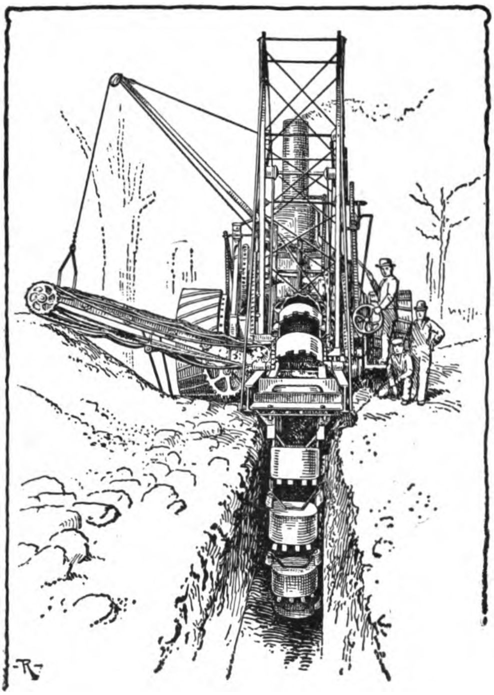


FIG. 103.—Buckeye traction ditcher.

A common type of trench machine is the Carson, Figs. 104 and 105, designed by Howard A. Carson for sewer trenches and first used upon work in the vicinity of Boston, Mass. It is particularly well adapted to this class of excavation.

Another type of trench machine is the cableway which is used frequently on many classes of work. A large cableway of the ordinary type is not particularly well adapted for sewer trenching. For this work, light towers which can be easily moved should be provided and the cableway should be relatively short, usually

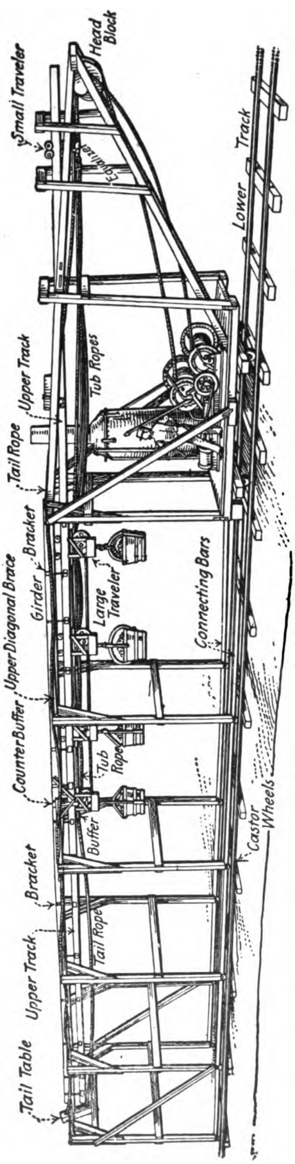


FIG. 104.—Carson four-traveler machine with single upper track.
A standard machine of four travelers has twelve trestles in the rear of the engine, although only six trestles are shown in this illustration.

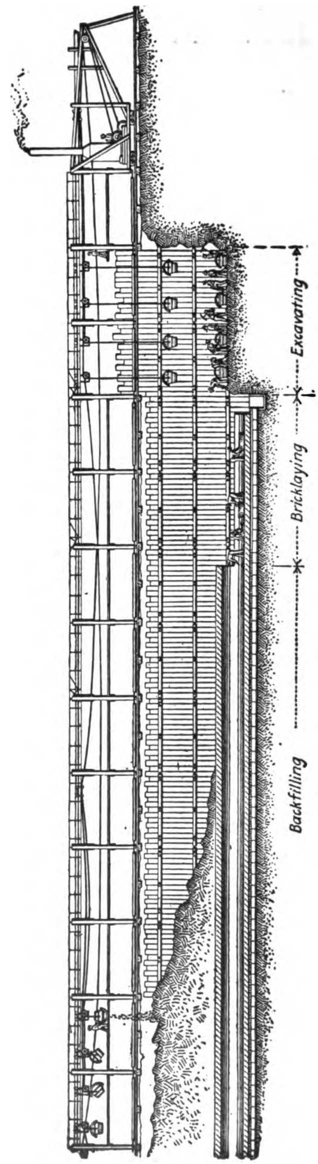


FIG. 105.—Longitudinal section of a sewer trench with machine.

not over 300 ft. in length, Fig. 106. Only one bucket is handled at a time and this has a capacity varying according to the require-

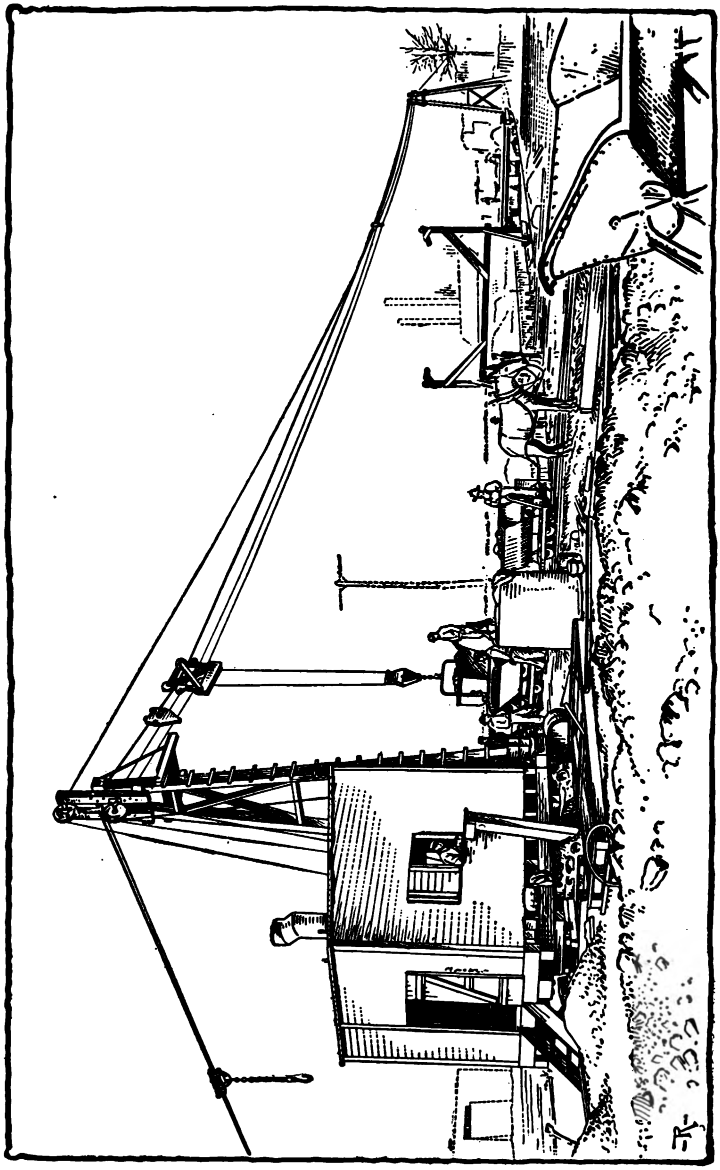


FIG. 106.—Cableway used on St. Louis sewers (Lambert).

ments of the work from ½ to 1½ cu. yd. As the cableway is relatively high above the ground and is supported and steadied only at the ends, 300 to 1,000 ft. apart, it is difficult to prevent the buckets from swinging and occasionally knocking out the timbering.

Rock Excavation.—The method to be adopted for the excavation of rock in sewer trenches depends upon the quality and quantity of rock to be excavated. Ledge varies greatly in character, from that which may be removed by picking and shoveling as readily as some kinds of earth, to rock which requires drilling and blasting for removal. Some kinds may be removed by picking, barring and wedging without the use of explosives, and some lie in nearly horizontal strata and may be removed by methods similar to those employed in the ordinary quarrying of building stone. In some cases the quantity to be removed is so small or the danger of damaging adjacent structures may be so great as to warrant its removal by picking, barring and wedging even where the nature and quality of rock are such as to warrant drilling and blasting under other conditions.

The methods commonly used for drilling ledge are hand drilling and power drilling. Hand drilling in turn may be subdivided into three classes, single hand, double hand and churn drilling. Power drilling may be classified in accordance with the type of machine drill used, which may be a rotary or core drill, a reciprocating or percussion drill or a hammer drill.

Hand Drilling.—For small holes and occasionally for large ones of moderate depth, the drill may be held in one hand and struck by a hammer held in the other hand, the hammer weighing about 4 lb. This method is largely used in quarrying, where a number of small holes are drilled in a line along which the rock is to be split by driving small wedges into the holes between tapered strips of steel. The wedges and strips of steel are called "plugs and feathers."

For larger and deeper holes it is customary to have one man hold and turn the drill, while two or three men strike the head of the drill in rotation with hammers weighing from 8 to 12 lb. In this method, the drills should be turned about 45 deg. each time in order to insure a nearly round hole.

In churn drilling the hole is drilled by the churning of a steel drill, which is raised by hand and allowed to drop of its own weight, or it may be thrust into the hole, although but little advantage is gained thereby. Each time the drill is raised, it is turned about one-eighth of a revolution so as to prevent its striking the bottom of the hole successively in the same position. By confining the turns to about 45 deg., the hole is maintained more nearly circular than would be the case if a larger turn were given each time. Where added weight is desirable, a ball of iron or steel is sometimes welded to the drill rod, at or slightly below its middle point, care being taken to have the center of gravity of the ball in the line of the axis of the rod.

The bit ordinarily used in double hand drilling is provided with a chisel edge and is made from octagonal steel 7/8 in. in diameter. The wedge-shaped edge varies in sharpness according to the quality of rock to be drilled. It should be made as sharp as possible and yet not break or become rapidly dulled. The angle of the wedge-shaped edge usually varies between 45 deg. for soft rocks and 90 deg. for hard rocks.

Bits are usually made up in sets, depending upon the depth of holes to be drilled. The bottom of the hole must be about 1 in. in diameter to receive the ordinary dynamite cartridge. The finishing drill must therefore have a face at least 1 in. in length, and as a matter of fact, it is usually made somewhat longer, in order to allow for wear. Each bit must be slightly smaller than the bit preceding it in order to allow for wear in drilling. A set of four bits will ordinarily vary approximately between 1 5/8 in. for the starting bit to 1 1/4 in. for the finishing bit. It is necessary to re-sharpen the drills at frequent intervals, not only that they may be reasonably sharp, but that they may be of proper gage.

Spacing of Drill Holes.—The spacing of drill holes and the quantity of rock loosened per foot of hole and per pound of dynamite vary greatly with the character of rock to be excavated, the strength of the dynamite used, the width of trench and the necessity of loading lightly to protect buildings and underground structures. Blasting in trenches requires much more powder than blasting on the surface, for it is possible to provide but one face, and that often a narrow one, toward which to blow.

Cleaning Holes.—During the process of drilling, water is poured into the holes, which tends to hold the powdered rock in suspension and forms a sludge which must be removed from time to time. This may be done by the use of a scraper or "spoon," made by attaching a semi-cylindrical strip of steel to the end of an iron rod about 3/8 or 1/2 in. in diameter. The bottom of the cylindrical portion of the spoon should be turned up much the same as the point of an ordinary tablespoon. Where such a spoon is not available, the broomed end of a round stick of slightly smaller diameter than the lower portion of the hole may be dropped into the hole and withdrawn, bringing with it a portion of the sludge, which may be removed by striking it over a nearby object.

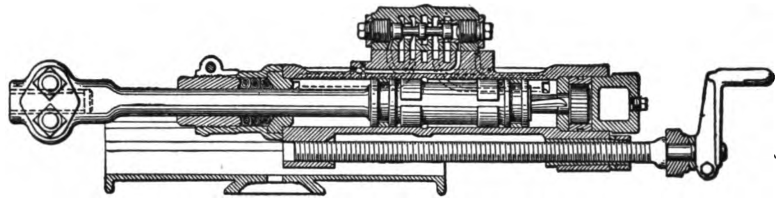
Drilling by Machinery.—When the depth of holes, hardness of rock or quantity of ledge to be excavated is such that the economical limitations of hand drilling are exceeded, drilling machinery should be used. Of the three different types of drilling machines in common use, the rotary or core drill is rarely if ever used for the drilling required on ordinary sewer work, although it may occasionally be used to advantage for making test borings.

The rotary drill rotates the bit which cuts or chips the rock at its base. The more common form is the so-called "core drill," by which an annular hole is cut by the bit, thus leaving inside the hollow steel a core of the rock which may afterward be broken off and withdrawn, thus furnishing a sample of the material encountered.

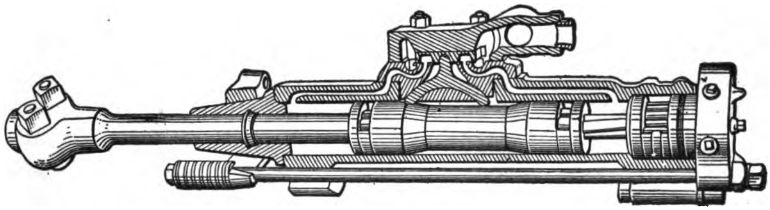
The reciprocating drill has been used upon all kinds of rock excavation for many years. It is driven by steam or compressed air and is used upon heavy work where hand or hammer drilling would not prove economical. It consists essentially of a steam cylinder in which a piston is driven forward and backward by the steam. The bit is attached to the lower end of the piston rod, which is rotated on the upward stroke in order that the hole may be drilled round and true. The bit is made to follow the rock as fast as it is cut away, by moving the entire cylinder toward the rock by means of a feed screw and crank. This type of drill withdraws the bit from the rock at each upward stroke and forces it back against the rock at each downward stroke, whence its old name of percussion drill.

There are several types of reciprocating drills, differing as to details of construction. The drills ordinarily used are often distinguished by the nature of the valve controlling the steam or air supply, as shown in Fig. 107.

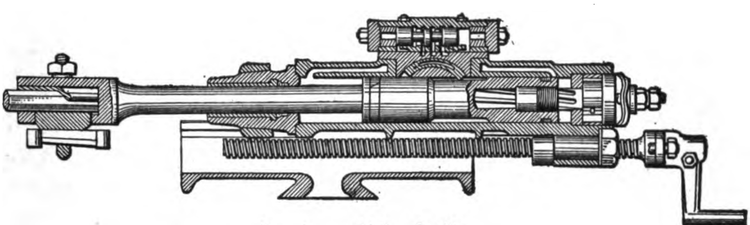
Hammer Drills.—The hammer drill came into general use after 1900. With it the bit remains at the bottom of the hole in contact with the rock, instead of reciprocating with the piston at each stroke, as in the case of the reciprocating drill. The piston, or hammer, strikes the upper end of the bit, which is rotated either by turning the drill by hand or automatically by a mechanical device.



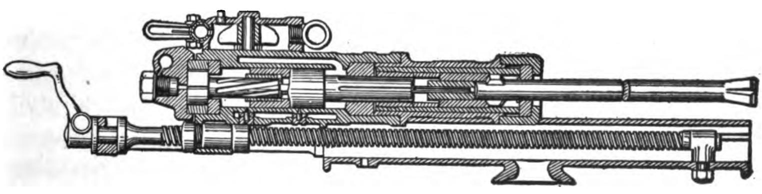
Differential Valve Drill.



Tappet Valve Drill.



Auxiliary Valve Drill.



Butterfly Valve Drill.

FIG. 107.—Four types of valves for percussion drills.

The advantages of the hammer drill over the reciprocating drill are that it is light, can be readily handled by one man and requires no tripod or other mounting, as shown in Fig. 108. It is particularly serviceable in narrow trenches where it is difficult to handle the reciprocating tripod drill. Holes may be drilled with this machine in any direction. There is considerable saving in time in changing bits, as with the hammer drill it is simply necessary to lift off the machine, take out the drill and replace it with a new one, replacing the machine on the new bit; while with the reciprocating drill it is necessary to crank back the feed screw, withdrawing the bit from the hole, remove the bit from the chuck, place the new bit, and finally turn down the feed screw to bring the bit to the required position.

FIG. 108.—Jackhammer drill (Ingersoll-Rand).

This drill occupies a sort of middle place between the double-hand drill and the reciprocating drill, superseding each under many conditions. The double-hand drill will still be used when there is insufficient drilling to be done to warrant providing an air compressor for operating power drills, but where such a machine is available there appears to be little field left for the hand drills. Similarly the reciprocating drill has been replaced upon work too heavy for hand drilling and yet within the practical limits of the hammer drill. Where power is available the hammer drill may be used advantageously for block-holing to break boulders, for trimming small outcroppings of ledge, and for following up reciprocating drills where shallow holes are required.

Explosives and Blasting.—Dynamite is the explosive now most commonly used on sewer excavation, although gunpowder or black powder, contractor's powder and TNT are sometimes used. The disruptive and shattering effects of an explosion are due to the instantaneous generation in a restricted chamber of large volumes of gases at high temperatures. The active and determining ingredient of dynamite is nitroglycerine. It burns quietly when ignited in the open air, explodes at a temperature of 388°F. and freezes at 41°F. When inhaled or absorbed through the pores of the body, it causes headaches and sickness. Nitroglycerine is a very unstable compound exploding very easily when subjected to detonation or percussion. Being a liquid it is readily absorbed by various substances such as sawdust, wood pulp, etc. and these are therefore used as carriers. True dynamite is a mechanical mixture of nitroglycerine and an inert absorbent. False dynamites are composed of nitroglycerine and absorbent mixtures which themselves enter into the chemical reaction causing the explosion and add to the strength and power which would be furnished by the nitroglycerine alone.

Freezing of Dynamite.—Several kinds of dynamite freeze at a temperature of 42 to 50°F. Dynamite which is frozen or

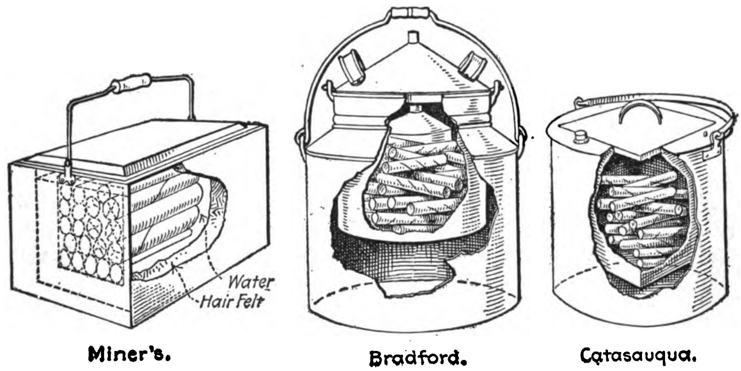


Fig. 109.—Commercial kettles for thawing dynamite.

chilled cannot be detonated easily. The first warning of this condition generally comes when a number of holes miss fire, while others appear to detonate in the ordinary manner. If a careful investigation be made, it will generally be found that the charges which appeared to detonate as usual were far less effective. Yet dynamite while chilled or frozen is exceedingly sensitive to friction and sticks should not be broken or cut while in this condition. Probably more blasting accidents occur in using or thawing frozen dynamite, than in any other way.

The thawing of dynamite requires special precaution to prevent explosion. Thawing by immersion in water is not only dangerous but objectionable for the reason that by diminishing the quantity of nitroglycerine, the explosive power is decreased. Thawing by holding over a hot stove, either in the hand, upon a shovel or in a container of any sort is foolhardy and dangerous as the nitroglycerine which may thus be freed by heat is liable to premature explosion, and there is danger of the explosion of the adjacent stores of dynamite.

Thawing kettles, especially designed for this work, Fig. 109, may be procured from dealers in contractors' supplies. They are

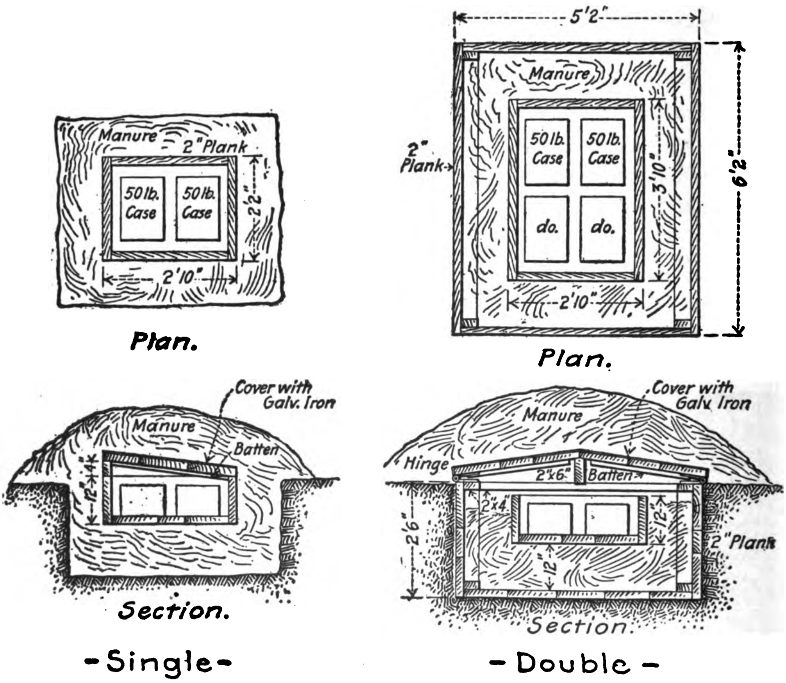


FIG. 110.—Thawing boxes for dynamite (du Pont).

designed for thawing relatively small quantities, as from one to two dozen sticks, and may be used with comparative safety provided the directions, usually accompanying the kettles, are strictly followed.

Perhaps the best method of providing a supply of thawed dynamite is to place the dynamite, immediately upon delivery on the work, in a box set in a suitable pit in the ground and entirely surrounded with green horse manure, as shown in Fig. 110.

Such a thawing box may provide for one, two, or possibly more, cases of dynamite at one time. The sticks may be taken from this pit from time to time as required upon the work, care being taken not to take the dynamite out until it has had ample time for thawing if it was delivered in frozen condition. Dynamite should not be left in such a pit longer than necessary, as dampness may in time penetrate and injure it. For larger quantities, regular thaw houses should be provided. The method of heating should be indirect, and such as will under no circumstances permit the temperature to be raised above 100°F.

Exploders and Fuses.—For exploding contractors' powder or dynamite, a powerful shock is required, for which purpose either electric fuses or blasting or fulminating caps are provided. The caps are made of copper shells $\frac{1}{4}$ in. in diameter and about 1 in. long, loaded with a little fulminate of mercury and carefully sealed. The fulminate of mercury occupies only about $\frac{1}{4}$ in. of the closed end, while the balance of the shell serves to hold the fuse. The electric fuse consists of a special cap containing fulminate of mercury, as in the blasting cap, and in addition, two electric wires projecting through a sulphur plug and connected by a platinum wire which becomes heated to a red heat by the current of electricity used, and thus explodes the fulminate of mercury.

In selecting exploders, care should be taken to purchase only those which are amply strong for the dynamite being used. In shallow holes one fuse only will generally be used and that will be placed in the top, or next to top, stick. Where deep holes are used, fuses should be placed about 5 ft. apart. The following instructions for priming dynamite and other high explosive cartridges are given in a bulletin published by the E. I. du Pont de Nemours Co.:

"To prime a dynamite or other high explosive cartridge with blasting cap and safety fuse, make a hole in the end of the cartridge after unfolding the paper shell, or in the side of the cartridge near one end, with a small pointed stick about the diameter of a lead pencil. This hole should not be much larger in diameter than the blasting cap, for an air space around it always detracts from the force of the shock that a detonating blasting cap gives the explosive surrounding it. The best results will be obtained if the blasting cap is pointed straight down into the primer cartridge.

"When the blasting cap has been put in the end of the cartridge, the paper must be folded carefully about the safety fuse, and tied securely with a piece of string. When it is inserted in the side of the cartridge near the end, the safety fuse is held in position by tying it to the cartridge with a double loop of string.

"If the work is wet, cover the safety fuse where it enters the blasting cap with soap or tallow to prevent water getting into the blasting cap. Oil or thin grease should never be used for this purpose, as they may penetrate the safety fuse, and destroy the efficiency of the powder in it.

"The correct way to prime a high explosive cartridge with an electric fuse is to follow the same method as when fuse and blasting cap are used. The common custom of taking one or more loops, or half hitches, around the cartridge with the wires themselves, after inserting the electric fuse cap in a hole made diagonally in the side of the cartridge near one end is always to be condemned. The principal objection is that the looping of the wires is very likely to break the insulation, causing short circuits, or leakage of current in wet work. Sometimes the wires themselves are broken."

When charging, the sticks of dynamite should be pressed firmly into place with a wooden tamper, having no metal parts. Where the holes are practically dry, it will be found advantageous to put a few slits in the side of the dynamite wrappers so that the cartridges may be pressed into the hole in such a way as to fill it completely. If the holes are wet, the cartridges should not be slit unless gelatin dynamite is being used. Care should be taken in pushing the top cartridge into place not to disturb or displace the exploder or damage the fuse or the connecting wires.

Pumping.—If good joints in a pipe sewer are desired, it is absolutely necessary to have a dry trench. Cement joints must not be submerged or exposed to running water until the cement has acquired its final set. It is undesirable to allow water to come into contact with mortar until the latter has been 24 hr. in place.

Diaphragm pumps are most often used in handling water in trenches. Such a pump, operated by hand, will lift from 30 to 50 gal. per minute from a depth of 16 ft. The usual size for such work has a 3-in. suction hose, which costs about as much as the pump and should be cared for attentively, in order to prevent damage by kinking or abrasion, with the attendant danger of failure when most needed. Larger sizes of diaphragm pumps are made for operation by gasoline motors. Good portable kits mounted upon wheels are now available.

When the quantity of water to be handled exceeds the capacity of a diaphragm pump a centrifugal pump or steam vacuum pump is generally employed, for a reciprocating pump is likely to have the plungers and valves cut by the grit in the water and is therefore not suitable for such use. Where the drainage of a trench can be accomplished through underdrains discharging into a sump which can be kept in service for a considerable time, it may be advisable to install a centrifugal pump at the sump, but the low lift and dirty water to be handled make high efficiency less important than reliability. Where the water is very dirty and the position of the pumps must be changed frequently steam vacuum pumps are now largely used when the diaphragm type has inadequate capacity.

Underdrains.—Underdrains, usually constructed with socket and spigot vitrified pipe, are of two classes; construction underdrains put in to facilitate the building of a sewer and generally abandoned after its completion, and permanent underdrains provided with permanent outlets. They are seldom less than 4 in. or more than 8 in. in diameter. As they are laid below the sewer they are likely to receive any leakage from the latter and care should be taken to prevent the discharge of their contents into water which should be kept free from contamination.

If a construction underdrain is laid in the bottom of a rock trench care should be taken to reduce the amount of excavation to a minimum. In gravel trenches there is little danger of fine sand entering the underdrain if the trench in which it is laid is backfilled with clean gravel or broken stone. Great care must be taken in laying the underdrains in fine water-bearing sand; here the tile should be carefully surrounded with gravel and sand, the coarser material next the tile. The joints are wrapped with one thickness of cotton cloth in strips 6 in. wide securely tied, to keep the sand out. Where careful precautions of this nature are not taken an underdrain may become completely clogged in a few hours. Even with the greatest care fine sand will sometimes enter an underdrain and where this is likely to occur it is customary to thread a manila rope through the underdrain as it is laid. This rope can be pulled back and forth to stir the sand and allow the water to carry it off. When the rope proves inadequate a chain is sometimes attached to it, which is similarly pulled back and forth until the drain is cleared.

If the underdrain is properly constructed little of the sand which causes trouble enters along its length but comes in at the upper or free end. It is difficult to prevent this, as the work of excavation keeps the sand stirred and the water carries it into the underdrain. It is possible to intercept much of the sand by digging sumps where it can settle, but these greatly delay the laying of the underdrain and it is generally considered better to keep the drain laid close to the excavation, and to keep it free from sand with a rope or chain.

Where the water is removed by hand pumps only, a comparatively short length of trench bottom should be worked at one time and the underdrain should be laid as soon as a portion of the bottom is ready for it. The upper end of the drain should be kept protected all the time by a substantial strainer to keep out the fine sand. One or more hand pumps are used to remove the sand-laden water from this short stretch of trench bottom, while other pumps take care of the water gathering in advance of this portion of the trench.

Quantities of Excavation.—The cost of sheeting, bracing and pumping will be about the same whether the trench is of the

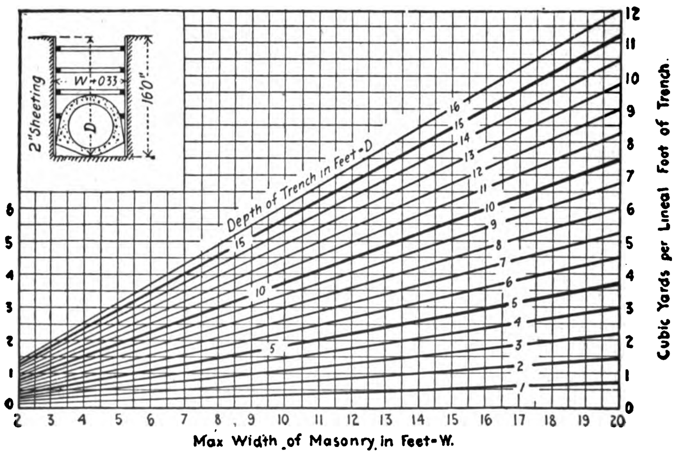


FIG. 111.—Theoretical quantity of excavation for trenches 1 to 16 ft. deep, for sewers of 5 to 20 ft. outside diameter.

exact width needed or a foot or two wider. The cost of picking, shoveling, backfilling and hauling surplus material to the dump will be closely proportional to the quantity of material actually excavated. Where some types of machinery will be used, the width of the trench must sometimes be wider than necessary for the structure. In hardpan and dry clay, the banks may stand without bracing, thus making it possible to dig the trench of practically the width of the structure to be built in it, while the same sewer in a trench in running sand may require heavy timbering. In rock excavation it is very difficult to excavate exactly to the line theoretically required.

To prevent controversy as to the quantity of excavation to be paid for on contract work, it is customary to prescribe certain limits, called "lines of excavation," to which the quantity to be paid for shall be calculated, but the contractor may, if necessary,

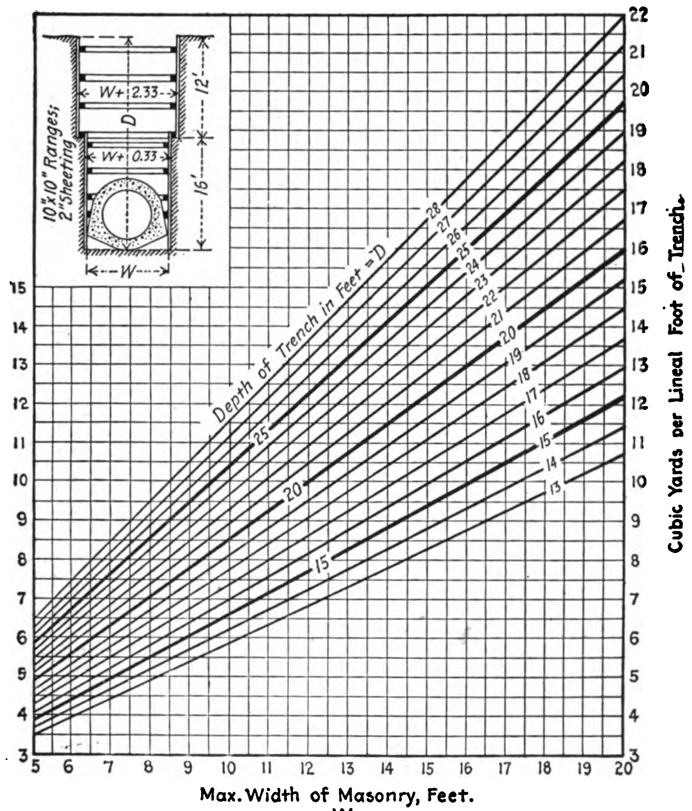


FIG. 112.—Theoretical quantity of excavation for trenches from 13 to 28 ft. deep for sewers of 5 to 20 ft. outside diameter.

excavate beyond these and if the structure can be built in narrower excavation he is not required to excavate the full width shown upon the drawings. Some engineers provide in the contract that earth excavation shall be paid for per linear foot of sewer construction. This is an excellent method, provided the preliminary engineering has been done with sufficient care and thoroughness so that there is no possibility of a change in plans.

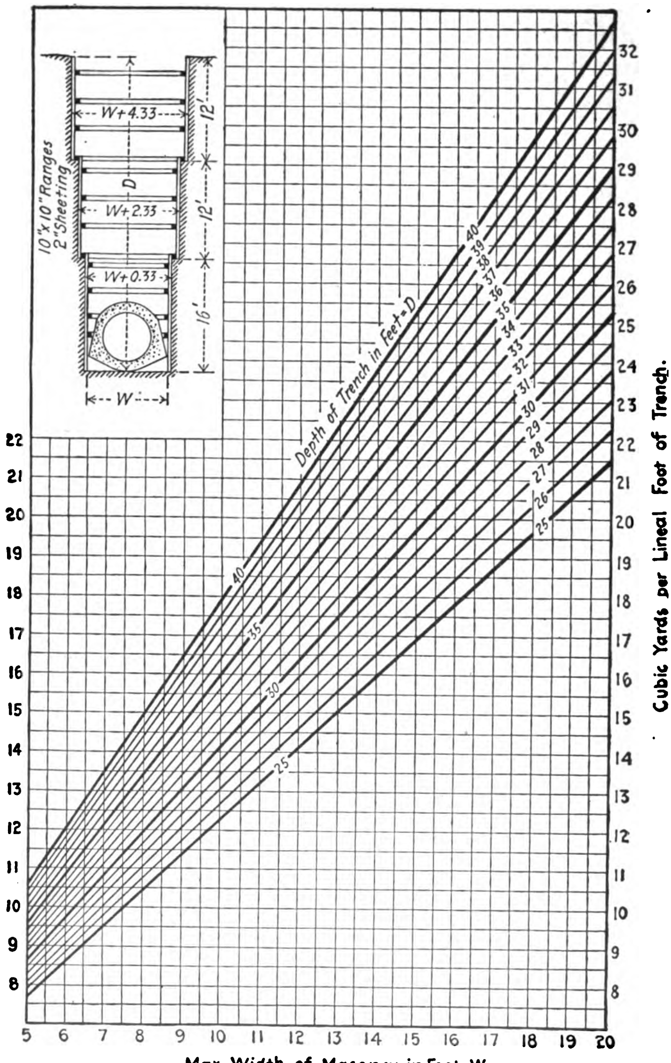


FIG. 113.—Theoretical quantity of excavation for trenches 25 to 40 ft. deep for sewers of 5 to 20 ft. outside diameter.

The quantity of excavation required for trenches for masonry sewers from 1 to 40 ft. in depth and up to 20 ft. in width may be obtained from Figs. 111, 112 and 113. These diagrams are based upon the assumption that the top set of sheeting will be 11 or 12 ft., the bottom set 16 ft., and the intermediate set 12 ft., that 2-in. sheeting will be used, and that 10 by 10-in. rangers and braces will be employed. The quantities of excavation obtained by the use of these diagrams should be regarded as approximate only, for special calculations will be needed for any case where exact quantities are required. The averages of actual cross-sections of about 25 sewer trenches in Louisville showed that the excess excavation for small sewers having an extreme outside width of 6 ft. was nearly 33 per cent, whereas the excess excavation for sewers from 12 to 16 ft. wide, outside, was 1 to 5 per cent, which indicates the decreasing importance of excess width as the width of the sewer increases.

CHAPTER VII

SHEETING AND BRACING OF TRENCHES

Shoring Trenches.—Trenches may be excavated in most materials to a depth of 3 to 6 ft. without much danger from caving banks. In some materials, as hardpan, excavations may be carried down to greater depths, 12 to 20 ft., without sheeting and without much danger if the trench is not kept open long. Usually, however, it is necessary to shore the sides of trenches thoroughly for the protection of the workmen and to make progress possible. It is well to have a trench so protected as to be safe at all times, and to anticipate storms which may cut away the sides unless they are well shored, and other causes of failure. Under some conditions it is desirable to place low banks of earth along the sides and across the ends of the trench to prevent surface water from flowing into it and from cutting away the earth behind the sheeting.

The method of shoring trenches is usually left to the judgment of a foreman in charge of the work. Good judgment based on wide experience is essential to determine when shoring is needed and what system shall be adopted. There are no rules by which an inexperienced man can determine the depth to which a trench can be excavated before sheeting should be put in place. In many cases the work is inadequately done, resulting in delays and unnecessary expense. On the other hand, it is not unusual to find an uneducated foreman or superintendent, with many years of practical experience, who is capable of rendering a better judgment upon this subject than that of an educated engineer without wide experience in this subject. This is largely because the practical application of theoretical principles of earth pressures involves many assumptions which can only be made correctly by an experienced engineer. It is for this reason that the young engineer should cultivate the habit of observing the slopes taken by all exposed materials, the methods of bracing used in all trenches and excavations he sees, and the behavior of this bracing.

Trenches are shored by placing braces across them to hold their sides in place. This often involves sheathing the sides with planks, called "sheeting" when so used, supported by longitudinal timbers called "rangers" or "wales," which are held by "cross-braces." Sometimes the sheeting is used vertically, sometimes it is placed horizontally and is then often called "box sheeting," and sometimes special forms of shoring are employed which will be described later.

Behavior of Different Materials in Excavation.—Little or no shoring may be required in trenches in hardpan. It usually caves in large masses so that an occasional brace is enough to hold it.

While new clay banks stand fairly well for a time after excavation, this material is liable to swell and in many cases water will gradually seep toward the trench, which makes it such a treacherous material that it is desirable to provide close sheeting and heavy timbers.

In dry alluvium sheeting will generally be required, although the banks usually stand well for a time. Occasionally it may be possible to leave spaces between the planks, for the material is not likely to ravel.

Close sheeting is almost always required in trenches in gravel because of the tendency of this material to ravel and roll through small openings. It is usually necessary to shore trenches in gravel when they are 3 or 4 ft. in depth.

Sand behaves in different ways, depending upon its size, uniformity and the moisture in it. Close sheeting is generally necessary and should be placed as soon as the trench is down 3 to 5 ft. Where the sand is fine and contains considerable water, high pressures are likely to be exerted against the shoring, approaching and perhaps exceeding a corresponding hydrostatic pressure. In such material the sheeting must be close, strongly braced and driven a considerable distance below the bottom of the trench to help hold the planks in their proper position below the lowest ranger as well as to prevent the sand in the trench from being forced up by the pressure of the soil outside the sheeting.

The intelligent handling of quicksand is a highly special problem. The following definition of quicksand was given[1] by Col. Charles R. Gow, a contractor of wide experience in building foundations and the handling of difficult work:

[1] "Quicksand, Its Nature, Behavior and Methods of Control," *Proc.* N. E. Water Works Assoc., vol xxxiv, 171; September, 1920. Fig. 114 has been reproduced from this paper by courtesy of the New England Water Works Association.

"Any material of a granular nature may become a quicksand if there is an upward movement of ground water through it, of sufficient velocity to lift and carry the individual particles. On the other hand, no such material will become 'quick' in character unless there is such a flow of water."

Another definition of quicksand was given by Allen Hazen, in a discussion of the subject before the American Society of Civil Engineers, as follows:

"Sand containing for the time more water than would normally be contained in its voids, and, therefore, with its grains held a little distance apart so that they flow upon each other readily."

Fig. 114, rearranged from diagrams in the paper by Col. Gow just quoted, illustrates methods of dealing with quicksand.

While trenches in rock may sometimes be excavated safely without shoring, it should be kept in mind that rock is treacherous material if seamy and the seams lie at an angle exceeding 30 deg. with the horizontal. This is particularly true when the seams contain clay or other material on which the rock will slide easily. It is often difficult to determine how far back from the face of the trench the rock has been cracked in blasting, and cracks are likely to permit large masses to slide into an unshored trench. When the banks are very treacherous it may be wise to provide close sheeting, but usually where the trenches are of moderate depth frequent stay-braces or skeleton sheeting, described later, will be sufficient. Blasting makes shoring troublesome to maintain, and good judgment is therefore necessary in directing the blasting.

The methods ordinarily used for calculating earth pressures disregard the cohesion of the particles of soil. It is this property which makes it possible to excavate a trench to a considerable depth without shoring, and the constructing engineer should study it well. A given soil will often show different cohesive qualities under different pressures. These qualities will increase with the increase in depth of the excavation, but an increase in the water content of the soil is likely to be encountered as the trench is carried down which will tend to reduce the cohesion. These assumptions should not be carried too far, however, as soils are rarely of uniform character from the top to the bottom of the trench.

The adhesive qualities of soil are usually small. If a trench has at some previous time been excavated parallel and near to a new trench, the soil, if allowed to yield, will usually break away to the line of the first trench, showing the lack of adhesion

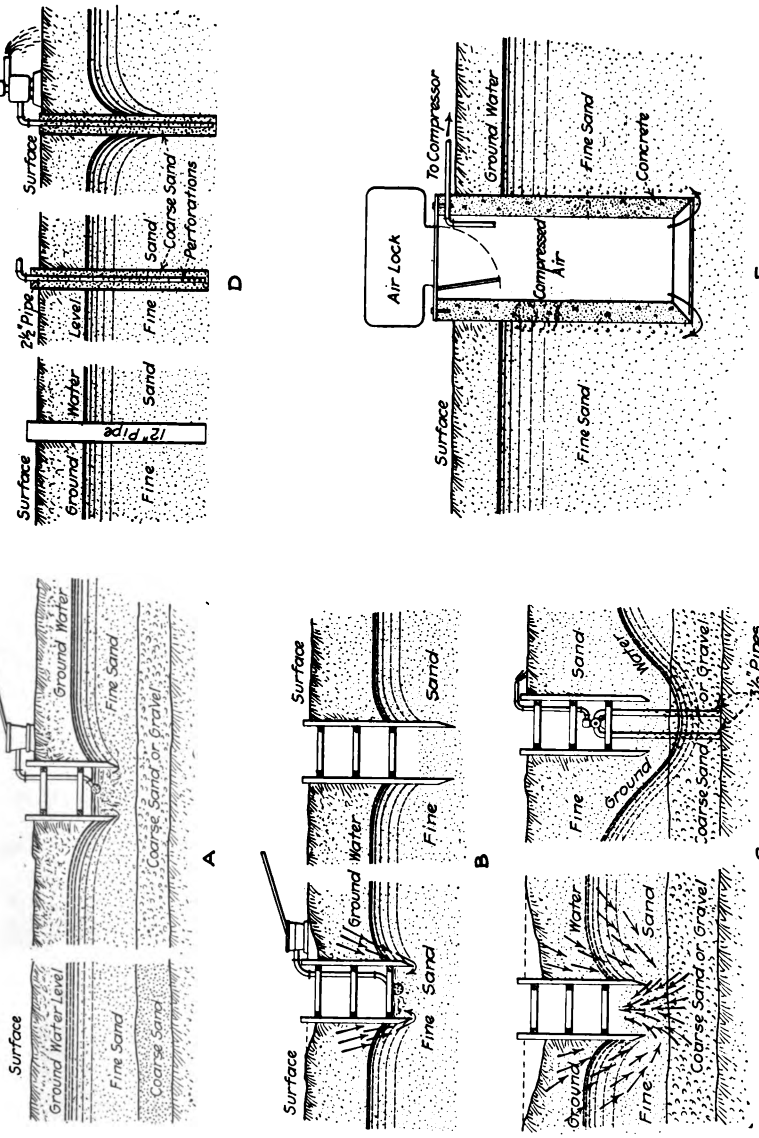


FIG. 114.—Methods of dealing with quicksand.

between the original soil and the refilled material. It is often necessary to sheet trenches solely because at some previous time the soil near them has been excavated, the cohesion broken and no adhesion established since then. For this reason the bracing and shoring of trenches in virgin soil is often much simpler than the work needed for similar trenches in city streets that have been excavated frequently.

The time element is important in shoring. Usually the sides of an excavation will be found to exert no marked lateral pressure when first cut. Gradually, sometimes quite rapidly, the soil changes in character, ravels or flakes, and cracks appear in the surface. Sudden and great changes may be caused by rains. Leakage from water mains may soften the material. Voids behind the sheeting may cause a gradual loosening of the soil under the vibration of passing vehicles or when heavily loaded with excavated material. Movements of the earth due to any of these causes result in changed and often greatly increased pressures on the sheeting, and may also change the point of application of the resultant earth pressure upon it.

Earth Pressures.—Earth pressures are usually calculated by Rankine's formula, which takes no account of cohesion, generally an important property of most soils. Consequently in applying the pressures calculated by this formula to practical problems they are usually divided by 2, 3 or 4; according to the engineer's judgment. The formula is:

$$P = \frac{wh^2}{2} \cos\theta \times \frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}$$

Notation:

w = weight of a unit volume of earth.

P = resultant pressure of the mass of earth against a vertical surface.

H = horizontal component of P.

ϕ = angle of repose of a particular earth.

θ = angle which the surface of the ground makes with the horizontal.

h = depth of any point on the vertical surface resisting the earth pressure.

p = intensity of the horizontal pressure at any depth, h, below the surface.

In evolving this formula it is assumed that the plane against which the lateral pressure is exerted, or the sheeting, is vertical and that the resultant earth pressure, P, is parallel to the surface of the ground.

Ordinarily in cases where the earth is sloped away and up from the vertical plane it may be assumed that the angle θ between the surface of the ground and the horizontal plane is equal to the angle of repose ϕ, in which case the formula becomes,

$$P = \frac{wh^2}{2} \cos \phi$$

If there is no surcharge and the surface is horizontal θ will equal zero, and the formula becomes,

$$^1P = H = \frac{wh^2}{2} \times \frac{1 - \sin \phi}{1 + \sin \phi}$$

In all cases if it is assumed that the direction of the resultant pressure P is parallel to the surface of the ground

$$H = P \cos \theta$$

If the angle of repose is reduced to zero, as in the case of a liquid, the formula becomes,

$$P = H = \frac{wh^2}{2}$$

In all cases P varies as the square of the height of the bank, as in the case of a liquid, and it has therefore been assumed by most engineers that the resultant earth pressure, P, is applied to the sheeting at a point one-third the height from the bottom. The intensity of pressure at any point may be found by the formula,

$$p = \frac{2P}{h}$$

It is generally assumed in calculations of this kind that the angle of repose is 30 deg. The formula then becomes,

$$P = H = \frac{wh^2}{6}$$

and

$$p = \frac{wh}{3}$$

These last formulas are simple and for practical purposes are all that need be used to obtain a general idea of the forces which sheeting and bracing are likely to have to withstand.

If an additional load is placed upon the banks, other than that due to earth cast out of the trench which, if sloped back at an angle equal to the angle of repose, or say 30 deg., can be cared

[1] Values of the last factor are given in Table 27.

for by the formula, the weight of such additional material should be calculated and reduced to an equivalent depth of superimposed earth, thus correspondingly increasing *H*, in which case the surface should be assumed to be horizontal. Table 28 will be useful in solving problems. Due consideration should also be given to the possibility of vibrations caused by passing vehicles affecting the pressures.

TABLE 27.—ANGLE OF REPOSE AND WEIGHT OF DIFFERENT KINDS OF EARTH

Kind of earth	Angle of repose		$\frac{1 - \sin\phi}{1 + \sin\phi}$	Weight, lb. per cu. ft.
	ϕ	Slope[1]		
Alluvium	18°	3 to 1	0.53	90
Clay, dry	26°	2 to 1	0.39	110
Clay, damp	45°	1 to 1	0.17	120
Clay, wet	15°	3.2 to 1	0.59	130
Gravel, coarse	30°	1.7 to 1	0.33	110
Gravel, graded sizes	40°	1.2 to 1	0.22	120
Loam, dry	40°	1.2 to 1	0.22	80
Loam, moist	45°	1 to 1	0.17	90
Loam, saturated	30°	1.7 to 1	0.33	110
Sand, dry	35°	1.4 to 1	0.27	100
Sand, moist	40°	1.2 to 1	0.22	110
Sand, saturated	30°	1.7 to 1	0.33	120

[1] Ratio of horizontal to vertical dimension.

Occasionally water will accumulate back of sheeting so that it may influence the pressure, but no definite information about such pressures has been obtained. It is known from repeated observations made in trenches in very wet, fine sand, which runs freely through even minute cracks in the sheeting, that if this sheeting remains in place the running sand rarely rises more than a few feet above the bottom of the trench. This indicates that the material is not free to act like a fluid, else the trench would become filled with it. Consequently it is generally considered that the maximum pressure which sheeting will be called upon to withstand is hydrostatic pressure.

The angle of repose of many different earths has been determined, but the reported values are of only general applicability because of the absence of any method of describing the characteristics of earth so definitely that exactly what material is meant by a description is clearly known. Rough average values are given in Table 27, taken from Baker's "Treatise on Masonry Construction," supplemented with a column of factors for use

TABLE 28. —INTENSITY OF HORIZONTAL PRESSURE AND TOTAL HORIZONTAL PRESSURE OF EARTH AT VARIOUS DEPTHS

(w = 100 pounds per cubic foot; ϕ = 30°. H and P are per foot of trench)

Horizontal surface			Surcharges: θ = 30°		
Depth h ft.	$p = \frac{1}{3} wh$ pounds	$H = P = \frac{1}{6} wh^2$ pounds	Depth h ft.	$p \cos \theta = \frac{3}{4} wh$ pounds	$H = P \cos \theta = \frac{3}{8} wh^2$ pounds
1	33	17	1	75	38
2	67	67	2	150	150
3	100	150	3	225	338
4	133	267	4	300	600
5	167	417	5	375	938
6	200	600	6	450	1,350
7	233	817	7	525	1,838
8	267	1,067	8	600	2,400
9	300	1,350	9	675	3,038
10	333	1,667	10	750	3,750
11	367	2,017	11	825	4,538
12	400	2,400	12	900	5,400
13	433	2,817	13	975	6,338
14	467	3,267	14	1,050	7,350
15	500	3,750	15	1,125	8,438
16	533	4,267	16	1,200	9,600
17	567	4,817	17	1,275	10,838
18	600	5,400	18	1,350	12,150
19	633	6,017	19	1,425	13,538
20	667	6,667	20	1,500	15,000
21	700	7,350	21	1,575	16,538
22	733	8,067	22	1,650	18,150
23	767	8,817	23	1,725	19,838
24	800	9,600	24	1,800	21,600
25	833	10,417	25	1,875	23,438
26	867	11,267	26	1,950	25,350
27	900	12,150	27	2,025	27,338
28	933	13,067	28	2,100	29,400
29	967	14,017	29	2,175	31,538
30	1,000	15,000	30	2,250	33,750

in the Rankine formula for pressures when the ground surface is horizontal.

Vertical Sheeting.—Vertical sheeting, although usually more expensive where other methods can be adopted with equal safety, is the type most commonly used on sewer work and particularly on deep and difficult work, and in trenches in dry running sands, gravels, and quicksand. Where trenches are excavated under conditions making it important to prevent settlement of pipes and other underground structures, pavements, and abutting buildings, it is often unwise to attempt to use other methods of shoring.

It is desirable to excavate the trench as deep as possible without endangering the banks before placing the sheeting. When the trench is excavated to this depth the first set of rangers (which should usually be about 1 ft. below the surface of the ground) and, if possible, the second set, are put in position against planks placed vertically against the properly trimmed banks at each end and in the center of each ranger. The end planks should be so placed as to come flush with the ends of the rangers. Having placed the rangers in position and supported them temporarily, the cross-braces are driven to hold the two opposite rangers in place. Braces should be cut so as to require reasonably hard driving and to set the planks back slightly into the bank, as otherwise they may become loose and ineffective. The banks should then be trimmed plumb and to a line, which will require the rest of the sheeting to be driven lightly, to get it down between the banks and the rangers to the bottom of the excavation thus far made. Some sewer builders maintain that it is better to hold the top and second sets of rangers temporarily in place until all the sheeting planks have been placed behind them against the bank, and then drive the braces, thus securing a somewhat tighter hold on the banks than where the planks are slipped in behind the rangers after the latter have been finally braced. The former method will generally prove more convenient and will give equally good results if care is taken to so trim the banks that each plank will have a driving fit.

The second set of rangers should be placed as soon as the first set has been secured in position, or better, if conditions will warrant, at the same time that the first set is placed. The vertical distance between the several sets of rangers will depend to some extent upon the local conditions and the character of material, but generally, upon work of moderate size, they should be spaced about 4 ft. on centers. The rangers should be directly over one another, thus allowing the sheeting to be driven plumb. After the rangers are securely braced, lap blocks or cleats made of 2-in. lumber the width of the braces and about 18 in. in length, should be placed on top of and across the joints between the ends of the braces and the rangers and securely nailed to the braces to prevent them from falling if they become loosened. In some cases it may be convenient to attach the cleats to the braces before the latter are driven, thus supporting them while they are being placed between the rangers and driven into their final position. The rangers should generally be placed level, although in side-hill work where the excavation is light and requires the use of only one or two sets of rangers, it may be advantageous to place them parallel to the surface of the street. This should never be done upon deep excavations or where the trenches are in treacherous material.

Rangers will vary in length according to the character of the work being done and particularly the class of machinery used upon the excavation. Common practice, especially in the eastern part of the country, is to make the rangers 16 ft. long, thus fixing the spacing of the braces at a trifle less than 8 ft. on centers, which allows for the convenient use of certain types of trench machines. Upon heavy work it may be advisable to use somewhat longer rangers, thus giving more room between braces for the operation of excavating machinery.

It will generally be found advisable to use three braces upon each set of rangers, one at the center and one at each end, although some practical sewer builders use but a single brace to hold the ends of two abutting sets of rangers, a 2-in. block being used at the end of the brace to cover the joint between the two rangers. By the former method each section of trench will act independently of the adjacent sections, whereas if the end braces engage upon the abutting sets of rangers, a slipping of one set is likely to affect the next set. Where two braces are used at the joints between rangers, some saving in lumber may be effected by making them of somewhat thinner stock than the center braces, as for example, by using 4 by 6-in. braces at the ends and 6 by 6-in. braces in the center.

After the sheeting has been placed in a section and tapped down to the bottom of the excavation, the driving is commenced.

One man, sometimes called the "plank lower," should always be stationed in the trench to dig out for and guide each plank as it is being driven. He casts the dirt to the center of the trench and where the material is such as to permit, digs slightly in advance of the plank. After the plank has been driven as far as possible (usually in good digging about 2 ft. below the normal depth of the trench before driving began) it is left and the next one is treated in a similar manner.

As at the beginning of the driving the sheeting usually stands several feet above the ground, platforms constructed of planks loosely laid upon wooden horses are provided at a suitable height upon which the plank drivers may stand and work to advantage. These horses may be conveniently made about 6 ft. high with horizontal cleats about 18 in. apart, so that the platform may be lowered from cleat to cleat as the planks are driven.

In trenches from 3 to 8 ft. in width, it will often be found advantageous to alternate the excavation and the placing and driving of the sheeting between contiguous sections of the trench, as the room in the trench in which the two classes of workmen must carry on their labors is somewhat restricted. This method of procedure involves the excavating of two sections of trench at the same time, thus increasing the length of open trench, which in some cases may be impracticable. On the other hand it often happens, especially in trenches in loose gravel and running sand, that it is necessary to drive planks short distances as the excavation progresses, in which case the driving must be done in connection with the excavation.

The position of the bottom of the sheeting relative to the excavation depends largely upon the character of the material being excavated. If the trench is in hardpan or alluvium, the excavation can often be carried considerably below the sheeting without endangering the stability of the banks. In sand and gravel it is necessary to drive the sheeting down at least as low as the bottom of the trench, and in many cases several inches below. In such cases it is always well to have the "plank lower" dig the earth away from the ends of the planks as they are driven, thus making it possible to keep the sheeting at all times somewhat below the general elevation of the bottom of the trench. In wide trenches, where such material is fairly stable, it may be possible to carry the excavation in the central part of the trench from 1 to 2 ft. below the sheeting, care being taken not to dig too close to the sheeting. This aids the "plank lower" in his work of digging the earth away from the planks as they are being driven.

The general method of shoring by means of vertical sheeting, is illustrated by Figs. 115 and 116, which are companion drawings

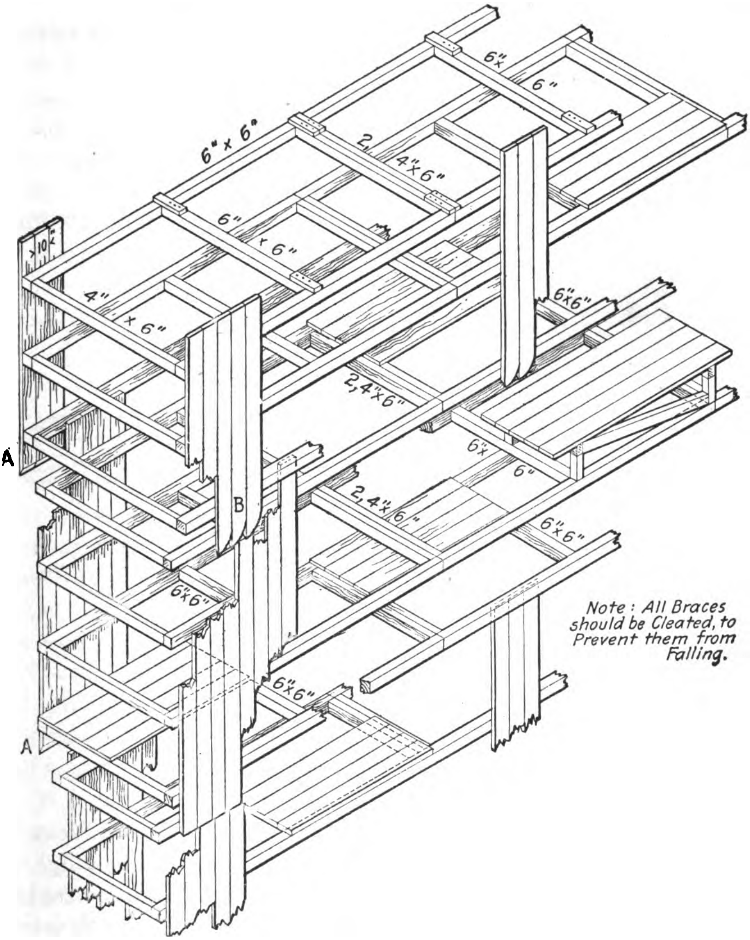


FIG. 115.—Isometric sketch of typical trench sheeting.

of a narrow trench carried to such depth as to require three sets of vertical sheeting. The braces are shown cleated, which is good practice upon work of this kind. Staging platforms are shown extending half way and all the way across the trench, as both methods are commonly used.

The most common practice is to use sheeting planks sharpened with "chisel" edges, placed with the beveled side toward the inside of the trench as shown at *A*. In some cases, especially in difficult work where it is necessary to drive the sheeting long distances ahead of the excavation, the planks are beveled in two directions, as shown at *B*, the object being that the planks shall drive straight and that they shall tend to hold tightly against the planks previously driven.

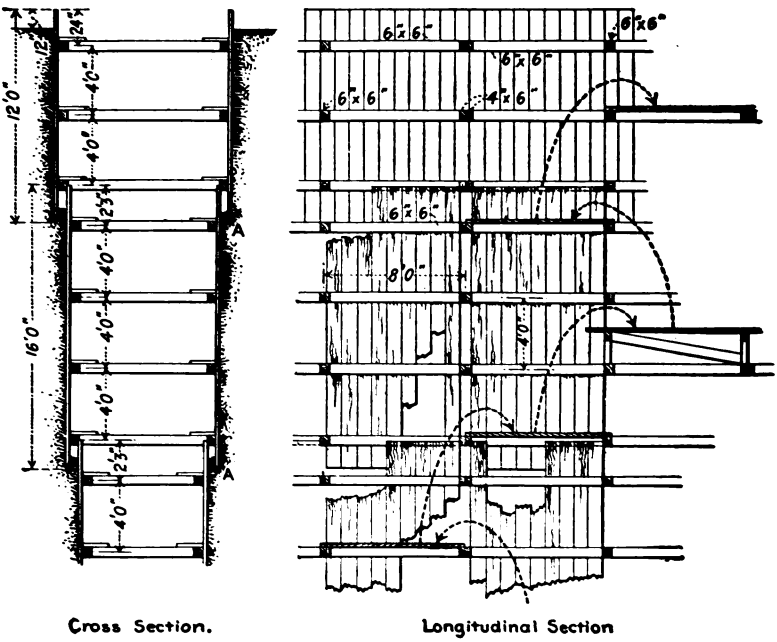


FIG. 116.—Typical trench sheeting.

In practice the rangers are rarely spaced with exactness or the planks driven to a uniform depth. In many cases lighter rangers are used, as 4 by 6-in. timbers, and occasionally where the excavation is not to be deep, 2-in. planks are used for rangers. Upon extensive work, however, where the depth of excavation will exceed 10 ft. or where the material to be excavated is such as to make it necessary to shore the banks securely, it is wise to use rangers at least as heavy as 4 by 6-in. timbers, and heavier stock as may appear to be necessary from the local conditions encountered.

As the upper portion of the trench, supported by the first set of sheeting, is the widest, it is wise to make the first set of sheeting the shortest, thus effecting a saving in excavation. Furthermore, the use of relatively short sheeting for the first section avoids the necessity of using horses to any great extent, in connection with the driving of the planks. Stages can be placed upon the rangers in the trench, from which the second and third sets of sheeting can be conveniently driven, even though they are relatively long.

In many cases the length of sheeting adopted for the several sets will be governed by the stock of lumber on hand when the work is begun. If, however, new lumber is to be purchased, care should be taken to order only those sizes which, by avoiding waste, will be economical. While very long sheeting is sometimes used, it is common practice to limit the lengths even of the longer sets to 16-ft. plank. Braces should ordinarily be of the same depth as the rangers.

After the first set of sheeting has been driven to the full depth to which it is intended to extend, guard rails made of 2 by 6-in. planks are spiked to the lowest set of braces at a distance from the rangers approximately equivalent to the thickness of the sheeting. The second set of sheeting is then passed through the space between the guard rail and the adjacent ranger and placed in its proper position ready for driving. If the sheeting is more than 10 ft. long, it may be well to place a second guard rail on the second set of braces above the first guard rail. This will hold the upper ends of the planks rigidly and prevent their springing, so that they will be more readily driven than if held simply by the guard rail near the bottom. Additional sets of sheeting should be started in the same manner.

It is not possible to place and drive planks in a second set of sheeting immediately under the braces of the first set. Therefore, as the excavation is carried down, a space equivalent to the width of the braces above will be left unshored. This space, sometimes called a "window," is frequently closed by placing short horizontal planks behind the adjacent vertical planks. In cases where the material is likely to run, if exposed for a short vertical distance, notched planks are used. These "notches" are usually made of 12-in. planks by cutting out about 6 in. from the upper end of the plank, leaving the lower portion, about 4 ft. in length, the full width of 12 in. These notches are placed under the braces and are driven like the other planks. As they are driven a "window" opens below the braces which must be closed from time to time with horizontal planks set in behind the notches or vertical poling boards fastened to them. Notched planks must be both "rights" and "lefts" so they may be fitted under the double braces from both sides.

Box Sheeting.—Horizontal or "box" sheeting is sometimes used where the material excavated is such that the banks will stand untimbered for a considerable length of time without great danger of their caving. With it less care need be taken in trimming the banks of the trench plumb than in the case of vertical sheeting, as it is not absolutely necessary, though it is preferable, that each plank be placed directly under the one above. After the excavation has been carried as deep as practicable without bracing (usually the width of three or four planks), the planks are placed along the sides of the trench and held in position by 2-in. cleats placed vertically at each end and usually in the center of the planks. These cleats are in turn held firmly by cross-braces. As the width of the trench between the opposite planks is likely to vary, extension braces are to be preferred to the ordinary driven brace, as they are more readily adjusted and by their use the trench may be more quickly braced. The extension braces are also preferable, as there is little or no jar in connection with placing them, while if driven braces are used the jar resulting from the driving may be sufficient to endanger the banks.

Box sheeting has been used in some cases where the material excavated was very unstable, such as coarse gravel and dry sand. In such cases the excavation can be carried only deep enough to accommodate a single plank at a time. Each set of planks is temporarily braced until the trench has been carried deep enough to accommodate uprights supporting three or four planks, after which the uprights, with one or two braces against each pair, may be substituted for the individual plank braces. In this class of material box sheeting may be more expensive and troublesome than vertical sheeting, but it is quite useful under certain difficult conditions, as for example in trenches between car tracks or under structures which interfere with the placing of vertical sheeting. The use of the wooden horses and temporary stages required by plank drivers is also avoided by the use of box sheeting, which is often advantageous, especially in narrow streets.

Sometimes the first few feet of excavation may be in dry clay, alluvium or other material which will stand fairly well, while beneath this stratum sand or gravel may be encountered. In such cases a combination of horizontal sheeting on top and vertical sheeting below, may be used with success.

Fig. 117 shows a trench about 12 ft. in depth, the banks of which were supported by two sets of box sheeting, each three planks in depth. The planks used were 2 in. thick, 12 in. wide,

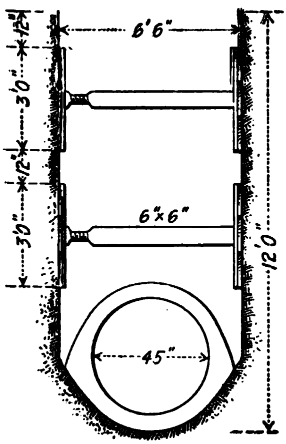


FIG. 117.—Box sheeting for 45-in. sewer.

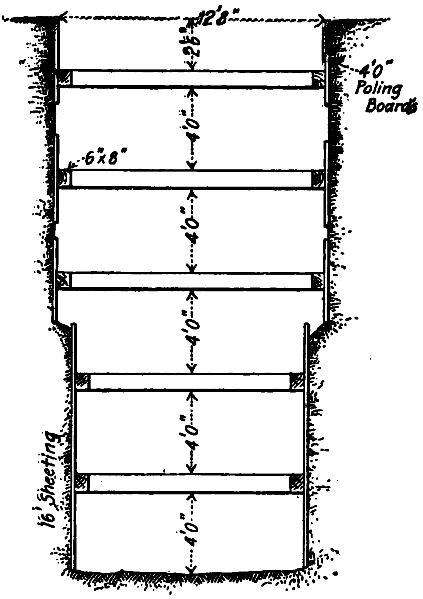


FIG. 118.—Poling boards and vertical sheeting for sewer 7 ft. 6 in. by 6 ft. 8 in.

and 18 ft. in length. The cleats were 2 by 12 in. and 3 ft. in length. Extension braces were used, as shown in the sketch. The material excavated was a dry alluvial clay.

Poling Boards.—This method of timbering is similar to that of vertical sheeting, except that short planks 3 or 4 ft. in length called "poling boards" are used and the sheeting is placed against the banks from time to time as the excavating proceeds instead of being driven. The planks are placed vertically against the properly trimmed banks, and are supported by means of rangers and braces, as in the case of vertical sheeting. In some cases where the banks stand well it is possible to get along with a single set of rangers and braces for one set of poling boards, as illustrated in Fig. 118. This, however, is not as safe as the method shown in Fig. 119, where two sets of rangers and braces are used for each set of poling boards. The nature of the material in which the trench illustrated by Fig. 118 was excavated was such that a space of from 6 to 12 in. could be safely left unsupported between the bottom of one set of poling boards and the top of the set next below.

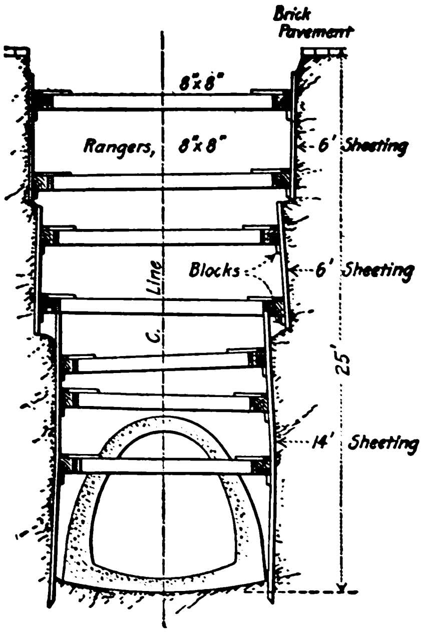


FIG. 119.—Poling boards and vertical sheeting for 7-ft. sewer.

Where this method of sheeting can be used, economy in excavation will result, because the trench will be of the same width from top to bottom, regardless of depth. Thus with a trench sufficiently deep to require three sets of vertical sheeting, a saving of about 3 ft. in width at the top and about 1½ ft. opposite the second set of sheeting, will result from the use of poling boards. Short pieces of plank which accumulate on sewer work requiring some vertical sheeting can be advantageously used for poling boards, so that a combination of poling boards and vertical sheeting often proves economical. The use of poling boards has the same advantages as the use of box sheeting, in that they do not project above the surface of the ground at any time.

The chief disadvantage in the use of poling boards lies in the fact that if there is a collapse in the timbering of any portion of the excavated section, resulting from the running out of the material behind the sheeting, serious deformation or even collapse of the entire timbering of the section may result. The danger of this is much greater with poling boards than with vertical sheeting.

Stay Bracing.—When the excavation is in hardpan, dry clay or dry loam, it is often unnecessary to provide close sheeting, although some support for the banks may be needed. In such cases stay bracing, or skeleton sheeting, will prove economical. The simplest form of stay bracing involves the placing of pairs of vertical planks braced tightly against the banks of the trench at intervals of 6 to 10 ft. The braces may conveniently serve as supports for staging platforms in trenches which are sufficiently deep to require that the material be staged out. If a trench must be left open for several days, the use of stay bracing is not to be

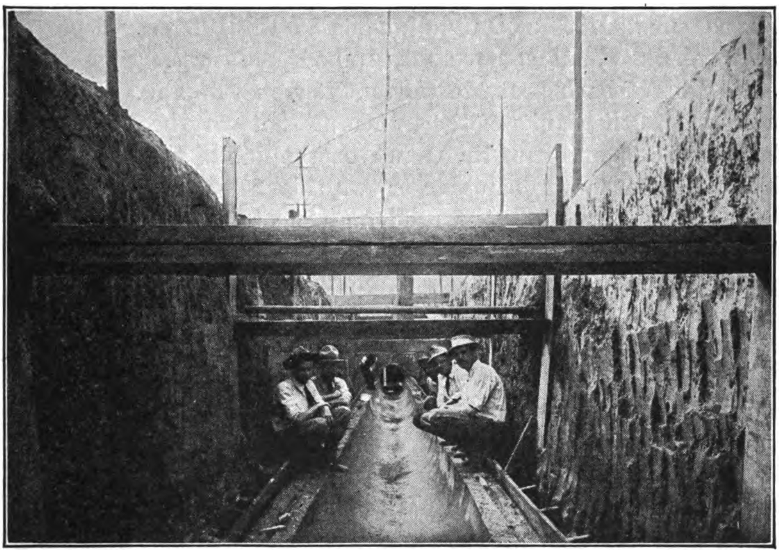


FIG. 120.—Trench in alluvium; stay bracing.

recommended unless conditions are especially favorable, as heavy rains are likely to soften the banks and cause expensive and dangerous caving.

Fig. 120 clearly shows the method of supporting the banks of a trench by means of stay braces. In this case, the pairs of planks were supported by single braces, some of which were 6 by 6-in. timbers, and others consisted of two 2 by 6-in. planks.

Skeleton Sheeting.—In some cases where it is difficult to determine prior to the completion of the excavation, whether or not it will be necessary to use vertical sheeting, skeleton sheeting is used. This consists of the usual rangers and braces with planks between the rangers and the banks of the trench opposite the braces. By putting in these frames, it is possible to slip planks in later without delay wherever found necessary. It is often desirable to place such planks at intervals of 2 or 3 ft. where it is unnecessary to provide close sheeting.

Bracing Curves.—When a sewer is to be constructed around a curve, the trench is generally laid out on chords of a length dependent upon the degree of curvature. The ends of the rangers are cut on radial lines, thus making the outer rangers longer than the inner rangers. It is desirable to have the braces nearly at right angles to the rangers but where the curve is very sharp this is not always practicable. Whenever the braces are not placed substantially at right angles to the rangers, cleats or short planks should be securely spiked against the rangers on both sides of each brace to prevent it from slipping. In some cases where the banks of the trench are not very heavy, it may be possible to excavate the trench on the line of the curve and, instead of heavy timbers, one or two thicknesses of 2-in. planks bent to the curve may be used for rangers. In such cases it will usually be helpful to scarf with a saw the inner side of each plank at intervals of 4 to 8 in. to a depth of from 1/4 to 1/2 in. to aid it in taking the curve without cracking or breaking, the inner side referred to being the side of shorter radius of curvature.

Jetting.—Where sheeting must be driven much ahead of excavation, it is occasionally found impossible to make substantial progress in the driving of sheeting by ordinary methods. In such cases a jet of water may be forced into the soil just below the bottom of the plank so that as the soil becomes softened the plank can be driven with comparative ease.

In some cases the bottom of each plank is fitted with a special appliance by means of which the jet of water is introduced into the soil immediately below the plank. In other cases the water may be conveyed to the lower end of the sheeting by a 3/4- or 1-in. pipe, independent of the sheeting itself, and pushed down by hand. Where the planks are fitted with special jetting shoes, it is economical to drive them a considerable distance below the bottom of the trench at one operation.

Practical Suggestions on Timbering.—Good workmanship is essential to the successful timbering of trenches. If it is desired to practice strict economy, it is wiser to adopt a less expensive method and to do the work in a first class manner than to adopt a more expensive method and resort to careless workmanship.

TABLE 29.—QUANTITIES OF LUMBER REQUIRED FOR SHORING AN ORDINARY NARROW SEWER TRENCH, 16 FT. LONG, AS SHOWN IN FIG. 121, PER LINEAR FOOT OF TRENCH AND PER SQUARE FOOT OF BANK SURFACE

(In Feet Board Measure)

Width of trench at bottom (outside of sheeting), feet	Size of braces and rangers, inches	Depth of trench																	
		8 Ft.						12 Ft.						16 Ft. (1 set)					
		6 braces	4 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank	9 braces	6 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank	12 braces	8 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank
4	4×6	36.00	128.00	512.00	676.00	42.25	2.64	54.00	192.00	768.00	1014.00	63.37	2.64	......	......	......	......	......	....
4	6×6[1]	37.33	192.00	512.00	741.33	46.33	2.89	56.07	288.00	768.00	1112.07	69.50	2.89	74.67	384.00	1024.00	1482.67	92.66	2.89
8	6×6[2]	93.33	192.00	512.00	797.33	49.83	3.12	140.00	288.00	768.00	1196.00	74.75	3.12	186.67	384.00	1024.00	1594.67	99.67	3.12
12	8×8[3]	275.55	341.33	512.00	1128.88	70.55	4.41	413.33	512.00	768.00	1693.33	105.83	4.41	551.11	682.67	1024.00	2257.78	141.11	4.41
		16 Ft. (two sets)						18 Ft.						20 Ft.					
		12 braces	8 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank	15 braces	10 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank	15 braces	10 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank
4	6×6[1]	93.33	384.00	1152.00	1629.33	101.83	3.18	112.00	480.00	1280.00	1872.00	117.00	3.25	112.00	480.00	1408.00	2000.00	125.00	3.12
8	6×6[2]	205.33	384.00	1152.00	1741.33	108.83	3.40	252.07	480.00	1280.00	2012.07	125.75	3.49	252.07	480.00	1408.00	2140.07	133.75	3.34
12	8×8[3]	595.55	682.67	1152.00	2430.22	151.89	4.74	733.33	853.33	1280.00	2866.66	179.16	4.97	733.33	853.33	1408.00	2994.66	187.16	4.68
		22 Ft.						24 Ft.											
		18 braces	12 rangers	2 in. plank	Total, 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank	18 braces	12 rangers	2 in. plank	Total 16 ft. trench	Per linear ft. of trench	Per sq. ft. of bank						
4	6×6[1]	140.00	576.00	1536.00	2252.00	140.75	3.20	140.00	576.00	1664.00	2380.00	148.75	3.10						
8	6×6[2]	308.00	576.00	1536.00	2420.00	151.25	3.44	308.00	576.00	1664.00	2548.00	159.25	3.32						
12	8×8[3]	893.33	1024.00	1536.00	3453.33	215.82	4.90	893.33	1024.00	1664.00	3581.33	223.81	4.66						

[1] All end braces 4 in. × 6 in., middle braces same as rangers.

[2] All end braces 4 in. × 6 in., center braces 6 in. × 6 in.

Note. The sizes of rangers and braces must be increased for very heavy banks.

[3] All end braces 6 in. × 8 in., center braces 8 in. × 8 in.

The banks should be carefully trimmed to a plane surface before placing the sheeting. As the trench is deepened the

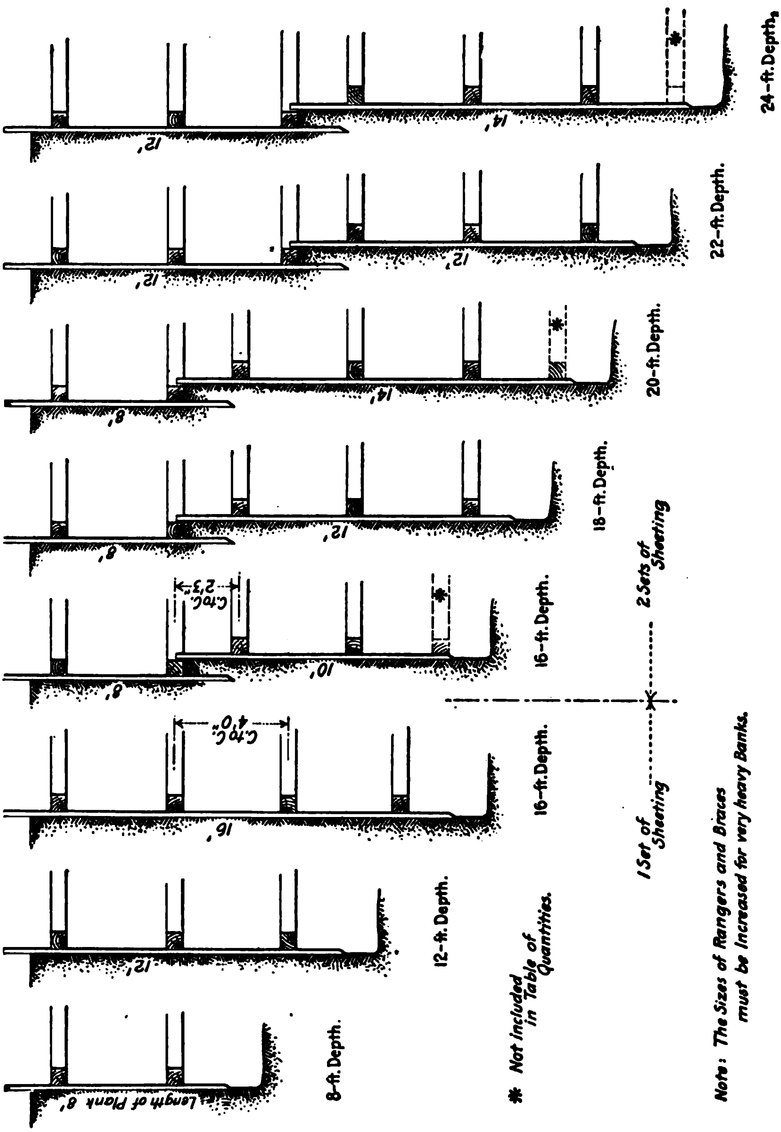


FIG. 121.—A method of shoring ordinary narrow sewer trenches.

material of the bank should not be permitted to run out or ravel or be trimmed back to allow the sheeting, when driven down, to slip over it without friction, inasmuch as this practice, particularly in running sand or gravel, is likely to lead the workmen to carelessness in allowing the formation of pockets behind the sheeting. Such pockets are undesirable not only because they may result in serious settlement of the ground and increase in pressure upon the bracing, but because they are likely to result in deformation of the finished sewer by spreading of the arch when the trench is backfilled.

In excavating a trench in loose material, hay or meadow grass will be found useful to prevent the material from running through the cracks in the sheeting.

The banks of a trench should never be allowed to move, if it is possible to prevent movement. The sheeting and bracing should be placed in the trench as soon as practicable, in order to prevent any movement of the top layers of the soil. The sheeting should be driven as fast as the excavation proceeds and in quicksand it is, of course, necessary to drive the sheeting in advance of the excavation. Great care should be taken to prevent the banks from raveling where notches or "windows" are required. The pressures exerted upon timbering increase greatly as the soil back of the sheeting becomes cracked, loosened, or in any way shifts its position. Such movements of the banks are likely to cause the breakage of adjacent water pipes and sewers, resulting in the partial or complete saturation of the soil with the water leaking from them, and producing pressures approaching hydrostatic pressure.

Braces should always be tight. Where they are short and of wood they may be cut to correct length, and driven, but where they are long they should be tightened by hard wood wedges driven to refusal. Upon the lighter work extensible screw braces will be found more convenient than wooden braces.

Except upon very heavy and expensive work it will generally be found more convenient to use rangers and braces of the same size from the top to the bottom of the excavation, than to attempt to vary their size in accordance with theory. If, as is frequently the case, it seems desirable to increase the strength of the timbering toward the bottom of the excavation, or at any other point, this can be done by decreasing the space between the rangers, and adding braces.

Trenches should always be braced in such a manner as to give as much room as is possible for working between the rangers and between the braces. For this reason, as well as to reduce the quantity of excavation, it is common practice to use rectangular rangers on edge or with the long side vertical. While this is not the most economical method of utilizing the strength of the timber, it does save room within the trench and makes it possible to construct the trench slightly narrower than if the rangers were turned so that the long side would be horizontal. Moreover, this method gives greater bearing area for the braces, which may be required as the transverse crushing strength may control the section.

It is generally desirable to use two braces at the contiguous ends of two sets of rangers, so that each set may act independently of the other and a settlement of one section of bracing will not affect the adjacent set. Center braces should generally be of the same size as the rangers. End braces should be of the same depth as the rangers but may generally be somewhat narrower, because the loads put upon them are smaller than those put upon the center braces.

All braces should be securely cleated to the adjacent rangers so that they will not fall to the bottom of the trench in case the banks yield and they become loosened.

Lumber and Tools.—Ordinarily the first cost of timber is an important if not the chief consideration in its selection. If sheeting requires hard driving, it is advisable to get either a soft, tough wood or a hard, strong wood. Spruce is very satisfactory and oak stands driving fairly well. Hemlock is usually unsatisfactory because it splinters. Oak is heavy and likely to warp. Sheeting planks should not be more than 10 in. wide, except notched planks, nor less than 6 in. It requires about as much labor to drive either a 4-in. or an 8-in. plank; two 8-in. planks can generally be driven in about the same time as one 12-in. plank. Many sewer builders prefer planks 10 in. wide for sheeting.

Sheeting is generally square edged, sawed stock. Rangers and cross-braces are generally similar stock, although round timbers may be used. Planks for poling boards and box sheeting require no dressing. Rangers should be of uniform length, generally 16 ft., so that the cross-braces may be set directly under each other at each place where they are used. Braces should be beveled along their vertical edges for about 1 in. each way, to facilitate driving and reduce splitting at the ends. The bevel at the bottom of a sheeting plank should be about 6 in. long, with the edge about ½ in. thick. The tops of the sheeting planks should be dressed to receive plank caps, placed over them to protect the ends against brooming under the driving maul. The cast-iron caps should fit loosely on the plank ends or they will be cracked in driving.

On account of the rapidly rising price of lumber, steel sheet piling has come into extensive use for heavy engineering foundation work. It is not as easily handled, driven or pulled as plank, but its useful life is much longer, and hence its economic value has been constantly rising as the price of lumber has increased.

A number of special tools are used in trench work. The sheeting is driven with wooden mauls or beetles, made of elm or

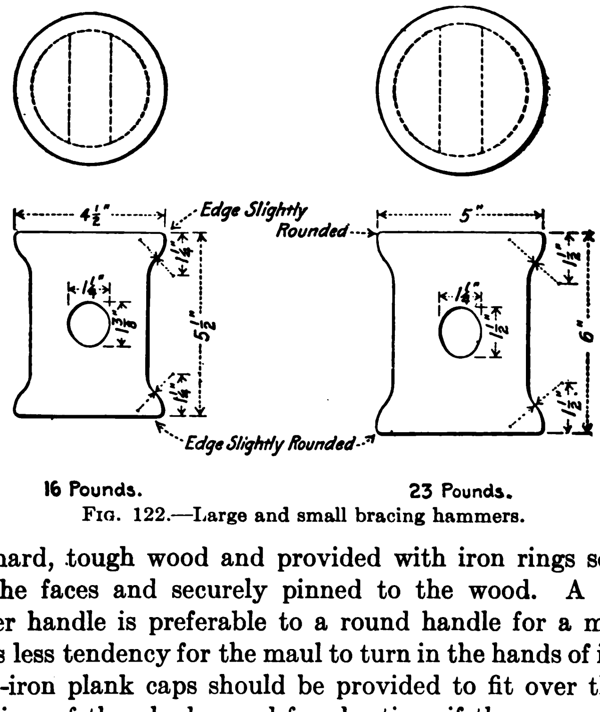


FIG. 122.—Large and small bracing hammers.

other hard, tough wood and provided with iron rings set back from the faces and securely pinned to the wood. A pick or hammer handle is preferable to a round handle for a maul, as there is less tendency for the maul to turn in the hands of its user.

Cast-iron plank caps should be provided to fit over the tops of all sizes of the planks used for sheeting; if they are not used the damage to the planks may be very great.

For driving cross-braces hammers of the type shown in Fig. 122 are convenient. The 16-lb. size is most useful and permits a blow to be delivered close to the end of a brace more effectively than by a sledge hammer. A lighter hammer is undesirable and a heavier one is so clumsy for ordinary work that the 23-lb. size is used only for heavy bracing.

Extension braces, Fig. 123, are often used in light trench work, being more quickly applied than timber struts and requiring no driving, an advantage where banks are unstable. These braces may be provided with lugs on both ends, which overhang the rangers and prevent them from falling. The extension brace is usually fitted with lap-welded iron pipe, and its length

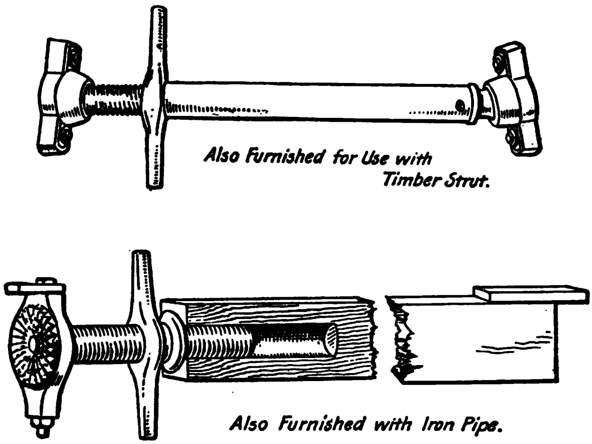


FIG. 123.—Extensible trench braces.

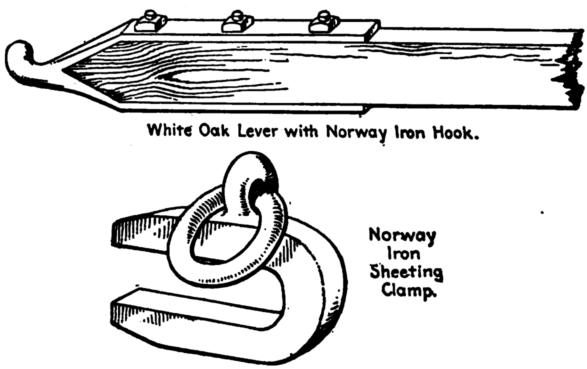


FIG. 124.—Sheeting or plank clamp and lever.

may be varied by substituting different lengths of pipe; sometimes timber braces with screws on one end are used, in which case one end of the strut is bored out to receive the screw.

Sheeting planks are pulled by means of a wrought-iron clamp slipped over them and raised by a long timber lever with a wrought-iron hook at the end, shown in Fig. 124.

CHAPTER VIII

PIPE, BRICK AND CONCRETE SEWERS

For many years engineers in charge of the maintenance of sewerage systems observed that lines of sewer pipe and large brick and concrete sewers sometimes showed signs of cracking only after a long period of service. There has been abundant evidence that in many cases the maximum load on a sewer was not imposed until heavy rains saturated the backfilling of the trench, and possibly also, put the sewer under some internal pressure at the same time. Tests of sewer pipe have shown that when the horizontal diameter of a sewer pipe lengthens about 0.04 in. under vertical loading, the pipe cracks or breaks. It is practically impossible to ram earth around the sides of a pipe so firmly that it will prevent such a small movement. Consequently where the pipe may be exposed to loads approaching a dangerous amount it is necessary either to use pipe of exceptionally high strength, bed it in a concrete cradle, or construct a sewer of some other material. No matter what type of construction is adopted, the design should be worked out, as a rule, without relying on the backfilling for lateral support, because such support will amount to little until the sewer has become so deformed that cracks must appear in it.

The pressures coming upon sewers were measured in experiments by Marston and Anderson, made by weighing the load on different lengths of pipes hung at different depths in trenches, from a system of levers ultimately ending on the platform of scales. Particular care was taken to avoid test conditions likely to cause uncertainty regarding the accuracy of the results, and where there was any question about the tests they were repeated, with or without modification, until uncertainty was eliminated. These tests checked satisfactorily results obtained by the use of the formula $W = CwB^2$, in which W is the total weight on a unit length of pipe, C[1] is a coefficient in which allowance is made for the ratio of the depth to the width of the trench and for the friction of the backfilling against the sides of

[1] Approximate values of C for use in the above formula are given in Am. Sew. Practice, Vol. i, p. 333.

the trench, w is the weight of a unit volume of the backfilling, and B is the width of the trench a little below the top of the sewer. The final practical data, the approximate maximum loads imposed on sewers in trenches by the filling materials above them, are given in Table 30. From their experiments and theoretical studies Marston and Anderson concluded that a 12-in. pipe will have to carry the same load as an 18-in. pipe, if each is

TABLE 30.—APPROXIMATE MAXIMUM LOADS, IN POUNDS PER LINEAR FOOT, ON PIPE IN TRENCHES, IMPOSED BY COMMON FILLING MATERIALS (MARSTON AND ANDERSON)

Depth of fill above pipe	Breadth of ditch at top of pipe									
	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.
	Partly compacted damp top soil; 90 lb. per cubic foot					Saturated top soil; 110 lb. per cubic foot				
2 ft.	130	310	490	670	830	170	380	600	820	1,020
4 ft.	200	530	880	1,230	1,580	260	670	1,090	1,510	1,950
6 ft.	230	690	1,190	1,700	2,230	310	870	1,500	2,140	2,780
8 ft.	250	800	1,430	2,120	2,790	340	1,030	1,830	2,660	3,510
10 ft.	260	880	1,640	2,450	3,290	350	1,150	2,100	3,120	4,150
	Dry sand; 100 lb. per cubic foot					[1]Saturated sand; 120 lb. per cubic foot				
2 ft.	150	340	550	740	930	180	410	650	890	1,110
4 ft.	220	590	970	1,360	1,750	270	710	1,170	1,640	2,100
6 ft.	260	760	1,320	1,890	2,480	310	910	1,590	2,270	2,970
8 ft.	280	890	1,590	2,350	3,100	340	1,070	1,910	2,820	3,720
10 ft.	290	980	1,820	2,720	3,650	350	1,180	2,180	3,260	4,380
12 ft.	300	1,040	2,000	3,050	4,150	360	1,250	2,400	3,650	4,980
14 ft.	300	1,090	2,140	3,320	4,580	360	1,310	2,570	3,990	5,490
16 ft.	300	1,130	2,260	3,550	4,950	360	1,350	2,710	4,260	5,940
18 ft.	300	1,150	2,350	3,740	5,280	360	1,380	2,820	4,490	6,330
20 ft.	300	1,170	2,420	3,920	5,550	360	1,400	2,910	4,700	6,660
22 ft.	300	1,180	2,480	4,060	5,800	360	1,420	2,980	4,880	6,960
24 ft.	300	1,190	2,540	4,180	6,030	360	1,430	3,050	5,010	7,230
26 ft.	300	1,200	2,570	4,290	6,210	360	1,440	3,090	5,150	7,460
28 ft.	300	1,200	2,600	4,370	6,390	360	1,440	3,120	5,240	7,670
30 ft.	300	1,200	2,630	4,450	6,530	360	1,440	3,150	5,340	7,830
	Partly compacted damp yellow clay; 100 lb. per cubic foot					[1]Saturated yellow clay; 130 lb. per cubic foot				
2 ft.	160	350	550	750	930	210	470	730	1,000	1,240
4 ft.	250	620	1,010	1,400	1,800	340	840	1,330	1,870	2,370
6 ft.	300	830	1,400	1,990	2,580	430	1,140	1,900	2,630	3,410
8 ft.	330	990	1,720	2,500	3,250	490	1,380	2,360	3,360	4,400
10 ft.	350	1,110	2,000	2,920	3,880	520	1,570	2,760	3,980	5,270
12 ft.	360	1,200	2,220	3,320	4,450	540	1,730	3,100	4,560	6,050
14 ft.	370	1,280	2,410	3,650	4,950	560	1,850	3,410	5,050	6,760
16 ft.	370	1,330	2,570	3,950	5,400	570	1,940	3,660	5,510	7,440
18 ft.	380	1,380	2,710	4,210	5,810	570	2,020	3,880	5,930	8,060
20 ft.	380	1,410	2,830	4,450	6,180	580	2,090	4,070	6,280	8,610
22 ft.	380	1,430	2,920	4,640	6,500	580	2,140	4,240	6,610	9,130
24 ft.	380	1,450	3,000	4,820	6,800	580	2,180	4,380	6,910	9,590
26 ft.	380	1,470	3,060	4,980	7,080	580	2,210	4,500	7,160	10,010
28 ft	380	1,480	3,120	5,100	7,310	580	2,240	4,610	7,380	10,430
30 ft.	380	1,490	3,170	5,230	7,530	580	2,260	4,700	7,590	10,780

[1] These two sub-tables contain the most important figures for practical use.

placed in the bottom of a 24-in. trench, other things being similar. When it is necessary to use a wide trench and the soil is firm, they recommend stopping the wide trench a few inches above the top of the pipe and then excavating the narrowest trench in which it is practicable to lay the pipe, making special enlargements for the bell, if necessary.

If sheeting is left in the trench but the rangers are removed, the friction between the backfill and the sides of the trench is manifestly decreased and the load on the sewer increased. Marston and Anderson found the increase was about 8 to 15 per cent and confirmed earlier experiments by Barbour. The latter found that if the rangers are left in place the load coming on the sewer is about the same as in unsheeted trenches.

TABLE 31.—PROPORTION OF LONG SUPERFICIAL LOADS ON BACKFILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH AT TOP OF PIPE (MARSTON AND ANDERSON)

Ratio of depth to width	Sand and damp top soil	Saturated top soil	Damp yellow clay	Saturated yellow clay
0.0	1.00	1.00	1.00	1.00
0.5	0.85	0.86	0.88	0.89
1.0	0.72	0.75	0.77	0.80
1.5	0.61	0.64	0.67	0.72
2.0	0.52	0.55	0.59	0.64
2.5	0.44	0.48	0.52	0.57
3.0	0.37	0.41	0.45	0.51
4.0	0.27	0.31	0.35	0.41
5.0	0.19	0.23	0.27	0.33
6.0	0.14	0.17	0.20	0.26
8.0	0.07	0.09	0.12	0.17
10.0	0.04	0.05	0.07	0.11

Sometimes the new backfill carries loads like piles of lumber, brick or concrete materials, or heavy trucks and road rollers pass over it. If the load is a long one, like piles of construction materials, Marston and Anderson found from a theoretical analysis checked by experiments that if this extra load, per running foot of trench, is regarded as unity, the decimal part of it which is transmitted to the pipe in trenches of different dimensions is approximately that given in Table 31. The theoretical analyses of the case of short loads, like those imposed by rollers, could not be checked by experiment but the maximum and minimum values of the proportion of the load reaching the sewer given in Table 32 are believed to be fairly correct.

TABLE 32.—PROPORTION OF SHORT SUPERFICIAL LOADS ON BACKFILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH (MARSTON AND ANDERSON)

Ratio of depth to width	Sand and damp top soil		Saturated Top soil		Damp yellow clay		Saturated yellow clay	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.5	0.77	0.12	0.78	0.13	0.79	0.13	0.81	0.13
1.0	0.59	0.02	0.61	0.02	0.63	0.02	0.66	0.02
1.5	0.46	....	0.48	....	0.51	....	0.54	....
2.0	0.35	....	0.38	....	0.40	....	0.44	....
2.5	0.27	....	0.29	....	0.32	....	0.35	....
3.0	0.21	....	0.23	....	0.25	....	0.29	....
4.0	0.12	....	0.14	....	0.16	....	0.19	....
5.0	0.07	....	0.09	....	0.10	....	0.13	....
6.0	0.04	....	0.05	....	0.06	....	0.08	....
8.0	0.02	....	0.02	....	0.03	....	0.04	....
10.0	0.01	....	0.01	....	0.01	....	0.02	....

PIPE SEWERS

Strength of Pipe.—The theoretical analysis of the strength of sewer pipe is that of thin elastic rings. For all practical purposes three assumed conditions of loading are sufficient to guide the engineer, first, concentrated loads at the top and bottom of the vertical diameter of the pipe; second, uniformly distributed loads above and below the horizontal diameter; and third, uniformly distributed loads on the top quarter and bottom quarter of the circumference of the pipe. Talbot and Marston have shown that the bending moment at the top and bottom of a pipe of diameter d under a concentrated load at the crown of Q is $0.159\,Qd$; under a load uniformly distributed, W, it is $0.0625\,Wd$; under a uniformly distributed load, W, on the top quarter of the circumference with the pipe supported on the bottom quarter of its circumference, it is $0.0845Wd$. The bending moments at the ends of the horizontal diameters under these conditions of loading are $0.091Qd$, $0.0625Wd$ and $0.077Wd$ respectively.

The thickness of the barrel of a pipe to resist pressures applied under the three given conditions is $t = 0.976\sqrt{Qd/f}$ for concentrated vertical loading, $t = 0.612\sqrt{Wd/f}$ for a load uniformly distributed over the top of the pipe, and $t = 0.71\sqrt{Wd/f}$ for a load on the top quadrant and supported only on the bottom quadrant. In these expressions f is the unit stress in the outside fiber, the other letters being used as before.

The modulus of rupture in many lots of sewer pipe tested by Marston at Ames ranged from 910 to 1,940 lb. per square inch for single-strength pipe and from 790 to 1,720 lb. for double-strength pipe. The modulus was about two to three times the tensile strength of the material. The same is true of the results of tests of small cement pipe, in which the modulus of rupture exceeded 1,000 lb. in some cases.

The investigations of the strength of pipe carried out by Talbot showed that the factor of safety of sewers and culverts built of pipe was probably very low, and the experiments of Marston and Anderson showed that the pressures upon sewer and drain pipe were probably greater in many cases than the pipe could reasonably be expected to carry. As a result of this work the state drainage and engineering associations of Iowa have adopted specifications providing that no drain tile or sewer pipe not strengthened by bedding in concrete may be used in a trench where the average bearing strength of the pipe is not equal to 165 per cent of the ordinary maximum load on the pipe, as determined by Table 30, plus the weight of the pipe itself. No sewer pipe may be used having a bearing strength under 1,250 lb. per linear foot for sizes under 15-in., 1,500 lb. for 15- to 20-in. pipe, 1,900 lb. for 21- to 27-in. pipe, and 2,400 lb. for 28- to 36-in. pipe. The adoption of these regulations was due to the many failures of drains and sewers in that state due to cracking. Although similar requirements have not been endorsed generally by engineering associations, it is an undoubted fact that skilful workmanship and careful compliance with the usual specifications regarding the bedding of pipe is necessary to prevent cracking. The apparent rude simplicity of the work misleads the observer who thinks it does not require great care and attention to details.

Vitrified Sewer Pipe.—Vitrified salt-glazed pipe are made from clays and shales, pulverized and screened, and mixed together so as to give a product which will burn well and be free from cracks and surface defects. The prepared clay is placed in the hopper of a press, this hopper being a cylinder about 24 in. in diameter, with the wall drawn in at the bottom to the shape of the outside of the bell of a pipe. A rod is held in the axis of the cylinder by a spider and to its bottom is attached a core die or bell, at the elevation where the wall of the cylinder is drawn in. An annular space as thick as the green shell of the pipe is left in this way in the bottom of the hopper, and the pipe is formed by pressing the clay through this space, the inside of the bell being formed by a mold on the top of a moving platform on which the green pipe is lowered. The thickness of both standard and double-strength pipe from a number of makers is given in Fig. 125.

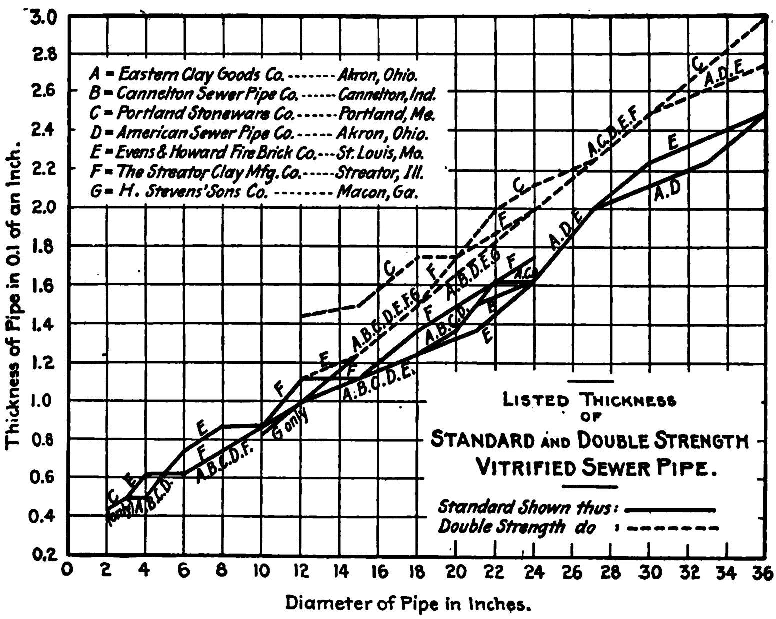


FIG. 125.—Listed thickness of standard and double-strength vitrified sewer pipe (1914).

The pipes are generally seasoned under cover for some time to allow as much water to evaporate as will naturally pass off in this way. They are then placed in a kiln and burned, the burning being a slow process in which most of the defects of pipe appear. These defects may be cracks, or blisters, or warping. The extent to which they are permissible without becoming cause for rejection should be stated in the specifications.

The perfection of manufacture required is best indicated by the specifications[1] for clay sewer pipe prepared by the American Society for Testing Materials[2] and revisions thereof, from which the following has been taken.

[1] *Proc.* A. S. T. M., vol. xix, 1919, pp. 543–568.
[2] *Proc.* A. S. T. M., vol. xx, 1920, pp. 264–265.

Rejection.—Pipes shall be subject to rejection on account of the following:

"(*a*) Variations in any dimension exceeding the permissible variations given in Table 33.

"(*b*) Fracture or cracks passing through the shell or socket, except that a single crack at either end of a pipe not exceeding 2 in. in length or a single fracture in the socket not exceeding 3 in. in width nor 2 in. in length will not be deemed cause for rejection unless these defects exist in more than 5 per cent of the entire shipment or delivery.

TABLE 33.—PERMISSIBLE VARIATIONS IN DIMENSIONS OF CLAY SEWER PIPE

Normal size, inches	Length, inch per foot (−)	Lengths of two opposite sides, inches	Internal diameter, inches		Depth of socket, inches (−)	Thickness of barrel, inches (−)
			Spigot (±)	Socket (±)		
6	1/4	1/8	3/16	1/4	1/4	1/16
8	1/4	1/8	1/4	5/16	1/4	1/16
10	1/4	1/8	1/4	5/16	1/4	1/16
12	1/4	1/8	5/16	3/8	1/4	1/16
15	1/4	1/8	5/16	3/8	1/4	3/32
18	1/4	3/16	3/8	7/16	1/4	3/32
21	1/4	3/16	7/16	1/2	1/4	1/8
24	3/8	1/4	1/2	9/16	1/4	1/8
27	3/8	1/4	5/8	11/16	1/4	1/8
30	3/8	1/4	5/8	11/16	1/4	1/8
33	3/8	3/8	3/4	13/16	1/4	3/16
36	3/8	3/8	3/4	13/16	1/4	3/16
39	3/8	3/8	3/4	13/16	1/4	3/16
42	3/8	3/8	3/4	13/16	1/4	3/16

NOTE.—The minus sign (−) alone indicates that the plus variation is not limited; the plus and minus sign (±) indicates variation in both excess and deficiency in dimension.

"(*c*) Blisters where the glazing is broken or which exceed 3 in. in diameter, or which project more than 1/8 in. above the surface.

"(*d*) Laminations which indicate extended voids in the pipe material.

"(*e*) Fire cracks or hair cracks sufficient to impair the strength, durability or serviceability of the pipe.

"(*f*) Variation of more than 1/8 in. per linear foot in alignment of a pipe intended to be straight.

"(*g*) Glaze which does not fully cover and protect all parts of the shell and ends except those exempted in Section 31; also glaze which is not equal to best salt glaze.

"(*h*) Failure to give a clear ringing sound when placed on end and dry-tapped with a light hammer.

"(*i*) Insecure attachment of branches or spurs.

"**Salt Glaze.** Section 31.—The glaze shall consist of a continuous layer of bright or semi-bright glass substantially free from coarse blisters and pimples. If present none of these shall project more than ⅛ in. above the surrounding surface. Not more than 10 per cent of the inner surface of any pipe barrel shall be bare of glaze except the socket, where it may be entirely absent. Glazing will not be required on the outer surface of the barrel at the spigot end for a distance from the end equal to two-thirds the specified depth of socket for the corresponding size of pipe. Where glazing is required there shall be absence of any well-defined net work of crazing lines or hair cracks. All glazing shall be equal to that produced by the best salt-glazing process."

The specifications of the American Society for Testing Materials limit the clay sewer pipe lengths to 2 ft. for pipes 6 in. in diameter; 2, 2½ and 3 ft. for 8- to 24-in. diameters, and 3 ft. for 27-in. and larger diameters. But one type of socket and one standard thickness for pipe of each nominal diameter is prescribed. The tendency is in the direction of the use of these single standards rather than to the multiplication of types. The 3-ft. lengths cost a little more than the 2-ft. lengths, but are preferable because of the small number of joints to be made with them. The difference in thickness between the standard and the double-strength sewer pipe, as made heretofore by different manufacturers, is shown in Fig. 125, with which the requirements of the standard specifications of the American Society for Testing Materials (Table 34) can readily be compared.

TABLE 34.—DIMENSIONS OF CLAY SEWER PIPE, A. S. T. M.

Internal diameter, inches	Laying length, feet	Inside diameter at mouth of socket, inches[1]	Depth of socket, inches	Minimum taper of socket	Thickness of barrel, inches	Thickness of socket
6	2	8¼	2	1:20	5	The thick-
8	2, 2½, 3	10¾	2¼	1:20	¾	ness of the
10	2, 2½, 3	13	2½	1:20	⅞	socket ¼
12	2, 2½, 3	15¼	2½	1:20	1	in. from
15	2, 2½, 3	18¾	2½	1:20	1¼	its outer
18	2, 2½, 3	22¼	3	1:20	1½	end shall
21	2, 2½, 3	26	3	1:20	1¾	not be
24	2, 2½, 3	29½	3	1:20	2	less than
27	3	33¼	3½	1:20	2¼	three-
30	3	37	3½	1:20	2½	fourths of
33	3	40½	4	1:20	2⅝	the thick-
36	3	44	4	1:20	2¾	ness of the
39	3	47¼	4	1:20	2⅞	barrel of
42	3	51	4	1:20	3	the pipe.

[1] When pipes are furnished having an increase in thickness over that given in last column the diameter of socket shall be increased by an amount equal to twice the increase of thickness of barrel.

The deep- and wide-socket pipe are favored by some engineers because they give more space for jointing, but the general belief is that the standard sockets, if made true to form so as to give the specified space for joints, are preferable because tighter joints can be made with them. Difficulties with the usual socket are largely due to imperfections in manufacture and inspection which result in jointing spaces being too small at times to allow the gasket and mortar of the joint to be put into place properly.

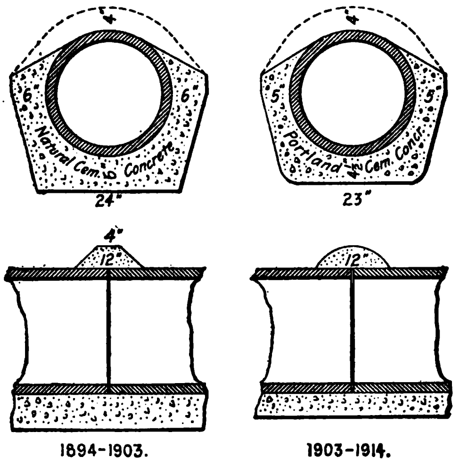


FIG. 126.—Cradle and joint of Washington pipe sewers.

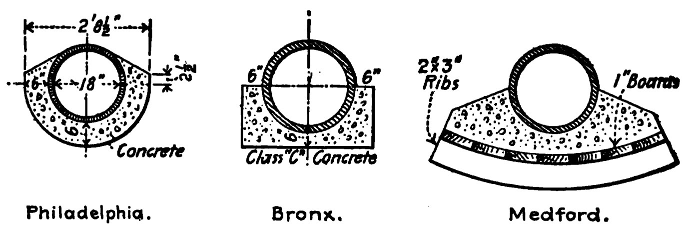


FIG. 127.—Types of cradles.

Vitrified pipe without any socket is used in the sewerage system of the District of Columbia, but not to any extent elsewhere in the United States. The joints are made by a ring of mortar over the top while the sides and bottom are made tight by the concrete cradle which supports the pipe. Fig. 126 shows two types of cradle and joint used in the District of Columbia, and Fig. 127 shows three types of cradles used with bell and spigot pipe.

Concrete Sewer Pipe.—The sand and gravel or broken stone used in cement pipe are generally proportioned to give a mixture of about 1:4 for sizes up to 20 in. to be used under heads exceeding 15 ft., while a 1:3 mixture is perhaps more used for pipe over 20 in. in diameter. Pipe 10 in. or less in diameter are preferably made without any coarse aggregate, on account of the difficulty of producing a dense, uniform material with coarse aggregate in a thin shell. The coarse aggregate used for larger pipe is from $\frac{1}{4}$ in. to half the thickness of the wall of the pipe. Dieckmann recommends that the fine aggregate should be graded so that not more than 10 per cent will remain on a 10-mesh sieve and not more than 30 per cent pass a 50-mesh sieve.

Two methods, dry and slush, have been used in making cement pipe. The mixture used in the dry method contains only enough water to leave web-like markings on the concrete when the forms are removed and to ball up when pressed in the hand. This mixture is tamped into molds and the quality of the pipe depends on the thoroughness of the tamping as well as character of the mixture and the care taken in curing. With the slush method, used for large pipe, the materials are so wet that they will flow into every part of the mold with little tamping, and as a result the pipe must remain in the molds longer than with dry mixtures. Small sizes of pipe are generally made by machines, which are sometimes used for larger sizes.

Cement pipe are sometimes made without sockets and are jointed like the straight vitrified clay sewer pipe used in the District of Columbia or by placing pre-cast cement bands and mortar over the joints. If they are without sockets there is a decided tendency to give one end an inside bevel and the other end an outside bevel, in order to make a closer joint than is possible with square ends.

Reinforced-concrete sewer pipe have the advantage over reinforced concrete sewers built in place of doing away with forms in the trench and permitting the manufacture of the pipe under the most favorable conditions to produce a dense, uniform material in which the reinforcement is placed accurately.

Laying Sewer Pipe.—Great care must be taken in handling sewer pipe to prevent injury to it. Pipe up to 24 in. in diameter are usually taken from the cars by hand, but where larger sizes must be handled in quantities a guy derrick will probably save time and breakage, although these large pipe can be handled with the aid of skids by hand if care is taken. If the pipe is placed in storage it should be laid in piles, the sockets of one row alternating with the spigots of the next lower row, and all pipe securely held by blocking, to prevent a run of the pipe in the lower rows. If pipe has to be carried through the winter, which is undesirable, it should be supported so that it cannot become frozen to the ground and that water will not collect in it. The pipe should not be taken from the yard to the street, as a rule, until a day or so before they are needed, owing to the greater danger of breakage at the site of the work.

Pipe are generally lowered into the trench by hand but when they are 24 in. in diameter or larger a three-leg derrick which can be set up across the trench is useful in handling them. Where a sling is desirable in lowering them what is known as a

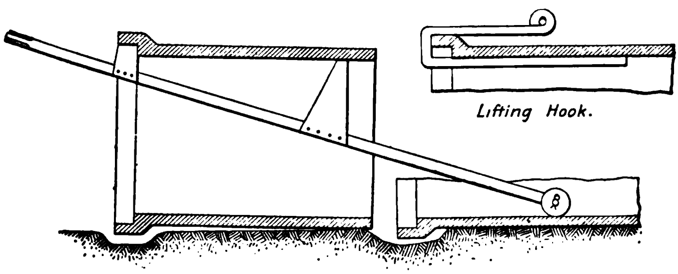


Fig. 128.—Pipe buggy and lifting hook.

"hook rope" is often used. This is a rope with a hook spliced in one end and a chain and hook spliced in the other end. The chain is passed through the length of pipe and the hook fastened over the chain at the spigot end of the pipe, thus forming a loop in which the pipe hangs as it is lowered into the trench.

After being lowered into the trench the pipe is usually shoved into place by the pipe layer, his helper lifting the spigot end by means of a wire or strap passed under the pipe. If the pipe is so large as to be hard to move in this way a lifting hook or pipe buggy, Fig. 128, may be used.

If, when the pipe is in place and tested for line and grade, it is found to be slightly too high, it is sometimes possible to force it down to grade by lightly tapping it, but otherwise it must be lifted and just enough earth scraped out to give it a uniform and firm support. The bottom of the trench in earth should be shaped as nearly as possible to receive the bottom quadrant of the pipe, with a deeper hole under each socket to enable the joint to be made. If the trench is in ledge the rock is generally taken out 6 in. below the invert of the sewer and the space refilled with good earth or gravel. If the pipe is found to be too low when tested it must be raised and earth carefully spread below it so as to bring it to grade when it is again lowered.

In most sewer pipe laying three or four men constitute a working gang. One handles the pipe on the bank and lowers it into the trench, another receives it and places it in position, another makes the joint and, with the fourth, carefully tamps the earth under and around the sides and fills the socket hole completely. The filling and tamping until the backfill is a foot above the pipe must be done very carefully to avoid disturbing or cracking the pipe. It must proceed equally on each side of the pipe and the tamper should have a face not exceeding 5 by 1½ in. and should not weigh over 7 lb. For work about the sides of the pipe a tamper of even smaller size is useful; it has a face about 4 in. long and ½ in. wide and a handle of ⅝-in. iron set at an angle so that the filling under the pipe can be reached easily.

Cement Joints.—Most sewer pipe are laid with cement joints. The cement may be used neat or in a 1:1 or 1:2 mortar with sand. The more cement there is the more easily can the joint be filled but the more liable the joint will be to crack. More time is required for a mortar joint to harden than one of neat cement. A 1:2 mixture is most commonly used, except when rapid setting is desired.

It is usual to begin a joint by placing a gasket formed of a piece of jute or oakum soaked in cement grout around the end of the spigot of the pipe to be laid. Its purpose is to keep the pipe centered in the socket of the pipe already in place and prevent the squeezing out of the mortar in the bottom of the joint by the weight of the pipe. If no gasket is used, mortar is spread in the lower part of the socket of the pipe already in place in order to hold the new pipe in place, but this mortar is liable to be forced out to some extent and joints made in this way are not so good as those made with a gasket.

After the pipe is in position, a man with a trowel and rubber glove fills the joint carefully with mortar, working it into place with a wooden calking tool. The mortar is usually left with a slope of about 45 deg. from the outer edge of the bell to the pipe with which the joint is made. This is termed "overfilling" the joint and is generally believed to give a tighter joint than finishing off the mortar flush with the end of the bell. Where unusually good joints are required, they are sometimes only partly filled with mortar, then another gasket is rammed into the joint flush with the end of the socket, and the joint is finally overfilled with mortar.

After a joint has been made it is desirable to wrap about it a piece of cheese cloth, securely tied at the top, to hold the mortar in place, for there is a tendency for the mortar to sag away, particularly if it is rather wet, and thus open the joint somewhat. The use of the cheese cloth also permits the use of a rather moist mortar with which a better joint can be made than with a stiff, dry mortar.

Except where a gasket is used there is a tendency for mortar to work through the joints into the interior of the pipe, where it will harden into an obstruction to the flow of sewage if not removed. It is therefore desirable to scrape the inner surface of a pipe sewer as it is laid, for which purpose a "go-devil" can be used. This consists of two wooden discs slightly smaller than the bore of the pipe. Each disc is made of two thin pieces of wood clamped together with a piece of cloth-insertion rubber packing between them, cut to fit tightly in the bore of the pipe. The discs are fastened together about 2 ft. apart by three ¼-in. iron rods. A ¾-in. manila rope passing through and fastened to the discs is threaded through the pipe as laid, after which the go-devil is pulled along, effectually cleaning the sewer pipe. A poor substitute for this is the swab or "half moon" made from a piece of board cut to the curve of the interior of the pipe but not filling it. This board, fastened to a stick a little longer than pipe, is drawn along first on top and then on the bottom, to clean the pipe.

Sulphur and Sand Joints.—A composition of sulphur, tar and sand has been used for many years in England in making the Stanford pipe joint, which has found little or no favor in the United States.

Joints of sulphur and sand have been used to some extent in the United States for many years, however, particularly in connecting several lengths of pipe on the ground before lowering them into the trench. The purpose of the sand is to keep the jointing material at a constant volume while it is cooling and prevent the shrinkage which occurs if little or no sand is used with the sulphur. The sand must be very fine and without a gritty feeling when rubbed between the fingers. Equal parts of sulphur and sand are generally used. The temperature of the material must not be raised above that necessary to keep it liquid, for too high a temperature will change the liquid to a plastic mass, which cannot be used until it cools again to the correct degree. The material is heated and poured like lead in jointing water mains, an asbestos jointer being useful in retaining it in the joint, except that the funnel should be higher, ½ in. at the throat and 3 in. high. The joint is absolutely rigid when finished and for this reason some engineers prefer a more plastic joint.

Asphalt and Tar Joints.—These are generally made with asphaltic preparations of the general character used in pavement work for joint fillers. The joint is first calked with oakum and then the hot filler is poured into it as lead is poured into the joint of a cast-iron water main. When the joint is filled it is overfilled with cement mortar. The material has a low melting point which prevents its successful use in places where large quantities of hot water or steam find their way into the sewer, as is the case in some large cities.

In a large mileage of sewers in very wet soil where it was particularly desirable to prevent the infiltration of ground water into the sewers through the joints, Alexander Potter used the following methods: The inside of the bell and the outside of the spigot were swabbed with tar and a gasket of jute soaked in tar was forced into the joint. Into a bucket full of portland cement tar was slowly poured and kneaded by hand until the mixture was of the consistency of rather stiff dough. This was then rolled on a board into a roll of proper size to be forced into the joint, where it was calked until the space was filled to within ½ in. of the end of the bell, after which an overfilled cement finish was given to the work. In this work the spigot had to be supported while the joint was hardening or the weight of the pipe would cause the filling material to exude from the joint, and some difficulty was experienced at first in conducting the work so as to avoid such troubles.

Linseed Oil Jointing Preparations.—Pipe joints have been made of several preparations in which linseed oil is the characteristic material. The G-K compound is described by its makers as "a homogeneous plastic mass composed largely of vulcanized linseed oil and a binder of anhydrous clay." Marbleoid is composed of linseed oil, sulphur and marble dust. Jointite and Filtite are other plastic jointing compounds, the latter being made in several grades for poured joints and also in a grade so stiff that it must be molded and pressed into the joints, a property which is claimed to make it of value for joints that must be filled under water.

The G-K compound is furnished in barrels of about 400 lb. The staves must be knocked away and the material broken up with a hammer or hatchet into pieces small enough to melt in a pail or kettle. The melting must be done carefully, for if the material is overheated it becomes carbonized and worthless. It bubbles and foams when first heated but after it has been kept in a melted condition for 15 to 30 min., the foaming disappears to a great extent and the material is ready for pouring. The joint is first calked with jute and the filler is then poured like melted lead in a cast-iron pipe joint, using an asbestos jointer to keep it in place. On work done under the authors' direction, a number of joints were poured in this way while water was standing in the trench half way to the crown of the pipe, without any imperfections resulting in the joints, but they do not recommend this practice under ordinary conditions.

Platforms and Cradles.—The necessity of supporting sewer pipe securely is met by the use of timber platforms or concrete cradles, Figs. 126 and 127. Where a single plank laid directly under the pipe is used as a platform, the water in the bottom of the trench can flow along either side without undermining the sewer itself but there is some danger of its washing the sides of the trench. To meet this objection, longitudinal timbers are laid a few inches apart, each length of pipe being supported by a transverse timber near each end, cut to conform with the shape of the pipe. (Wedge blocks must not be used as they produce a concentrated load on the pipe which is likely to cause rupture.) With such a platform the water flows through the spaces between the longitudinal timbers. The objection to it is most evident where there is an underdrain below the pipe, for the water flowing under the sewer and over the underdrain may wash earth from below the sewer into the underdrain, causing settlement or cracking of the sewer. It is for this reason that concrete cradles are generally employed, even when a platform must be constructed to receive the concrete.

In very wet trenches in fine sand it is necessary in many cases to hold the bottom down until the pipe or concrete cradle can be placed. Frequently gravel and coarse sand spread over the bottom will be of some service; brickbats, straw, broken stone and cinders have been used for this purpose with more or less success. The best results are undoubtedly obtained when concrete is used, but even this material cannot be relied upon at times to hold down the bottom unless a platform is first constructed to receive it. The planks should be worked down through the sand and water until they are 6 to 8 in. below the bottom of the sewer, and the material above them should then be cleaned off as well as possible before placing the concrete.

In quicksand and similar material, it is necessary to carry down the bottom of the trench by special expedients, and in carrying them out it is highly important to do the work rapidly and reduce the walking about on the bottom of the trench to a minimum, for the more such material is stirred up the more difficult it is to control. In order to put in a cradle under such conditions, two planks are placed on edge far enough apart to include the cradle between them, thus forming a kind of bottomless box. The ends of the planks are fastened together so as to hold them vertical and the planks are then pushed down into the quicksand. When they are in place they can be securely held there by braces between them and the sheeting of the trench. Planks cut to form a bottom for this box are then put in place one by one, by men standing on each plank and working it down to its final position, where it is held by cleats. Brief reference has already been made to methods of handling quicksand in Chapter VII under "Excavation."

Branches.—A branch must be set in the sewer at every house, well down the line of the sewer from the point where the main drain passes through the wall of the building. Branches must also be provided for all vacant lots, usually every 30 to 40 ft., depending upon the character of the building operations in that vicinity.

The branches have the side outlets closed with stoppers until the house connection between the sewer and soil pipe of the building is put in. The stoppers should be put in place before the branch is lowered into the trench and should be made as nearly water tight as possible. The work can be done conveniently by placing a number of Y's on a horse constructed to hold the branch openings in a horizontal plane. The stoppers are then placed in position, jute is calked around them, Fig. 129, and considerable mortar is placed around and over the jute and stopper. Then a piece of wood, cut in the form of a truncated cone, is twisted around on top of the stopper. This forces the mortar to the outside, forms a circular ring of cement in and over the joint, and leaves the central part of the stopper clean. Afterward, when it is desired to make a connection with one of the branches, it is necessary only to break the center of the stopper with a hammer and chip out the remainder of the stopper and mortar with a chisel.

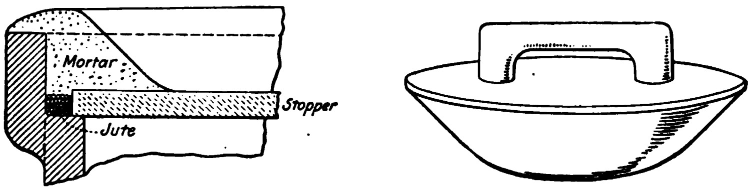


FIG. 129.—Stopper in branch and former for sealing it.

Backfilling.—Backfilling the trench is one of the most important steps in sewer construction. The work of this nature about the pipe, up to the level of its crown, must be done with selected material from which hard lumps and large stones have been removed. This is usually done by the pipe-laying gang.

The backfilling of the trench above this point is of much importance, because if not thoroughly done settlement is likely to take place. Only loose dirt should be placed around the pipe until the backfill is carried to a height of 6 to 12 in. above the crown of the pipe, or higher if filling is dumped from a height in large quantities, as from a bucket. Above this filling the coarser material may be used, but care should be taken to have rocks and lumps of hard material completely surrounded with fine material, so that there shall be no large voids. Usually one rammer to one or two men shoveling the earth into the trench, will accomplish satisfactory work.

With some kinds of material the backfilling may be consolidated by the use of water, the process being known as puddling. Water should usually not be put into the trench until the backfilling has been carried up a short distance above the top of the pipe; then the trench should be partly filled with water and the earth thrown into it. The use of water is particularly well adapted to materials through which the water can be readily drained away. Water should not be used in clayey materials which will hold it for a long period of time and which tend to shrink as the water dries out.

By thorough ramming it may be possible to put more earth into a trench than by puddling with water. With some classes of material, such as loam, it is possible to put more material back into the trench than came out of it, and with sizes of pipe up to 15 in. usually all such material taken out can be put back. With other material, such as gravel, clay and hardpan, there will be a surplus left after backfilling, in excess of the cubic space taken up by the pipe and manholes.

It is often desirable to fill the trench completely full of earth or even overfill it and then to roll it with a steam roller. If the trench is narrow this may best be done by so operating the roller that one of the rear wheels will pass over the trench and follow the backfilling down below the surface of the street. If sufficient material is put into and over the trench to completely fill it after rolling, it will be necessary to remove a portion of the surface to make room for the macadam or paving to be put back later. When a roller is used it is important to take care not to hit the manholes, as the frames may be displaced and the brickwork injured.

In very deep and wide trenches it may sometimes be less expensive to repair the street after settlement than to go to the expense of very thorough tamping during construction. The engineer in drawing his specifications for the consolidation of backfilling must use judgment as to the expense to which he cares to go to have the backfilling tamped or puddled, basing his judgment upon the character of the work and the nature of the locality through which the sewer is to be built. Where it is not desired to spend sufficient money to thoroughly tamp the backfilling much can be accomplished with little labor by spreading the dirt to make sure that the voids around the stones and hard lumps of earth are filled. The use of water in puddling is generally less expensive than tamping. Upon some large work it is not practicable to fill the trench with water and throw the material into it. If this is impracticable considerable good can be done by the intelligent use of a stream of water from a hose.

The cost of backfilling may in part be offset by the saving in the amount of material to be hauled away. Where it is necessary to haul the surplus material a long distance, the cost of hauling may be fully as great as the cost of the tamping necessary to put the material back into the trench.

Backfilling should always be done as soon as possible after the sewer is built, in order to clean up the street and prevent inconvenience to the public due to large piles of dirt left in the street unnecessarily long. Where there is a surplus of material, the finer material and that which can be consolidated to best advantage should be used for backfilling, the coarser part being carried to the dump. In freezing weather it is sometimes difficult to get enough fine, unfrozen earth for backfilling immediately around the sewer. This should be done, however, even if necessary to thaw the earth thrown out upon the bank for the purpose. Filling the trench with frozen earth is always unsatisfactory and it is never possible to secure a well-compacted fill under such conditions. Settlement follows thawing in the spring, when further filling becomes necessary. Moreover, the integrity of the sewer pipe may be endangered by such practice.

Where it is not necessary to reopen the street to travel, as when the sewer is built through open fields, the amount of labor devoted to tamping may sometimes be reduced or wholly eliminated and the fill allowed to settle and become compacted by rains.

Tamping is a portion of the work which is very likely to be neglected. It is not excessively expensive and is usually well worth the cost, especially in city streets. The cost of tamping varies greatly, but in ordinary city trenches where the sewers are small, it may involve from one tamper per shoveler, to one tamper to three shovelers.

CONCRETE AND BRICK SEWERS

Circular Sections.—The circular section encloses a given area with the least perimeter and, therefore, gives the greatest velocity when flowing full or half full. Under ordinary conditions, such sections are economical in the quantity of masonry required, although in flat-bottom trenches or under conditions requiring pile foundations or timber platforms, additional masonry is required to support the arch. In combined systems, where the dry-weather flow of sewage is very small in comparison with the storm-water flow, the velocity for the low flows is greatly reduced in the circular section and on that account this section may not be so advantageous as the egg-shaped section.

The standard circular concrete section built at Louisville is shown in Fig. 130 and Table 35. The dimensions were based on what experience had shown to be a safe thickness of masonry under the conditions there. The minimum thickness at the crown and at the invert was fixed at 5 in. because of the practical difficulty of obtaining with certainty a first-class wall of monolithic concrete of less thickness. The soft-ground section was used where the earth would not stand when trimmed to the shape of the firm-ground section. Other circular sewers are illustrated in Figs. 58, 59 and 60. The circular and egg-shaped sections are usually preferred for sewers under 5 ft. in diameter.

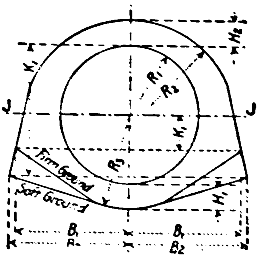


FIG. 130.—Louisville standard concrete section.

Egg-shaped Sections.—In combined sewers where the dry-weather flow of sewage is small compared with the capacity of the sewer required for storm water, or in sanitary sewers for a district where the present population is but a small

TABLE 35.—DIMENSIONS OF PLAIN CIRCULAR CONCRETE SEWERS, LOUISVILLE

Dimensions of the sections										Quantity of concrete cu. yd. per lin. ft. sewer	
Diameter	H_1	H_2	B_1	B_2	K_1	K_2	R_1	R_2	R_3	Firm ground	Soft ground
24"	5"	5"	1' 7¼"	1' 8⅜"	6"	1'10½"	1' 0"	1' 6"	1' 5"	0.13	0.15
27"	5"	5"	1' 9⅝"	1'10⅞"	6¾"	2' 1 5/16"	1' 1½"	1' 8"	1' 6½"	0.15	0.18
30"	5"	5"	2' 0"	2' 1½"	7½"	2' 4⅛"	1' 3"	1' 10"	1' 8"	0.18	0.21
33"	5"	5"	2' 2⅜"	2' 4⅛"	8¼"	2' 6 15/16"	1' 4½"	2' 0"	1' 9½"	0.19	0.23
36"	5"	5"	2' 4¾"	2' 6⅝"	9 "	2' 9¾ "	1' 6"	2' 2"	1'11"	0.22	0.26
39"	5"	5"	2' 7¼"	2' 9⅛"	9¾"	3' 0 9/16"	1' 7½"	2' 4"	2' 0½"	0.25	0.29
42"	6"	6"	2' 9⅝"	3' 0 "	10½"	3' 3⅜ "	1' 9"	2' 6"	2' 3"	0.29	0.35
45"	6"	6"	3' 0"	3' 2¼"	11¼"	3' 6 3/16"	1'10½"	2' 8"	2' 4½"	0.33	0.40
48"	6"	6"	3' 2⅜"	3' 5¼"	1' 0"	3' 9"	2' 0"	2' 10"	2' 6"	0.38	0.45
51"	6"	6"	3' 4¾"	3' 8 "	1' 0¾"	3'11 13/16"	2' 1½"	3' 0"	2' 7½"	0.41	0.49
54"	6"	6"	3' 7¼"	3'10¼"	1' 1½"	4' 2⅝"	2' 3"	3' 2"	2' 9"	0.43	0.53
57"	6"	6"	3' 9⅝"	4' 1⅛"	1' 2¼"	4' 5 7/16"	2' 4½"	3' 4"	2'10½"	0.47	0.57
60"	6"	7"	4' 0"	4' 4 "	1' 3"	4' 8¼ "	2' 6"	3' 6"	3' 0"	0.53	0.65
63"	6"	7"	4' 2⅜"	4' 6⅛"	1' 3¾"	4'11 1/16"	2' 7½"	3' 8"	3' 1½"	0.57	0.71
66"	6"	7"	4' 4¾"	4' 9⅛"	1' 4½"	5' 1⅞"	2' 9"	3' 10"	3' 3"	0.61	0.77
69"	6"	8"	4' 7¼"	5' 0 "	1' 5¼"	5' 4 11/16"	2'10½"	4' 0"	3' 4½"	0.66	0.84
72"	6"	8"	4' 9⅝"	5' 2¼"	1' 6 "	5' 7½"	3' 0"	4' 2"	3' 6"	0.70	0.88

proportion of the ultimate development, the ideal sewer section is one in which the hydraulic radius remains constant as the depth of flow decreases. It is impracticable to obtain the ideal, but the egg-shaped or oval section, Figs. 51 and 63, comes nearer to it than any other thus far devised.

As explained in Chapter IV, the advantage of the egg-shaped section is that for small flows the depth is greater and the velocity somewhat higher than in a circular sewer of equivalent capacity.[1] The depth of flow in the egg-shaped sewer is always greater than in the equivalent circular sewer for equal discharges, and for small flows this increase in depth produces better flotation for the solid matter. In deep trenches there will be a saving in total excavation by using the egg-shaped sewer, due to the decrease in width of trench, which may more than offset the small increase in depth.

The disadvantages of the egg-shaped section are that it is less stable, more liable to crack, requires more masonry, and in general is more difficult to construct than the circular section. In very stiff earth or in rock it is sometimes possible to excavate the bottom of the trench to conform to the shape of the invert of the sewer, but in general, in yielding earth or where foundations are poor and piles or platforms are needed, the egg-shaped section requires more masonry backing under the haunches to support the arch than does the circular sewer. Hence the egg-shaped section may be more expensive than the circular section in many cases, and much more expensive than some of the other sections.

Semi-elliptic Sections.—The arch of this section, Fig. 131 and Table 36, is either a true semi-ellipse or is made up of three circular arcs approximating a semi-ellipse. As the center of gravity of the wetted area is lower than in a circular sewer, the normal flow line will be much lower, which may be of some advantage in locating lateral connections lower or in raising the invert of the main sewer. This, of course, contemplates the possible operation of the lateral sewers under a head at times when the main sewer is running full. Although this is of material advantage where the allowable difference in water level is small, it should generally be avoided.

The chief advantage of this section is that the shape of the arch more nearly coincides with the line of resistance of the arch under

[1] This is shown in detail in "American Sewerage Practice," vol. i, page 384.

actual working conditions than is the case with other sections. Because of this the arch can be made relatively thin and still keep the stresses in the masonry within allowable limits. The section is dependent to only a small extent on the lateral pressure of the earth to prevent failure, and does not depend upon the resistance of the earth filling on the sides, as is very often the case with a horse-shoe section.

TABLE 36.—DIMENSIONS OF AUTHORS' SEMI-ELLIPTICAL SEWER SECTION

(See Fig. 131)

1	2	3	4	5	6	7	8	9	10	11	12
			Thickness of concrete		Interior radii			Exterior radii			
Inside vertical diameter "D"	Area of waterway	Hydraulic mean radius	Crown	Center of invert and spring line	Crown intrados and side wall	Side intrados	Invert and side extrados	Crown extrados	Invert	Area of concrete	Quantity of concrete cu. yd. per lin. ft.
ft. in.	sq. ft.	ft.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	sq. ft.	
6 0	28.2	1.442	0 6	0 9	2 0	6 3	7 6	2 6	8 3	14.12	0.523
6 6	33.1	1.562	0 6½	0 9¾	2 2	6 9¼	8 1½	2 8½	8 11¼	16.58	0.614
7 0	38.4	1.683	0 7	0 10½	2 4	7 3½	8 9	2 11	9 7½	19.21	0.712
7 6	44.05	1.803	0 7½	0 11¼	2 6	7 9¾	9 4½	3 1½	10 3¾	22.08	0.817
8 0	50.1	1.923	0 8	1 0	2 8	8 4	10 0	3 4	11 0	25.10	0.930
8 6	56.6	2.043	0 8½	1 0¾	2 10	8 10¼	10 7½	3 6½	11 8¼	28.44	1.054
9 0	63.4	2.163	0 9	1 1½	3 0	9 4½	11 3	3 9	12 4½	31.80	1.177
9 6	70.7	2.284	0 9½	1 2¼	3 2	9 10¾	11 10½	3 11½	13 0¾	35.41	1.311
10 0	78.3	2.404	0 10	1 3	3 4	10 5	12 6	4 2	13 9	39.24	1.453
10 6	79.3	2.525	0 10½	1 3¾	3 6	10 11¼	13 1½	4 4½	14 5¼	43.26	1.602
11 0	94.75	2.646	0 11	1 4½	3 8	11 5½	13 9	4 7	15 1½	47.48	1.757
11 6	103.5	2.764	0 11½	1 5¼	3 10	11 11¾	14 4½	4 9½	15 9¾	51.89	1.921
12 0	112.75	2.884	1 0	1 6	4 0	12 6	15 0	5 0	16 6	56.51	2.092
12 6	122.4	3.005	1 0½	1 6¾	4 2	13 0¼	15 7½	5 2½	17 2¼	61.31	2.270
13 0	132.4	3.125	1 1	1 7½	4 4	13 6½	16 3	5 5	17 10½	66.32	2.456
13 6	142.7	3.245	1 1½	1 8¼	4 6	14 0¾	16 10½	5 7½	18 6¾	71.51	2.649
14 0	153.5	3.365	1 2	1 9	4 8	14 7	17 6	5 10	19 3	76.91	2.849

Area of waterway = $0.7831D^2$. Area of concrete section = $0.3924D^2$.

The semi-elliptical section depends to a larger extent upon the stability of the invert than do the circular, egg-shaped, catenary, gothic, basket-handle and horseshoe sections. Where the semi-elliptical section is constructed in compressible earth and the structure is built monolithic, with reinforcing bars running continuously from the center of the invert to the crown of the sewer, there will be a large bending moment at the center of the invert. Under such foundation conditions, the invert should be made as thick as the arch at the springing line and should be heavily reinforced to withstand the stresses. Unless this is done, cracks are likely to occur in the center of the invert.

As in the horseshoe type, the invert of the semi-elliptical section readily conforms to the bottom of the trench excavation and for that reason the quantity of masonry below the springing line is not excessive.

This section is not as advantageous for low flows as the circular, because of the wide and shallow invert in which there is a very low velocity. However, for sewers where the quantity to be carried is not subject to wide variations and the normal flow is as much as one-third of the total capacity of the sewer, this

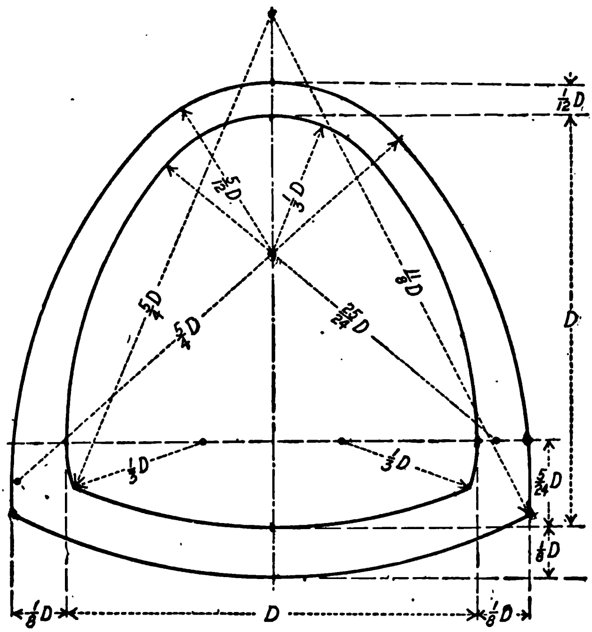


FIG. 131.—Authors' standard semi-elliptical section.

disadvantage may be neglected. The hydraulic properties of the semi-elliptical section are very good in general, which, with the very desirable structural features, make this type one of the best for sewers over 6 ft. in width.

Other Sections.—The catenary section, Figs. 54 and 64, was used extensively on the Massachusetts North Metropolitan sewerage system under the direction of Howard A. Carson. Its principal advantage is that it conforms so nearly in shape to the available space inside the wooden timbering of earth tunnels. The line of resistance keeps fairly well within the arch section, the hydraulic properties are fairly good, and the center of gravity of the wetted section is low with respect to the height.

The gothic section, as developed on the Massachusetts North Metropolitan system, has a horizontal diameter about 17 per cent less and a vertical diameter about 8 per cent more than the diameter of the equivalent circle.

The horseshoe and basket-handle sections are much alike. The latter has a horizontal diameter about 6 per cent less than the vertical. The arch is slightly pointed and the invert is flatter than a semicircle. The horseshoe section, Fig. 53, has been widely used. Above the springing line it has a semi-circular arch, while the side walls below the springing line are vertical or incline inward, sometimes with a plain and sometimes with a curved surface. The invert varies in section from a horizontal line to a circular or parabolic arc, or other design calculated to concentrate the low flows near the center of the invert. The invert conforms to the trench bottom, saving masonry. For a given width, this section will have less height than the equivalent circular sewer. Its main disadvantage is that it depends upon the ability of the back-filling to resist the lateral thrust of the arch transmitted to the side walls near the springing line.

The parabolic or delta section, Fig. 55, has a somewhat larger carrying capacity than a circular section of the same height. It is both economical and strong, the normal flow line is lower than in the circular section, and the sloping invert is well adapted for low flows. It requires a wider trench than the semi-elliptical section for equal capacity and height.

The U-shape section, Fig. 66, has fairly good hydraulic properties until it becomes filled, when the width of the roof causes a large increase in the wetted perimeter. The invert adapts it for low flows and is easily constructed. It requires considerable masonry in proportion to its area, and its main field is for sewers about 3 ft. wide and considerably over 3 ft. in depth.

The rectangular section, Fig. 52, has been steadily increasing in favor because of its excellent hydraulic properties until it is filled, simplified form work, economy of masonry and space in the trench, and ease of construction. The V-shape invert is frequently used with the rectangular section on account of its suitability for low flows.

The semi-circular section, Figs. 57 and 65, was formerly used for large sewers on low land, where the natural ground surface was below the top of the sewer. The rectangular section has largely displaced it as being less expensive and possessing better hydraulic properties.

Analysis of Masonry Arches.—The complete analysis of the stresses in masonry arches is a highly complex problem which few engineers are ever called upon to solve. A considerable proportion of the large masonry sewers in the country were designed without anything more than a very elementary investigation of their arches. The increasing use of reinforced concrete sewers is responsible for a more general effort by designers to analyze the stresses in these structures, and now that a complete, accurate method[1] of doing this has been developed, the design of large sewers can be made to depend upon both theory and experience instead of on the latter mainly. There are three general methods of analysis[2] now in use, as follows:

The first method, called the "voussoir arch method," based on the so-called "hypothesis of least crown thrust," is applicable only to that portion of the sewer section above the springing line of the arch. Either the sewer must have very heavy side walls or the thrust of the arch must be carried by the sides of a rock trench, in order to make this method strictly applicable.

The second method, based on the elastic theory of the arch and following the method described by Turneaure and Maurer in their "Principles of Reinforced Concrete Construction," is applicable to all sewer sections and can be used to cover all conditions. It has some mechanical disadvantages when applied to the analysis of the entire sewer structure, invert included.

The third method, also based on the elastic theory but using the so-called method for indeterminate structures, is of special advantage in the analysis of the entire sewer section as it permits a more suitable division of the axis in the side wall and invert. It does, however, require some additional labor over the second method when applied to an arch with fixed ends. For large sewers constructed in compressible soil and built of monolithic reinforced concrete, the third method is the most desirable.

Influence of Construction Conditions on Section.—The method of construction of a brick or concrete sewer, whether in open cut or in tunnel, may have an important influence on the selection of the cross-section, as well as considerations of strength

[1] "American Sewerage Practice," vol. i, p. 488.
[2] "American Sewerage Practice," vol. i, p. 471.

and of the hydraulic elements of the different sections. In a tunnel it is desirable to use a section which will utilize to the best advantage all of the space inside the tunnel bracing. In earth tunnels with the usual timbering, the catenary or semi-elliptical sections conform readily to the available space, while in rock tunnels the circular or horseshoe sections are likely to be advantageous. When the sewer is built in an open cut, the choice of the section will be influenced by the bearing capacity of the trench bottom and the ability of the arch to carry the load of the backfilling.

In rock or firm soil, it is possible to shape the bottom of the trench to the profile of the bottom of the sewer and thereby save considerable masonry when using circular or egg-shaped sections. If the excavation is in soft material and the bottom of the sewer must be flat or the sewer is to be carried on piles or a timber platform, considerable additional masonry will be required for the circular or egg-shaped sections, which is not needed if other sections are employed.

Sometimes the cross-section is influenced by the approach of the crown of the sewer to the surface of the street, sometimes the side room is limited by adjacent subsurface structures, and the available depth may be limited by tidal influence on the ground water or other conditions controlling the permissible depth of the hydraulic grade line. The rectangular section has proved one of the most useful for such conditions, although the horseshoe section, with the horizontal or vertical diameters adjusted to meet the conditions, has been largely used. In a few cases the full elliptical section has also been used in restricted places. Where the hydraulic grade line depth is limited, it is desirable to use a sewer section which will carry the maximum and minimum flows with the least variation in depth of flow. The catenary, parabolic, semi-elliptical and rectangular sections are especially suitable for this purpose, as the center of gravity of the wetted area is comparatively low down from the crown in contrast to the circular section. The semi-circular section has also proved useful in this connection, although the rectangular section is being used instead in the more recent work of this character.

Cost of Excavation and Materials.—The cost of excavation required by one type as compared with another should be carefully considered, for if the excavation is in earth in a deep trench, it will probably be cheaper to use a narrower deeper section and thereby save considerable width of excavation, even though the depth of excavation be slightly increased. This will be especially true in a deep rock trench where it may be found of advantage to use a narrow rectangular section having a height 1½ to 2 times the width. For a sewer built in very shallow cut, or practically on the surface of the ground, a wider section will be advantageous, because little additional cost is incurred by increasing width whereas greater depth may increase materially the cost of excavation. Furthermore the cost of an embankment over a wide section will generally be less, because of reduced height and narrower side slopes. The parabolic or delta section is especially useful for crossing low lands where the sewer is largely out of the ground and must be covered by an embankment. The semi-circular section has also been much used for this same purpose, but has been superseded more recently by the rectangular section, having a width about 1½ times its height.

In former years a great many sewers were constructed of quarry stone or large cobbles, but in recent years other materials have proved less expensive and better adapted for the work. The cost of brick varies greatly in different localities and this may influence to a large extent the type of construction selected. In general, concrete is more desirable than brick, but where brick masonry can be had much cheaper than concrete it may be advisable to build the sewer of brick. The object in designing a sewer section should be to obtain one in which the quantity of the masonry and other materials is a minimum consistent with the requisite stability, hydraulic properties and other considerations.

For sewers in which the normal flow is at least one-third of the maximum flow, it has been found that the semi-elliptical section is very economical in masonry, and at the same time provides for the other requirements.

Stability.—Where a sewer is constructed in open trench, the structure must be designed to carry the earth or trench load as well as any superimposed load. The circular arch is not as strong as either the gothic, the parabolic, or the semi-elliptical arch. The semi-circular arch depends to a great extent upon the lateral pressure of the sides of the trench and also to a certain extent on the lateral resistance or passive pressure of the earth backfilling, although this can be obviated by increasing the thickness of the side walls or abutments. The semi-circular sections obviate part of this difficulty by omitting the side walls and resting the springing line of the arch directly on the invert or foundation. In a rock trench the resistance of the sides of the trench is so great that the side walls of the sewer can be greatly reduced in thickness, the thrust of the arch being carried directly into the walls of the trench. In this latter case a very flat arch can be used to advantage.

Imperviousness.—Where a sewer is to be constructed under a river bed or below the water table, it may be of particular importance for the walls of the sewer to be impervious. To this end, if the sewer is built of concrete, it is desirable to insert longitudinal reinforcing bars in the concrete, with a total area of 0.2 to 0.4 per cent of the sectional area of the concrete, in order to distribute the stress throughout the length of the sewer barrel and thereby prevent the formation of large cracks which would permit leakage. Unless the cracks are very small there may be some danger of corrosion due to the water passing through them and coming in contact with the reinforcement. This might in time weaken the structure.

While the possibility of leakage or infiltration does not ordinarily determine the shape of the waterway of a sewer, it is worthy of consideration when the selection is to be made. For example, if a sewer is to be built below the water table it may be well to adopt a section which is least likely to crack, whereas under other conditions the advantages of a different section might be sufficiently great to warrant its use even though small arch cracks are to be expected. The stability of the horseshoe section depends to a certain extent on the lateral pressure of the earth backfilling, and on that account, the semi-circular arch is apt to crack and may produce unsatisfactory conditions, not only because of leakage into the sewer, but especially on account of the rusting of the steel reinforcement.

Materials for Arches.—In constructing brick arches, three general types of bonding have been used. In the first, "rowlock bond," the arch is built of concentric rings of brick, all laid as stretchers. In the second type, part of the brick are laid as stretchers and part as headers, with radial joints in which the outer end of the joint is widened by increasing the thickness of the mortar or by inserting thin pieces of slate. In the third method, Fig. 59*f*, the masonry is divided into blocks or sections.

Plain concrete arches have been used to a considerable extent in recent years, and have an advantage over the stone or brick masonry arches in that the structure is somewhat more elastic and may withstand tensile stresses to a slight degree although they should not be designed with this in view. In the design of such arches, as well as those of stone and brick, the line of pressure should fall within the middle third of the section, in order that no tensile stresses may be developed. If all the loads acting on the sewer were known exactly, it would be possible to design the section so that at no time would the line of pressure lie outside the middle third, but practically this is impossible, as our knowledge of the action of earth pressure is a matter of approximation only. On that account, under special conditions the stresses in the arch section may not be entirely due to direct compression, but in addition bending stresses may be developed.

Arches of reinforced concrete are not subject to the limitations just mentioned, but can be made to withstand heavy bending moments by reinforcing the section with steel bars to carry tensile stresses. In arches in which the line of pressure lies within the middle third, the stresses in the arch are mainly due to compression and the concrete must of necessity carry the principal part of the load, so that the steel cannot be stressed to the allowable limit. On the other hand, the presence of the steel reinforcement furnishes a sort of insurance to the structure, to care for tensile stresses which may occur on account of unequal settlement of the foundations, temperature changes and many other conditions which cannot be foreseen. The steel is also an additional factor of safety against careless and defective construction. On account of its presence, it is possible to increase slightly the allowable working stresses in the concrete over those which should be used for plain concrete masonry. Because of these considerations the authors believe that for large sewer arches reinforced concrete offers greater advantages than plain concrete, even though an analysis of the section shows that the line of resistance for the conditions considered will remain within the middle third of the masonry section.

Lining for Concrete Construction.—From the observations made by the authors of the condition of sewers after some years of service it appears that on all slopes in which the estimated velocity of the sewage will be 8 ft. per second or greater, the invert may advantageously be paved with hard-burned or preferably vitrified paving brick with square edges, laid with the edges projecting as little as possible and with full portland cement mortar joints. This invert paving should extend well up on the sides of the sewer, on straight sewers covering, in general, the bottom quadrant of a circular sewer. The use of brick paving is preferable to concrete on account of the greater ease of making repairs and further on account of the probability that vitrified or even hard-burned brick will withstand the wear better than concrete of average quality. It is desirable when sewers are to be built of concrete to use hard aggregates, especially for inverts, and a first-class granolithic finish where the surface is subject to greatest wear is better than the ordinary concrete finish.

Brick Laying.—The brick should be laid by shoving them into a full bed of mortar, forming full joints on all sides at one operation. The longitudinal face joints should be as narrow as practicable, usually $\frac{1}{4}$ to $\frac{3}{8}$ in. The joints between the rings should not exceed $\frac{1}{2}$ in. unless it is desired to put in a thicker joint to prevent infiltration. The exterior of the arch is often covered with a plaster of cement mortar $\frac{1}{2}$ in. thick. After the centers have been removed the interior surface of the sewer should be cleaned and pointed as smooth as possible. In some cases one or two coats of cement wash have been applied to the inner surface for the purpose of reducing roughness.

Where the trench is composed of suitable material, the bottom is trimmed to conform to the desired outside surface of the invert masonry, and each brick laid directly on a bed of mortar placed on the bottom. Templates or "profiles" are placed in position and a mason's line is tightly drawn between them. Bricks are then laid in a single row in the center of the bottom of the invert for the full length of the section, about 12 ft. The several rows of bricks are then laid on either side, care being taken to lay each brick accurately to line at the time it is slid into place. A skillful mason will rarely touch the brick after the joint is made, and hammering a portion of the wall to bring it to line should be discouraged, as the adhesion of mortar to brick may be destroyed in this way. Care should be taken to see that the outside ring of brick is well supported by the bank. It frequently happens that the trench cannot be excavated exactly to the line of the proposed brickwork, in which case masons will often fill the space between the brickwork and the bank with loose material insufficiently rammed. In fact it is very difficult to ram the material filling this space without injuring the green masonry. It seems probable that many of the cracks which have been found in brick sewers may be attributed to loose and improper filling between masonry and the solid bank of the trench. Care should be taken in laying the arch rings to see that all joints are properly filled and not unnecessarily thick. The key should be laid by an experienced mason who realizes the importance of tight joints and a well-fitting key properly forced into place, that there may be no injury to the arch when the centers are struck.

After the first few courses in the invert are laid, it is wise to provide a few loose boards or planks upon which the masons and tenders may walk, thus protecting the green brickwork from injury, due to movement of the brick resulting in a lack of adhesion of mortar to the surface of the brick. Upon large sewers the invert may become so high that it will be necessary to provide a working platform for masons and tenders. In building such platforms care should be taken to protect the green masonry from injury.

In trenches in wet or loose material when the soil will not maintain any given shape, the brickwork may be laid in a cradle. Such a cradle may be constructed of ribs cut to the shape of the outside of the invert, making an allowance for the thickness of the plank which are to form the cradle proper. These ribs are set in the bottom of the trench at the proper elevation and 2-in. planks are nailed to them, earth being carefully tamped behind each one as it is put in place. The brick are laid, as already described, on this cradle.

Profiles and Centers for Brick Sewers.—A profile should be made as light as possible and at the same time strong and rigid enough to always hold its true shape. As shown in Fig. 132, nails are driven in the profile, properly spaced for the several courses. Upon large work and where skilled masons are employed, two or sometimes three or more courses of brick may be laid to one stretching of the line. This is especially true upon relatively flat inverts, as those of horseshoe-shaped sewers. Profiles should extend high enough to enable the mason to carry the invert one or two courses above the springing line to allow room for the centers to fall after being "struck." Two profiles are usually spaced 16 ft. apart when a new invert is begun, and thereafter one profile only is used and it is generally placed about 16 ft. ahead of the finished work, a longer spacing being unwise because of the danger that the line may sag between the nails.

The dimensions and method of constructing a profile suitable for use in building a two-ring brick sewer 36 in. in diameter, are shown in Fig. 132. Such a profile should be made preferably of clear soft stock, such as white pine, planed on both sides and of uniform thickness. Provision should be made for the mortar joint between the outer and inner rings and the nails should be so spaced as to allow for joints of proper thickness between courses. It is generally well to send the profile from the shop without having the line nails driven, so that the mason may space them to conform to the thickness of the brick actually being used. The profile represented in Fig. 132 is designed for brick 4 in. in width and 2 in. in thickness.

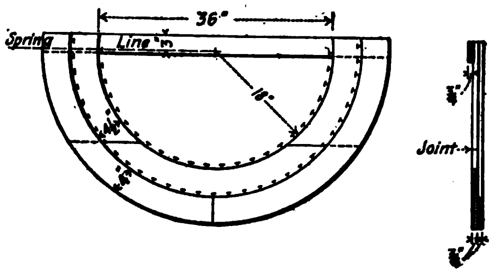


FIG. 132.—Profile for 36-in. brick sewer.

A profile may be secured in place by two vertical boards nailed to its sides and to a trench brace or other support above, and should be accurately set to line and grade and carefully leveled before any brick are laid.

Arch forms or centers, generally built of wood, are usually 8 or 10 ft. in length. It is common practice to build them in one piece, although some prefer to have them in two or more segments to facilitate handling, especially inside the completed sewer. They are set on chairs or legs and can be adjusted to the proper height by means of wedges. After the arch has been completed and the forms have been in place the required length of time, the centers are struck by loosening the wedges. It will be found convenient to have enough centers for a full day's work, the length required varying greatly, but commonly ranging from 24 to 48 ft. Arches turned one day may generally have centering removed the next morning; in fact, centers are sometimes struck without apparent injury to the work as soon as the masonry is completed and backfilled to a depth of 1 or 2 ft. above the crown. Centers are usually set as soon as the inverts are completed.

The size of the lagging and thickness and spacing of the ribs are governed by the size of the sewer. Centers should be made as light as possible to make them easy to handle, but they must be strong and rigid enough to receive and support the masonry without deformation.

When sewers are constructed in tunnel, block centers are more convenient than full centers, although the latter may be used if very short, say 3 or 4 ft. in length. A block center is shown in detail in Fig. 133, and derives its name from the key block. The two large segments are first set in position and through the space at the top between them the brick and mortar are passed to the masons, after which, as the work progresses, the short key blocks are placed and the arch is completed.

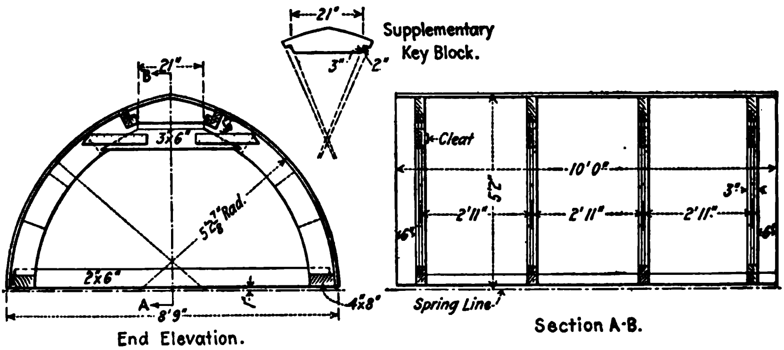


FIG. 133.—Details of block center.

Placing Concrete.—Upon small work, where excavations are shallow, it will usually be most satisfactory to shovel the concrete into the forms from the mixing bed. Where the work is deep and it is not convenient to place the mixing bed over it, the concrete can be conveyed from the mixing bed to the forms through chutes, which may be made of wood lined with sheet iron, or preferably entirely of sheet iron. Such troughs are frequently U-shaped, although, and especially where they must be placed in a nearly vertical position, they may be made nearly cylindrical in form. In either case they should be so constructed as to telescope, the lower end of the upper trough fitting into the upper end of the trough below. It will usually be found convenient to provide a metal hopper at the top, into which the concrete may be thrown or dumped and from which it will flow into the troughs.

Concrete should be made of such a consistency that it will flow readily through a trough inclined 1 ft. in 3 ft., and at least this inclination should be provided, as concrete will not flow readily through troughs laid upon a flatter slope. It is necessary many times to hang the troughs or pipes upon a much steeper slope and difficulty is often experienced when the angle of inclination is at or somewhat more than 45 deg. by the tendency of the stone, and particularly of rounded gravel, to separate from the mortar in passing through the trough or pipe. Where the angle of inclination is nearly vertical, this tendency is again reduced. Where there is a decided tendency for the stone to separate from the mortar, it may be necessary to catch the concrete in a box and re-mix it before finally placing it in the forms. If this is not done, care must be taken to prevent "pockets" of cobbles or broken stone in which there is insufficient mortar and fine aggregate.

Concrete is now usually mixed wet and does not require tamping, but does need more or less spading and slicing with thin metal blades. These are of assistance in working the concrete through reinforcement and for scraping and prying the larger stones away from the forms, so as to leave a good mortar finish when the forms are removed. It is to be remembered, however, that excess of water produces a weaker concrete than a well filled thoroughly tamped drier mixture. Two or more men, depending on the size of the work, are required for placing and cutting the concrete, and one man, when wheelbarrows are employed, to assist in emptying them into the chutes.

Small circular sewers, perhaps up to a diameter of 4 ft., can in many cases be advantageously built in one operation, full cylindrical forms being placed before any concrete is put into the trench. Upon larger sewers there is a great variety of forms available for use, some being made of wood and others of steel. Much ingenuity may be exercised in the design of large forms to reduce the cost of labor in removing and carrying them forward and resetting them.

Sewers more than 4 ft. in diameter are generally built in two, three, or more operations. The invert is first placed for a considerable distance. Upon this the side walls are built and after these become set the arch is turned. This method of progressive construction requires that a considerable length of trench be kept open at all times for concreting, and as time is required for the concrete to become thoroughly set and acquire sufficient strength so that the forms may be removed, it is a source of considerable delay which should be taken into consideration in planning the rate of progress to be attained.

Inverts are generally laid and screeded to templates. In some cases the concrete is placed in alternate blocks, from 8 to 10 ft. in length, between bulkheads. Where the concrete is placed continuously it is necessary to provide templates at frequent intervals, which may conveniently be constructed of ½ by 1½-in. steel on edge and hung from the bracing overhead, Fig. 134. As the concrete is screeded these templates are moved forward.

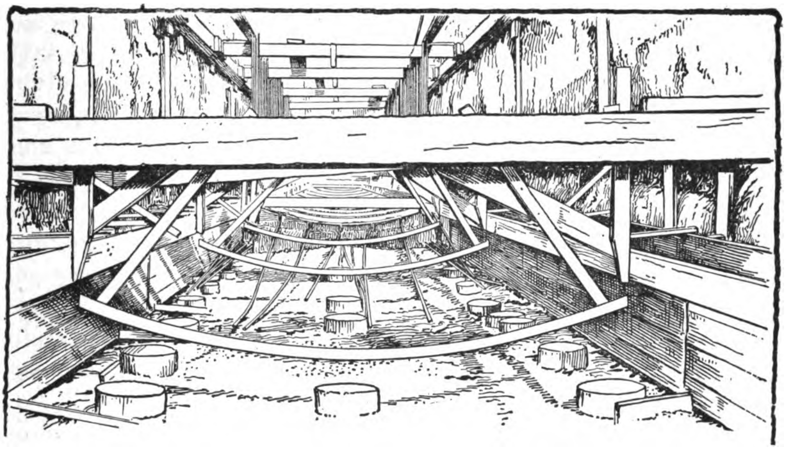


FIG. 134.—Templates to which invert was screeded, Louisville.

Where the concrete of which the invert is constructed is very wet and contains a large quantity of mortar, it may be possible to so work it as to bring the mortar to the surface and provide a satisfactory and smooth invert surface by screeding. If the concrete is lean or dry it may be necessary to use a small quantity of mortar to secure a smooth finish, usually not more than ½ in. This, of course, should be applied before the concrete has acquired its initial set.

If good concrete work is to be secured it is absolutely essential to have watertight, well-made and accurate forms, and to mix the concrete of the proper consistency. If these two points are given careful attention, little difficulty should be experienced in providing a satisfactory concrete structure.

Placing Reinforcement.—In the construction of reinforced concrete sewers, placing and holding the reinforcement in its proper position is a troublesome as well as an important matter. This is particularly the case where expanded metal or woven fabrics of fairly coarse mesh are used. The fabric acts as a screen, preventing the free flow of concrete around the forms. In such cases, as with larger circular sewers, it is desirable to lay part of the invert under the more inaccessible part of the form, by means of templates and screeds. Part of the reinforcing material, bent to shape and extending above the springing line, is held in proper position by fastening it to longitudinal timbers on either side of the trench, and, if necessary, it is also supported by blocks underneath it, removed as the concrete reaches them. When the invert is more or less flat, the concrete may be first placed up to the position of the steel, which is then put in position and the concreting continued, or the whole system of reinforcement may be first placed in position and held by blocking and fastening to the trench braces, before any concrete is placed. After the sewer is completed to the springing line and the centers placed, the arch steel is set and fastened by wiring it to the projecting ends of the invert steel, a proper space between the centers and the steel being maintained, if necessary, by blocks which are removed as the concrete reaches them. Metal and concrete chairs or supports may also be fashioned or used advantageously in some kinds of work.

Forms in General.—The forms and centers used for concrete sewers differ from those used for brick sewers in that they must have smooth surfaces and be watertight. In selecting the type to be used and the material of which it is to be built, the chief considerations are lightness and economy, for substantially as good workmanship can be obtained with the several types and materials provided the forms are properly built and maintained. The first cost, cost of upkeep, and cost of placing, striking or collapsing, and pulling ahead, are some of the essential items to be considered.

There are no arbitrary rules by which to determine whether wooden forms, wooden forms covered with metal or steel forms are most economical for a given case, for this depends largely upon local conditions. The design of forms which can economically and conveniently be erected, struck and moved ahead under the varying conditions encountered in the construction of sewers, especially in deep trenches, has taxed the ingenuity of many experienced sewer builders.

The life of wooden forms is comparatively short. In many instances after being used 8 or 10 times they have to be practically rebuilt, although often the ribs can be used again, the lagging only requiring renewing. When new and well built, wooden forms give a smooth and satisfactory finish, but after repeated use, patching and repatching, they become so rough that the interior surface of the concrete is likely to be unsatisfactory.

Building Concrete Sewers.—Where concrete sewers are built in two or more operations there is difficulty sometimes in securing good joints between the horizontal sections, and there is usually some finishing required because of the rough concrete where the centers do not fit the concrete of the invert. It has therefore been found advantageous in constructing sewers from 2 to 4 ft. in diameter to pour the concrete of both invert and arch at one operation, thus reducing to a minimum the cost of chipping, patching and finishing the inner concrete surface.

On small sewers where the quantity of concrete per linear foot is only a small fraction of a cubic yard, the cost of preparing for mixing and pouring concrete, such as placing mixing boards, hoppers and chutes, which is a considerable item, is the same whether half or all of the sewer is built at one time. The cost of preparation per cubic yard is therefore half as much when all the sewer is built at once as when it is built in two operations. By completing the sewer in one operation the length of open trench required is less than when the invert is first laid and allowed to set before the arch is poured. The cost of handling the forms for a sewer built in one operation is also considerably less than the cost of handling the half-round forms used when the work is done in two operations.

It is usually necessary to construct sewers having flat bottoms in at least two operations, because of the difficulty encountered in securing the desired smooth finish of the invert if the concrete for both invert and arch is poured at one time. The sewer shown in Fig. 138 was built in this way. The invert and side walls were run first, the concrete extending back to the sheeting so that outside forms were not needed. The lowest set of trench timbers, which came 4 in. above these walls, was removed 48 hr. later, the concrete thereafter serving to brace the sheeting. Four days after the arch was poured the centers were collapsed and moved ahead, through other forms in place, and set up ready for another run.

Full Round Wooden Forms.—Figs. 135, 136 and 137 illustrate the wooden forms used in constructing a 3-ft. sewer in one operation. Fig. 135 shows the cylindrical forms and outside jackets in place. The sketch also shows how these forms

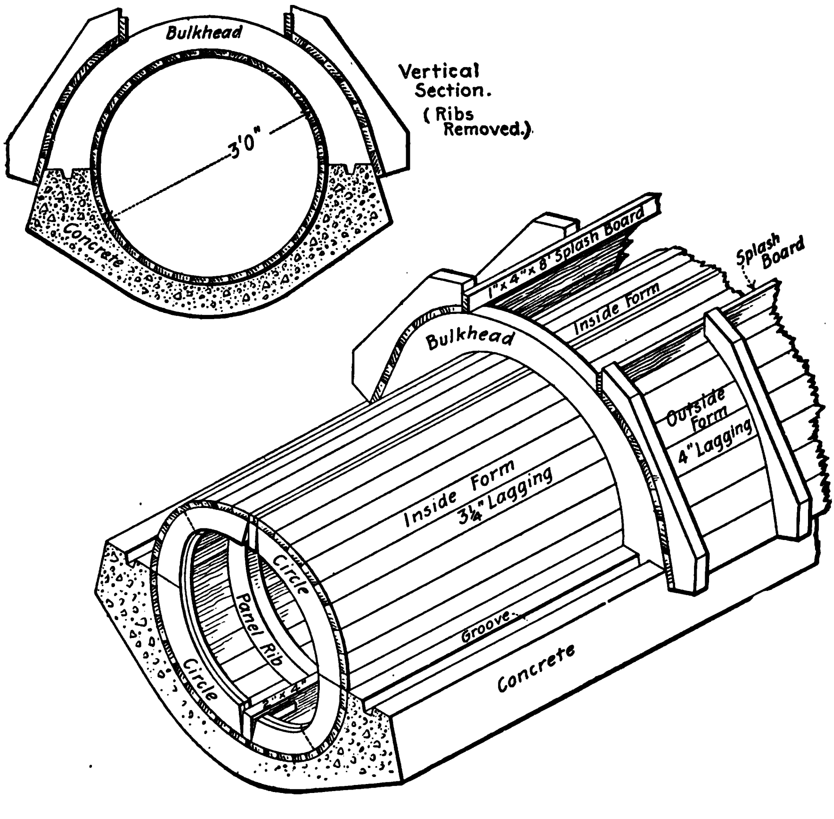


FIG. 135.—Wooden forms for building circular sewer in one or two operations, Louisville, Ky.

can be used for building the sewer in two operations, if that method of procedure is desirable, the concrete of the invert being shown as completed for a portion of the distance and the bulkhead being shown in place to receive the wet concrete of the arch.

The inner forms are built in sections of four panels, shown in detail in Fig. 137. A skeleton support consisting of rings held together by two stringers, one at the top and one at the bottom, is first placed in position in the trench and the panels are placed upon and attached to this support as shown in Fig. 136. When the concrete has acquired sufficient hardness, the stringers holding the skeleton ribs in place are removed, allowing the ribs and panels to be collapsed and carried out through other forms supporting fresh concrete. There is no internal bracing required by these forms, so that the space inside the skeleton ribs is entirely free.

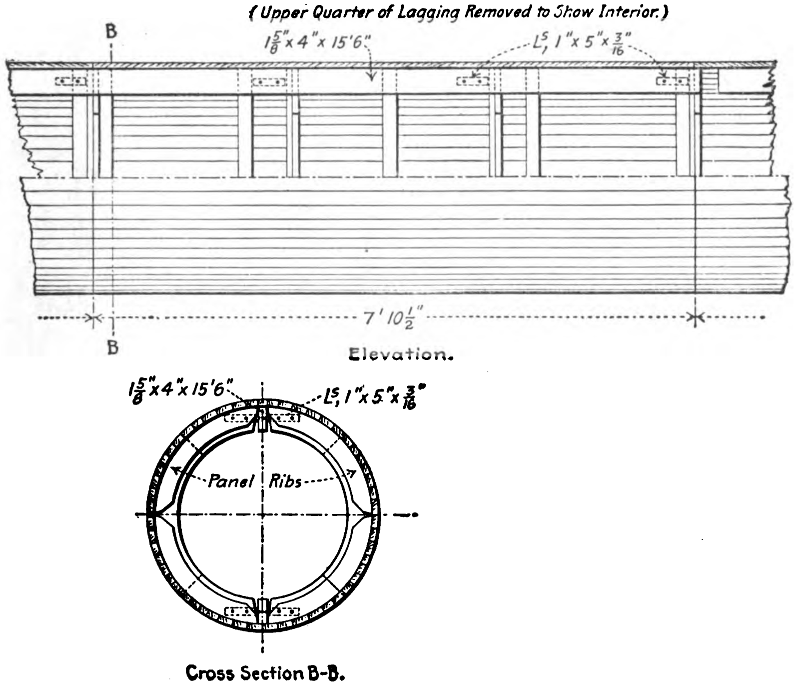


FIG. 136.—Full round wooden forms, Louisville, Ky.

When full round wooden forms are used they should be thoroughly soaked with water before they are placed in the trench, otherwise the water in the concrete will cause them to swell and crack the sewer the whole length of the run. Wooden forms, particularly when new, must be protected from the sun or they will dry out and the joints will open so that when concrete is poured the thin mortar will run through the joints and leave the concrete with a rough surface which must be chipped, pointed and smoothed, involving considerable expense and leaving a sewer not so satisfactory as that built with tight forms. After being used a few times the forms will give no further trouble from swelling. They should be thoroughly cleaned and oiled before each use.

Forms for Sewer Built in Two Operations.—The forms used in the construction of a 12 by 12-ft. semi-elliptical reinforced concrete sewer are illustrated in Fig. 138. It was first intended

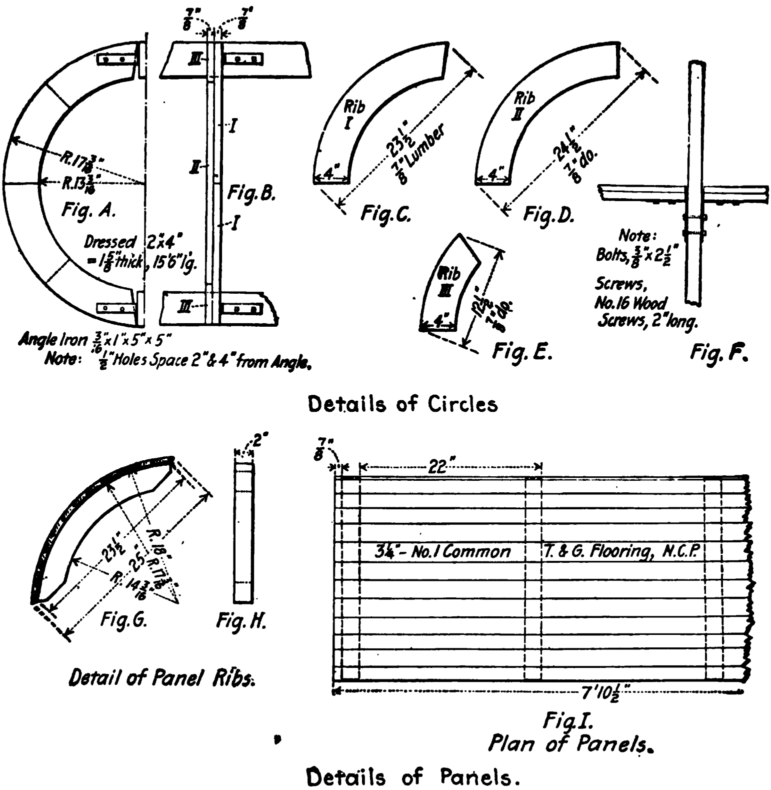


FIG. 137.—Details of full round forms, Louisville, Ky.

to build this sewer in three operations— invert, side walls and arch—and the forms were built with that method in prospect. It was later decided to build the invert and bench walls to the height shown on the drawing in one operation, after which the arch was to be poured. The supplementary bench wall form, *A*, Fig. 138, was therefore provided, but, as the lower portion of the arch form, *B*, had already been constructed, it was used

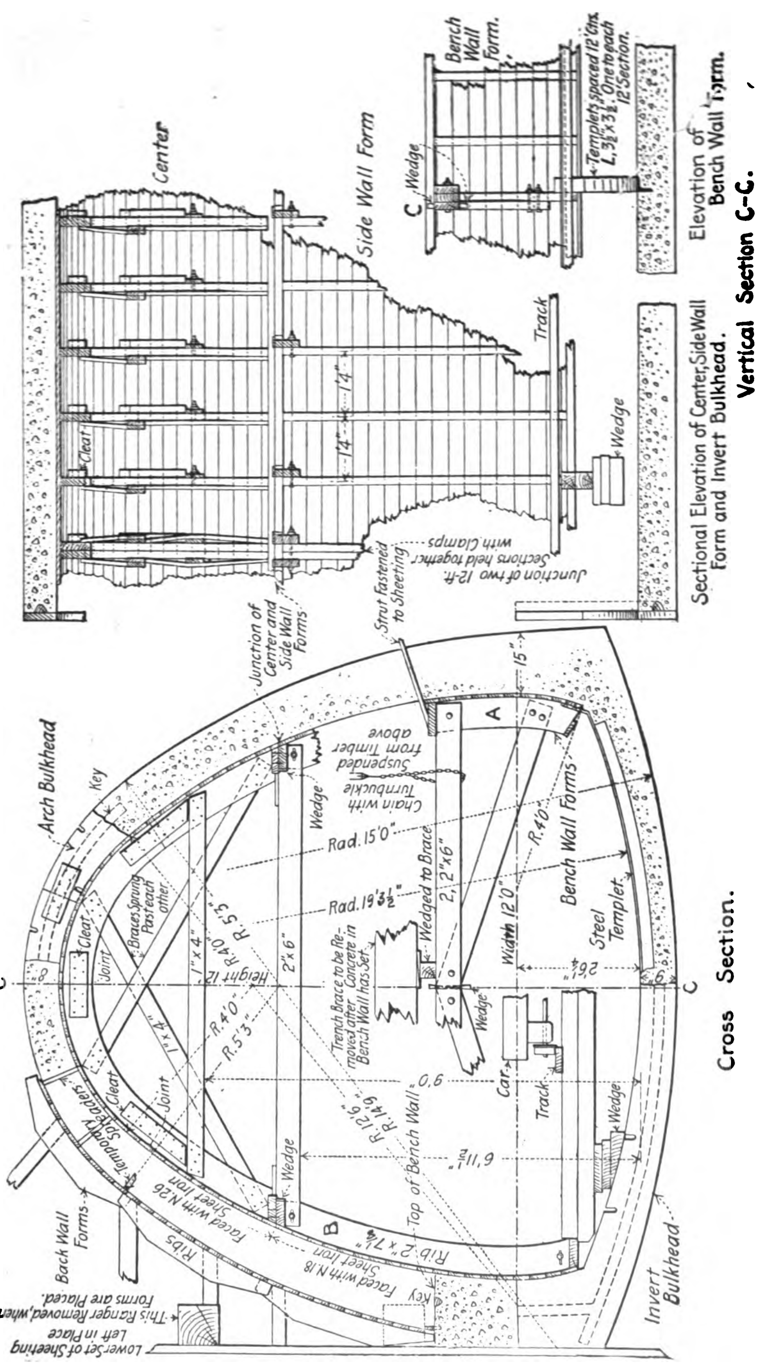


FIG. 138.—Details of wooden forms covered with sheet-iron for semi-elliptical sewer, Louisville, Ky.

throughout the work although had the original intention been to pour the concrete in two operations, this portion of the form would not have extended more than 2 to 4 in. below the top of the bench wall. The sewer was built in sections 48 ft. long.

The bench wall forms with the angle iron templates attached were first placed in correct position as to line and grade, the forms being hung.

Manholes.—Where 8-in. work is used, the brick may be laid as headers with the long sides forming radial joints, or in common

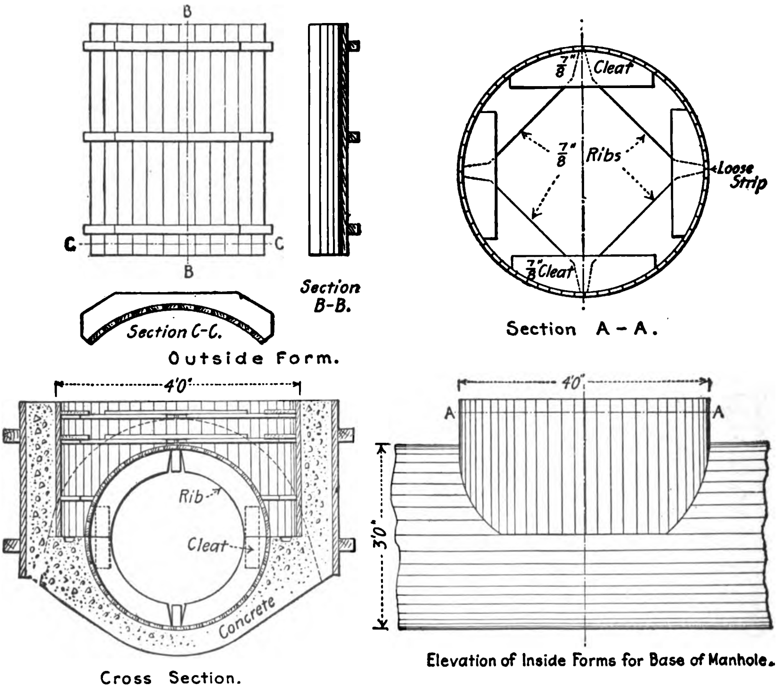


FIG. 139.—Form for manhole base, Louisville, Ky.

bond with several courses forming double rows of stretchers to be followed by a course of headers laid with the long sides making radial joints. In manholes requiring 12 in. of brickwork, the bond is complicated by the fact that if the joints are made radial the outer portion will become very wide. This may be obviated by laying the outer rings with ordinary $\frac{3}{8}$ or $\frac{1}{2}$-in. joints and introducing stretchers or headers, as the case may be, when the opportunity offers, filling in the gaps with broken or half brick.

A good mason can lay the brick by eye so as to conform with accuracy to the prescribed line. On small sewers the pipe are usually left out where manholes are to be built. A foundation of concrete or brick is first laid up to the invert of the sewer, which is then formed either with brick, concrete or a split tile of the same diameter as the sewer.

Great care should be taken in the construction of manholes to have all joints in the brickwork well filled with mortar. A fruitful source of leakage is the joints around the pipes entering the manhole. These should be filled and where possible the hub of the pipe should be laid in the masonry, forming a cut-off.

Fig. 139 illustrates the forms used for building a concrete base for a brick manhole on the 36-in. concrete sewer built with the forms shown in Fig. 135. The section in that illustration shows the inside manhole form in place, attached to the sewer forms, after pouring the concrete of the sewer and the manhole base.

CHAPTER IX

CHEMICAL AND BIOLOGICAL CHARACTERISTICS OF SEWAGE

The successful disposal of sewage depends upon the extent to which the changes that take place in its organic matter and bacterial flora are controlled, so as to prevent the development of undesirable conditions. These changes are chemical and bacteriological, closely interwoven in their nature, and affect the water-borne solids which constitute less than one thousandth part (0.1 of one per cent) of the total weight of the sewage, in most cases.

Many sanitary investigations, the operation of sewage treatment plants, and sometimes the intelligent control of the disposal of sewage in large bodies of water, must depend to a considerable extent upon the information furnished by chemical analyses.

Among the living things which play a part in the disposal of sewage are rats, the scavengers of sewers, in which many live; gulls and other birds which feed upon floating organic matter discharged from sewers; fish, which often congregate about sewer outlets for food; certain plants which thrive on some nitrogen compounds of sewage, and organisms too small to be seen without the aid of a magnifying glass or microscope. The last class of living things appears to be most important, and is separated into two groups, bacterial organisms and plankton. Bacteria are plants, studied with difficulty with a microscope and investigated mainly by their behavior when cultivated under different, carefully standardized conditions. Plankton comprises both plants and animals, is easily studied with the microscope and does not require special culture.

If sewage causes offense, certain bacteria are responsible; if it is oxidized to odorless substances possessing fertilizing value, certain other bacteria are the cause. Chemical analyses show what changes have been wrought in the sewage by the treatment, but they do not reveal the living cause of these changes, upon which successful sewage disposal depends. Chemical analysis of polluted water discloses its general character but not the presence or absence of disease germs. Where sewage has contaminated water not subsequently purified, the resulting ill effects upon the health of the drinkers of the water have been due, so far as is now known, to disease-producing bacteria. For this reason a study of bacteria is essential to a correct understanding of the sewage disposal problem.

The changes taking place in sewage in filter beds are probably due in a very large degree to bacteria. When the sewage is discharged into rivers, ponds or the sea, the changes in it may be largely due to the plankton of these waters. This plankton lives to some extent upon the organic matter and certain parts of the inorganic matter of the sewage. The extent of the work accomplished by the plankton is still undetermined, for little is known of the fundamental principles upon which depend the development and control of these organisms. That their mission is to convert substances like those in sewage into living tissue is clear, but how to take advantage of this action, to accomplish the greatest good under the many conditions encountered, is not evident from present knowledge.

Sampling.—The general character of sewage is best ascertained by mixing samples taken every half hour for 24 hr. and analyzing the composite sample. For very accurate determinations, the quantities of half-hourly samples mixed together should be proportionate to the flow of sewage at the time of sampling. As some of the suspended matter may be quite coarse, it is desirable to take 1 qt. or more of sewage every half hour and pour it into a clean tub or barrel. The contents of the barrel are thoroughly mixed at the end of 24 hr., and from 2 to 4 qt. of the composite are taken for analysis. Care must also be exercised that the small samples taken for laboratory test are fairly representative of the large sample sent in.

Polluted water should ordinarily be analyzed within 12 hr. after sampling, and raw sewage and sewage effluents within 6 hr. Certain determinations, like that for dissolved oxygen, should be made on the spot. If the samples are kept at a low temperature, as in an ice box, the extent of the changes taking place in them will be lessened materially. If samples are shipped to a distant laboratory or are combined in a weekly composite, they must be sterilized by adding chloroform, formaldehyde, mercuric chloride or some other germicide. Certain sterilizing agents interfere with some analytical determinations, so it is sometimes desirable to sterilize two different portions of the same sample, each with a different germicide, as formaldehyde and sulphuric acid. For strong sewage it may be necessary to use the equivalent of 1 to 2 c.c. of the concentrated chemical solution to each 500 c.c. of the sample, while one-half this quantity will suffice for good effluents and for polluted waters.

MEANING OF CHEMICAL ANALYSES

General Characteristics of Sewage.—"Concentration" is a term commonly used to designate the proportion of sewage matter to water in a given sewage. A strong or concentrated sewage contains a relatively larger proportion and a weak or dilute sewage a smaller proportion of sewage matter. There is no recognized standard by which a sewage may be classed as strong or weak. In sewage disposal problems, the organic matter is more important than the mineral matter, and is therefore the usual basis for the classification, which takes into account the organic nitrogen, albuminoid ammonia, oxygen consumed, oxygen demand and volatile solids, the significance of which will be explained later in this chapter. Where any of these determinations is regularly made, it may be taken as a standard of strength although it is much wiser to base judgment upon a consideration of all of them, giving to each the proper weight under the circumstances.

"Composition" is a term used to designate the different ingredients in a given sewage and their amounts. If these amounts in the case of certain of the ingredients are high or low as compared with certain assumed standards, then this sewage can be termed strong or weak. Composition must be determined before concentration can be judged.

The "condition" of a sewage depends upon the changes which have taken place in it since it entered the sewerage system. Although no new ingredients are added to it as it flows toward the treatment or disposal site, changes take place in its composition, due largely to biological action. The changes are mainly in the organic matter, which is made up of complex compounds of carbon, hydrogen, oxygen, nitrogen, sulphur and other elements, more or less readily broken up into other and usually simpler compounds by chemical and biological action. Methods of chemical analysis are largely dependent upon such chemical action, while it is believed that some sewage treatment processes are mainly dependent upon biological action. The objectionable characteristics of sewage must be overcome by removing the matter causing them and disposing of it apart from the rest of the sewage; by transforming it into unobjectionable matter; or by killing directly or indirectly the objectionable living organisms. Frequently all three methods are utilized in a single treatment plant.

Solids or Residue on Evaporation.—This residue is determined by evaporating down a known volume of sewage, and drying the residue at 212°F. In the drying, all gases and small quantities of very volatile substances escape, but their relative quantity is usually very small. The weight of this dry residue is compared with the weight of the original sample of sewage to determine the solids in parts per 1,000,000, the usual method of recording the results of such analyses.

Suspended and Dissolved Solids.—The total residue on evaporation contains the solids which were dissolved in the sewage and those which were in suspension or carried along by it without being dissolved. In many sewage treatment problems, the solids in suspension are more important than those in solution. Their quantity is determined by filtering the sewage, evaporating the filtrate to dryness and determining the dissolved solids by weighing the residue thus obtained, and finally subtracting the quantity of dissolved solids thus determined from the total solids as determined in the manner described in the previous paragraph.

The suspended solids are often determined directly by filtering a known volume of sewage through a Gooch crucible, which consists of a crucible with a perforated bottom provided with an asbestos filter. The residue collecting on the asbestos mat is dried at 212°F. to constant weight and the suspended solids are calculated. This method usually gives higher results than those obtained by the difference of the total solids and dissolved solids found by filtering through ordinary filter paper described above.

Extreme care must be taken in all the steps from the beginning of sampling to the final weighing in order that the proper proportion of all the suspended matter may be included in the final residue and that changes shall not take place in the colloidal matter during the period of sampling and analysis. (See following paragraph on "Colloidal Matter".)

Settling Solids.—The determination of the suspended solids in sewage and in the effluent from the plant where the sewage is treated shows the effect of the treatment on this important part of the matter which the sewage contains. If the sewage is passed through a sand filter practically all the suspended solids are removed, but if the sewage is merely allowed to settle for a few hours in a sedimentation basin smaller proportions of the suspended solids will be removed. The efficiency of sedimentation is sometimes expressed by noting the percentage of the suspended solids removed, but this may be misleading because it is impossible to remove all the suspended solids by any practical method of sedimentation. It may be preferable, therefore, in some cases to state what percentage of the solids removed in the laboratory under standard conditions by sedimentation are actually removed in the settling basins, since this figure will show the practical efficiency of the process. In the operation of the large sewage treatment works at Providence, R. I., this determination is made in the following manner: The sample of the sewage which represents its character during each 24-hr. period is well shaken and a portion of this is analyzed to determine the suspended solids in the raw sewage as it arrives at the works. The 24-hr. sample is allowed to stand quiet for 4 hr., the time taken by the sewage to pass through the sedimentation basins, and another portion of the sample is then decanted and its suspended solids determined. The difference between the two determinations is assumed to be the quantity of suspended matter in the Providence sewage which will settle in 4 hr., and the actual performance of the settling basins is referred to this quantity in order to ascertain how closely the sedimentation in the basins approached the practicable maximum.

Another method of determining the quantity of settling solids is to place a measured volume of sewage in a conical glass, 4 in. in diameter at the top, the size of a thimble at the bottom, 17 in. tall and graduated at the lower part to show the number of cubic centimeters below each mark. A liter sample of sewage is placed in one of these cones, called Imhoff glasses, and allowed to stand for the number of hours which the sewage is assumed to remain in the sedimentation basins. At the end of this time there will be a certain amount of sludge in the bottom of the glass and the quantity is measured by the graduations. This method of determination is liable to serious errors if the suspended matter is coarse, for the larger pieces tend to clog the narrow tapering bottom of the glass and thus prevent the solids from filling the space below them. For this reason such glasses are more useful in determining the settling solids in effluents than in raw sewage.

Colloidal Matter.—As a practical matter, a substantial portion of the solids usually reported as being in suspension when the determination is made by filtration, evaporating and weighing, may be considered as being in a colloidal state. Although there is no standard method at present of determining the quantity of colloidal matter, it is certain that such substances are of much importance in sewage treatment, especially where industrial wastes are discharged into the sewers. Colloids are substances, it should be kept in mind, which while soluble as judged by ordinary physical tests will not pass through a parchment membrane. Substances which, while in solution, will pass through such a membrane are termed crystalloids.

Practically speaking, the term "colloids" is often used in discussing sewage treatment problems to include the very finely divided suspended matter which will not readily settle in ordinary sedimentation basins, and true colloid substances, which can hardly be classed as finely divided suspended matter in the sense that they are visible to the naked eye, but consist of substances which, under certain conditions, may be thrown out of their state of pseudo-solution and retained mechanically upon the surfaces of tank walls or filtering materials.

Fixed Residue, Fixed Solids or Mineral Matter.—A definite quantity of sewage is evaporated to dryness, the residue weighed to give the total solids, and this residue is then ignited at a relatively low temperature to burn off the organic matter. The weight of the remaining matter gives approximately the amount of mineral matter in the original sample.

Loss on Ignition, or Organic Matter.—The difference between the weights of the total residue and the fixed residue represents approximately the quantity of organic matter in the original sample. The test merely gives an idea of the quantity but not of the character of the organic matter present in the sample, and thus does not show how this matter will behave under artificial or natural conditions. This behavior is very important in sewage treatment and can be best illustrated by explaining the nitrogen cycle of change in organic matter.

Nitrogen Cycle.—The nitrogen cycle is shown diagrammatically in Fig. 140. Dead organic matter is decomposed through the process of decay, sometimes accompanied by putrefaction, and the nitrogen it contained appears first as free ammonia. Later oxidation proceeds and the ammonia is first changed to nitrites and then to nitrates. The nitrates are good food for plants, which convert them into living complex nitrogenous organic matter or proteins. Eventually the plants die and decay or they are eaten by animals, a part being converted into animal tissue which will later die and decay. Another part is converted into waste substances like urea, which is easily oxidized, and is thus changed into ammonia without going through the complete cycle. There is another deviation from the complete cycle due to the fact that certain bacteria reduce nitrates to nitrogen gas, a process termed "denitrification." If there were no compensation for this the amount of nitrogen in the soil available for vegetation would gradually become exhausted, but fortunately there are other bacteria which have the power to use the atmospheric nitrogen for the formation of their own substance, a process termed "nitrogen fixation."

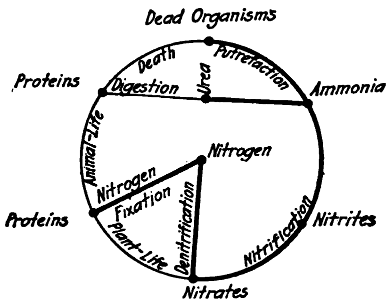


FIG. 140.—Nitrogen cycle (after Rahn). (From "Microbiology," edited by Marshall, page 126.)

As discharged into the sewers, some of the organic nitrogenous substances are relatively stable and others only loosely bound together. Some of the nitrogen is even present in inorganic form, as ammonia and ammonium compounds. The organic substances are easily broken up directly or indirectly by bacterial action, into other substances. Some of the nitrogen which at first was loosely bound is liberated from its organic combination and may escape as gaseous nitrogen or it may be changed into some inorganic form such as ammonia. The nitrogen of other nitrogenous substances may first become part of less complex and less stable matters only to be changed later into gaseous nitrogen or into inorganic matter if the process is allowed to proceed far enough. However, some nitrogenous organic compounds will persist for a very long time.

In the treatment of sewage by bacterial action, much depends upon the conditions under which the processes are carried on. Sometimes the action is conducted in the absence of air, as in airtight or air-trapped tanks, and the changes are confined to an interchange of the elementary chemical substances present in the sewage. This process is often termed "hydrolysis," because it is generally accompanied by a breaking up of the molecules of water and the combination of the hydrogen and oxygen of the water with carbon, nitrogen and other substances from the organic matter. In other cases, the bacterial action goes on in the presence of an abundant supply of oxygen, furnished either directly from the atmosphere or from that dissolved in relatively pure water. Under such circumstances the action is one of oxidation, the organic substances being split up and combining with the oxygen, the nitrogen finally appearing as nitrites or nitrates of some base or alkali, as sodium nitrite ($NaNO_2$) and sodium nitrate ($NaNO_3$).

There are accurate, convenient methods for determining the quantity of nitrogen present in sewage and of ascertaining whether it is in relatively stable or unstable combination as organic matter or in any one of the three combinations just mentioned as indicating progressive change from the organic substance to the highly oxidized mineral compound, nitrates. It is this characteristic of the determination of nitrogen compounds which has given them so much importance in sewage analysis, rather than any particular significance in the quantity of nitrogen present. If specific information is desired as to the quantity of trouble-making matter present, the determination of organic sulphur may throw more light on the problem than the nitrogen determination.

Organic or Kjeldahl Nitrogen.—This is the nitrogen combined with carbon and other elements in the form of organic matter; it does not include nitrogen as free ammonia, nitrites and nitrates. The total quantity of nitrogen is determined by prolonged boiling with strong acid and finally distilling the ammonia thus formed. This treatment is severe enough to break up the more stable organic nitrogenous substances and to liberate the nitrogen in the form of free ammonia. By determining the total organic matter in a filtered sample information is obtained concerning the quantity of dissolved nitrogenous organic matter, and, by difference, the quantity of such matter in suspension in the sewage.

Albuminoid Ammonia.—Organic nitrogenous substances may be divided into two classes, those readily yielding ammonia by boiling with alkaline permanganate of potash for a short time, and those which are not so decomposed. The nitrogen from the latter class is sometimes called "residual organic nitrogen." The ammonia formed from the former class by the boiling process mentioned is distilled off and accurately measured, the result being usually reported as "nitrogen as albuminoid ammonia" because the nitrogen of albumin may be driven off in this form by similar treatment. Albuminoid ammonia does not exist as such a compound in sewage, but the test results are a measure of the quantity of the less stable nitrogenous organic substances present. If the results are reported as ammonia, as they sometimes are, they may be expressed in terms of nitrogen by multiplying the amount of albuminoid ammonia by the factor $^{14}/_{17} = 0.82$.

A knowledge of the quantity of nitrogen as albuminoid ammonia in sewage and effluents may be of considerable value to the operator of treatment plants, and in many cases it is well worth determining and may be more instructive than the determination of total organic nitrogen. In studying sewages from different cities or the condition of sewage from the same city at different times, it is often desirable to determine both the nitrogen as albuminoid ammonia and the total organic nitrogen. By deducting the former from the latter some idea may be formed of the relative quantity of the more stable forms.

Free Ammonia.—For many years free ammonia has been determined by distillation, but the method is uncertain because some of the unstable substances in sewage, such as urea, give off their nitrogen when heated to the degree required for distillation. It is practicable in many cases to determine the free ammonia directly in the sample, without distillation, and where the results are reliable this method is preferable.

As free ammonia is a decomposition product, the quantity present is a valuable indication of the freshness of the sewage. The fluctuations in the quantity of free ammonia in the effluents from some treatment processes are a helpful guide to the efficiency of the biological action upon which the treatment depends. While free ammonia may or may not indicate an unfavorable condition of the treatment plant, it does not of itself constitute a source of offensive odor and it does not ordinarily represent organic matter which will decompose and create offensive conditions.

Nitrites and Nitrates.—The determination of nitrogen as nitrites and nitrates is instructive as regards the efficiency of the last step in the complete biological purification of sewage.

The quantity of nitrites is determined by adding to the sample quantities of acetic acid solutions of sulphanilic acid and *a*-naphthylamine, which produce a pink color if nitrites are present. The depth of color thus produced is compared with a set of standards containing progressively increasing quantities of sodium nitrite and treated in the same way. The sample contains the same quantity of nitrites as the standard it matches.

Nitrates may be determined by a colorimetric method similar to that used for determining nitrites. The sample after evaporation is treated with phenolsulphonic acid and made slightly alkaline. If nitrates are present a yellow color is produced, the depth of which, when compared with similarly treated standard solutions of potassium nitrate of known strength, indicates the quantity of nitrates in the sample. This method is not applicable to water containing more than 30 parts per million of chlorine, as is sometimes the case with sewage and often with industrial wastes. Such samples should be treated by the reduction method, by which the nitrates are reduced to ammonia by nascent hydrogen generated from strips of aluminum foil placed in the samples after they have been made alkaline with potassium hydrate. After reduction the ammonia is distilled off and measured as in the determination of free ammonia.

Oxygen Consumed.—Determinations of nitrogen do not give any information about the quantity of organic matter in which nitrogen is absent. As a measure of the carbonaceous as well as nitrogenous organic matter, the so-called determination of "oxygen consumed," "oxygen absorbed" or "oxygen required" has long been used. The sample for this test is accurately measured, acidulated and treated with a known quantity of a solution of potassium permanganate of standard strength. It is then placed in a bath of boiling water and digested for exactly 30 min., in the standard method of the American Public Health Association, although the practice in some laboratories differs at this stage of the test. The quantity of the oxidizing agent remaining after digesting is measured and the quantity of oxygen used up or absorbed by the sample is computed. This determination is, in a way, like the determination of the loss on ignition, except that it is wet instead of dry combustion, and is carried out in such a manner that the quantity of oxygen absorbed is under control and subject to accurate measurement.

Unfortunately the combustion is never complete, so that the quantity of oxygen absorbed is only that used up by the more easily attacked substances and the quantity consumed depends largely upon the length of time the digestion is continued. Thus many different results may be obtained from the treatment of the same sample for different periods of time, and different sewages may yield widely different results with the same treatment, depending upon the proportion of the readily oxidizable substances contained in them. This determination is, therefore, of most service when used for comparing the quality of sewages from the same community from day to day, or comparing a sewage with the corresponding effluent from a plant in which it has been treated. It must not be forgotten, however, that the organic matter indicated by the oxygen-consumed test of an effluent may not be in the same form as that in the original sewage.

Putrescibility Tests.—The consumption of oxygen in the oxygen consumed test, due to chemical action, is paralleled in nature when the original organic substances are broken up, largely by the action of living organisms, and the carbon and oxygen are united in a stable combination, substantially unattended by disagreeable odors. The oxygen required to oxidize the dissolved and suspended matters is derived from the supply dissolved in the water, including the oxygen combined with nitrogen in the form of nitrites and nitrates. If there is an insufficient supply of this oxygen, an entirely different biological process will set in, due like that of oxidation to the work of living organisms, but of a different kind. The organic matter will be broken up as in the other process, but the products of this process will contain among them offensive smelling matter such as hydrogen sulphide, not formed in the presence of an abundant supply of oxygen.

As oxygen is required to prevent putrefaction and as water can dissolve only a limited quantity of oxygen, an effort has been made to determine the stability or putrescibility of samples by determining the quantity of oxygen dissolved in them after standing for different periods of time, adding thereto the oxygen available from nitrites and nitrates, although the quantity which can be derived from the latter is often negligible. By observing the loss in oxygen due to the changes going on in the sample, a knowledge of its putrescibility may be obtained. If the oxygen is not all used up the sample will prove stable.

The standard putrescibility test of the American Public Health Association is made by placing a sample of the sewage in a 150- to 200-c.c. glass-stoppered bottle to which a small quantity of methylene blue is added. The sample is then incubated, preferably at 20°C. during 4 days, observations of the color being made at least once a day. If the sample is stable the blue color of the chemical will persist. If the sample is putrescible, the color will disappear on account of the reducing action of the sulphur compounds, provided the sewage or effluent contains no trace of hypochlorites which may have been used in disinfecting it. This test is based on the action of the organisms in the sample. If there is sufficient organic matter of the right kind to maintain their existence in adequate numbers, they will exhaust the supply of oxygen and the biological action taking place will be changed from oxidation to putrefaction. If the quantity and nature of the organic matter are unsuited for the rapid growth of the bacteria, there will be an ample supply of oxygen to meet the requirements of such changes as go on in the organic matter, and the process will be one of oxidation throughout and will not be attended by the production of offensive odors.

Table 37 gives the relative stability numbers corresponding to the time of incubation at both 20°C. (68°F.) and 37°C. (98.6°F. blood temperature). The relative stability number is assumed to indicate the ratio of available oxygen in the sample to that required for the complete oxidation of the organic matter. For example, if a sample of effluent when incubated at 20°C. retains its color only until the end of the third day, it is said to have a stability number of 50, which means that it contained, as disolved oxygen and oxygen in nitrites and nitrates, one-half of the oxygen required for complete oxidation of the organic matter. The American Public Health Association's committee which recommended this test stated that if a sample retains its color for 4 days it may be considered as practically stable, except where great accuracy is desired, but the authors believe that the additional information obtained by continuing the incubation for 14 days at 20°C. is well worth the labor involved.

TABLE 37.—RELATIVE STABILITY NUMBERS
("Standard Methods of Water Analysis," American Public Health Association)

t_{20}	t_{37}	S	t_{20}	t_{37}	S	t_{20}	t_{37}	S	t_{20}	t_{37}	S
0.5	...	11	3.0	1.5	50	8.0	4.0	84	13.0	6.5	95
1.0	0.5	21	4.0	2.0	60	9.0	4.5	87	14.0	7.0	96
1.5	...	30	5.0	2.5	68	10.0	5.0	90	16.0	8.0	97
2.0	1.0	37	6.0	3.0	75	11.0	5 5	92	18.0	9.0	98
2.5	...	44	7.0	3.5	80	12.0	6.0	94	20.0	10.0	99

S = Relative stability or ratio of available oxygen to oxygen required for equilibrium; expressed in percentages.
t_{20} = Time in days to decolorize methylene blue at 20°C.
t_{37} = Time to decolorize at 37°C.

This methylene-blue test requires only simple, inexpensive apparatus and can be carried out by persons without a knowledge of chemistry. It has been quite generally adopted as a routine test for the control of sewage treatment and dilution. Where merely treatment to insure freedom from offensive conditions is needed, it will be found a useful test and perhaps, in some cases, the only one required.

Biochemical Oxygen Demand.—The putrescibility test shows the *relative* amount of oxygen required to prevent putrefactive conditions. It is often important, however, to measure the *actual* oxygen requirements of a sewage, effluent or trade waste.

The amount of oxygen used up by a sewage or waste, in a stated period of time at a definite temperature, usually 20°C., is called the "biochemical oxygen demand." It has been defined by the U. S. Public Health Service as follows:

"The biochemical oxygen demand of a polluted water, a sewage or an industrial waste, is defined as the milligrams of oxygen per liter required for the stabilization of its organic matter by aerobic bacterial action." This may be determined by the following methods:

The relative stability method consists in adding to the sample to be examined a proper volume of aerated tap water of known dissolved oxygen content, a small amount of methylene blue, and incubating the mixture at 20°C. until the blue color disappears. The biochemical oxygen demand is then calculated from the initial dissolved oxygen content, the relative stability number, the ratio of available oxygen to oxygen required for equilibrium or stability, and the dilution.

The method described above is often modified by omitting the methylene blue and adding to the sample sufficient aerated tap water to ensure residual dissolved oxygen (of about 30 per cent saturation) at the end of a 10-day period of incubation. The difference between the oxygen in the initial diluted sample, and in the sample after incubation, gives the amount of oxygen required by the amount of the original sample that was used for dilution.

The nitrate method is based on the biochemical consumption of oxygen from sodium nitrate. In this case, a measured amount of sodium nitrate solution is added to the sewage in tightly stoppered bottles and incubated at 20°C. for 10 days. At the end of the period of incubation, the residual nitrite and nitrate is determined in the sample. The biochemical oxygen demand is the difference between the available oxygen added as sodium nitrate and that found at the end of the period of incubation as nitrates and nitrites.

In the determination of the biochemical oxygen demand of sewage or of certain trade wastes, it is usually not necessary to include in the initial oxygen content, the amount of oxygen present in the sample in the form of dissolved oxygen, nitrites or nitrates, as the amount present is relatively small as compared with the amount required. However, in the determination of the biochemical oxygen demand of effluents, polluted streams, or relatively weak trade wastes that have a comparatively low demand for oxygen, it becomes necessary to determine the amount present in the form of dissolved oxygen, nitrites and nitrates and include it in the initial oxygen content.

It should be noted here, that in making this determination a sterilized sample cannot be used and if the sample is acid or possesses hydrate alkalinity, bacterial activity will be inhibited.

In order to obtain comparable results by any one method, care must be taken that the temperature of incubation remains practically constant, that no oxygen is liberated during incubation and that the time of incubation is the same in all cases.

It is not possible to obtain results that check closely on all samples when determined by the different methods, due possibly to the selective action of the bacteria as well as to the variation in the speed of the reaction as the supply of available oxygen is diminished.

Dissolved Oxygen.—It is important not to confuse the determination of dissolved oxygen with that reported as oxygen "consumed," "required" or "absorbed." The results reported as dissolved oxygen are obtained by the determination of the quantity of atmospheric oxygen which is dissolved in a given sample of water or sewage. Pure water constantly in contact with the atmosphere is capable of dissolving only a definite quantity of oxygen at any temperature and atmospheric pressure. The quantity varies inversely with the temperature. Pure water dissolves more oxygen than does impure water. Soft surface water will dissolve more than hard ground water of sea water. Water is said to be saturated with dissolved oxygen when it contains all the quantity it is capable of dissolving when intimately mixed with air at the specified temperature and pressure. Water usually contains less than this amount and the results of dissolved oxygen tests are reported in percentages of saturation, as 60 per cent saturation. This method of stating the results may cause confusion unless the facts just stated are kept in mind. For example, if the oxygen has been used up by biological processes in two waters at temperatures of 15 and 27°C. respectively, until the percentage of saturation has fallen to 50 in each case, it is not true that the same quantity of oxygen has been used up in the two cases nor that the same quantity remains. The dissolved oxygen remaining is 5.07 and 4.03 parts per million respectively. The water at the lower temperature contains about 25 per cent more oxygen than the other in spite of the fact that its loss of oxygen has been 25 per cent greater.

Chlorine.—Common salt, sodium chloride, is present in all water supplies and is also an ingredient of food. It occurs in relatively large quantities in kitchen wastes, wash waters, urine and feces. Salt is therefore found in greater quantities in sewage than in most natural waters, and its presence in the latter in excess of that normal to them indicates their probable contamination by sewage. As the chlorine which is combined with the sodium in salt can be easily and accurately determined, it is customary to analyze the water or sewage for this only, disregarding the sodium and reporting the result in terms of chlorine.

The quantity of chlorine per inhabitant reaching the sewers in a given unit of time is practically uniform; therefore the chlorine content of the sewage may be used to measure its concentration. To do this it is necessary to know the quantity of chlorine in the water supply and also any possible source of an unusually large quantity of chlorine discharged into the sewers. Similarly the quantity of water entering the sewers may sometimes be ascertained by the degree of dilution afforded the sewage, the chlorine content of the ground water being known. The proportion of ground water in the effluents of filters and irrigation fields can be ascertained in the same way.

Appearance.—The turbidity, sediment and color of the effluents from sewage treatment works and of water into which sewage and effluents are discharged are frequently observed and recorded along with the results of chemical analyses. It is generally sufficient to designate the turbidity as "slight," "distinct," "decided," "milky" and the like. Sediment is more rarely reported, and then it is sufficient to record it as "slight," "distinct" or "decided." Where industrial wastes enter the sewage it may be important to report the color in general terms, but this is not usually desirable.

Odor.—The odor of an effluent or of water containing sewage or effluent may furnish valuable information, but this is rarely the case with sewage. The odor is best determined by half filling a bottle holding 2 to 4 qt. with the water or effluent, corking it, shaking it vigorously for about a minute, and then applying the nose quickly to the mouth of the bottle. Where the odors are due to gases dissolved in the water they are sometimes so completely liberated by the shaking that a second shaking will produce no further odor, and for this reason a second sample should be used in case of doubt. Some samples which yield only a slight odor when cold will give off a decided odor when heated.

While it is difficult to convey to others the exact meaning of arbitrary terms adopted for designating the character and strength of odors the authors have found the observation of odors helpful, particularly to the operator of filters of the types commonly used. Many years ago they adopted in connection with the operation of intermittent sand filters at Worcester, Mass., a scale of odors of the effluent to indicate the efficiency and condition of the filters. This scale was: 0, no odor; 1, slightly musty; 2, distinctly musty; 3, decidedly musty; 4, offensive; 5, very offensive. With these observations of odor there were parallel tests of the putrescibility of the effluent. Such samples as had no odor or but a slight musty odor were always non-putrescible. Samples which were 3 or higher on the scale of odor were putrescible almost without exception, while samples observed as 2 were generally non-putrescible. Such a scale of odors can be worked out by which the quality of the effluent from other treatment plants can be judged fairly well without waiting for the results of chemical analyses.

CHANGES IN SEWAGE CAUSED BY BACTERIA

Comparatively little study has been devoted to the isolation, identification and life processes of the species of sewage bacteria. The natural biological treatment of sewage appears to be through such control of conditions as will cause the vigorous predominant growth of species capable of bringing about desired changes. Such treatment is governed by more or less accurate knowledge of the work accomplished by masses of bacteria and the results to be accomplished by providing different working conditions for them, rather than by a knowledge of bacteria themselves and of their individual characteristics. For such purposes, the grouping of bacteria into three classes originally proposed by Pasteur is more helpful than some of the later classifications made by bacteriologists. Pasteur called those bacteria "aerobic" which can exercise their functions only when air is present; others, called "anaerobic," can function only when all oxygen is absent; the third class, called "facultative," can function under either aerobic or anaerobic conditions although not always with equal vigor.

Number of Bacteria in Sewage.—The organic and mineral matter in sewage usually affords suitable food for bacteria, which multiply in it with extraordinary rapidity at a favorable temperature, such as 20°C. Experiments made at Lawrence[1] with a sample of sewage placed in a bottle and kept at a temperature of 16° to 20°C. showed an increase in the number of bacteria from 1,190,000 per cubic centimeter after a lapse of 2 hr. to 23,475,000 after 25½ hr. Then the number began to decrease and at the end of 192 hr. it was 2,341,000. This decrease was probably due in part to the diminution in the food

[1] *Report* Mass. State Board of Health, 1894, p. 461.

supply and the dissolved oxygen, which disappeared entirely in less than 24 hr. after the sample was taken, and in part to the increase in the quantity of those products of bacterial action which check such action.

Such figures show that the number of bacteria to be expected in sewage will vary with the age of the sewage at the place where samples are taken. Manifestly it will also be affected by the dilution of the sewage by water. It was found at the Lawrence Experiment Station that fresh sewage from toilet rooms contained an average of 2,850,000 to 3,160,000 bacteria per cubic centimeter, while sewage sampled after flowing from the city to the Station averaged 1,136,000 to 2,110,000 bacteria. At Columbus, Ohio, George A. Johnson found the number was 320,000 to 27,000,000, averaging 3,600,000. In Philadelphia, George S. Webster found from 1,100,000 to 5,800,000, the average being 3,000,000. The more concentrated sewage of English cities contains from 10,000,000 to 100,000,000 bacteria, according to the Royal Commission on Sewage Disposal. The number varies also with the hour at which the samples are taken.

The number of bacteria may also be affected by industrial wastes discharged into the sewers. Wastes from packing houses and some tanneries contain materials stimulating the increase of bacteria, whereas pickling liquids from galvanizing works, being germicides, tend to reduce the number. Paper-mill wastes are practically free from bacteria in some cases, and if organisms are introduced into them, the bacteria will not multiply sufficiently in a reasonable length of time to be effective in causing biological changes, unless the wastes are diluted with water or sewage or treated chemically to overcome their germicidal nature.

Breaking Down of Complex Substances.—The bacterial decomposition of sewage is often carried out by several species of organisms working in relays, so to speak. Each utilizes in turn some of the decomposition products of the life processes of the preceding organisms. If it were possible to control these processes, sewage might be passed from tank to tank or filter to filter, in each of which only one process would take place.

The multiplication of bacteria in sewage is attended by the formation of ammonia, carbon dioxide, and sometimes hydrogen, nitrogen and methane. Some organisms produce hydrogen sulphide. In all these changes, it is important to remember, there is no loss of matter, only a change in its form and characteristics. The changes in nitrogenous matter were shown in Fig. 140, page 286. A somewhat similar change occurs in carbonaceous matter, illustrated in Fig. 141. At the death of the organisms the first stage in the cycle of change is that of decay, when putrefaction may perform an important function. This stage ends with the oxidation of the carbon into carbon dioxide gas or carbonates. In the form of carbon dioxide the carbon is food for green-plant life and is converted by it into such complex organic substances as carbohydrates, fats and proteins. The mature plants die and the elements of which they are composed may at once recommence the cycle or the plants may serve as food for animals, by which they are converted into animal tissue such as fats and proteins.

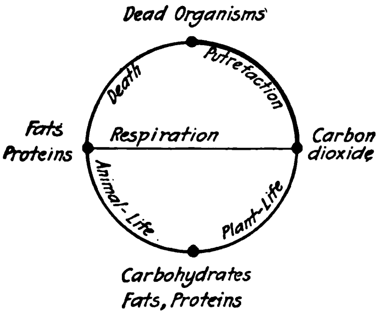


FIG. 141.—Carbon cycle (after Rahn). (From "Microbiology," edited by Marshall, page 125.)

Another process of evolution important in sewage disposal problems is the sulphur cycle, Fig. 142. Sulphur is always present in sewage ready to enter into any biological process, although varying greatly in quantity. The oxygen dissolved in sewage is soon exhausted by the bacterial processes going on and the organisms then appear to appropriate the oxygen of some of the mineral and organic substances in the sewage. If sulphates are present they will yield their oxygen, and the resulting sulphides may be decomposed with the formation of hydrogen sulphide and the production of offensive odors. These odors may be due not only to hydrogen sulphide but also to volatile substances such as indol, skatol, cadaverin and mercaptan, about which little is known. If sewage can be supplied with dissolved oxygen, its decomposition will not be attended by the offensive odors so evident when it

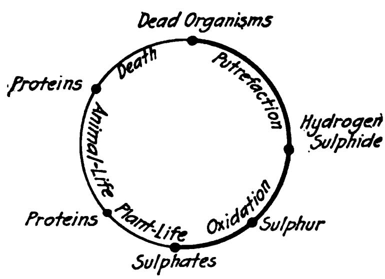


FIG. 142.—Sulphur cycle (after Rahn). (From "Microbiology," edited by Marshall, page 127.)

is undergoing anaerobic changes, and therefore an effort is usually made to bring about the desired changes in sewage under aerobic conditions.

After the organic sulphur-bearing compounds become mineralized, with the formation of hydrogen sulphide, which does not appear suitable for the nutrition of higher plants, another transformation is brought about by the so-called sulphur bacteria, capable of oxidizing the hydrogen sulphide, provided oxygen is present for this process. These organisms produce sulphates, which are taken up by plants, and the cycle thereafter continues like those already explained.

Decomposition of Sewage with Diminution of Oxygen.—There are several terms much used in connection with the treatment and disposal of sewage whose significance is based upon the bacterial changes described in this chapter. The sewage which reaches the public sewers contains free oxygen so that aerobic bacteria thrive, and in this condition is termed "fresh." Unless sewage is vigorously agitated it absorbs oxygen from the air very slowly, and in the course of several hours the oxygen is greatly depleted or exhausted, if any aerobic bacteria are present their vitality is waning, and the sewage is termed "stale." The facultative bacteria have been thriving while these changes were taking place. After the last traces of oxygen disappear the sewage becomes very dark in color and various sulphides, including hydrogen sulphides, are formed. Sewage in this stage is termed "septic."

When sewage stands for a long time or flows very slowly through tanks, the anaerobic organisms gain the ascendancy, and ammonia, marsh gas, carbon dioxide, hydrogen, nitrogen and other gases may be formed as the complex organic substances are broken down into simpler ones. This is a reduction or septic process.

Oxidation Processes.—The process of oxidation of free ammonia to nitrites and nitrates, already explained in its chemical features, is called "nitrification." That it is due to bacterial action is proved by the fact that it is checked or stopped entirely by low temperatures, by heating to the temperature of sterilization, and by germicides. Furthermore, temperatures known to be favorable to bacterial activity cause nitrification to proceed rapidly.

In treating sewage by irrigation and intermittent filtration the production of high nitrates in the effluent is evidence of practically complete oxidation. Where sewage is applied to beds of very coarse material and conditions do not closely simulate those of natural soil, nitrates are not always found in the effluent in such large quantities and consequently organisms producing them are not always regarded as so essential as formerly.

A number of species of bacteria called nitrite-formers have been isolated, but apparently only one species is present in one locality. Like the sulphur-oxidizing bacteria, they thrive on inorganic matter and are unfavorably affected by organic matter. Their power to convert ammonia into nitrite is apparently dependent upon the presence of carbon dioxide.

The nitrate-formers are also capable of growing upon inorganic media. They can obtain their carbon from carbon dioxide or from loosely bound carbonates. While not so sensitive to organic matter as are the nitrite-formers, its presence interferes with their development although not to such a degree with their ability to oxidize nitrites. A small quantity of ammonia will arrest their development.

Under certain conditions a reduction or deoxidation process may become established in some types of sewage filters. The nitrates and nitrites appear to lose oxygen, and ammonia or nitrogen or both may be liberated. These changes may be due to strictly chemical reactions or may be due to the action of some species of bacteria; they are termed "denitrification" and the organisms producing them grow under both aerobic and anaerobic conditions provided nitrates or nitrites are present.

Detection of Contamination by Test Bacteria.—Bacterial analysis is capable of disclosing the presence of much smaller proportions of sewage in water than can be detected by chemical analysis. The contamination is traced by species of bacteria originating in the human intestinal tract. Of the three important groups of sewage bacteria, B. coli, B. enteritidis sporogenes and sewage streptococci, the first is universally found in human urine and feces and is the commonly accepted test organism.

There are many varieties of coli bacteria, none of which is considered harmful to man. Some of these varieties are much more significant of the contamination of water by sewage than others, and in stating the number of B. coli present in a sample of water, only those varieties which presumably indicate sewage contamination are reported. The importance of these tests is due to the fact that sewage contamination may mean that the germs of water-borne diseases are finding their way into water used for drinking purposes. The relative degree of contamination is indicated by the numbers of B. coli and of all species of bacteria in the different samples of water. A rough classification of different degrees of contamination, made for the International Joint Commission[1] by A. J. McLaughlin, John W. F. McCullough, John A. Amyot and Frederick A. Dallyn is as follows:

1. Relatively pure waters; under 2 B. coli per 100 c.c., under 10 bacteria of all kinds per cubic centimeter on agar at 37°C.
2. Slight pollution of relatively pure water; 2 to 10 B. coli per 100 c.c., 10 to 25 bacteria of all kinds per cubic centimeter.
3. Considerable pollution; 10 to 20 B. coli per 100 c.c.; 25 to 50 bacteria of all kinds per cubic centimeter.
4. Serious pollution; 20 to 50 B. coli per 100 c.c.; 50 to 100 bacteria of all kinds per cubic centimeter.
5. Gross pollution; over 50 B. coli per 100 c.c.; over 100 bacteria of all kinds per cubic centimeter.

Typhoid Fever.—B. typhosus, the bacillus of typhoid fever, is present in enormous numbers in the feces and urine of persons having that disease. Some of these persons continue to discharge the bacilli long after they have apparently returned to normal health and are called "typhoid carriers" in consequence. When the bacilli are discharged into water some of them may live for weeks and months, and if this water is drunk the drinker is in danger of becoming infected. In a general way, B. coli and B. typhosus show much the same life characteristics, the former being somewhat more resistant to unfavorable conditions.

While the germs of the disease are transmitted by milk, shellfish, flies, new vegetables and fruits, by contact, and by various means now unknown, the chief mode of transmittal in the past has been by water. It is certain that the contamination of municipal water supplies by sewage is likely to result in typhoid epidemics[2] because of the large number of persons exposed to the danger of infection. Sewage should be disposed of and water supplies procured in such a manner as to reduce so far as

[1] "Progress Report," Jan. 14, 1914, p. 20.

[2] A summary of epidemics traced to this cause down to 1908 is given in "American Sewerage Practice," vol. iii, p. 122. The typhoid fever death rate in 51 American cities in 1 00 to 1912 inclusive is given on pages 118 and 119 of that volume. The death rate in all leading American cities is published annually in the *Jour.* Am. Med. Assoc.

practicable the danger of spreading this justly dreaded disease and others, such as cholera and dysentery, similarly communicated.

PLANKTON

The present condition of our knowledge of the life processes of plankton is very unsatisfactory. It is known that at least some kinds of these organisms thrive best in waters rich in nitrogen, and thus are found in great numbers in sewage-polluted streams and ponds where there is also enough oxygen to support them, but vigorous growths of them are also found in ponds far removed from any possible contamination by sewage. Free ammonia has been repeatedly found high in waters containing large numbers of such organisms and low where they were few, but this ammonia may represent the products of decomposition of the organisms instead of their food. For the present, therefore, the chief practical value of our knowledge of plankton is the information afforded by observing the growths in streams and ponds receiving sewage and effluents, for these growths indicate certain desirable conditions.

The algæ and diatoms are the forms of plankton which apparently perform the most important part played by micro-organisms in changing inorganic into organic matter. There are some species which appear as thick mats or strands of light- or dark-green color in shallow streams, ditches and ponds; there is a blue-green group which occurs in rivers and lakes, and other groups which are found in heavily polluted pools and channels and leave black or red scum on the ground when the water dries up. These plants break up the carbon dioxide in the water, liberate the oxygen and convert the carbon into organic matter, and this in turn is converted within their cells into other organic substances with the help of nitrogen from the ammonia, nitrites and nitrates in the water. Some of these plant forms are also apparently able to absorb dissolved organic matter directly.

An example of what such plankton can accomplish is afforded by the authors' observations of a small stream at Cincinnati. A sewerage system serving a population of about 1,700 persons discharged sewage into the upper reaches of this stream, which has a sluggish flow for about a mile below the sewer outlet and then spreads out into a shallow creek with occasional pools. The odors from the water just below the sewer outlet were very offensive at times, causing much complaint. At the time of the inspection, the water near the outlet contained about 70 per cent of its saturation value of dissolved oxygen. Farther down stream green algæ appeared and this growth increased in luxuriance as the distance from the outlet increased. Where it was heaviest enormous numbers of gas bubbles were thrown off by the green masses, which, when collected in a bottle and tested by ignition, appeared to be pure oxygen. At a distance of 1¾ miles below the outlet, the water was 130 per cent saturated and free from objectionable appearance. While, therefore, the green growth was indicative of pollution it also showed that active purification of the stream was taking place.

An entirely different group of organisms, the euglenas, also produces a green scum on polluted water. It is sometimes classed as animal because it absorbs organic matter directly and sometimes as vegetable because it gives off oxygen in the sunlight. It appears on pools of sewage and in stagnant parts of polluted streams.

The animal micro-organisms live on bacteria and other organic matter, and if they are abundant in water it may be inferred, not only that many bacteria are present but also that the organic matter which is food for bacteria is present. An abundance of such organisms is, therefore, strong evidence of the contamination of water. The larger animal organisms feed to a considerable extent on the smaller forms, and young fish feed on these larger animalculæ. If, however, the quantity of sewage discharged into the water steadily increases, a time comes when there is so much nitrogenous matter and so little available oxygen in the water that the micro-organisms decrease in numbers. The work is too hard for them. With the disappearance of these minute oxygen-producing plants, the capacity of the water to absorb sewage is still further decreased and offensive anaerobic conditions are imminent. This was well shown by the way in which the progressive pollution of the head of Narragansett Bay, in Rhode Island, shifted seaward the frontier line of the region where algæ and diatoms lived.

COMPOSITION OF SEWAGE

Sources of Total Solids.—Part of the total solids in sewage come from the water which is the principal constituent of the sewage (upwards of 99.9 per cent). The whole of the water supplied to consumers does not reach the sewers and the portion that does is supplemented by water furnished by private water supplies and by the ground water which enters the sewers. If the combined yields from all these sources form a very hard water it will contribute a considerable proportion of the solids in the sewage, while soft water will furnish an insignificant proportion. Solids are also contributed from kitchen sinks, laundries, baths and other wash waters. Feces, urine and paper furnish further solid matter, and industrial wastes contribute still more. The actual amounts of the contributions from some of these sources depend rather closely upon the population and those from other sources depend rather upon the character of the water in the sewers and the industries of the city, so that there is naturally a considerable variation in the solids in sewage of dif-

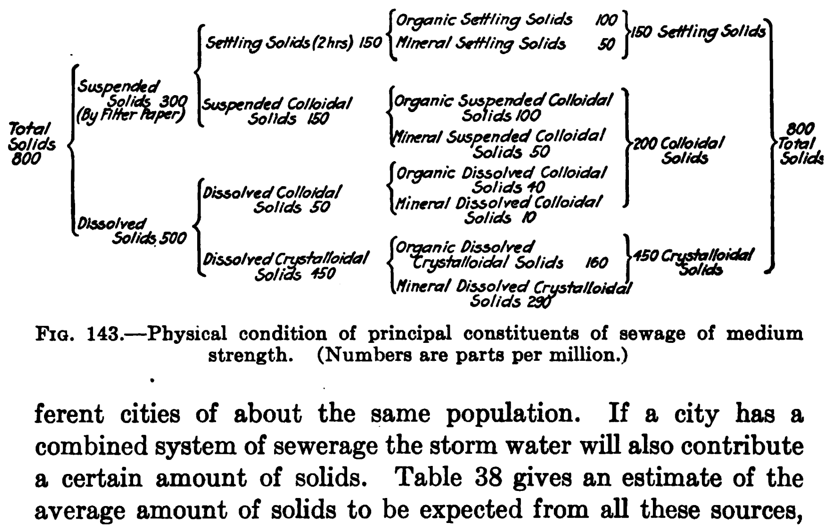


FIG. 143.—Physical condition of principal constituents of sewage of medium strength. (Numbers are parts per million.)

ferent cities of about the same population. If a city has a combined system of sewerage the storm water will also contribute a certain amount of solids. Table 38 gives an estimate of the average amount of solids to be expected from all these sources, and Fig. 143 gives an outline of the physical condition of the principal constituents of sewage of medium strength, and the average quantities of each.

The data given in Table 39 are averages of analyses of the sewage of numerous cities, grouped according to the characteristics of the cities. Their chief value is illustrative and as guides in estimating the probable quality of the sewage from a community of known size and character.

Grams per Capita.—The different amounts of water in sewages result in the average analysis of the sewage of one community being sometimes of little help in predicting the composition of another sewage. If, in addition to accurate analyses, however, the quantity of sewage and contributory population are known, it is possible to calculate the weight per capita of each chemical constituent, and from such results reasonably accurate estimates of the composition of sewage of other similar places can be made. Even by this method, the prediction is uncertain because of the lack of accurate data; greater accuracy is possible in the case of a residential city than in that of an industrial community, but even then, where a water supply is very hard, it may have a marked effect on the solids in the sewage.

TABLE 38.—ROUGH ESTIMATE OF TOTAL SOLIDS IN THE SEVERAL CONSTITUENTS OF SEWAGE

Constituents	Grams per capita per day	
	Items	Total
Water supplies and ground water, assumed to be soft	12.7	
Feces	20.5	
Urine	43.3	
Toilet and news paper (suspended)	20.0	
Sinks, baths, laundries and other sources of domestic wash waters	86.5	
Total for residential sewage from separate sewerage system		183.0
Industrial wastes	200.0	
Total from industrial city with separate sewerage system		383.0
Storm water	25.0	
Total from industrial city with combined sewerage system		408.0

Suspended Matter.—In chemical analyses all solids floating or suspended in sewage are reported as suspended solids, which average about one-third of the total solids in sewage. As a rule, the older the sewage the more finely divided the suspended matter. When considering the results of analyses in which the quantities of suspended matter are given it should be borne in mind that it is impossible to take samples which represent fairly the coarser particles. Where suspended matter is removed from

TABLE 39.—COMPARATIVE AVERAGE SEWAGE ANALYSES

Constituents	Parts per million						Grams per capita					
	Large American cities (combined sewers)	American mfg. cities	Small American mfg. cities	American residential and rural communities	Large English mfg. cities	German communities of the Emscher district	Large American cities	American mfg. cities	Small American mfg. cities	American residential and rural communities	Large English mfg. cities	German communities of the Emscher district
Sewage flow, gal. per cap. per day.	178.0	95.0	69.0	80.0	49.0	....	....	....	....	....	....	....
Nitrogen as:												
Free ammonia	10.6	26.5	38.9	27.2	36.5	33.7	7.8	8.3	9.5	7.5	6.9	7.5
Albuminoid ammonia	7.0	11.9	11.3	7.8	14.8	....	....	2.7	2.8	2.4	2.9	3.6
Organic nitrogen	8.0	24.1	23.8	18.0	....	16.9	6.4	8.0	5.8	5.5	....	11.1[1]
Nitrites	0.11	0.26	....	....	....	....	0.05	....	....	....	....	....
Nitrates	0.44	1.19	....	....	....	....	0.21	....	....	....	....	....
Oxygen consumed	59.0	133.0	107.0	71.0	266.0	....	35.6	48.0	27.0	21.6	49.0	....
Chlorine	48.0	109.0	83.0	47.0	162.0	209.0	37.0	44.9	21.0	13.4	30.0	....
Alkalinity	153.6	129.0	....	....	....	....	161.0	....	....	....	....	....
Solids:												
Total	1355.0	1058.0	730.0	603.0	1896.0	2044.0	567.0	263.0	185.0	183.0	362.0	550.0
Volatile	452.8	635.0	448.0	393.0	....	623.0	185.0	147.0	112.0	120.0	....	151.0
Fixed	902.8	423.0	282.0	210.0	....	1421.0	382.0	116.0	73.0	63.0	....	399.0
Suspended solids:												
Total	214.2	384.0	242.0	342.0	668.0	601.0	127.0	131.0	58.0	105.0	126.0	153.0
Volatile	144.3	288.0	203.0	260.0	....	332.0	72.0	88.0	49.0	82.0	....	82.0
Fixed	69.3	96.0	39.0	82.0	....	269.0	45.0	43.0	9.0	24.0	....	71.0
Dissolved solids:												
Total	1052.5	608.0	488.0	261.0	1228.0	1443.0	441.0	167.0	126.0	77.0	236.0	397.0
Volatile	241.5	270.0	245.0	133.0	....	291.0	99.0	74.0	63.0	39.0	....	69.0
Fixed	811.0	338.0	243.0	128.0	....	1152.0	342.0	93.0	63.0	39.0	....	328.0
Fats	25.3	37.0	....	....	....	....	25.0	....	....	....	....	....

[1] Total nitrogen.

Note.—German figures are based on flow per capita of total population, therefore probably somewhat lower than strictly comparable figures.

the sewage by grit chambers or sedimentation tanks, the quantity and character of the material so removed may in some cases be more accurately determined by careful measurements and

TABLE 40.—RELATIONS EXISTING BETWEEN GRAINS PER GALLON, PARTS PER 100,000, AND PARTS PER 1,000,000

(From "Standard Methods of Water Analysis," 1912, p. 14)

	Grains per U.S. gal.	Grains per Imperial gal.	Parts per 100,000	Parts per 1,000,000
1 grain per U. S. gallon	1.000	1.20	1.71	17.1
1 grain per Imperial gallon	0.835	1.00	1.43	14.3
1 part per 100,000	0.585	0.70	1.00	10.0
1 part per 1,000,000	0.058	0.07	0.10	1.0

TABLE 41.—COMPARATIVE ANALYSES OF WATER AND SEWAGE

(Parts per million)

Place	Residue on evaporation	Nitrogen as			Chlorine	Nitrogen as		Oxygen consumed	Hardness
		Free ammonia	Total albuminoid ammonia	Suspended albuminoid ammonia		Nitrates	Nitrites		
Brockton, Mass.									
Water supply (1905-09)	32.7	0.013	0.103	0.019	6.2	0.009	0.000	2.2	5.0
Sewage (1905-09)	2210.0	47.8	24.5	19.3	144.3	.....	.....	433.7	......
Worcester, Mass.									
Water supply (1905-09)	31.4	0.019	0.120	0.021	2.1	0.052	0.001	3.0	8.7
Sewage (1905-09)	871.0	17.7	7.2	4.3	113.4	.....	.....	122.0	......
Providence, R. I.									
Water supply, filtered (1910).	56.0	0.009	0.078	0.002	6.3	0.12	0.00	3.5	14.0
Sewage (1909)	1715.0[1]	15.40	7.00	3.50	496.5	.....	.....	89.3	......
Chicago, Ill.									
Water supply (1908)	156.0	0.034	0.104	......	5.0	0.29	0.00	2.6	......
Sewage (1909-10)	471.0	8.8	7.6[2]	......	40.0	0.35	0.11	38±	......
Mansfield, Ohio.									
Water	391.0	0.081	0.012	......	8.5	1.5	0.0	0.05	285.0[3]
Sewage	876.0	13.3	31.7[2]	......	108.7	0.2	0.0	51.6	......

Brockton sewage from Brockton city reports. Worcester sewage from Worcester city reports. Brockton and Worcester water analyses from Mass. State Bd. Health Rept., 1909, page 198. Chicago, Langdon Pearse and Report of Lake Michigan Water Commission, 1909, page 45. Providence water analysis, Pratt. Providence sewage analysis, Bugbee. Mansfield, Ohio, Dittoe.

[1] Estimated.

[2] Organic nitrogen (about twice the nitrogen as albuminoid ammonia).

[3] Average of Hedges spring supply and supply from main pumping station wells. Alkalinity plus incrustants.

analyses of the sludge produced than by analyses, however carefully made, of the sewage and effluent.

TABLE 42.—TYPICAL MASSACHUSETTS STRONG, MEDIUM AND WEAK SEWAGES

(Parts per million)

Character of sewage	Strong	Medium	Weak
Place	Brockton	Worcester	Stockbridge
Nitrogen as			
Free ammonia	61.2	20.3	12.4
Albuminoid ammonia	22.1	7.1	3.7
Total organic nitrogen	50.2	18.0	7.1
Oxygen consumed	247.0	193.0	26.8
Total solids	1584.0	882.0	298.0
Volatile solids	1074.0	429.0	177.0
Suspended solids	892.0	276.0	99.0
Chlorine	166.0	114.9	18.0

TABLE 43.—PERCENTAGE OF SUSPENDED MATTER IN SEVERAL SEWAGES SETTLING IN SHORT PERIODS

Location	Suspended matter in sewage, p.p.m.	Settling period, hours	Per cent settled	Authority
Chicago, Ill.				
At stockyards	1,775	2	89	Wisner and Pearse
At stockyards	560	2	56	Wisner and Pearse
At 39th St. Station	156	2	54	Wisner and Pearse
At 39th St. Station	72	2	31	Wisner and Pearse
At 39th St. Station	156	4	63	Wisner and Pearse
At 39th St. Station	72	4	43	Wisner and Pearse
Providence, R. I.	314	4	55	Bugbee
Columbus, O.	210	4.2	60	Johnson
Atlanta, Ga.	320	3	80	Hommon
	114	3	50	Hommon

In the treatment of sewage, the quantity of suspended matter which will settle during the period the sewage is retained in tanks is important. The proportion of the suspended matter in a sewage which may be classed as settling depends upon the strength of the sewage, the character of the suspended matter, the detention period in the tanks, and possibly other factors. The effect of the strength is shown by the results of the investigations summarized in Table 43.

CHAPTER X

SEWAGE DISPOSAL BY DILUTION

Methods of Sewage Treatment.—The most natural method of disposing of sewage is to discharge it into the nearest body of water. This is called disposal by dilution. It is not a makeshift, but just as thoroughly a scientific process of purification as any other, if it is intelligently practiced within the safe limits of the digestive capacity of the water. When the processes of nature by which the water is kept inoffensive are overworked, the sewage-laden waters become offensive and it is necessary to treat the sewage in some way to assist the forces of nature which had previously prevented any nuisance—the primary object in studying methods of sewage treatment being to find ways of changing the character of the sewage so that the effluents from the treatment works will neither cause offensive conditions nor be a menace to health when they are discharged into some body of water.

If the floating matter alone is objectionable, it is necessary to remove that only. If deposits of suspended matter are the source of complaint, the removal of the settling solids may be sufficient. Where dilution fails because of lack of oxygen, the treatment of the sewage may be carried far enough to reduce its oxygen demand to the point where the natural water is able to provide an adequate supply of oxygen. It is essential, therefore, before determining the exact type of treatment to be adopted, to ascertain to what extent it is necessary to remove the objectionable constituents from the sewage. Having ascertained the extent to which nature must be assisted in order that disposal of the effluent by dilution shall be permissible, to accomplish his object the engineer selects one or more of the following:

1. Screening.
   (a) Coarse.
   (b) Fine.
2. Sedimentation.
   (a) Rapid in grit chambers.

(*b*) Slow in settling basins.
Plain.
With chemical precipitation.
With septic action in single-story tanks.
With sludge digestion in two-story tanks.
(*c*) With aeration.
3. Contact bed treatment.
4. Trickling filter treatment.
5. Intermittent filtration.
6. Broad irrigation.
7. Disinfection.

These methods of treatment are explained in the following chapters.

Prior to 1890, little attention was given to objectionable conditions produced by the discharge of sewage into American rivers. Since that date, however, there has been a tendency toward progressively greater restriction upon the discharge of untreated or raw sewage into bodies of water and toward constantly raising the degree of improvement required of treatment plants. While there can be little doubt that in certain cases progress in these directions has exceeded logical requirements, it is equally true that over the greater part of the country the requirements are still quite lax and more care in disposing of sewage will be necessary in the future than has been exercised in the past. It is desirable, therefore, when adopting a plan for the disposal of sewage, to select for present use such process or processes as may meet the needs of the present and immediate future, but at the same time lend themselves to later development of more effective means of treatment when they shall be required.

Nature of Disposal by Dilution.—Disposal by dilution is the discharge of sewage or the effluent of sewage treatment plants into natural waters where it will be dispersed through the waters in such a manner as to be carried away so rapidly, or be so changed in composition and character, that it will not prove offensive or a menace to health. In 1905 Fuller estimated that 28,000,000 persons in the United States were served by sewerage systems and 96 per cent of the sewage was disposed of by dilution. Estimates made in 1915 by the authors indicated that about 41,800,000 persons were then served by sewerage systems and that the sewage of about 84 per cent of this population was disposed of by dilution.

The conditions favorable to successful disposal of sewage by dilution are: (a) freshness of the sewage; (b) freedom of sewage from floating matter and solids capable of settling, a condition attained by treatment; (c) thorough diffusion through the diluting water; (d) diluting water of high oxygen content; (e) swift currents to carry the sewage to points of unlimited dilution; (f) biological equilibrium; (g) absence of slips and coves tending to facilitate sedimentation accompanied by sludge deposits.

The United States Government has charge of navigation in New York Bay, and before it would permit sewage from a number of cities in New Jersey to be discharged into the Bay it required the Passaic Valley Sewerage Commission, which was building the joint outfall sewer to the Bay, to agree that the sewage should not produce visible suspended particles in the Bay, deposits objectionable to the Secretary of War, odors, grease or color at the point of dispersion, injury to the public health, public or private nuisance, or reduction in the dissolved oxygen of the waters of the Bay to an extent to interfere with major fish life. The Commission agreed to pass the sewage through coarse screens and grit chambers, then through screens with openings not larger than 0.4 in., and finally through sedimentation basins where the mean velocity of flow is not over 0.5 in. per second for average conditions and 0.75 in. for maximum flow conditions. The miminum period of detention of the sewage in the tanks is fixed at 1 hr. and the average at not less than 1½ hr.

The principal physical change which takes place in the sewage discharged into a body of water is due to sedimentation and dilution. It is particularly important, therefore, to pay attention to the suspended solids, for while they are similar in composition to the dissolved substances the changes they undergo are subject to somewhat different laws. Much of this suspended matter is inorganic and not food for living organisms. The organic portion, which is such a food, is usually greater than the birds, fish and other food consumers can remove immediately and a part of it is carried along by the current or deposited about the sewer outlet. The current of the body of water receiving the sewage may therefore be important in disposal by dilution, for if the current is so small that organic suspended matter is deposited on the bottom there is the possibility that offensive conditions will result from the decomposition of such deposits, and if the deposits are large objectionable shoaling may occur.

The decomposition of sludge deposits is relatively slow. In some rivers, sedimentation will remove the settling solids from the waters during normal conditions, throwing upon the rivers principally the burden of the dissolved and colloidal organic matter. But in time of freshet, when there is an ample volume of water and a swift current, such deposits may be scoured out and the rivers relieved of the burden of changing this settled organic matter also into more stable substances. Under such conditions sedimentation in the river performs a function similar to that in settling tanks, through which the sewage may be passed before its discharge.

Decomposition of Organic Matter.—If the ratio of diluting water to sewage is inadequate, bacteria may so thrive that they will exhaust the available supply of oxygen and anaerobic conditions likely to cause offensive odors will become established. In any event such processes are likely to go on in the sludge banks formed by the precipitation of organic sewage matter. On the other hand, if there is enough oxygen to meet the demands of the bacteria, the aerobic organisms will predominate and the organic matter will be oxidized as explained in Chapter IX.

The oxidation of sewage under conditions favorable to the growth of plants is certain to be followed by the growth of plankton. The oxygen given off by some of the plankton is an important factor in maintaining the supply of dissolved oxygen in water under such conditions. In the case of ponds and lakes the saturation of the water is usually confined to the upper portion of the water, because the plankton producing it thrive where the light is strong. At considerable depths bacteria thrive and make large demands on the dissolved oxygen there, which is still further depleted by contact with the products of the anaerobic decomposition of the mud at the bottom.

In addition to the results of chemical and bacterial studies, there are records of the effect of the discharge of known amounts of sewage into waters, prepared mainly by the investigations mentioned on page 314.

Stearns' Investigation.—After an investigation for the Massachusetts State Board of Health, Frederic P. Stearns, then its Chief Engineer, concluded ("Special Report on Water," 1890, p. 791) that if the flow is less than 2½ cu. ft. per second per 1,000 inhabitants an offense would be almost sure to arise.[1]

[1] Stearns estimated that each person contributed to sewage daily an average of 0.015 lb. free ammonia, 0.003 lb. albuminoid ammonia, 0.218 lb. dissolved solids and 0.042 lb. chlorine.

With larger volumes than 7 cu. ft. per second per 1,000 inhabitants, the pollution would be too small to cause any nuisance. Where the water is to be used for manufacturing purposes, the amount of dilution should be greater, he stated, and in the case of a stream used for domestic water supply he was unwilling to say that any degree of dilution would make the water entirely safe to use.

Goodnough's Investigations.—In 1902 another investigation was made for the Massachusetts State Board of Health by Goodnough, who succeeded Stearns as Chief Engineer of the Board. He narrowed the range of dilution fixed by Stearns and summarized the results ("Report State Board Health," 1902, p. 452) as showing that where the quantity of water available for the dilution of the sewage in a steam exceeds about 6 cu. ft. per second per 1,000 persons contributing sewage, objectionable conditions are unlikely to result. Where sewage was discharged at many outlets into a large body of water he thought that objectionable conditions might not result from somewhat less dilution than that named. In every case where the flow was less than 3.5 cu. ft. per second per 1,000 persons objectionable conditions had resulted.

In 1902, in a letter to the Committee on Charles River Dam, Goodnough made the following statement:

"The degree of dilution which has been found necessary to prevent unsanitary conditions where sewage is discharged into a stream, assuming 75 gal. of sewage per person, ranges between 20 to 1 and 60 to 1. In estimating the degree of dilution of the sewage, no account has been taken of the purifying effect of the water of the basin itself." (Evidence, etc., before Committee, page 108.)

J. Herbert Shedd testified in 1902 before the Committee on Charles River Dam that about 5 cu. ft. per second of the ordinary flow of the river, per 1,000 persons contributing sewage, would render the presence of the sewage unobjectionable (Evidence, etc., before Committee, p. 365).

In 1908 and 1913 Goodnough made for the Board investigations of the condition of the Merrimack River, the results appearing in special reports. The condition of other rivers was examined to obtain corroborative evidence regarding pollution, and it was found that wherever the dilution was 3.4 cu. ft. per second or less per 1,000 persons, a nuisance followed. Where serious pollution was observed with higher rates of dilution the nuisance was usually due to the discharge of large quantities of industrial wastes into the stream. No case of dilution was found between 5.8 cu. ft., where industrial wastes and sewage combined to cause offense at Webster, and 9.2 cu. ft. at Ware, where the effect of the sewage was noticeable for a considerable distance, except at cities along the Merrimack. The dilution in the latter river was 8.7 cu. ft. below Lawrence and 7.6 cu. ft. below Haverhill. Below Lawrence the surface of the river was reported to be covered often for a long distance with froth, scum, and oily and greasy matters, particularly in summer, when there was a noticeably disagreeable odor at times along the south bank. Except at the latter place, the pollution had not rendered the river offensive except near some of the large sewer outlets and where banks of sludge were exposed at low water, the report stated, and these conditions could be ameliorated by improving the methods of discharging sewage into the stream, so as to provide uniform diffusion through the river water.

Pollution of Ohio Rivers.—In 1897 the Ohio State Board of Health had an investigation of the condition of certain Ohio rivers made under the direction of Allen Hazen. In discussing the results he stated that in the case of sluggish streams, or of streams the waters of which are already somewhat polluted, the quantity required for proper dilution may become 6, 8 or even 10 cu. ft. per second per 1,000 population. ("Preliminary Report of an Investigation of Rivers," p. 32.)

New York Harbor.—The Metropolitan Sewerage Commission of New York estimated in its 1910 report that the sewage discharged into New York Harbor was diluted with 32 parts of water, and that this ratio would become 1 to 13 about 1940. The flow of diluting water in the harbor was estimated at 4.7 cu. ft. per second per 1,000 population, which would be reduced to 2.65 cu. ft. in 1940.

The permissible limit of pollution in sea water depends upon whether the object in setting the limit is the prevention of nuisances or the protection of fish. This has been emphasized in disputes over the permissible pollution of New York Bay, where the dissolved oxygen should not be reduced below 70 per cent saturation, according to Black and Phelps, in order that food fishes might continue to live in the waters. Fuller stated[1] on this topic:

[1] *Trans.* Am. Inst. Chem. Engrs., vol. iii, p. 392.

"With respect to guarding against objectionable odors I think it is clearly necessary for the chemists and bacteriologists to keep in mind that putrefaction does not exist so long as oxygen remains at all. In fact you can go further than that, and say that so long as oxygen is available from nitrates, nitrites, or other oxidized salts there is substantially no putrefaction. I am aware that that does not provide for one feature that may be of some importance, and that is the protection of major fish life. I believe, however, that the European custom in many places is sound in indicating that 30 per cent of the dissolved oxygen necessary for saturation provides a reasonable margin in the case of a majority of species of fish life of the larger kinds. Perhaps some may call for more, but so long as there is 30 per cent remaining at all places at all times, it is a matter of deduction from our well-established laws of biology and chemistry that there can be no putrefaction. The larger number of the principal rivers in this country serving as public water supplies do not contain as much as 70 per cent of the oxygen necessary for saturation. Among the rivers with which I have been personally familiar through analysis, I may mention the Merrimack River at Lawrence, Mass. Twenty years ago it had as low as 50 per cent and sometimes but 30 per cent of dissolved oxygen. In those days it served as the water supply for Lawrence without being filtered, and in the last 17 years, since filtering, it has been regarded as one of the good water supplies of the world. I believe that if this 70 per cent margin suggested by Dr. Soper were applied to Lawrence, it would show that the Merrimack River at that place was not providing a proper disposal for the sewage at Lowell and the cities above, notwithstanding the fact that it provides an excellent water supply at that point."

Temperature.—The effect of temperature upon disposal by dilution is important. A river may receive in winter, without causing objectionable conditions, a quantity of sewage which, in summer, would make it a nuisance. Bacterial action at low temperatures is relatively slow, and sewage may be carried by the river receiving it to a point where ample dilution is afforded before the bacteria have caused any offensive results. This has an important economic aspect, for, in summer, sewage treatment may be carried to a degree insuring satisfactory conditions at an expense which, if continued through the year, would be prohibitive. Advantage may be taken of the winter conditions by providing a less expensive treatment, care being taken to avoid sludge deposits which may prove objectionable during the succeeding warm season.

Fresh and Salt Water.—When sewage is discharged into salt water there is a greater tendency for the settling solids to form sludge banks than when it is discharged into fresh water. This is due in part to chemical action of the salt water on the sewage and in part to the inability of the salt water to carry as much matter in suspension as the fresh water can. Sea water normally contains about 20 per cent less dissolved oxygen than distilled water so that, other things being equal, fresh water is able to dispose of more sewage than sea water.

Tides.—If a river discharges into a tidal estuary, a mixture of fresh and salt water is produced, changing in character from hour to hour. If the estuary is short and steep, it clears itself with each ebb tide; if long and with complex entries, the water may oscillate backward and forward with a varying degree of salinity. Inasmuch as the tidal currents must be utilized to remove sewage, it is necessary to have a comprehensive knowledge of them before locating the outlet of a sewerage system, in order to be sure that offensive conditions will not arise at any stage of the tide.

At the turn of the tide, the incoming salt water follows the bottom, owing to its greater density, while the overlying brackish water is still traveling seaward. Gradually, as the depth of the incoming wave increases, the motion of the entire section changes, flowing inward with increasing velocity until the maximum velocity, or "strength of the tide," is reached. The velocity then falls off until ebb tide begins, when the whole mass of water flows outward with increasing velocity until the maximum run is reached. The velocity then decreases until the turn of the tide is reached and the cycle begins again. The velocity of the ebb tide is greater than that of the flood tide because the flow of fresh water is in the same direction as that of the outgoing salt water, instead of in the opposite direction, as on the flood tide.

The "tidal prism" of a tidal basin is the volume of water contained within it between the elevations of high and low water. It depends upon the mean range of the tidal rise and fall, which decreases in going from the northern to the southern limits of the United States, as follows:

East Coast City	Range, Feet	West Coast City	Range, Feet
Boston	9.6	Seattle	11.6
New York	4.4	Astoria	6.4
Baltimore	1.2	San Francisco	4.0
Charleston	5.2	San Diego	4.0
Savannah	6.5		
Key West	1.2		
Galveston	1.0		

Floats.—Observations of the movements of floats are made to ascertain the direction of currents. Fig. 144 illustrates two types employed by the New York Sewerage Commission in its investigations of the currents in New York Harbor and neighboring waters.

The small float had the advantage of low cost, ease of handling, and small area exposed to the wind. On the other hand it was destroyed by the paddles and propellors of steamers

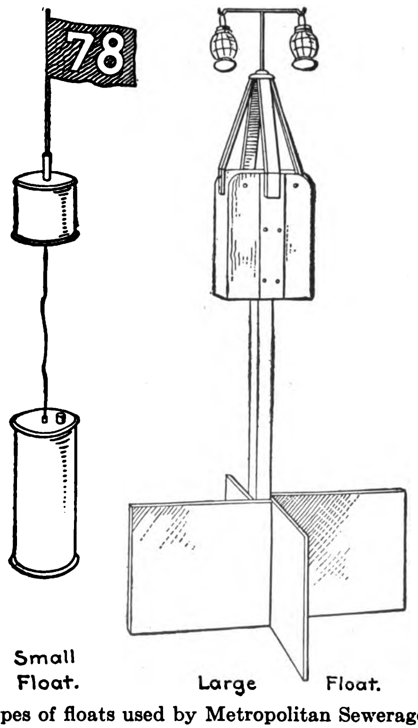


FIG. 144.—Types of floats used by Metropolitan Sewerage Commission.

and, as it was unable to support a lantern, it was unsuited for night work. The upper cylinder was 5⅜ in. in diameter and 5 in. long. The lower cylinder was 6¼ in. in diameter and 14 in. long, and was weighted with sand until the top of the upper can was nearly level with the surface of the water.

The large float was made in two sizes, 5 and 6 ft. long. The illustration shows the smaller size, which had an upright of 2 by 2-in. timber, buoyed by a cork float at the top and provided at the bottom with four 12 by 24-in. vanes.

In making use of floats to determine the direction and velocity of tidal currents it must be kept in mind that sewage discharged into tidal water will move over the surface more rapidly than floats. For example, the early investigations of the tidal currents in the vicinity of the Moon Island outlet of the Boston sewerage system were made with floats 8 ft. long. These floats showed the direction of the currents, but they moved with the mean velocity of the water and not with the surface velocity during the ebb tide, during which alone it was proposed to discharge the sewage. After this outlet was placed in service it was found that the sewage moved on the surface much more rapidly than was anticipated from the results of the float observations, and in later observations of this nature the Massachusetts State Board of Health used floats about 8 in. long for most observations and 24 in. long for the remainder, as it was found that the sewage tended to form in a shallow sheet on top of the water, and hence the surface and shallow depth velocities were of greater importance than the mean velocities of deeper sections from the surface down.

Depth of Outlet in Tidal Waters.—Experience with the discharge of sewage in tidal waters has shown that the outlet should be at a considerable depth, in order to disperse the sewage through as large a volume of water as possible before it rises to the surface. Not only do the large solids in sewage present an offensive appearance if they float on the surface, but the greasy substances cover the surface with a film or "sleek" which is not necessarily an indication of putrescible organic matter yet is unsightly and likely to be the cause of complaint against the disposal of the sewage in this way. These undesirable conditions are much reduced when the sewage is discharged in depths of 25 ft. or more. For example, the Peddocks Island outlet in Boston Harbor, the point of discharge of the sewage of a population estimated at 382,000 in 1912, is in 30 ft. of water at low tide, and the sewage is so thoroughly dispersed that the Massachusetts State Board of Health reported in 1910 that it would be difficult to detect its position but for the fact that it is the feeding ground for large numbers of gulls. The authors showed, however, that considerable quantities of sewage rise to the surface at this outlet, although not visible, by dissolving 2 lb. of eosine in water and discharging this dye into the shore end of the outfall sewer. On proceeding to the vicinity of the outlet the colored sewage could be seen rising with considerable velocity and spreading over an area of many acres, although the experiment was performed during a very heavy wind and rain storm.

Salinity.—Although the normal specific gravity of sea water, which depends upon its salinity, varies from about 1.022 to 1.028, it is practically constant near any one place in the ocean. This fact has been utilized in studies of the dilution of sea water by fresh water in tidal basins, and affords an easy method of determining certain tidal movements which it is difficult to investigate in other ways. For instance, off New York the normal specific gravity of the sea water is 1.025, corresponding to a chlorine content of 18,000 p.p.m. under the assumption that 88.6 per cent of the total excess specific gravity above unity is due to the chlorides. If a sample of water from New York Harbor is found to have a specific gravity of 1.015, it must result from a mixture of about 60 per cent sea water and 40 per cent fresh water. By conducting a series of observations of this nature in the East River at New York, the Metropolitan Sewerage Commission showed that there was a distinct flow southward through this tidal river, although float observations and studies of tidal movements, made mathematically, pointed to the absence of any such flow.

The investigations of this nature are made by a salinometer,[1] a form of hydrometer about 12 in. long, with a stem $\frac{5}{32}$ in. in diameter, carrying a 4-in. scale reading specific gravities from 1.00 to 1.03 by intervals of 0.0005. A sample of water is placed in a tall cylindrical glass and the salinometer lowered into it. Both the thermometer and hydrometer scales are read and the hydrometer reading is corrected by an amount ranging with the temperature from -0.0011 at 35°F. to $+0.0028$ at 82°F., which was determined by experiment.

Effect of Wind.—The wind blowing over the surface of a body of water tends to set up currents in it, owing to the frictional resistance of the water to the currents of air over its surface. On-shore and off-shore winds produce complementary currents in opposite directions at greater depths. Judson found[2] that on Lake Michigan the rate of travel of the surface currents was about 5 per cent of the velocity of the wind. The U. S.

[1] "Use of the Salinometer in Studies of Sewage Disposal by Dilution," Kenneth Allen, *Jour.* Assoc. Eng. Socs., April, 1911.

[2] *Report* Lake Michigan Water Commission, 1909.

Hydrographic Service reported[1] that a study of many observations in the North Atlantic indicated surface currents of about 3.2 per cent of the velocity of the wind. Whipple found[2] in Lake Erie that the surface currents were from 3 to 6 per cent of the velocity of the wind. It is believed that the effect of these surface currents may extend to depths of 30 to 40 ft.

Knowledge of the currents produced by the wind has been gained largely by studies of temperature changes in the water at different depths. The readings of temperatures below the

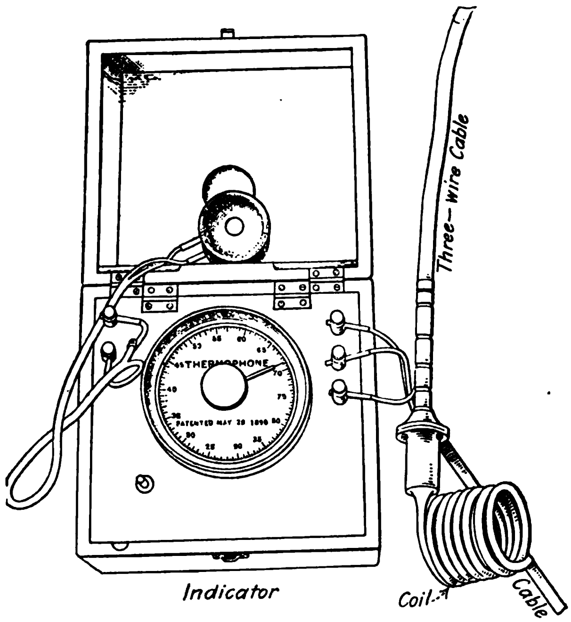


FIG. 145.—The thermophone. (From Whipple's "Microscopy of Drinking Water.")

surface are most easily made with the thermophone,[3] invented by Whipple and Warren. It is an electrical thermometer, Fig. 145, consisting of a sensitive electrical coil which may be lowered to any desired depth and an indicator connected with the coil by lead wires. An electric current is supplied by dry batteries and a telephone receiver is connected to the indicator by which the operator can detect, by a buzzing sound, when the movable

[1] *Monthly Weather Review*, 1902.
[2] Report on Cleveland Water Supply, 1905.
[3] "Microscopy of Drinking Water," Whipple, p. 88.

pointer is made to approach the correct temperature reading, at which the buzzing ceases. Thus by reading the dial at this point, the operator can ascertain the temperature of the distant coil. Thermophones adjusted for a range from 40 to 250°F. will give readings correct to within 0.2°.

Milwaukee Investigation of Lake Currents.—In 1909 to 1911 an investigation was made by Alvord, Whipple and Eddy of the methods of disposing of sewage and protecting the water supply of Milwaukee. This involved a study of the currents in Lake Michigan, which exemplifies the scope of such investigations.

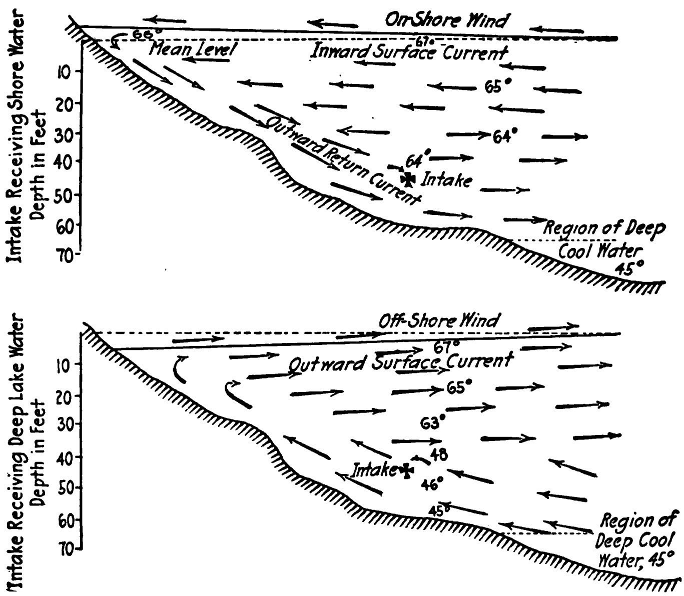


FIG. 146.—Effect of wind on lake currents at Milwaukee.

As the capacity of the lake is about equal to its discharge during 100 years, the lacustrine current toward its outlet at the Straits of Mackinac is inappreciable, and temperature and winds are responsible for both surface and deep movements of the water.

Fig. 146 illustrates the manner in which winds were found to cause sudden large fluctuations in the temperature of the water drawn in at the intake of the Milwaukee water works. Thermophone readings of the water every 5 ft. in depth taken at the intake and at points 1, 2 and 3 miles farther out in the lake, showed that 2 or 3 miles from the shore, where the lake becomes deep, the water below a depth of about 65 ft. is practically stagnant in the summer. Nearer the surface, however, the wind has a marked effect in circulating the water.

The temperature observations were supplemented by investigations of the number of bacteria in samples of water collected at

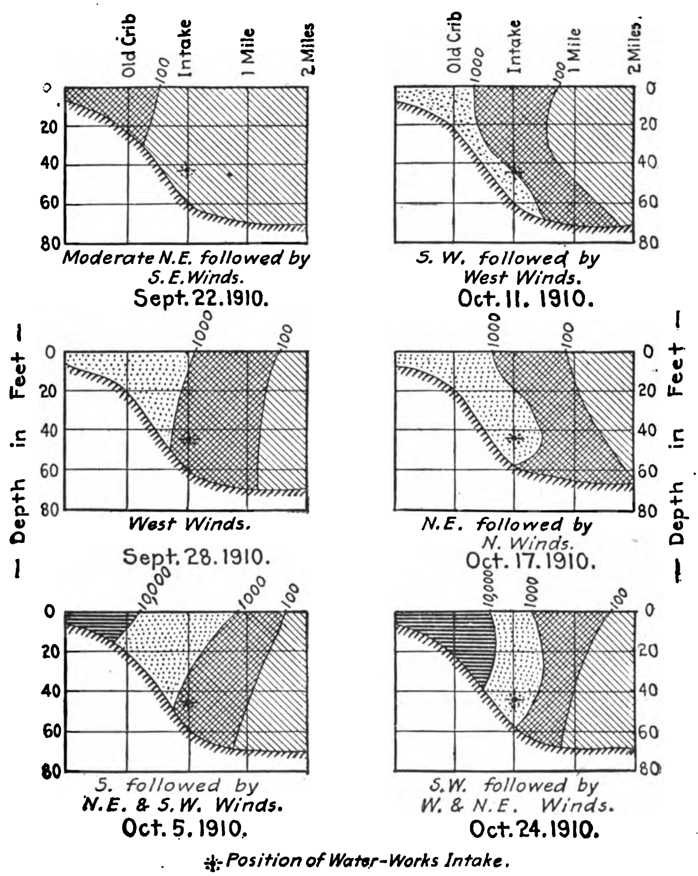


FIG. 147.—Effect of winds on vertical distribution of bacteria in a plane passing east and west through the intake of the Milwaukee water-works.

numerous points, at both the surface and the bottom, and by determinations of the chlorine in the water at the same points. The results obtained on 6 days are shown in Figs. 147 and 148, and illustrate how the wind affected the distribution of the sewage through the lake in the vicinity of the city.

On September 22 there was an on-shore wind. The bottom water had practically the same temperature as that at the surface and the turbidity of the water was the lowest observed at any time during the investigation. The surface water containing the largest amount of chlorine was driven to the northward and kept near the shore and the largest number of bacteria was found

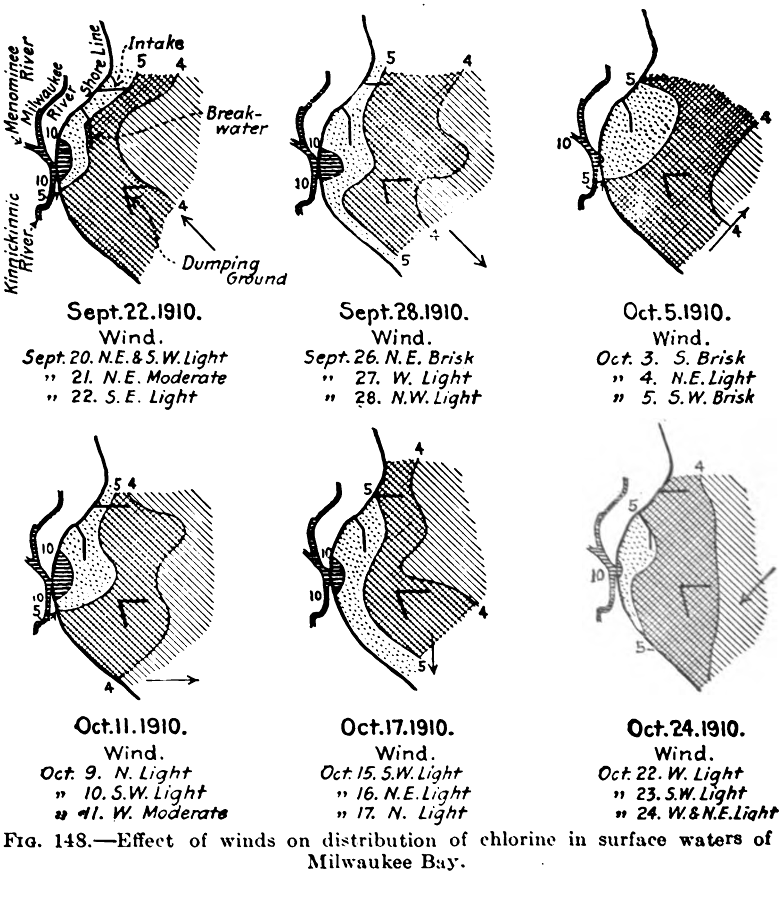


FIG. 148.—Effect of winds on distribution of chlorine in surface waters of Milwaukee Bay.

near the shore. The sewage of the city is discharged into the lake by the rivers shown in Fig. 148.

On September 28, there was a light off-shore wind, and little difference was found in the temperatures of the surface and bottom waters. The turbidity was high on this day. The chlorine extended more directly from the mouth of the river, whence the pollution came, and was more widely distributed in the bay. The surface water containing the largest number of bacteria was now spread out from the shore.

On October 5, the wind was off-shore and colder water was brought in at the bottom, the difference between the average surface temperature and the lowest bottom temperature being

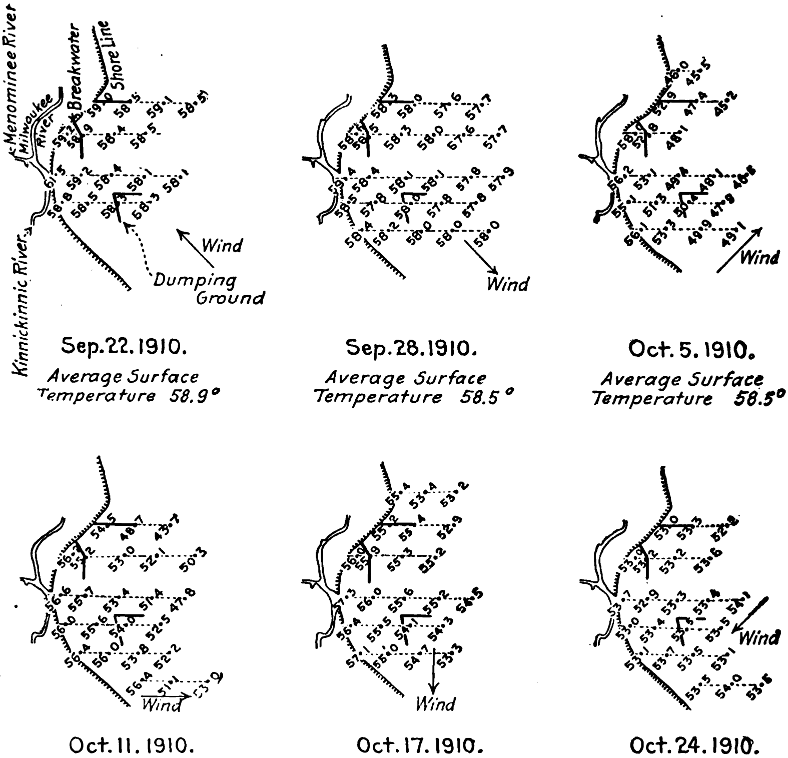


FIG. 149.—Effect of winds on temperatures of bottom waters in Milwaukee Bay.

13.3°, as is evident from Fig. 149. The surface water containing the largest amount of chlorine had been driven to the north and there was a well-marked surface drift of water with large numbers of bacteria toward the water works intake.

On October 11, with off-shore winds, the temperature conditions, chlorine and bacteria were about the same as on October 5.

On October 17, with a north wind, the difference between the average surface temperature and the minimum bottom temperature was 3.3°. The surface water with the largest amount of chlorine and numbers of bacteria had been driven southward.

On October 24, when there was an on-shore wind, the turbidity was high, the largest proportion of chlorine was near the shore, but the distribution of bacteria was unusual, probably due to heavy pollution of the river caused by the refuse washed from streets by a rainfall of 0.5 in. on October 21 after a long period of dry weather.

Temperature Changes in Deep Ponds.—Temperature changes in the water of deep ponds cause vertical currents due to differences in the specific gravity of the water. As the surface water cools in the autumn it becomes heavier and sinks, thus causing currents reaching to increasing depths until all the water acquires the temperature of maximum density, 39.2°F. If it were not for the wind, circulation would then cease, but the slight differences of specific gravity near this temperature facilitate movement of the water and it is possible for the entire contents of the pond to reach a slightly lower temperature than 39.2°. In the spring, when the icy surface water becomes heavier as it is warmed, vertical currents are again established and continue until the temperature of the lower strata has reached a point somewhat higher than 39.2°, due to wind action. There is thus an overturning and mixing of the water in the spring and autumn, while in summer and winter there is stagnation below the depths affected by winds. These conditions do not exist in shallow ponds because the winds mix the water throughout its depth. It is interesting to consider what would happen if the maximum density of water were below its freezing point.

Quiet and Running Water.—The absorption of oxygen by quiet water is very gradual, but when the surface is agitated the absorption proceeds more rapidly. Therefore a running stream absorbs oxygen more rapidly than a pond, and as oxygen is required for the aerobic change of organic matter into mineral matter there was a prevailing opinion for some years that running water was preferable to quiet water for receiving sewage, and that a polluted river purified itself in a distance of 20 miles. This theory has been exploded. It is now known that it does not do so and that there are other factors which influence the changes taking place in the organic matter, such as sedimentation, winds, sunlight, bacteria and plankton, which are more active in quiet waters than in running streams.

The sewage discharged into running water is broken up and altered so that the stream may appear to have been unaffected by the sewage discharged into it above the place of observation, whereas the organic matter in it may have undergone only part of the changes necessary to render it innocuous. The improvement has been in appearance rather than in composition. In 1899 Sedgwick stated[1] "that sedimentation and the destruction of micro-organisms by various agencies are more completely effected in standing than in moving water, so that modern sanitary science has reversed the tenet of 30 years ago and now unhesitatingly affirms that it is quiet water rather than running water that 'purifies itself.'"

Notwithstanding the importance of sedimentation and of the action of organisms in relatively quiet water, the great value of aeration of water by wave action, agitation by propellors, and falling over dams and riffles, must not be ignored. It is easily conceivable that a stream may be so seriously polluted as to become putrid if practically quiescent, whereas, if caused to fall over dams or pass down rough rapids at frequent intervals along its course, it will absorb sufficient oxygen to maintain the processes of oxidation and thus avoid putrefaction.

Basic Information for Planning Dilution Projects.—From what has been stated in this chapter it will be seen that in order to pass intelligently upon the disposal of large quantities of sewage in water, (*a*) hydrographic surveys may be required; (*b*) a study of the quality and temperature of the lake water and of the currents and winds affecting them may be needed, with chemical and bacterial surveys of the water at different points and depths, from which can be plotted zones of pollution of different degrees; (*c*) a study of possible locations for and forms of the sewage outlet may be desirable to ascertain any danger of pollution of a water supply, of nuisance by washing of the sewage back to the shores of the lake or other body of water, or of objectionable sleek on the surface of the water; and, finally, (*d*) the engineer will have to weigh carefully the character and extent of the preliminary treatment required to fit the sewage for dilution, the desirability of disinfecting the sewage or effluent before discharging it into the water, and the relative desirability of treating the sewage or purifying a water supply drawn from the lake or river.

[1] *Report* Pittsburgh Filtration Commission, p 17.

Sewage Dilution at Boston.—The first large outfall sewer discharging into Boston Harbor had its outlet near Moon Island, where float observations showed that the average velocity of the ebb tide was 0.74 mile per hour and that the floats traveled about 4 miles seaward during the whole ebb tide. The sewage is stored on the island and discharged during ebb tide. Experiments showed that when 22,000,000 gal. of sewage are discharged in 45 minutes, about 750 acres of harbor surface are plainly discolored on a comparatively calm day, and an objectionable appearance is presented by two-thirds of this area, although offensive odors arise from but a relatively small portion of it. Suspended matter is sometimes seen 1½ miles from the outlet, and sleek is occasionally observed at still greater distances. Generally speaking, the upper 2 to 3 in. of the sewage-covered area contain much the greater percentage of the sewage. The conditions described last for 2 to 3 hr., depending largely on the wave action.

The second large outfall sewer discharging into the harbor originally had its outlet near Deer Island Light but a few feet below the surface near the edge of the main ship channel. The sewage was discharged continuously and discolored the water over an area of 350 acres on ebb tide and for a distance of 1½ miles from the outlet, although sleek was sometimes observed farther out on calm days. In 1916 this outfall was extended 315 feet further out toward the ship channel, discharging through 14 openings the deepest being at a considerably greater depth than the discharge end of the original outfall. Of the multiple openings the outermost is 48 in. in diameter and the rest elliptical-shaped varying from 25 by 44 to 13 by 23 inches.

The third large outfall sewer has its outlet near Peddocks Island at a depth of 30 ft. at low tide; the sewage is discharged continuously. The sewage is diluted so quickly by the sea water that the percentage of sewage in the surface water directly above the outlet is very small.

Dilution at Cleveland.—Whipple found at Cleveland in 1904 that the general current in Lake Erie near that city was a mile in 6 days, which is negligible in comparison with currents due to winds. During 11 months of the year the prevailing winds are down the lake; the winds across the lake are practically equal in both directions. The strongly polluted water of the Cuyahoga River, see Fig. 3, was found to pollute the lake more than the outlets along the lake front. The turbidity of the water did not have an appreciable relation to the amount of sewage pollution, although the numbers of bacteria varied with the turbidity, decreasing rapidly off-shore. The chlorine determinations indicated a general outward sweep of the polluted water eastward away from the shore. Winds were found to have the same effects that have been described in connection with the Milwaukee investigation. The odor of the lake water near the shore was frequently moldy and oily, but 2 or 3 miles off-shore little odor was perceptible.

Chicago Drainage Canal.—The sewage of Chicago is carried to the Desplaines and Illinois Rivers by the Chicago Drainage Canal, and is discharged eventually into the Mississippi. The canal was designed to carry a minimum of 3.3 cu. ft. per second of lake water per 1,000 persons contributing sewage. The Canadian Government has not been favorably disposed to the diversion of water through the canal, and since it was opened there has been an enormous and unforeseen increase in the trade wastes discharged into it. The effect of the dilution of the sewage was investigated thoroughly in connection with an unsuccessful suit by the State of Missouri to obtain an injunction against the discharge of Chicago sewage into tributaries of the Mississippi. It was claimed that the sewage contaminated the water supply of St. Louis, while the defendants contended that the processes of self-purification in the canal and rivers removed all danger of such contamination. In 1911, Wisner reported that the dilution in summer was inadequate to prevent the reduction of the dissolved oxygen throughout the canal to less than 2½ p.p.m. In the lower 15 miles of the canal it averaged from 0 to 10 per cent saturation, the latter figure being less than 1 p.p.m. under summer conditions.

CHAPTER XI

GRIT CHAMBERS, RACKS AND SCREENS

Sewage, particularly sewage from combined systems while they are carrying storm water, may contain a large amount of suspended matter. Part of it is heavy mineral matter like sand, gravel, bits of coal and cinders, which is called grit. It can be removed, if necessary, in small settling basins called grit chambers. Sewage also contains cloth, paper, lumps of meat, bits of vegetables and pieces of wood ranging in size from matches to railroad ties, which can be removed by screens.

Object of Grit Chambers.—Among the objects which may make the removal of coarse mineral matter desirable are: (1) to prevent injury to pumping machinery by grit; (2) to keep mineral matter from the sludge of settling basins, which flows less readily if coarse grit enters it: (3) to prevent inverted siphons from becoming clogged with material often difficult to remove, (4) to prevent deposits of inorganic matter in bodies of water into which the sewage is discharged; (5) to keep heavy grit from settling on the sludge in septic tanks and tending to seal it and to make the ebullitions of the gases of decomposition, when they occur, very violent.

Object of Racks.—Racks are used for the removal from the sewage of the coarser materials, such as pieces of wood, cork, bottles, orange skins, cloth, cotton waste, wool, hair, fiber, and similar matter, which find their way into sewers, and which are likely to cause obstruction or injury. They may be placed upon the upstream or downstream side of the chamber and are in the nature of coarse screens.

Object of Screens.—Among the objects accomplished by screening the following are prominent: (1) the protection from injuries and clogging of appliances for conveying, pumping and treating sewage and sludge; (2) the prevention of unsightly matters floating on tanks at treatment works or on bodies of water into which raw sewage is discharged; (3) the reduction of the amount of sludge settling on the bottom of slow-moving bodies of water and liable to cause offense by decomposing; (4)

the prevention of heavy and extremely tough floating scum on the surface of septic and other tanks; (5) the removal, so far as possible, of the fine suspended matter in addition to the coarser matters already mentioned, either as a final treatment to prepare sewage for disposal by dilution or as a preliminary one to assist other methods of treatment. As a temporary expedient, for service while developing more complete methods of treatment and building the plant for applying them, screening may offer the advantages of low cost and considerable efficiency.

Theory of Grit Chambers.—The object sought in the grit chamber is to remove the grit, in general the inorganic material, without depositing any of the organic matter. Grit settles from flowing sewage according to its size and specific gravity, and to the velocity of the stream. A velocity which will just permit pebbles ¼ in. in diameter to settle will carry along sand. To provide a uniform separation of grit from other settling solids, a uniform velocity must be maintained, and this is difficult on account of the constant variation in the quantity of sewage and the large capacity of the chambers in proportion to the flow through them during the early years of their use. Yet the velocity of flow must not be too low as this will result in the deposit of the organic material.

Another element entering into the problem of removing grit is the depth of sewage through which it must settle. Upon this will depend the length of the grit chamber, for a definite period of time is required for a particle of given size and weight to fall a specified distance through a flowing stream. The grit chamber, therefore, must be long enough to provide time for the particle to reach a depth below which it will be prevented from passing out of the basin. The difficulties of operation will be reduced by retaining in the grit chamber only such matters as settle at velocities not lower than 1 ft. per second, if it is practicable to handle the remaining heavy materials in other portions of the treatment plant. The organic matter likely to be precipitated in grit chambers may become extremely offensive if the velocity of flow is reduced below 1 ft. per second, and wherever practicable it is more satisfactory to handle it with the sludge from settling tanks than to attempt its removal by grit chambers. Even with a velocity of 1 ft. per second, there is danger of intercepting with the grit such organic matter as coffee and tea grounds, raisin and fruit seeds, rags, paper, bits of meat and vegetables, and some feces. For this reason the real value of the removal of the small amount of grit which can be safely effected in such chambers should always be determined carefully before adopting them, for they may be found to have no useful purpose, particularly with a separate sewer system. They should not be expected to remove more than from 10 to 40 p.p.m. of the suspended matter.

Quantity and Character of Grit.—The quantity and composition of the grit collected in a grit chamber depend so largely on the composition of the sewage, the specific gravity of the grit, the dimensions of the grit chamber and the rate of flow through it that reports of the grit collected in different cities rarely show close agreement. In addition to these factors, part of the storm water, which carries much grit, escapes through storm-water overflows in many combined systems of sewerage, so that not all the grit reaches the grit chambers.

In the Emscher District in Germany Imhoff found that with a velocity of 1 ft. per second very little organic matter settled with the grit and the latter averaged about 0.24 cu. yd. per 1,000 persons annually. This is the quantity estimated by Fruhling[1] as a fair average of many German reports on grit; but it is to be remembered that the dilution of the sewage is much less and hence its concentration is substantially greater in Germany than in America. At Worcester, Mass., with velocities of 0.2 to 0.6 ft. per second, averaging 0.5 ft., 3.2 cu. yd. of grit were collected annually per 1,000 persons, but most of the storm water did not reach the grit chamber in this case.

The grit which collects in the Worcester chamber when removed after 4 to 8 weeks is extremely offensive and must be covered immediately after removal to prevent the escape of such odors. It averages about 2,014 lb. per cubic yard. The dry solids run from 53.3 to 75.4 per cent averaging 64.1; the loss on ignition from 18.9 to 28.1 per cent; averaging 22.7; the organic nitrogen in dried samples from 0.515 to 0.871, averaging 0.675 per cent. The grit collected in grit chambers at Hamburg was found to consist of about 33 per cent of water, 3 per cent of organic matter and 64 per cent of mineral matter.

Design of Grit Chambers.—The quantity of sewage reaching a grit chamber fluctuates so greatly that in order to keep the velocity of flow within the desired range it is generally considered desirable to provide at least two independent basins in such a

[1] "Entwässerung der Städte."

chamber, as shown in Fig. 150. One of these will suffice for the dry-weather flow and moderate rainfalls and will afford an opportunity for cleaning the other, while both may be used when large quantities of sewage reach the chamber. The standard design of the Emscher District provides three narrow, parallel basins, Fig. 151. There is some uncertainty in estimating the velocity of flow through grit basins introduced by the pit in which the grit is caught, which is below the invert level of the inlet and outlet conduits. It is sometimes assumed that the

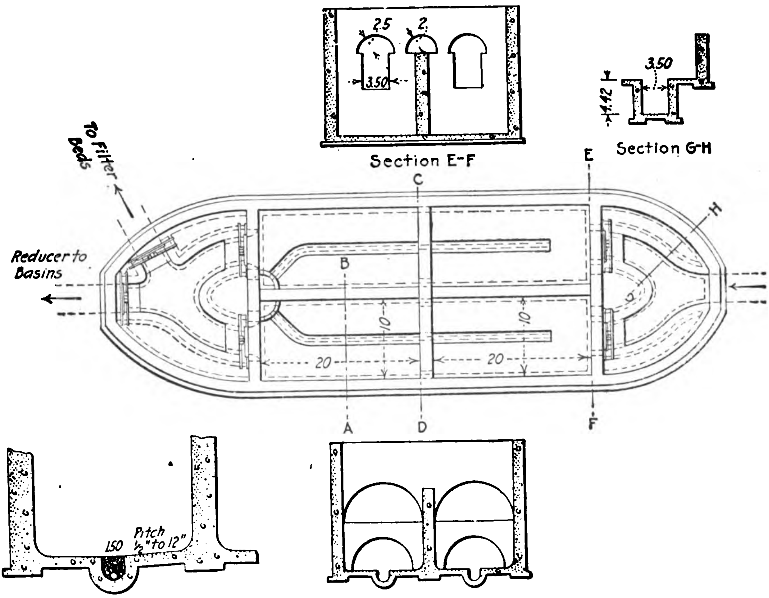


FIG. 150.—Grit chamber at Worcester, Mass.

liquid below this level is without motion, and experiments at several places have indicated the existence of a plane below which stagnation exists. Its position depends upon the shape and length of the chamber and is apparently below the invert level of the inlet and outlet in most cases, so that in estimating velocities some movement below this level should be provided for if the actual velocity of flow through the chamber is to be great enough to prevent the settling of organic solids.

A cross-section of the general shape shown in Fig. 52 is desirable for a grit chamber on account of its hydraulic properties. Where the grit is to be removed by shovel, the bottom slopes should be gentle, but if the grit is removed mechanically it is desirable to have the bottom form a long trough or succession of hoppers in the bottom of which the material will collect. There is some evidence to indicate that in order for the grit to slide to the bottom of such pits the sides must have a slope of at least 45 deg. Where mechanical cleaning is employed, it is desirable to round off all angles in order to facilitate the sliding of the grit to the bottom of the pits.

The longitudinal section of American grit chambers has generally had vertical end walls and a horizontal bottom. Experi-

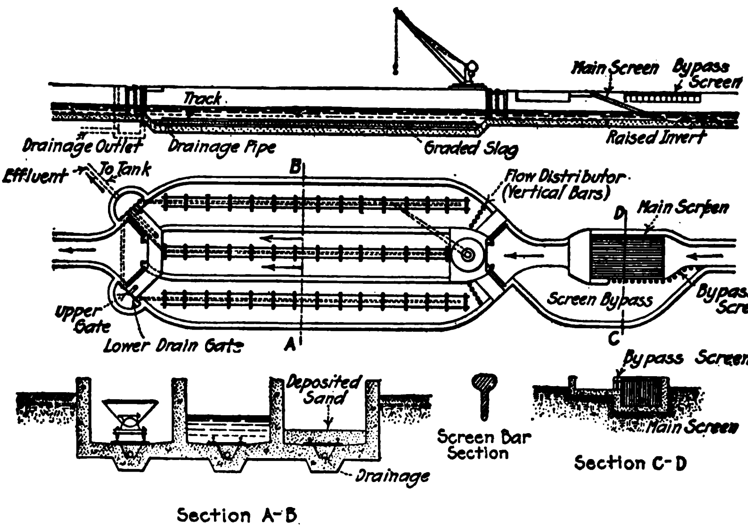


FIG. 151.—Imhoff screen and grit chamber.

ments have shown that there are vertical currents at these end walls. At Rochester, N. Y., for instance, Brown[1] found that the vertical wall at the outlet of the chamber caused such a strong eddy that submerged floats were raised by it into the currents moving downstream. Unless it is particularly desired to have these eddy effects, the outlet end of the channels or basins may be flared out so as to give a gradual change in the velocity of flow, although it is rarely possible in such short basins to use the most desirable angle of flare, 13½ deg.

There is a great difference of opinion regarding the depth of the pit in which the grit is collected. Fig. 150 shows a deep

[1] Report on Sewage Disposal System of Rochester, 1913.

pit, such as is used when it is desired to have a few inches of stagnant sewage over the largest quantity of grit which it is proposed to collect before cleaning. Fig. 151 shows a shallow form of construction used when it is desired to have sufficient velocity at all times in all parts of the chamber to prevent the deposit of organic solids. The length of the grit chamber should be governed mainly by the rate at which the grit settles in sewage flowing at the assumed minimum rate, a subject discussed in Chapter XII.

Cleaning Grit Chambers.—Provisions must be made for cleaning a grit chamber as soon as the deposits in it reach such an elevation that fluctuations in the velocity of the current above them disturb the surface of the grit. If there is but one basin in the chamber, there should be a bypass around it to carry the sewage during cleaning, unless use is made of some mechanical method of doing the work without interrupting the flow through the chamber. It is probable that mechanical cleaning can be employed on quite small basins, as buckets are now made in small sizes easily handled by portable derricks or gantry cranes. If a basin must be put out of service for cleaning, some provision must be made for unwatering it in order to get at the grit. Where this cannot be done by drains, as in Fig. 151, portable diaphragm or centrifugal pumps may be used.

Where hand cleaning is done, provision must be made for shoveling the grit into buckets which can be dumped into the carts which remove it. Inasmuch as rapidity in the removal of grit is desirable, even when the material is handled by laborers, the permanent installation of tracks and a pillar crane as shown in Fig. 151 has much to commend it. These chambers are most useful during the first part of a heavy storm, when their capacity may be taxed to the utmost, and the ability to clean a basin quickly may save considerable damage to a pump or the filling of valuable space in an Imhoff tank with mineral grit which should never be found there if practicable measures to prevent it can be provided.

Where the basins are cleaned by clamshell buckets, it is desirable to protect their floors with some hard covering, such as a granite block pavement or steel plates, to prevent the buckets from breaking them.

Theory of Screening.—There are two distinct services rendered by screening. The first is the removal of material likely to obstruct or injure machinery, to render diluting waters unsightly, and to cause deposits of coarse material in siphons and on the bottoms of bodies of diluting water. For this purpose coarse screens are used, generally made of parallel bars which are seldom less than ½ in. apart. Fine screening is carried on to remove a much larger proportion of these substances and is essentially a treatment process, whereas coarse screening is rather a precautionary process to insure the satisfactory operation of the sewage disposal or sewage-treatment system. Fine screens are commonly made of perforated metal or wire cloth, the perforations rarely exceeding ¼ in. in size. In using them care must be taken not only to keep them clean but also to prevent sewage passing through them with velocities high enough to macerate the organic matter which it is desired to intercept.

The claim that fine screening is the equivalent of sedimentation, which is sometimes made, is not justified. While most of the matter taken from sewage by fine screens can be removed by sedimentation, not all of the material which will be collected in sedimentation tanks is removed by fine screens. In sedimentation, weight determines the separation of solids from the sewage, whereas in screening the separation depends primarily upon size. Screening tends to break up the suspended solids whereas sedimentation leaves them unchanged.

From such data as are now available it appears that coarse screens will remove from less than 5 to perhaps 10 p.p.m. of coarse material and that fine screens may be expected to remove from 20 to 70 p.p.m., when the sewage is strong and fresh and where conditions are otherwise favorable for screening.

The disintegration of suspended matter during its passage through sewers and pumping machinery has an important bearing upon the efficiency to be expected from a screening plant. In this way much of the matter is converted into a colloidal condition, which enables it to pass the screen. It is important, therefore, when fixing the size of screen openings, to give careful consideration to the nature of the sewage which must be screened and the location of the screens. Under certain conditions it may be wise to provide for screening the sewage before it has time and opportunity to become disintegrated and colloidal.

Racks or Parallel-bar Screens.—Racks are often used in connection with grit chambers, and in such cases the best position for them depends to some extent upon the type of grit chamber and the kind of machinery, if any, used to remove the grit. Where the grit chamber is large and intercepts much coarse organic matter, the work of cleaning the racks will be less if they are placed at the outlet of the chamber instead of at the inlet. An objection, sometimes serious, to placing racks above grit chambers is that when partially clogged the racks cause the sewage to back up in the sewers, forming deposits there rather than in the grit chambers. As the sewage passes the racks it is given an irregular eddying motion likely to reduce the sedimentation in a chamber immediately behind the racks, which is another reason for placing them in the outlets of such chambers.

Racks are of two general types, those which are stationary or "fixed," and those which are in motion while in service and are known as "movable." The same classification applies to fine screens.

Fixed Racks.—Racks and screens are generally placed at right angles to the axis of the channel they cross. Their length should depend upon the quantity of sewage to be screened, the size of the openings and the permissible loss of head due to them. If only a narrow channel is available, the desired screening area may be obtained by curving the rack in plan, or giving it an angular shape in plan, or placing it on a very flat slope. Boston experience indicates that with combined sewers the rack should have an area of at least 150 per cent of the channel leading to it.

The size of the openings depends upon the purpose of the screening. Where the rack screens sewage passing to centrifugal pumps with closed impellers, it is necessary to take out material of rather fine size in case the pumps are small, this size being ascertained from tables supplied by the makers of the pumps. Where open impellers are used, the openings of the racks need not be so small, and a clear distance between bars of 5/8 in. has been found to work well. The size of the openings in racks guarding treatment works is gradually becoming standardized at 1/2 to 1 1/2 in. A 1-in. opening has become standard in the Boston district for the racks screening sewage before its disposal by dilution.

Where the screening devices have an important function which must be performed regularly, it is necessary to provide duplicate apparatus so that one unit may be in service while the other is being repaired or cleaned. The duplication is provided either by using two channels, each containing one or more racks, or by placing one rack behind another in the same channel. The preference is generally given to two channels because where there is but one an accident to a screen may render it difficult for the other to serve its purpose.

There is considerable variation in the degree of importance attributed by managers of sewerage works to keeping the racks clean. In some small separate systems, where no storm water enters the sewers, hardly any attention is given to the removal of screenings. In some large stations, like those of the Boston Metropolitan District, the removal of the screenings is occasion-

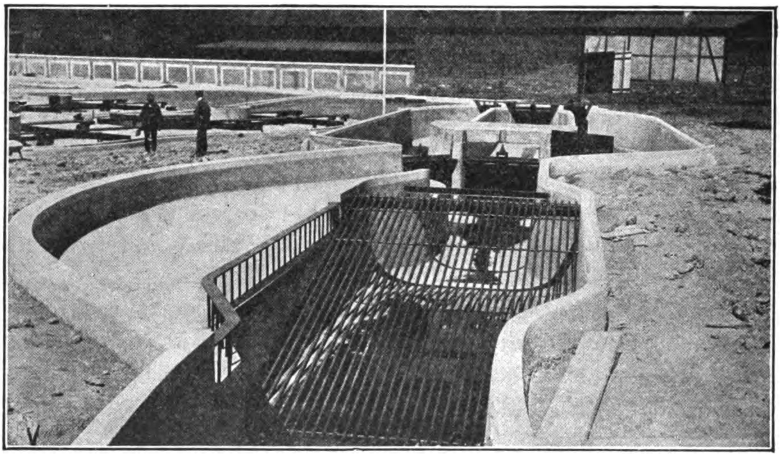


FIG. 152.—Waste-weir at rack of an Emschergenossenschaft plant.

ally an arduous task. There is apparently no way to foresee exactly what the conditions will be, and it is the designer's duty, therefore, to make the racks and their supports strong enough so that in case they become clogged and the sewage is dammed back by them, they will resist this extra pressure until their condition is relieved. If there is any likelihood of the development of such a head upon the rack, it may be desirable to install a float-operated alarm system which will automatically give notice when the sewage above the rack reaches a dangerous height, or this may be avoided by providing a bypass with a screened waste weir of sufficient length, as indicated in Fig. 152, to prevent the sewage from rising above a certain height.

Inclined racks are more easily cleaned than vertical racks and give a larger screening area, and for these reasons are most used. The inclination is from 30 to 45 deg. from the vertical, in most cases. In detailing the construction it is desirable to curve the top of the bars backward, so that the rake used in cleaning can be pulled back freely and carry the screenings into a trough or to a platform at the top of the rack without any difficulty. Where the bars forming the rack are of considerable length and must be supported at intermediate points as well as at the ends, they should have lugs or ears by which their rear edges can be attached to the supports in such a way that these supports will offer no obstacle to the tines of the rakes used in cleaning.

The details of the sides and bottom of the channel where the rack lies should be so worked out that there are no openings larger than the clear openings of the rack, particularly where it is desired to take out material less than 1 in. in size.

In all but very small installations grooves are left in the walls of the channel for the insertion of stop planks when it is desired to repair the racks. If stop planks or their equivalent are not provided for, it will be necessary to close the channel with sand bags or some other temporary expedient in order to repair the racks.

Cleaning Fixed Racks.—No matter whether it is expected that cleaning will be infrequent or frequent, provision must be made for it and for the expeditious removal of the screenings, which have usually a very offensive odor. When the racks are cleaned by hand, it is desirable to have some kind of trough or platform to receive the screenings as they are raked over the top of the racks. At a few European plants the screenings are raked over the top into a cart or small truck which is placed below the overhanging upper part of the rack, but such elaborate detailing is not common in the United States.

Mechanical cleaning is employed with some large installations of fixed racks. This is the case at Toronto, Ont., where there are six racks, two in chambers 33 ft. deep and four in chambers 14 ft. 3 in. deep. Each rack is made up of bars 10½ ft. long, ½ in. thick and spaced to give ½-in. openings. The width of a rack is 5 ft. 8½ in. They are cleaned by means of rakes attached horizontally to endless-chain belts driven by shafting and gearing at the top of the racks. The material clinging to the tines of the rakes is detached by a cleaning bar with four rows of teeth. The bar is placed horizontally in the headframe of the mechanism

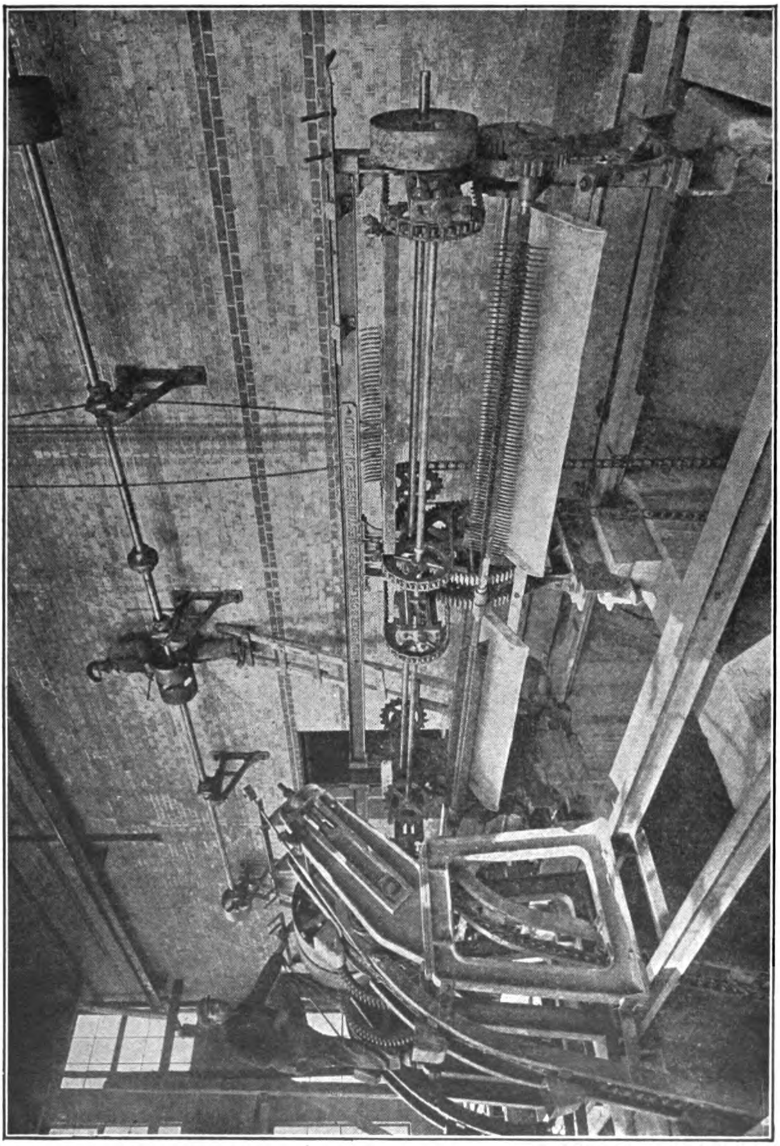


FIG. 153.—Headframes of rack-cleaning apparatus and grit elevator, Toronto.

for moving the chain belts and revolved so that the teeth pass between the tines of the rakes. All the refuse brought up in this way drops into a sloping tray or trough which discharges it into a screw conveyor. This conveyor also removes grit dropped into it by a bucket elevator used in cleaning the grit chamber in front of the screens. Fig. 153 shows this installation while it was being set up. On the right is the head frame for operating one of the endless-belt devices, with one of the rakes attached to the chains and provided with a part of the malleable-iron teeth which are used to drag the screenings to the surface. The

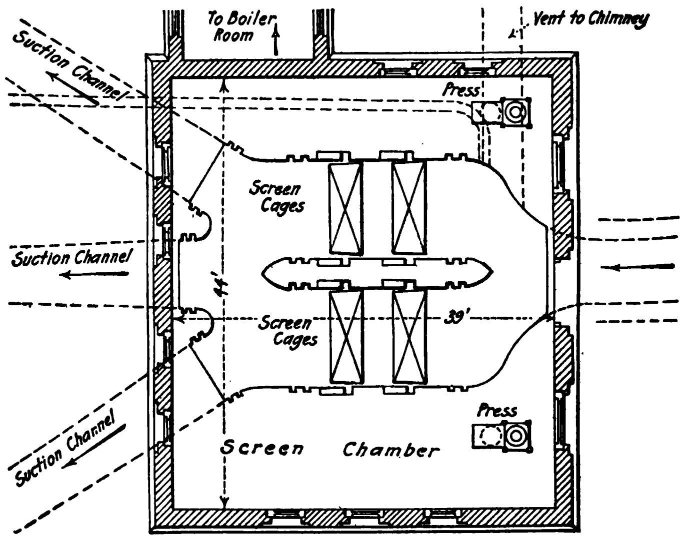


FIG. 154.—Screen chamber at Ward St. station, Boston.

revolving bar for cleaning the rakes is also shown in place. On the left are the head frames of some of the bucket elevators used in raising grit.

Cage Racks.—The only type of movable racks which has been used to any extent in the United States is shaped like a cage, and is raised vertically to be cleaned. It has a bottom of steel plates or a grid of iron bars and three sides formed of upright bars, the fourth side being the opening which is placed across the channel through which the sewage passes so that the rack resembles a basket. Fig. 154 is a plan of a typical screen chamber at Boston and Fig. 155 gives a general idea of the machinery by which a cage is operated. Each cage is about 9 ft. high and wide

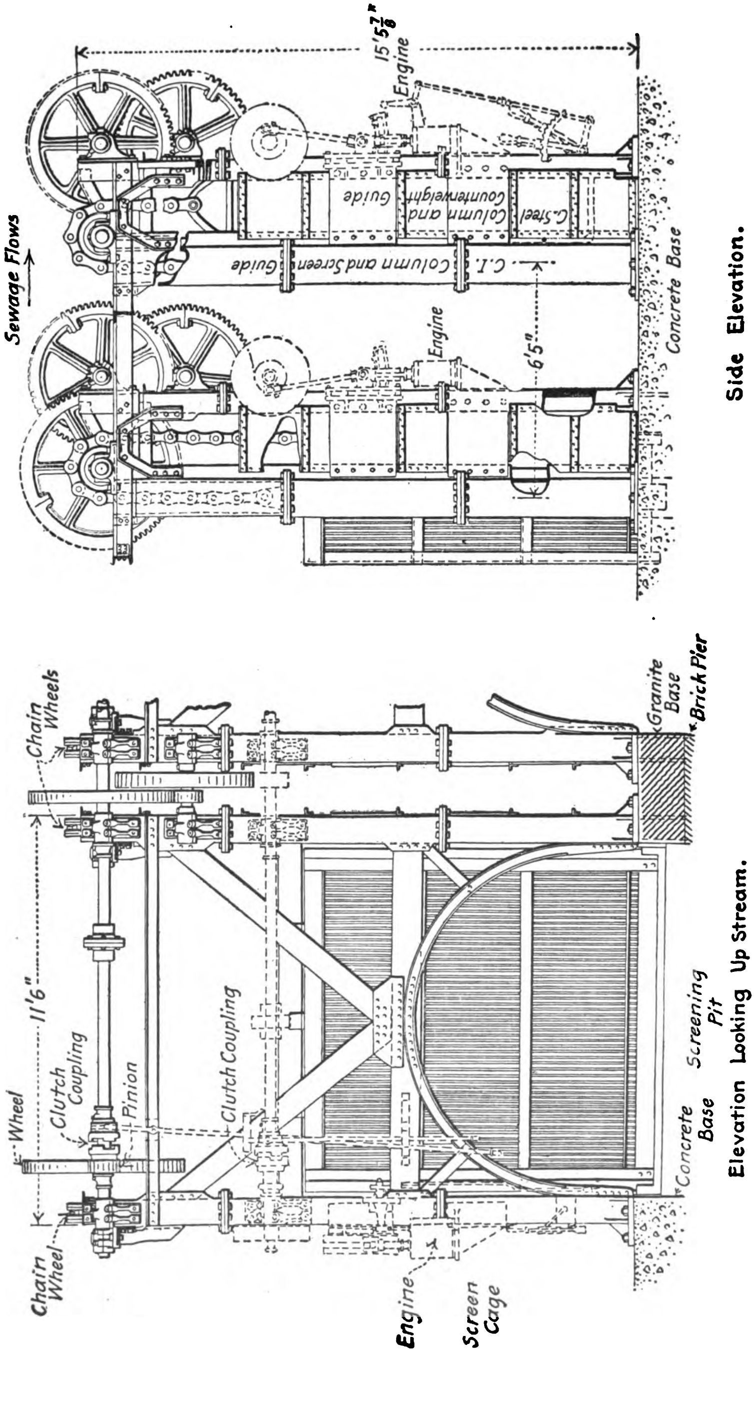


FIG. 155.—General features of screening machinery, Ward St. station, Boston, Mass.

and $3\frac{1}{2}$ ft. across from front to back. The bottom is of steel plates perforated with holes for drainage, and the sides are vertical $\frac{3}{4}$-in. bars with clear openings of 1 in. left between them. The cages are counterweighted and operated by small reversing engines. Similar cage racks more recently installed at Boston, Washington and Milwaukee are equipped with electric driving motors. During heavy storms the screens need constant attendance and cleaning. The screenings are pressed in a hydraulic press to drive out a large part of the moisture and are then burned in a boiler plant in the immediate vicinity.

Wing Racks.—The wing or Frankfort rack is a type of movable rack developed for the purpose of keeping all parts subject to frictional wear above the sewage. This is accomplished by placing three to six racks on a revolving horizontal shaft, as shown in Fig. 156. As it is necessary to have at least one rack intercepting the entire channel at all times, for otherwise part of the sewage will not be screened, a depression must be made across the bottom of the channel and at least three racks used. The larger the number of racks, the smaller the depression need be.

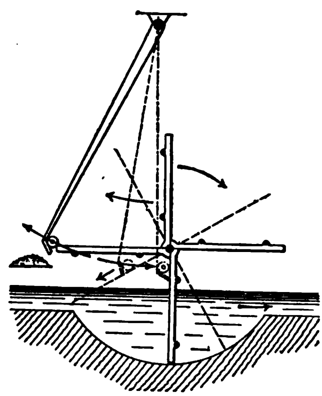


FIG. 156.—Frankfort wing rack.

The racks are cleaned mechanically. A pendulum arm hinged at the top carries a brush at the bottom, which pushes the screenings from the inner edge of each rack toward the outer edge, finally delivering them upon a belt conveyor. The face of the brush is protected by a light rake. The rack revolves in a direction opposite to the current, and the pressure tending to force material through the rack is somewhat greater than that due to the current itself. If the rack is built with small openings, the collection of screenings against its face may increase the pressure so as to make considerable power requisite for driving the apparatus.

A modification of this form of rack, in which the racks are curved instead of flat and a somewhat different method of cleaning them mechanically is used, has been introduced in a few European cities. It is called the Geiger rack.

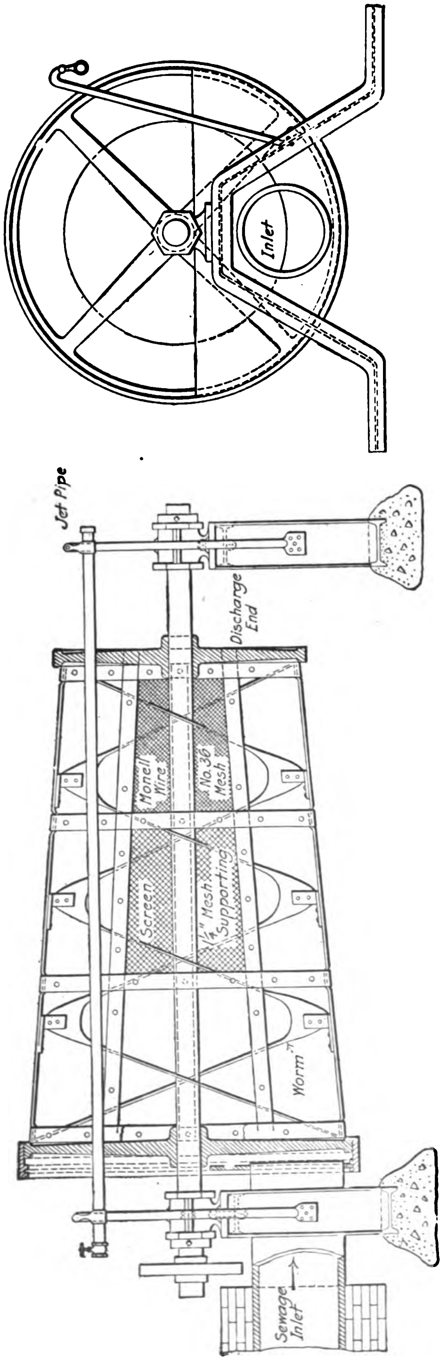


FIG. 157.—The Weand drum screen.

Link-bar Racks.—Very broad endless link belts are used as movable screens in several German cities, the most notable installation being at Hamburg. The latest of these plants there has belts made of links 14.2 in. long, held in angle-iron frames about 9.8 ft. long and 15 in. wide, so as to leave 0.4-in. openings between the links. Each rack consists of 46 of these frames, hinged together in such a way that the openings between the frames are very small. The links were first made of wood, then of hard rubber and finally of an aluminum alloy. The rack moves around sheaves at the top and bottom and is cleaned by a rake or stripper having a long row of rubber teeth passing across the entire width of the rack.

Drum Screens.—Drum screens are horizontal cylinders covered with perforated metal or wire cloth and revolved slowly while the sewage flows into them. The screenings are collected inside the cylinders and must generally be loosened from the screens by jets of water or air directed against the outside of the cylinders near their top. In Europe the cylinders are usually short and of large diameter, while the few used in the United States are longer and of smaller diameter.

This type was first developed in the United States by O. M. Weand for use at Reading, Pa. The first installation had a 40-mesh screen on a cylinder 6 ft. diameter and 12 ft. long, making about 8 r.p.m., Fig. 157. It was used for 4½ years screening an average daily flow of 5,500,000 gal. from an estimated contributory population of 40,000. From 5,000 to 10,000 gal. of water were needed daily to clean it and about 5 hp. was required for its operation. It removed about 1,500 lb. of wet screenings per million gallons of sewage, and these screenings were partially dried to a weight of 950 lb. in a centrifugal machine. About 20 per cent of the suspended matter in the sewage was removed in this way, at a total cost, including maintenance and repairs, of about $2 per million gallons of sewage (under pre-war prices).

Another screen of this type was placed in service in Brockton, Mass., in 1911, but has since been removed. It was 12 ft. long, 6 ft. in diameter and covered at first with 38-mesh wire cloth, later replaced by 30-mesh cloth. It was driven by a 20-hp. oil engine and about 20,000 gal. of water were used daily in washing it. From 1,500 to 2,000 lb. of screenings were removed from each million gallons of sewage, and were partly dried in a centrifugal machine. They were used for filling at first, being covered at once with ashes or soil, but later they were mixed with coal and burned under boilers, as was done at Reading.

A drum screen 11½ ft. in diameter and 9¼ ft. long is used to screen settled sewage at Baltimore. A 26-mesh wire cloth is used. The introduction of the screen made it possible to dispense with about 50 man-hours of labor daily for cleaning the nozzles of trickling filters and kept the filters in better operating condition than was practicable when that amount of labor was used.

The Dorrco screen, Fig. 158, was first developed for screening industrial wastes and was first employed in screening municipal sewage at the Dorr Company's experimental activated sludge plant at Mt. Vernon, N. Y., where it removed 12.2 per cent of the suspended solids and delivered screenings with 80 per cent average moisture content. The drum is a plate punched with $\frac{3}{64} \times \frac{1}{2}$-in. slots parallel with the shaft on which the drum rotates. The drum is partly submerged in the sewage, which passes through the slots and is discharged through an opening in one end of the drum. Except for this opening, both ends are closed. At one point on the circumference of the screen a 1 by 1 by ⅛-in. angle extends the whole length of the drum. The drum is driven so as to have a peripheral speed of

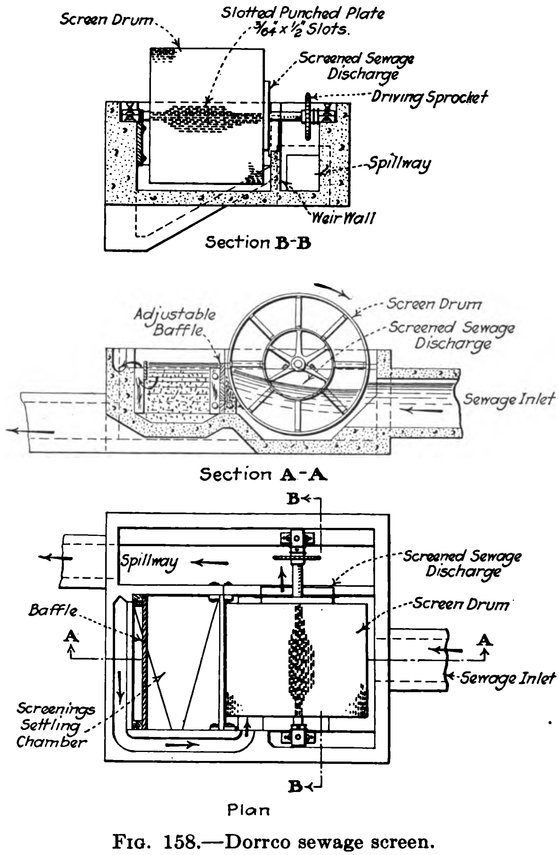
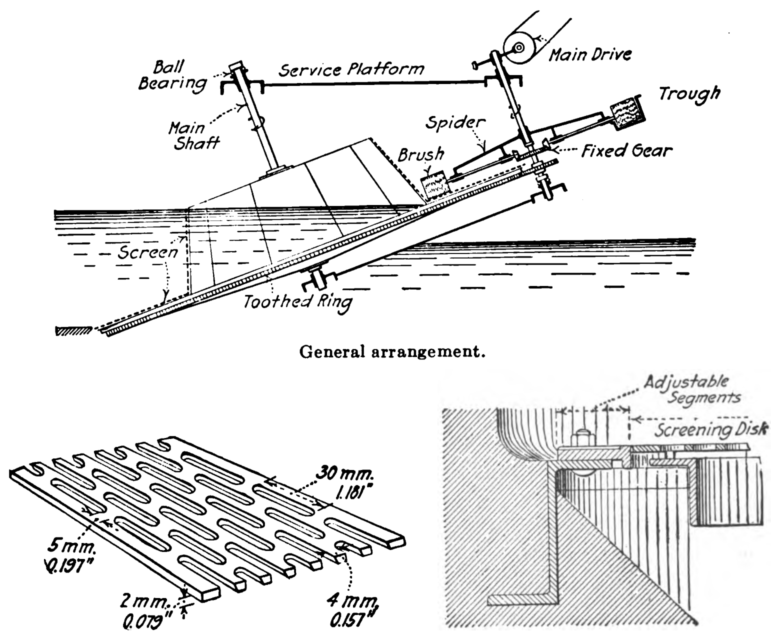


FIG. 158.—Dorrco sewage screen.

300 ft. per minute. About ½ hp. is required to drive a drum 4 ft. 10 in. in diameter and 4 ft. long. The makers rate the screen at 50,000 gal. per 24 hr. per square foot of screen area.

The raw sewage coming through the inlet strikes the screen as it rotates and the coarse particles are held against the outside of the screen by the pressure of the sewage. The rotation of the screen causes a depression in the water level at the point of submergence and an elevation of the water level at the point of emergence, thus causing an outward flow of the liquid at the latter point. The coarse particles are carried on the surface of the screen until they reach the point of emergence, where they are forced off by the outward flow of the liquid. By this action, the space at the point where the screen emerges from the liquid becomes filled with coarse solids, and as the angle on the circumference of the screen reaches this point it carries a portion of the solids up and throws them over the baffle into the screenings



General arrangement.

Screening plate. Detail of seal.

FIG. 159.—Riensch-Wurl screen.

settling chamber. Here the screenings settle and are drawn off by a pump. Any liquid carried with the coarse solids into the screenings settling chamber flows under the baffle and over the side wall into the shallow trough which returns it to the screen chamber.

Disc Screens.—The disc, separator, or Riensch-Wurl screen, provides an adequate screening area by placing the screening surface on a very flat slope, as shown in Fig. 159. It was developed in Germany and has been used to some extent in the United States. It consists of a disc made of sheets of perforated metal, with or without a frustum of a cone attached to its center, the whole mounted on a shaft whose inclination from the vertical determines the tilting of the disc. One-fourth to one-third of the area of the discs is cut out by the perforations through which the sewage passes. The weight of the discs in screens up to 16 ft. diameter is carried by a ball bearing at the top of the shaft. In larger sizes the shaft is stationary and carries an annular ball-bearing support on which the frame moves. In each case the bearing is above the level of the sewage.

As the screenings are raised above the surface of the sewage, they are swept into a circular gutter by brushes on the ends of the arms of a large spider. The brushes revolve and the arms

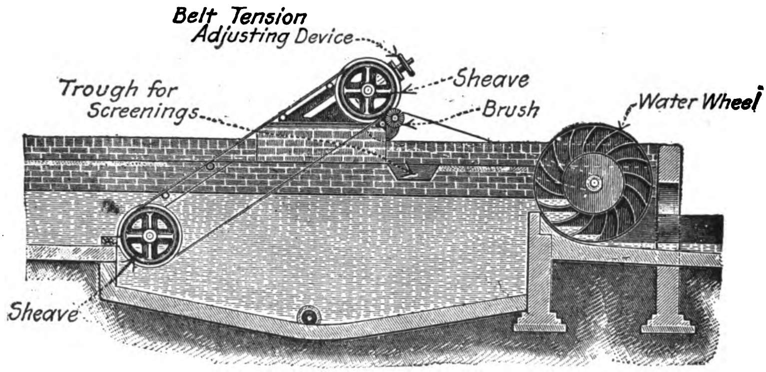


FIG. 160.—A typical English belt screen.

also revolve, the combined motion of the several parts being such that every portion of the screening surface of the disc is passed over at least twice. This cannot be accomplished, however, with the conical screen in the center of the disc, which is cleaned by a vertical brush of conical form. Where there is much fat in the sewage it has been found desirable to blow steam or hot water over the disc once or twice a day, and the condition of the brushes is improved by the same treatment, and by washing with kerosene oil.

Screens of this type have been constructed with discs from 4 to 26 ft. in diameter and with perforations from 0.03 to 0.2 in. wide. The largest plants installed down to the time of the world war were at Dresden and Bremen.

Belt Screens.—Endless-belt screens are used quite extensively in Great Britain. They are generally supplied by a manufacturer at Carshalton, in Surrey, whence the common British name of "Carshalton" screen used for them. The belt consists of twisted steel wires and plates perforated with ¼ to ½-in. holes, and is operated as shown in Fig. 160. It is not considered desirable to attempt to screen more than about 3,500,000 gal. per day with a single belt; two belts should be used for a larger quantity of sewage.

A belt screen used in screening sewage from the Union Stock Yards at Chicago consists of two parallel chain belts carrying removable frames to which 40-mesh wire cloth is attached. The cloth is cleaned by jets of compressed air, jets of water and a cylindrical brush. About 1,000 lb. of refuse per million gallons of sewage are removed.

Screenings.—Reliable information regarding the quantity of screenings is meager and somewhat incomplete. Table 44, however, gives typical data obtained with various kinds of screens at certain American cities.

The offensive character of screenings makes their prompt removal very desirable.

Coarse screening to remove large floating matter in sewage will continue to be practiced extensively. In this connection however the experience with screens at Worcester, Mass., is instructive. When the chemical precipitation works were put in operation in 1890, about 3,000,000 gal. of sewage was screened through racks with openings of about ¾ in. Cleaning them called for about all the time of a day laborer during dry weather and had to be carried on continuously through the 24 hr. in wet weather. The expense was such that the racks were taken out and the whole of the suspended matter in the sewage went to the sedimentation basins, until the grit chamber, Fig. 150, was built. At this time the sludge was pumped from the sedimentation basins by a centrifugal pump and it was found necessary to screen the sludge before it entered the pump. A laborer had to be stationed at the rack to keep it clean while the pumping was being done. Eventually a Shone ejector replaced the centrifugal pump, and as it was able to handle very coarse material screening was unnecessary. Thus, by practical experiment, the entire cost of screening was eliminated at Worcester, except for screening the portion of the sludge pumped to filter presses. This experience corresponds with that in Birmingham, England, where it has been found more desirable to screen sludge than sewage.

TABLE 44. QUANTITY OF SCREENINGS AT FIVE AMERICAN CITIES

City	Boston, Mass.		Milwaukee, Wis. Experiment Station	New York City, Dyckman St.		Rochester, N. Y.	Reading, Pa.
	North Metropolitan	South Metropolitan					
Date	1913 to 1919		2/3/19–7/13/19	6/9–10/19	6/18–19	1919	1911
Type of screen	Cage		Link belt	Disc	Disc	Disc	Drum (Weand)
				Riensch Wurl			
Screen opening, inches	1.0	1.0	1/8 by 2	3/64 by 2	1/16 by 2	3/8 by 2 (1) 1/8 by 2 (1) 1/16 by 2 (2)	40 meshes
Suspended solids in crude sewage, p.p.m.	....	....	294.0	137.0	122.0	...	133.0
Suspended solids removed p.p.m.	....	....	10.0	36.0	20.0	...	20.0
Suspended solids removed, per cent	....	....	3.4	26.3	16.4	2.8	15±
Screenings moisture, per cent	....	....	89.7	78.5	81.7	...	89.5
Cubic feet per 1,000,000 gal.	2.57	3.55	12.6	22.7	16.7	6.4	27
Pounds per 1,000,000 gal.	....	....	595.0	1,264.0	928.0	...	1,620.0

CHAPTER XII

SEDIMENTATION AND SEPTIC TANKS; CHEMICAL PRECIPITATION

Sedimentation tanks are basins in which the sewage stands quiescent or flows at a low velocity, permitting a portion of the suspended solids to settle to the bottom, forming a deposit of sludge which must be removed at intervals. While grit chambers and screens will remove the heavy, sandy matter and the larger objects floating and suspended in sewage, there are many cases where it is desirable to remove still more of the suspended solids containing organic substances. If these suspended solids are discharged into natural bodies of water they may form offensive sludge deposits. They tend to clog fine-grain filters and to impose an unnecessary burden on coarse-grain filters. Where the sewage is disposed of by dilution so that bathing beaches or shellfish beds are threatened, disinfection of the sewage is advisable, and disinfection is much more effective if applied to sewage after clarification by sedimentation than when it is applied to raw sewage or screened sewage.

In the hypothetical analysis of a typical sewage of medium strength, Fig. 143, it was assumed that the suspended solids amount to 300 p.p.m., of which 150 p.p.m. are capable of settling in 2 hr. Still larger portions are capable of settling in longer periods. By passing such sewage through grit chambers and fine screens, it appears that in general from 15 to 75 p.p.m. suspended matter may be removed, thus leaving from 75 to 135 p.p.m. capable of settling in suitable tanks. But it is probable that much grit and coarse suspended matter in the original sewage would not be reported in an analysis, and it is safe to assume that a large part of it is not included in the 300 p.p.m. assumed in the hypothetical analysis. Therefore a large portion of the 150 p.p.m. will pass through the grit chambers and screens.

The septic tank is a sedimentation basin in which an opportunity is intentionally given for anaerobic decomposition of the sewage and sludge, mainly for the purpose of decreasing the quantity of sludge.

Sedimentation.—The accepted theories of sedimentation assume as working conditions that the liquid is clear water and

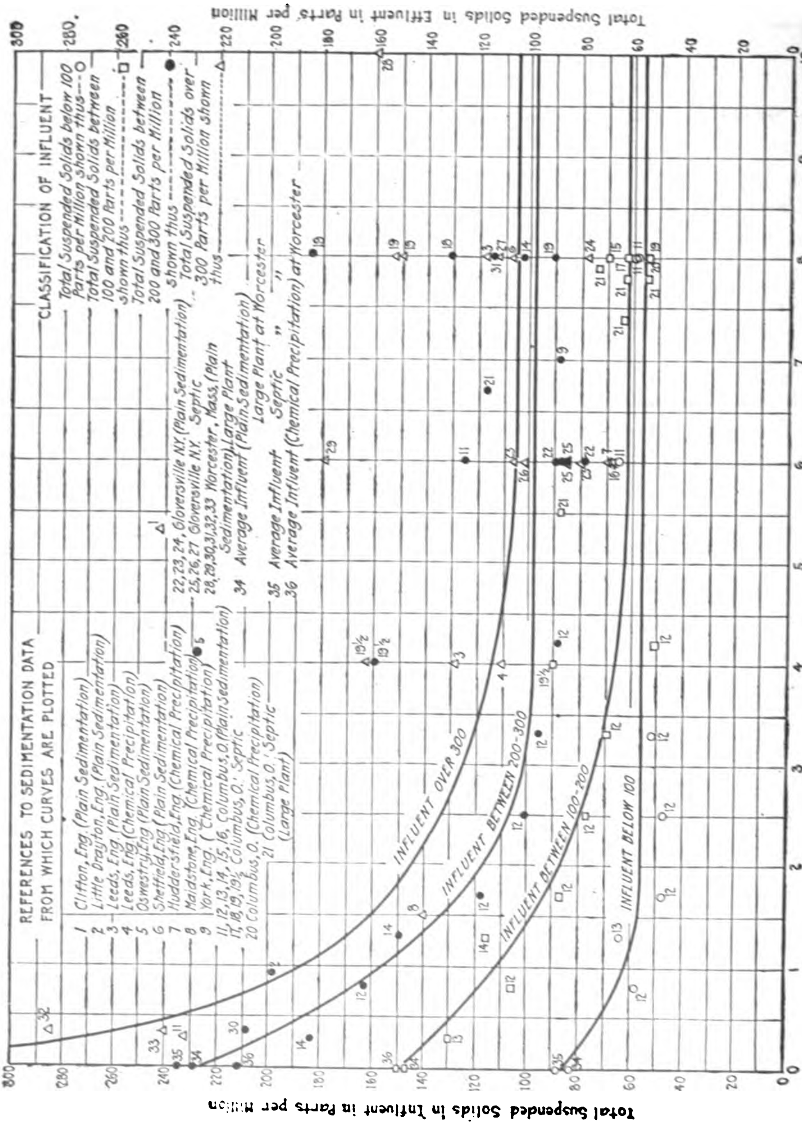


FIG. 161.—Removal of suspended solids in sewage by sedimentation.

that the particles are of about the same character. Practically, the sedimentation of sewage is greatly complicated by the changes constantly occurring in its composition, by the evolution of gas, by the decomposition of sludge, by the presence of extremely fine suspended matter and colloidal matter, and by temperature changes. Currents exist in basins at times to such an extent that the actual velocity of a particle of suspended matter may be several time the average velocity with which the whole volume of sewage passes through the tank. As a matter of fact the velocity of

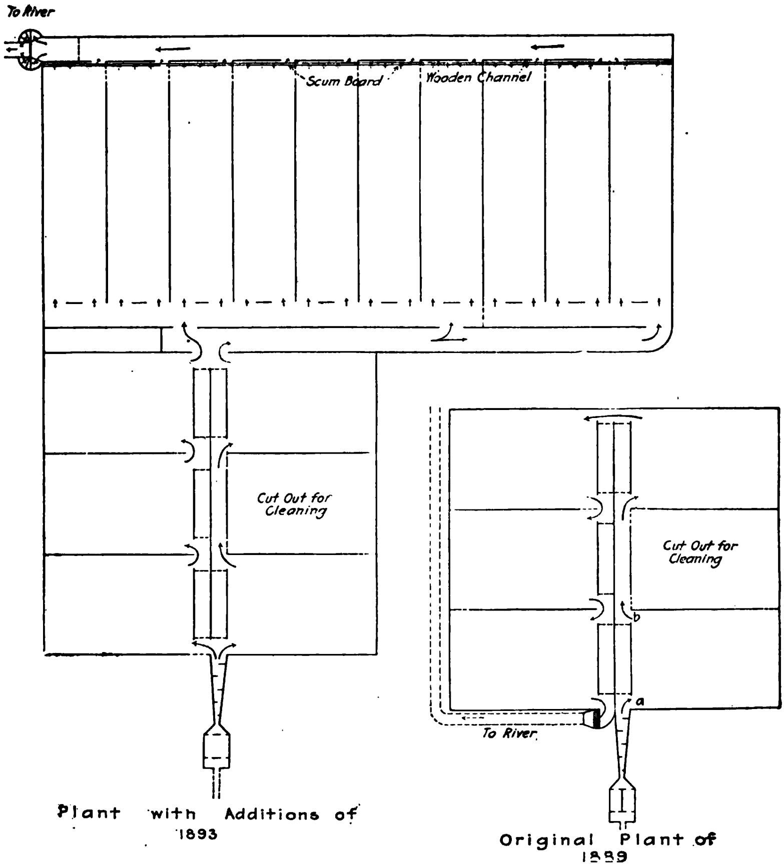


FIG. 162.—Sedimentation tanks at Worcester, Mass.

flow within practical limits appears to have very little if any effect upon the efficiency of sedimentation. The period of detention, however, is very important essentially governing the proportion of the suspended matter removed from a particular sewage. To attempt to estimate the sedimentation in a large settling basin on theoretical principles is very likely to prove unsatisfactory, and the authors prefer to employ as a general guide in such estimates the actual results obtained by settling basins in a considerable number of cities which furnish sewages of different chacteristics. Fig. 161 gives such data.

Types of Sedimentation Basins.—The older type of settling basins is a shallow, rectangular structure through which the sewage flows horizontally. Recognizing the fact that fine and light particles of suspended matter require a longer time in which to settle than do heavier particles, many of the older sedimentation plants were designed to provide a long route through which the sewage was obliged to flow. This was frequently accomplished by arranging the tanks in series, like the original chemical precipitation tanks at Worcester, Mass., Fig. 162. Here the sewage was admitted to the first tank at *a*, allowed to flow through the tank and out at *b*, then in the same way through each of the other tanks, unless one or more were cut out for cleaning. Relatively high velocities caused considerable agitation at the inlet and outlet weirs.

Additional tanks were subsequently provided and the old tanks were used for roughing basins to collect the heaviest part of the suspended matter. The sewage passed through them in series but was admitted to the new tanks in parallel, thus providing for a minimum rate of flow in the latter. This method of operation proved much more satisfactory than the old method, especially because it facilitated the handling of the sludge, a large portion of which settled in the roughing tanks. These were cleaned at frequent intervals, at times as often as once in 2 or 3 days. The secondary or finishing tanks received relatively little sludge and did not require cleaning more often than once in 2 or 3 weeks in summer, to prevent active putrefaction of the sludge, while in winter they were sometimes used for 5 or 6 weeks without being cleaned.

Some settling tanks are cylindrical, sewage entering somewhat above the bottom and overflowing at the top, Fig. 163. Sludge is drawn from the conical bottom without requiring the emptying of the tanks, and may be delivered by gravity at a considerable elevation above the bottom of the tank. In such tanks, the upward flow must be less rapid than the downward velocity of most of the settling particles, some of which tend to collect in a stratum between the influent orifice and the overflow weirs. Particles moving upward may be caught in this stratum, which acts as a sort of filter and is probably some aid to the sedimentation in such tanks.

Vertical-flow tanks are particularly adapted to the clarification of the effluent from trickling filters, because the suspended matter in such effluents, if allowed to accumulate on the sides and bottom of very shallow tanks, will reduce the quantity of dis-

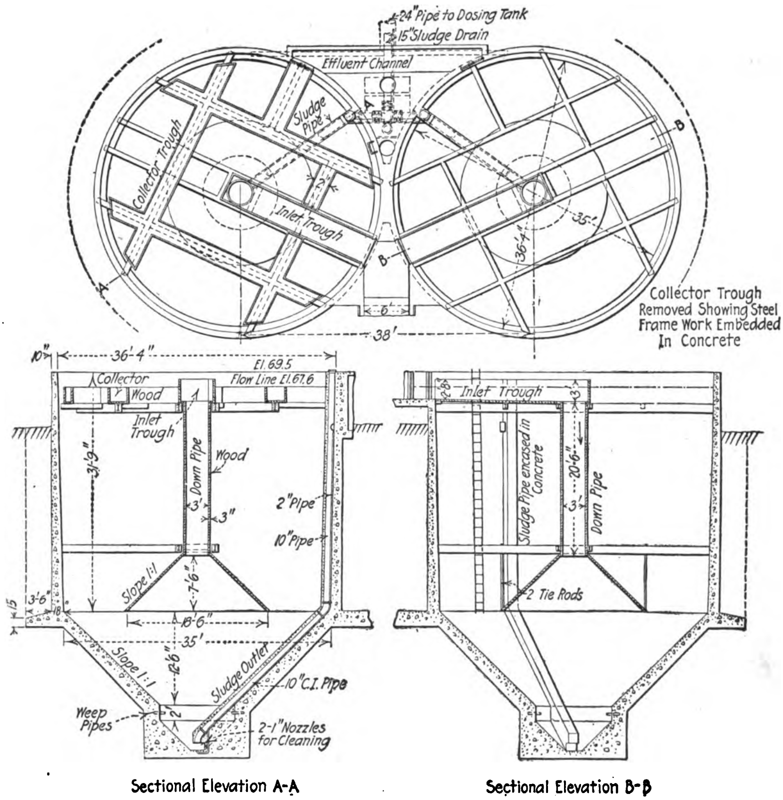


FIG. 163.—Vertical-flow settling tanks, Gloversville, N. Y.

solved oxygen in the liquid passing through them. This makes it desirable to reduce as much as possible the time of contact of the liquid and sludge, which is best accomplished in this type of tank.

Detention Period.—When sewage stands quiescent or moves with a very low velocity the suspended matter settles out gradually, the heaviest portions first. After prolonged standing, a part of the suspended colloidal matter may be thrown down gradually through coagulation due to physical contact or changes in the composition of the sewage, and eventually a part of the dissolved colloidal matter may be precipitated similarly. Such prolonged sedimentation is not practical in sewage treatment, not only because of the expense of providing sufficient settling basins but also because the changes which take place in sewage and sludge would generally render such a process inadvisable.

The quantity of suspended matter precipitated in various periods of time from the sewage to be treated should be ascertained if possible, and that period of sedimentation chosen which will cause the removal of the largest quantity required by the conditions. This fixes the detention period, or the length of time assumed to be required for the influent to displace a tankful of sewage. If the rate of sewage flow is 2,400,000 gal. a day and sedimentation for 1 hr. is considered sufficient, then the detention period will be 1 hr. and a tank capacity of 100,000 gal. must be provided. Such a period may be very short, perhaps not over 30 min. in some cases, and it is unlikely to exceed 4 to 6 hr.

In the case of horizontal-flow tanks, it was formerly the practice at some works to fill the tanks with sewage, allow it to stand for the desired period of time, and then draw off the supernatant liquid. This method of operation, called the fill-and-draw method, is very rarely so desirable as the continuous-flow method of operation. If the sewage passes through sedimentation tanks sufficiently slowly to provide a suitable period of detention the sedimentation efficiency is substantially as good as in the fill-and-draw method where a similar detention period is afforded and the operation of the tanks is much more convenient and economical.

Design of Horizontal-flow Tanks.—It is customary to build tanks from 6 to 10 ft. in depth and with a ratio of width to length of 1:2 to 1:6. It is desirable even though more expensive to keep the width small in comparison with the length, in order to promote uniformity of distribution of the sewage on entrance and uniformity in its flow through the basin. In large basins this uniformity of flow is affected by winds, oscillations of the sewage due to gases escaping from the sludge, and by differences of temperature of the sewage in different parts of the basin. The length is governed by the detention period, the quantity of sewage to be handled, and the area available for the basins. The number of basins should be sufficient to afford flexibility of operation by cutting in or out one or more basins. Even small works should be provided with at least two units, to allow cleaning and repairs without interrupting service.

The heavier and greater part of the settling solids subside quickly near the inlet end if velocities are not excessively high. Therefore it is convenient to provide a somewhat greater depth at this end, so that the sludge may be retained as close as possible to the sludge outlet sluices, which should be placed at the inlet end of the tank. This will result in saving considerable labor where tanks are cleaned by scraping and squeegeeing the sludge. The gates for drawing off the supernatant liquid when the basins are to be cleaned may well be placed at the outlet end of the tank,

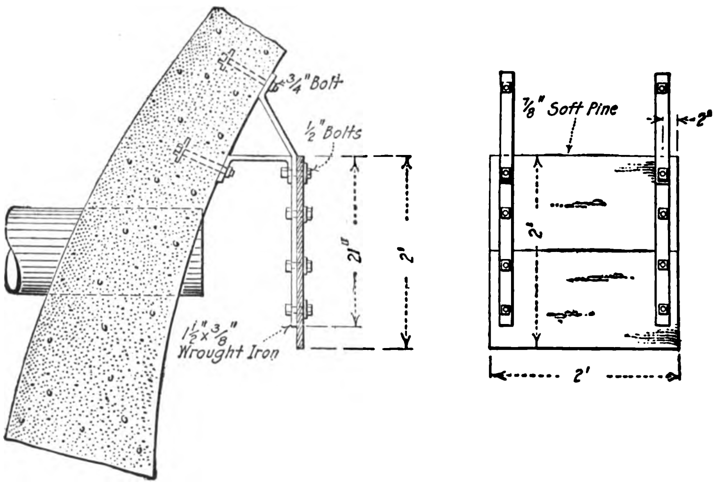


FIG. 164.—Baffle plate in front of inlet pipe of settling basin.

where the sludge is usually not so deep as at the inlet end and it is frequently possible to draw the water lower than at the other end.

The most obvious way to effect uniform distribution of the sewage entering and leaving a basin is by means of inlet and outlet weirs extending entirely across the ends of the basin. But, while such weirs are satisfactory for the effluent, they are not satisfactory for introducing the sewage into the tank. This is chiefly because of the tendency of suspended matter to settle in the influent channel and to collect on the edge of the weir. Attempts have been made to solve this difficulty by cutting orifices in the side of the influent channel, but they are generally unsatisfactory. The best method appears to be to provide a few relatively large openings through which the sewage will flow with moderately high velocity, which is checked almost immediately by small baffles placed in front of and close to the influent openings. Fig. 164 illustrates such a baffle used by the authors in a covered sedimentation tank.

Baffles and scum boards, used quite generally to assist in maintaining uniformity of flow, must be carefully arranged, for increasing the rate of flow by too much baffling will result in mixing and stirring up the sludge. Scum boards extending a few inches below the surface are usually placed near the outlets, and sometimes near the inlets also, to prevent movement of the scum.

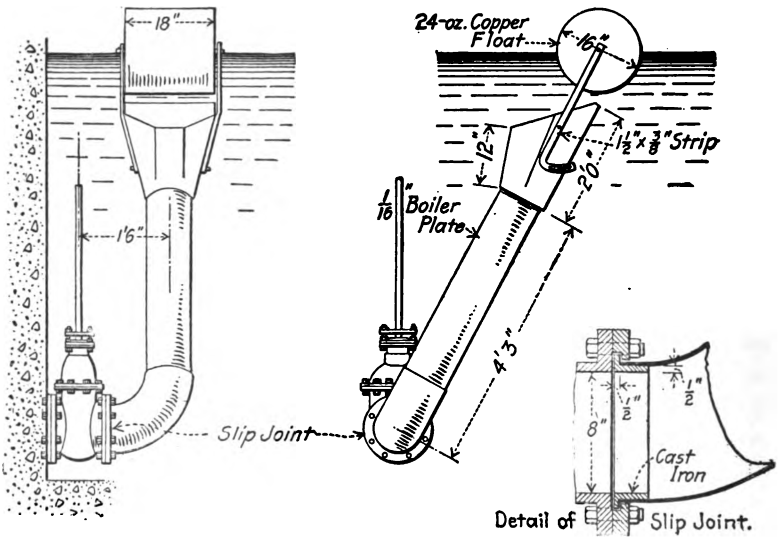


FIG. 165.—Swiveling outlet pipe for settling basins.

Sometimes transverse baffles extending to a considerable distance below the surface are placed at equidistant points throughout the length of the tank, these in turn alternating with sludge dams 1 ft. or more high, built across the bottom of the tank. Experiments made at Philadelphia show a marked superiority in baffled tanks over similar tanks without baffles.

In many small and medium-size plants the sedimentation basins have been roofed, sometimes on account of severe winter weather and sometimes to conceal the tanks and their contents from public view. There is a small but indeterminate increase in the efficiency of the tanks caused by roofing them but not enough to justify the expense of the covering. Open tanks should have their walls extend at least 6 in. above the surface of the liquid in order to prevent the latter from being blown over the coping by winds. They should also be surrounded by fences if there is any danger of children falling into them.

When a tank is to be cleaned, the supernatant sewage is drawn off by means of a pipe extending from the bottom to the top of the basin and provided at the bottom with a swivel joint, Fig. 165, which permits it to revolve through an angle of 90 deg. To the top of this pipe is attached a float of sufficient size to prevent the pipe from falling to the bottom in which case it would draw sludge. The float is so adjusted as to hold the pipe a few inches below the surface of the sewage as it is drawn down, and a stop is provided so that when the pipe has fallen to a predetermined point it can go no farther. A gate should be provided on the pipe line with which the swivel pipe is connected so that the upright pipe can be kept partly filled, because its buoyancy will throw it up out of the water if empty. The upper end of the pipe should be enlarged and cut back at an oblique angle so that as it falls the opening will gradually approach a horizontal position and the sewage will be drawn from the surface.

Sedimentation in Vertical Tanks.—In these tanks the influent pipe extends to a considerable depth below the surface, where the sewage is distributed at a relatively low velocity throughout the horizontal cross-section of the tank. The pipe ends several feet above the elevation of the sludge deposits, to avoid any agitation of the sludge by the incoming sewage, unless it is desired to have the sewage as it enters come in contact with the sludge, to take advantage of any influence the latter may have in promoting precipitation by attraction and coagulation, which is apparently of some value with certain industrial wastes.

After leaving the inlet orifice, the sewage spreads out as it rises in the tank, and its velocity is gradually reduced to a rate at which the particles of suspended matter are just held in equilibrium, neither rising nor falling. As more sewage passes this zone the suspended matter is mechanically filtered out by the floating mass, which increases in density, and coagulation occurs by the aggregation of particles. When this mass becomes sufficiently dense, portions drop out of the stratum and settle down to the sludge at the bottom.

The progress of sedimentation in such tanks was studied experimentally by the authors at Gloversville, N. Y., where the sewage contains large quantities of tannery wastes which often cause a fairly good chemical precipitation and thus aid materially in the efficiency of the sedimentation process. Sewage entered the test tank 16 ft. below the elevation at which it flowed out and samples of the sewage were drawn from the tank through faucets at different elevations. Table 45 gives the results of these tests, which indicate that the deeper the tank the more efficient the sedimentation at a given rate of flow. It will be seen that greater efficiency can be obtained by increasing the vertical height through which the sewage must flow than by increasing the area of the tank and decreasing the velocity and the height through which the sewage must rise.

TABLE 45.—PERCENTAGE OF SUSPENDED MATTER REMOVED AT DIFFERENT DEPTHS IN VERTICAL TANK AT GLOVERSVILLE, 1909

(Averages of 4 experiments except as otherwise stated)

Upward velocity, ft. per hour	Depths of sedimentation				
	4 ft.	7 ft.	10 ft.	13 ft.	16 ft.
6	46	53	59	62	67
8	37	46	55	59	65
10	56	63	67	70	80
12	52	62	68	70	73
14	49	64	68	73	79
16[1]	37	51	63	64	65
16	....	....	55[2]	62[2]	66[2]
18	32[3]	51[3]	59	63	68
20[3]	....	27	36	54	67
22	....	33[2]	57	64	74
24[1]	....	43	49	54	62

[1] One experiment only.
[2] Average of 2 experiments.
[3] Average of 3 experiments.

Design of Vertical-flow Tanks.—The form of construction of these tanks lends itself particularly to the removal of sludge without drawing off the supernatant liquid, and they are invariably built with this method of sludge removal in view.

The most extensive use of these tanks in North America is in Toronto, Ont., where the installation was built from the plans of John D. Watson, who built and operated a number of them in the sewage works at Birmingham, England. The Toronto tanks, Fig. 166, were built to handle 240 gal. of sewage per capita from a population of 550,000. The sewage is screened before entering these tanks, which are 24 in number. Each is 25 ft. wide and

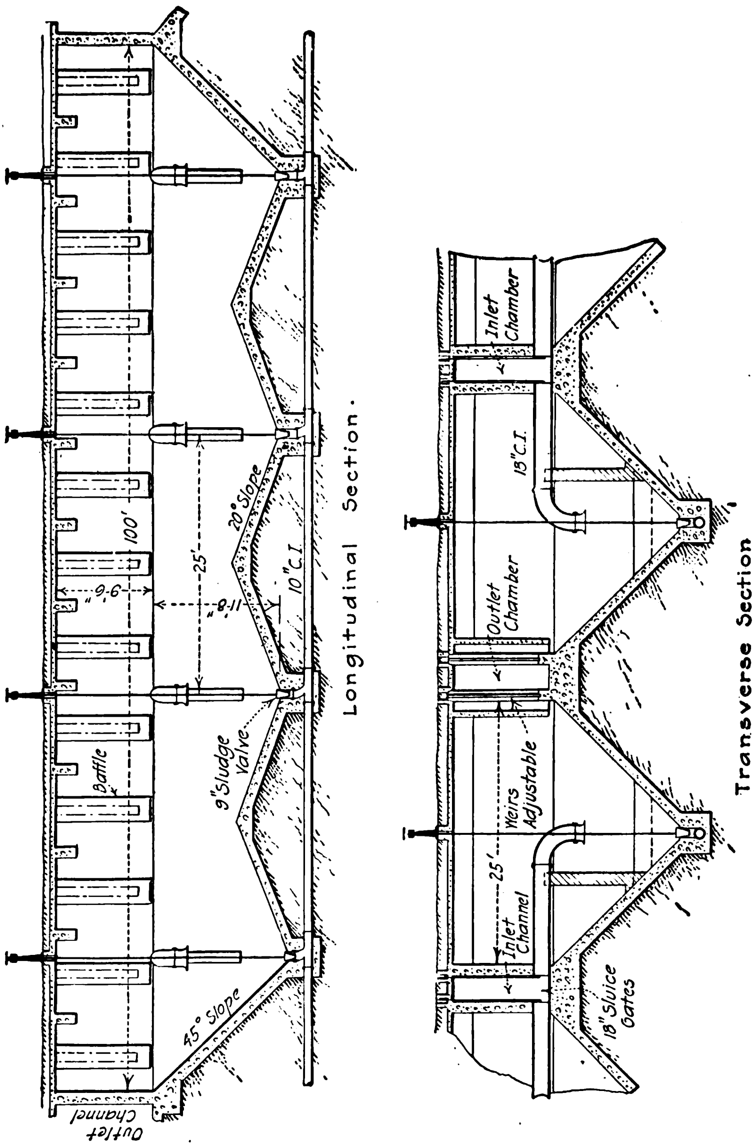


FIG. 166.—Vertical-flow settling basins, Watson type, Toronto, Ont.

100 ft. long with four hoppers in the bottom. The flow line is about 16 ft. above the lowest point of the hopper and the capacity

of a tank filled to this elevation is about 18,000 gal. The tanks remove about half of the suspended solids, which range from 170 to 400 p.p.m. in the screened sewage.

The cost of tanks of the vertical type is generally greater than that of the horizontal-flow type and less than that of the Imhoff type, described in Chapter XIII. Studies made by Wisner at Chicago in 1911 of the cost of the three types, designed for a sewage flow of 200 gal. per capita, indicated that if the cost of a horizontal-flow tank with a detention period of 6 hr. is taken as 1.00, that of a horizontal-flow tank with a detention period of 8 hr. will be about 1.33, that of a vertical-flow tank with a detention period of 4 hr. about 1.45 and that of an Imhoff tank with a detention period of 3 hr. about 2.49.

Septic Action.—The bacterial action in septic tanks is similar to, and a continuation of, that going on in sewers hitherto explained. It differs from the action in the sedimentation tank owing to the longer detention period, the degree to which bacterial action is carried, and the type of bacteria cultivated. When a septic tank is first filled, the bacteria in the sewage increase rapidly in numbers, any dissolved oxygen or nitrates which are present in the entering sewage become exhausted quickly, the sewage becomes staler and staler and anaerobic decomposition begins. The suspended matter which accumulates as sludge in the bottom of the tank is also attacked by organisms and part of the solids are disintegrated, coarse matter becoming finely divided. In these changes a large amount of gas is produced by the decomposition of the sludge and is retained in it until parts of the sludge become so light that they break away from the remainder and rise to the surface. The gases are occasionally given off in such volume that the liquid appears to boil. At temperatures of 15° to 20°C., such conditions may become established in 3 to 6 weeks. If, however, a new tank is seeded with a small quantity of sludge from an active septic tank the period required to develop a septic condition can be materially reduced.

The effluent from septic tanks generally has a greater avidity for oxygen than fresh or stale sewage and it is probable that under certain conditions over-septicization causes conditions which are inimical to the bacterial life upon which further bacterial treatment of the sewage depends. The danger can be overcome to some extent by thorough aeration of the septic tank effluent. The effluent is likely to be more offensive than fresh sewage, and its aeration is frequently accompanied by offensive odors.

Septic Sludge.—The organic matter in the sludge in septic tanks undergoes such changes that a reduction in its volume is inevitable, but this reduction is by no means so large as was claimed by early advocates of septic treatment. It amounts to 10 to 40 per cent, the average being about 30 per cent. A considerable part of the reduction in volume is due to an increase in its density, due to its disintegration and the compacting, resulting from prolonged stay in the tank. The sludge removed from a septic tank does not exceed 20 to 25 per cent of the volume removed from sedimentation tanks operated so as to prevent septic action. There is a great variation in the character of the sludge, due to variations in the quality of the sewage, temperature, construction of the tanks and methods of operation. Some sludge from septic tanks is reported to have very little odor but usually it is decidedly offensive.

Scum.—Scum forms on some septic tanks. It consists of the coarse suspended matter which floats and its quantity depends mainly on the character of this suspended matter. If the sewage is fresh and the suspended matter not much disintegrated, large quantities of scum are probable. The suspended matter brought to the surface of the sewage forms such a compact mass that the entrained gases can be liberated but slowly. Meantime the formation of more gas in the remaining sludge carries more suspended matter to the surface, increasing the thickness of the scum perhaps to 2 ft. or more in extreme cases, and it often projects 2 to 6 in. or more above the surface of the sewage. Under such conditions, especially in open tanks, the surface of the scum is likely to become dry and leathery, thus forming a fairly tight roof, sometimes cracked by the gas pressure below it. Molds and fungi often develop in this mass, binding it together, and eventually weeds may grow over its surface. Much trouble has been experienced under some conditions, particularly in the south, with the breeding of flies in this mat, necessitating screening and treatment with oil or other insecticide.

Volume of Sludge for Which Provision Should Be Made.—In computing the operating capacity of septic tanks, the space occupied by the sludge must be estimated by the same methods followed in the case of a sedimentation basin and the result modified by the reduction of volume already explained. Inasmuch as the dissolution of a part of the solids takes place only in the organic matter, it may be desirable where combined sewage must be treated by anaerobic methods to pass the sewage through grit chambers before it reaches the septic tanks, but there may be some cases where combined sewage will not readily part with the suspended mineral matter in a grit chamber unless the velocity is checked so that organic matter also settles out, forming an offensive mass troublesome to dispose of. If the grit is first removed, the solids settling from combined sewage during dry weather in septic tanks will probably not differ materially in character and amount from those obtained from separate sewage. The additional organic matter washed into combined sewers by storm run-off can be estimated only from local conditions, among which the efficiency of the street cleaning service is prominent.

Provision should be made for storing about 60 to 70 per cent of the suspended matter not removed by coarse screens and grit chambers, plus its accompanying water. The total storage space to be provided will be fixed by the assumed lapse of time between tank cleanings, which depends in turn upon the condition of the sewage reaching the works and the method of disposal of the tank effluent. In some cases during very warm weather sewage reaches such tanks in such a septic condition that they can be operated only as sedimentation basins, in order to prevent over-septicization and the production of an effluent which is not satisfactory for further treatment.

There is always the probability that septic tanks built for small American towns will not receive proper attention, and the small basins are usually given large sludge storage capacity in consequence. With the fresh, weak sewage likely to be received at such plants, there is more danger of large amounts of suspended matter in the effluent, due to high velocities of flow, than of over-septicization, and consequently it is well to provide storage for at least 9 months' accumulation of sludge consisting of 60 per cent of the total suspended matter and enough water to constitute about 90 per cent of the total volume. Liquefaction of the solids may be expected to keep tanks designed on this basis in fair working condition for a somewhat longer period. With plants for large cities, where fairly competent supervision is likely to be given, the provision for sludge storage may be reduced by designing the tanks so that a small amount of the oldest part of the sludge is drawn off at frequent intervals. This procedure is much favored in England, and the bottom slopes, sludge channels and sludge gates are designed to facilitate it. With frequent removal of the sludge, the percentage of water in it will be higher than when a long period of digestion is permitted. The designer

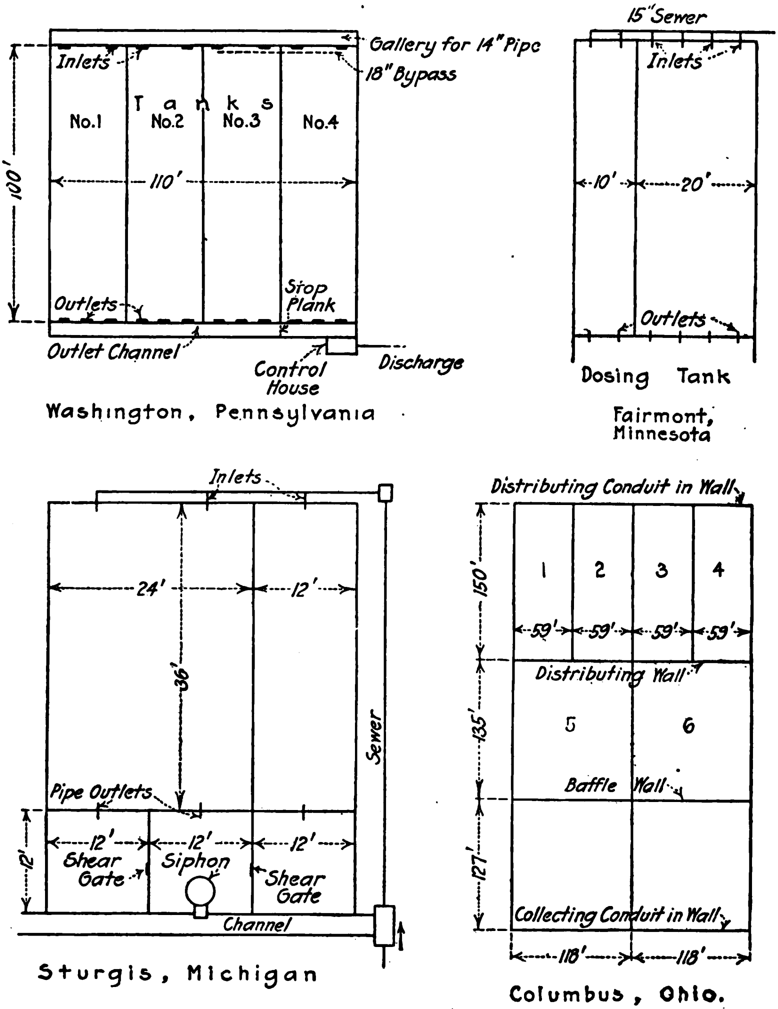


FIG. 167.—Different arrangements of septic tanks.

must keep in mind the fact that sludge with 95 per cent of water occupies twice the space required to store sludge consisting of the same amount of solids but with only 90 per cent of its volume composed of water.

Number of Septic Tanks.—The size of septic tanks depends upon the desired detention period, character of sewage, space desired for sludge accumulation in the tanks, and local topography. The best size having been determined, the number of tanks depends upon the quantity of sewage to be treated. Subdivision of tank capacity not only gives to an intelligent operator the means of adjusting his plant to the fluctuations in flow, but also permits him to dilute trade wastes, avoid over-septicization, clean out his tanks in rotation so as to avoid great variations in the quality of the effluent, and carry on experiments with different operating conditions.

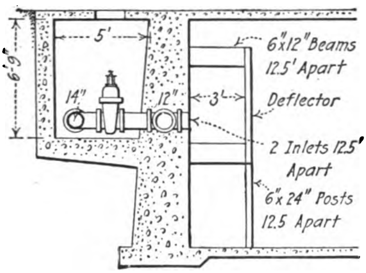


Fig. 168.—Inlet of septic tank, Washington, Pa.

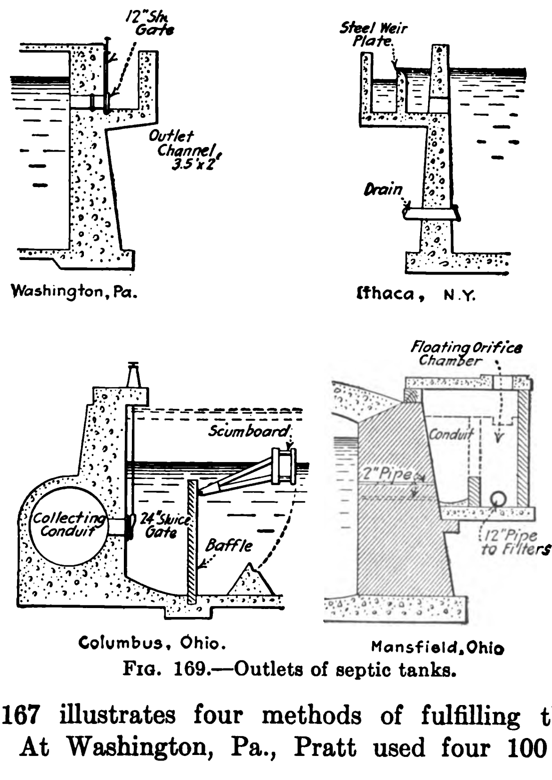


Fig. 169.—Outlets of septic tanks.

Fig. 167 illustrates four methods of fulfilling these conditions. At Washington, Pa., Pratt used four 100 by 25-ft.

tanks with an average depth of sewage of 8 to 10.5 ft. The inlet detail is shown in Fig. 168. The outlet detail is shown in Fig. 169. The height of the sewage in the outlet channel is controlled by automatic apparatus. If it is desired to employ series operation, stop planks are placed in the outlet channel between tanks 3 and 4. Then tank 1 or tank 2, or both, will discharge sewage through the outlet channel into tank 3. Here the direction of flow is reversed from that for parallel operation. On reaching the further end, the sewage enters three outlets in an 18-in. pipe bypass, which conveys it to the same number of outlets in tank 4, where the normal direction of flow is resumed. The two tanks at Fairmont, Minn., designed by Marston, have a combined capacity of 80,000 gal. and their effluent is delivered to two intermittent sand filters. The five tanks at Sturgis, Mich., designed by Pierson, are 8.5 ft. deep and can be employed, by various settings of the valves and gates, to give capacities of 31,600, 37,700, 46,600, 52,700, 58,800, 72,100 or 78,200 gal.

The large installation of tanks[1] at Columbus, Ohio, designed by Gregory, consisted of four primary tanks, numbered 1 to 4, and two secondary tanks, numbered 4 and 5. The sewage entered each primary tank through four 24-in. sluice gates. In the distributing wall at the lower end of these tanks was an overflow channel at the top, a 5-ft. collecting and distributing conduit in the center and a 5 by 3.3-ft. blow off conduit at the bottom. The sewage passed from each tank into the collecting and distributing conduit through four 24-in. sluice gates, and passed from it through eight 24-in. sluice gates into either or both of the secondary basins. The sewage was drawn from each of the latter basins through eight 24-in. sluice gates into a collecting channel in the end wall, Fig. 169.

One secondary and one or two of the primary basins were used continuously, except in winter, until septic action became violent enough to interfere seriously with the subsidence of suspended solids. Then these tanks were put out of service and the other secondary and primary basins were put in service. The tanks shut off remained full until the sludge had become fairly well digested, which was indicated by the comparatively little boiling and a change in the color of the supernatant liquid from jet black to a clearer greenish color. When the tanks were shifted the clear liquid in those which had been standing full

[1] Later converted into Imhoff tanks.

was drained into the river, the sludge remaining in the tanks. The liquid was non-putrescible, contained less than 10,000 bacteria per cubic centimeter and was very low in suspended matter.

Roofs.—It was claimed by some of the early advocates of septic tanks that they should be roofed in order to ensure proper anaerobic changes in the sewage and sludge, but this opinion is no longer general. The main reason for roofing them at present, particularly small tanks, is to reduce the odor often arising from them. If the wind is prevented by a roof from agitating the surface of the sewage, the odor is rarely so strong as to cause annoyance beyond the immediate vicinity of the basin. Occasionally a covering is desirable to keep the sewage warm or to keep off children.

The roofs are sometimes wood but more often concrete. When the latter is used the surfaces exposed to damp air should be finished as dense and smooth as possible in order to prevent deterioration of the concrete by the action of the hydrogen sulphide given off in the gases from the decomposing sewage and sludge. Vent pipes must be provided so that the sudden filling or emptying of a tank will not cause any large pressure above or below the roof. As the gases are sometimes explosive, a notice to this effect should be posted near the entrance to any roofed tank.

Inlets and Outlets.—Action in septic tanks, in liquefying organic matter, is apparently less active at mid-depth than at the level of the sludge or just below the scum, and it is therefore desirable to draw off and admit the sewage near mid-depth. This also avoids disturbing the scum by currents. Where weirs are used they are usually guarded by scum-boards from 2 to 4 in. in front of them, extending to a depth of at least 2 ft. The narrow openings between the boards and weirs are easily kept free from scum, and the sewage is compelled by the depth of the boards to take the same course it would follow with submerged openings. Fig. 168 is an inlet used at Washington, Pa. In some cases of small basins, the sewage enters and leaves the basins through pipes running horizontally across the basins a few inches from the walls. These pipes are provided with tees at intervals and from each tee a pipe is carried down to the desired depth, where it ends in a second tee from which short branches run horizontally and end in quarter-bends, opening toward the wall.

Four types of outlets used with septic tanks are shown in Fig. 169. At Washington, Pa., there are three of the outlets illustrated used in each tank 25 ft. wide. In the tanks at Mansfield, Ohio, Barbour used two horizontal rows of 2-in pipe running through the wall, 6 in. apart vertically and 12 in. apart horizontally. The upper row is 2 ft. below the surface of the sewage. These pipes discharge into a concrete conduit running along the outside of the end wall, 4 ft. deep, 2 ft. wide at the top and 18 in. wide at the bottom. Opposite the center of each tank, this conduit passes through a chamber where the sewage is drawn off through a floating orifice. Each orifice is 2 by 12 in. and supported by an iron frame held in place by two 21 by 36 by 8-in. floats. The orifices can be adjusted for submersion to any depth down to 1.5 ft. The other two outlets shown are self-explanatory.

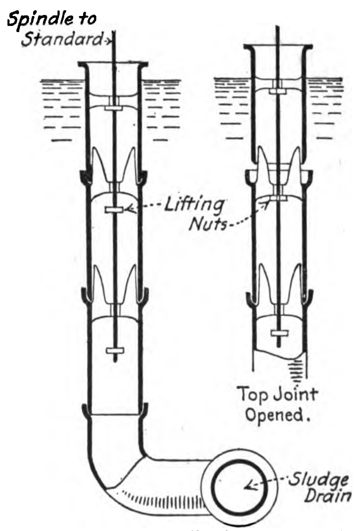


FIG. 170.—Draw-off pipe for septic tanks.

In drawing off the contents of a septic tank by means of a swiveling arm of the type shown in Fig. 165, the lip must be fitted with a scum board, and if the scum is tough or thick this form of draw-off pipe may not be so serviceable as that shown in Fig. 170. This has a vertical standpipe consisting of sections about 1 ft. long. Inside each section is a cross-arm with a hole in the center, through which passes a spindle raised and lowered by a gate standard of the rising stem type. Lifting nuts are fixed on the spindle at such points that the top joint must be opened before the one next it, and the second joint must be opened before the third. By using this detail there is a minimum disturbance of the scum and sludge. In case it is desired to operate with very thick scum, the first and second lifting nuts can be adjusted so that the top joint will not open.

Bottom Slopes.—The slopes of the bottoms of septic tanks do not ordinarily differ from those employed with sedimentation basins. Septic sludge free from grit can be moved with scrapers on slopes of 1.5 to 3 per cent without difficulty, but thick scum, which settles on top of the sludge when tanks are drained, must be broken up before it can be handled easily on such slopes.

The general arrangement of the bottom will usually be influenced considerably by the method of disposing of the sludge. Wherever practicable it should be discharged by gravity into a well from which it can be pumped or into a sludge drain. There is little information about the flow of sludge through pipes, but it is significant that at the septic tanks at Mt. Vernon, N. Y., no difficulty was experienced for several years in draining off the sludge through a 16-in. pipe on a 1 per cent grade. Here the tanks are of the half-hopper type, with the sludge outlet in one of the side walls, so that the floor slopes toward it at the rate of 1:12 to 1:24 from both end walls and the other side wall. In Great Britain a slope of about 1:15 is considered desirable for the discharge of septic sludge by gravity.

The sludge outlets are usually closed by shear gates unless they are so large that sluice gates are necessary for convenient operation; the latter are generally more satisfactory as they are less likely to leak. All conduits for sludge should be run in straight lines between manholes, if possible, in order that any stoppage can be removed without serious difficulty.

Small septic tanks which collect only a small quantity of sludge can best be drained into sludge beds, trenches in earth, or composting beds. If on account of local conditions, they can be drained by gravity only by the construction of expensive accessory works, they can often best be cleaned by hand. The supernatant liquid which cannot be drained off by gravity and sludge of ordinary density can be pumped by diaphragm pumps into carts, as nightsoil is handled. This causes odor in the vicinity, but is preferable in the case of some small installations to expensive excavation in rock or to far distant sludge beds. The floors of such small tanks may be left flat if the cost of excavation to give easy grades to one point would be heavy.

Chemical Precipitation.—Chemical precipitation is a method of increasing the sedimentation of suspended and colloidal matter by the addition of chemicals which form floc in the liquid, the floc drawing to itself the substances it is desired to remove. Calcium oxide or lime, CaO, sulphate of alumina, $Al_2(SO_4)_3$, ferrous sulphate, $FeSO_4$, and ferric sulphate, $Fe_2(SO_4)_3$, are used to produce chemical reactions; clay and soil have been added to weight the precipitate, and charcoal has been added as a deodorizer.

The theories upon which the chemical reagents act differ somewhat, but are fairly represented by the action of copperas and lime. The lime is added to the sewage to give it the alkalinity necessary for the reaction and then the copperas or ferrous sulphate is added. The chemical reaction is:

$$FeSO_4 + Ca(OH)_2 = Fe(OH)_2 + CaSO_4$$

The soluble ferrous sulphate is converted into the very bulky flocculent ferrous hydrate, which strains out the suspended matter as it precipitates.

The degree of clarification depends largely upon the quantity of chemicals used. If enough is employed in an intelligent way, the effluent will generally be clear, substantially free from suspended and colloidal matter and without pronounced color. In many cases the quantity of chemicals required to produce this result is so great as to entail serious expense, and consequently the process is not carried far enough to give a clear effluent. The handling and disposal of the sludge from chemical precipitation tanks is also a costly matter, for the volume of the sludge often reaches 0.5 per cent of the volume of the sewage treated. Furthermore, although the effluent may have a moderately satisfactory appearance it is ordinarily putrescible and not comparable with good filtration effluents. These drawbacks to the use of the method have prevented its more general adoption and even led to its abandonment in some cases. The greatest field for usefulness of chemical precipitation at present seems to be in the treatment of industrial wastes.

Quantity of Lime Required.—Normal domestic sewage is slightly alkaline. Where lime alone is used as the precipitant, it is necessary to add a sufficient quantity to combine with all the free carbonic acid and the carbonic acid of the bicarbonates, producing normal calcium carbonate, which acts as the coagulant. Much more lime is generally required when it is used alone than when sulphate of iron is also employed. Where industrial wastes introduce mineral acids or acid salts into the sewage, these must be neutralized before precipitation can take place and may increase the quantity of lime required.

The required degree of alkalinity where lime is the only precipitant may be determined by titrating one portion of the sewage with erythrosine and another portion with phenolphthalein. The erythrosine alkalinity must be more than twice that shown by the phenolphthalein test. In case copperas and lime are the precipitants, it is necessary to add enough lime to provide a normal carbonate alkalinity. If enough lime is used, a pink color will appear in a glass of the treated sewage to which a few drops of an alcoholic solution of phenolphthalein is added. Only enough lime should be used to produce the pink color. If sulphate of alumina or ferric sulphate is used, the sewage may not need lime, because bicarbonate alkalinity is capable of precipitating these salts. Bicarbonate alkalinity is indicated in this case by the pink color produced when erythrosine is added to a sample of sewage in the presence of chloroform; no color is produced by erythrosine with neutral or acid solutions.

If too much lime is used in the treatment of sewage, some of the suspended organic matter will be dissolved and the effluent may be worse than the raw sewage. If not enough lime is used, the effluent will not be well clarified, and if an acid sewage is treated the effluent may remain acid and be poorly prepared for treatment in contact beds or trickling filters.

In Worcester, Mass., the quantity of lime used varies greatly, the dose being determined by the phenolphthalein test. In 1893, 1,233 lb. per 1,000,000 gal. of sewage were used, the amount gradually being decreased to 905 lb. in 1912. In 1908, when the general business depression caused a great reduction of output in the local foundries and wire works, but 871 lb. of lime per 1,000,000 gal. of sewage were used.

The pre-war practice in Providence, R. I., is indicated in Table 46.

In London the sewage was treated with 475 lb. of lime and 119 lb. of copperas per 1,000,000 gal. When a basin is closed off for cleaning, its contents are allowed to stand for 2 hr. The top 4 ft. of supernatant liquid is then drawn off through telescopic weirs and the remaining contents of the basin are then drawn into a sump and allowed to stand for 24 hr., when the top liquid is decanted, treated with 2,370 lb. of lime and 1,190 lb. of copperas per 1,000,000 gal., and returned to the basins, the sludge stored being carried to sea by sludge steamers.

Slaking Lime.—Lime does not dissolve readily in water and on this account it is best applied to sewage in the form of milk of lime. This is prepared by slaking quicklime in a small amount of

TABLE 46.—COST OF CHEMICAL PRECIPITATION AT PROVIDENCE, R. I.[1]

Year	Pounds per 1,000,000 gal.		Water in sludge, per cent.	Cost per 1,000,000 gal.		
	Lime	Copperas		Chemical precipitation	Sludge disposal	Total
1903	606	65.0	94.63	$3.31	$2.44	$5.75
1904	683	58.0	92.46	3.42	2.57	5.99
1905	559	51.5	92.11	3.13	2.66	5.79
1906	638	72.1	92.58	3.50	3.10	6.60
1907	654	83.1	92.15	3.54	3.07	6.61
1908	726	....	91.77	3.42	3.43	6.85
1909	700	....	91.29	3.36	4.22	7.58
1910[2]	486	....	92.07	3.11	4.06	7.17
1911	438	....	92.57	....	....	....
1912	......	....	90.03	2.85[3]	2.49	5.34
1913	......	....	89.60	2.50[3]	2.30	4.80

[1] Since this chapter was prepared, lime treatment has been resumed.

[2] In 1910, the city began to experiment with the disinfection of its sewage, and the following year these experiments entailed so many modifications of operation that all the records could not be abridged into the usual summarized form. In 1912 chemical precipitation was abandoned and plain sedimentation followed by disinfection of the effluent was adopted. During the period when chemical precipitation was practiced, the removal of suspended organic matter, as determined by albuminoid ammonia, was from 80.5 to 86.4 per cent, while during 1912 and 1913, it was 41.0 and 40.2 per cent respectively. Lime treatment was resumed for a time about the year 1915 and subsequently discontinued. It is now (1921) being used again.

[3] These figures include the cost of disinfecting the sewage.

water and allowing it to stand over night, and longer if possible. This is then usually mixed with about 20 times its weight of water, although a thinner mixture is sometimes used.

Lime of fair quality will take up about one-third of its weight of water in slaking, but poor grades will not require so much The amount necessary for the lime to be used should be determined by experiment. A quick-slaking lime will require more water than hard, close-grain grades. In general the lime will increase from 50 to 100 per cent in bulk in slaking.

The temperature during slaking is an important index of the character of the operation. It should be as near 200°F., or just below the boiling point, as possible. If the temperature stays below 200°, too much water has been added and the slaking is less vigorous and thorough. If not enough water is added, the temperature will become too high and the lime will be in danger of becoming "burnt."

Pipes carrying milk of lime may become choked with precipitated carbonate of calcium, and it is desirable to have facilities for flushing them with very weak acid and water. It is sometimes helpful to blow steam or air through the feed pipes.

Mixing Chemicals and Sewage.—The precipitants must be mixed with the sewage before the latter enters the tanks, in order to obtain the best results from this method of treatment. It is inadvisable to add precipitants before pumping sewage, because the floc formed by the precipitants may become more or less broken up and charged with air, which interferes with their

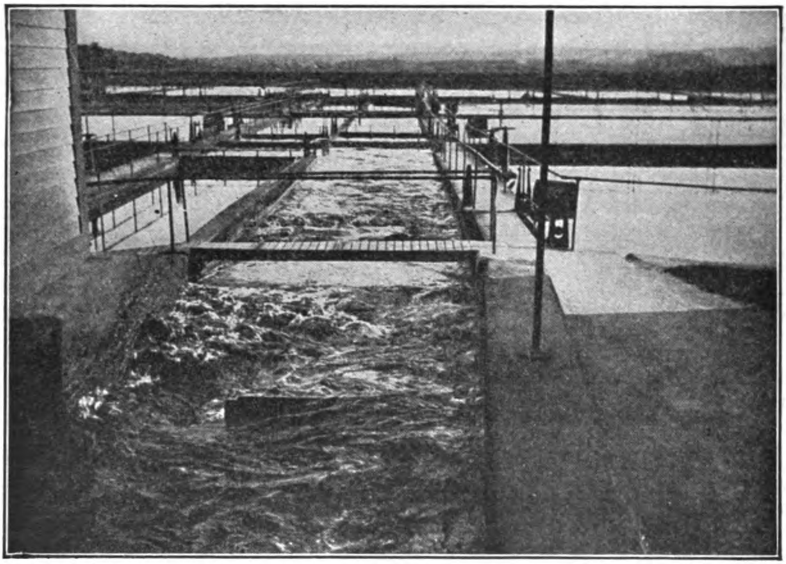


FIG. 171.—Baffled mixing channel or "fish ladder" leading to precipitation basins, Worcester, Mass.

sedimentation. One of the most satisfactory means of mixing the dose of chemicals with the sewage is the fish ladder, Fig. 171.

Detention Period.—The best detention period depends upon the amount and character of the suspended matter in the sewage, the amount and nature of the trade wastes which may be discharged into the sewage, the method of operating the tanks, and the treatment of the effluent after leaving the tanks. British experience favors a detention period of 8 hr. for tanks operated continuously and 2 hr. for tanks operated on the fill-and-draw method, in the case of domestic sewage. At Worcester, Mass., where chemical precipitation has been employed longer than elsewhere in the United States, the detention period is about 7 hr.

Design of Precipitation Tanks.—After many years' experience in supervising the operation of chemical precipitation works at Worcester, Mass., and Providence, R. I., Bugbee has summarized his opinions regarding the design of tanks for chemical treatment as follows:

The depth of the tank should be between 6 and 9 ft. Since the tanks should be cleaned very frequently, it is unnecessary to provide space for the storage of sludge at the bottom of the tanks, as was done in some of the older precipitation plants.

The length of British tanks was formerly made about two and one-half times the width, but it has been found that better results were obtained with a greater proportional length.

The bottom of the tank should rise toward the outlet and also slope from each side wall toward a drain running longitudinally through the tank. If the bottom could be given a grade of at least 10 per cent, the sludge would all flow to the drain. This is obviously impossible with large tanks, in which it is necessary to force the sludge adhering to the bottom into the drain by means of wooden scrapers. Hence narrow tanks, involving the least travel for the men employed in this work, are most economical in operation.

The inlets for the sewage should be as nearly the full width of the tank as possible, in order to reduce the entrance velocity to the minimum and to avoid stagnant corners where septic action is likely to occur in a short time. For the same reason no dead ends should exist on the inlet and effluent channels, and the bottoms of these channels should be constructed to retard the sedimentation of suspended matters, just as is done in sewers carrying variable quantities of sewage.

The sluice gates through which the sludge is drawn off should be at the inlet ends of the tanks where most of the sludge will be deposited.

There should be a floating arm and valve for drawing off the supernatant liquid before removing sludge from a tank. The valve should be at the extreme end of the tank in order to remove the greatest possible quantity of water from the sludge.

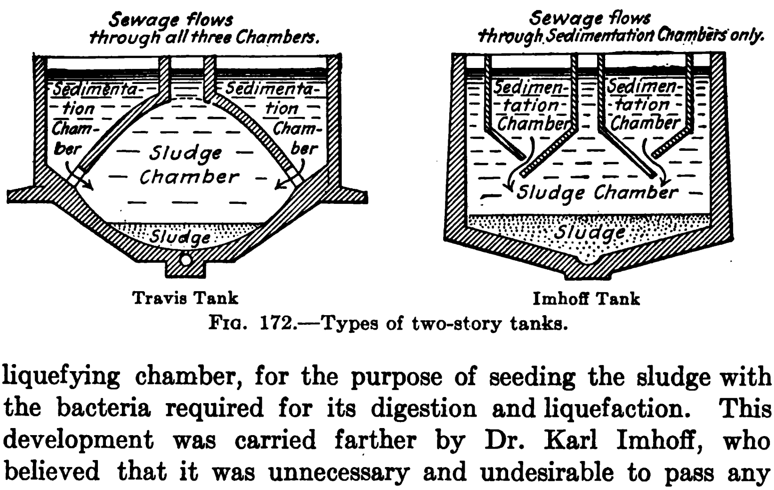
Provision should be made for supplementing the work of the scrapers used in sludge removal by means of gates in the side walls, through which supernatant liquid may be introduced from adjoining full tanks for flushing.

The size of the individual tanks should be so proportioned to the flow as to give at least four units, in order that the tanks may be cleaned as often as once a week in hot weather, without reducing the desirable working capacity of the plant. Frequency of cleaning is an important operating duty, because of its value in preventing septic conditions in the effluent and also because of its effect on the work of sludge pressing.

CHAPTER XIII

THE IMHOFF OR EMSCHER TANK

If the settling solids can be removed from sewage and their digestion take place under anaerobic conditions which will not affect the sewage, the exhaustion of the dissolved oxygen and nitrates, if any, in the sewage will be retarded. By keeping the sewage as fresh as possible in this way, odors are reduced to a minimum and the subsequent treatment of the sewage or its disposal by dilution is generally aided. Dr. William Owen Travis developed a type of two-story sedimentation tank, Fig. 172, in which this separation of the processes of sedimentation and sludge digestion is largely accomplished, only one-sixth to one-eighth of the sewage being sent through the lower or sludge-



Travis Tank Imhoff Tank

FIG. 172.—Types of two-story tanks.

liquefying chamber, for the purpose of seeding the sludge with the bacteria required for its digestion and liquefaction. This development was carried farther by Dr. Karl Imhoff, who believed that it was unnecessary and undesirable to pass any portion of the sewage through the liquefying chamber, as the solids were already sufficiently seeded. He therefore developed the tank known as the Imhoff or Emscher tank, Fig. 172, the latter name coming from the district in Germany in which the tanks were first installed on a comparatively large scale.

Sewage flows through the sedimentation chambers only, and the function of these chambers is solely the removal of the settling solids. These solids drop to the inclined surfaces of the bottom of a sedimentation chamber and slip through a slot into a sludge chamber. The slot is trapped so that no gases from the sludge chamber can rise into the sedimentation chamber. In the sludge chamber the solids undergo septic decomposition as in a septic tank. The gases given off by this decomposition rise through the portions of the sludge chamber extended to the top of the tank, called gas vents, and scum collects on the surface of the sewage in these vents just as it does on the surface of septic tanks, and often to a greater depth. On account of the depth of the sludge chamber and the long period of time the sludge is allowed to digest, the sludge from tanks of this type which are operating normally is more dense than that from plain sedimentation tanks or septic tanks. Theoretically, therefore, the tank carries on simultaneously and independently by means of its two-story construction, the functions of both plain sedimentation tanks and septic tanks.

One of the most important advantages of this form of tank is its production, when operating properly, of sludge which is easily disposed of. As the sludge is withdrawn from the tank, it contains large quantities of gas held mechanically in it. When relieved from the pressure of the considerable depth of supernatant liquid in the tank (when the latter are of adequate depth) and spread out upon sludge-drying beds, the solid matter will be buoyed up by the entrained gas, thus permitting the relatively clear liquid below to pass quickly into and through the beds. As the water disappears the sludge gradually settles down on the surface of the bed, and the gas is liberated and replaced by air, leaving the mass quite porous and in a condition favoring rapid drying. Good sludge from such tanks is dark colored, not sticky or pasty but rather granular, free from offensive odor, somewhat frothy and resembling dark garden soil mixed with water.

A peculiar feature of the operation of some Imhoff tanks is the production of so much gas in the sludge chamber that the sewage in the gas vents seems to foam or boil. Occasionally the gases have brought up with them so much sludge that the scum has overflowed the tops of the gas vents. The early tanks of this type in the United States were designed on the basis of experience gained in operating tanks with the strong sewage of German industrial towns, and it was soon learned that the areas of the gas vents which were adequate for German conditions were not large enough for American conditions, particularly where the climate is warm.

Apparently the sludge chamber requires alkaline conditions for satisfactory operation. When the Imhoff tank does not operate satisfactorily the sludge is relatively light colored, has an offensive odor, is somewhat sticky or pasty, and does not dry well. In some cases the addition of lime to the liquid in the sludge chamber has been tried as a cure for the troubles but its efficacy is doubtful. In other cases the troubles may have been due to filling the tanks with fresh sewage late in the fall, when the weather is not adapted to develop bacterial activity, resulting in a long period of very moderate bacterial activity in the sludge chamber. It is considered desirable by some to fill the tanks with hard water before any sewage is turned into them, so that operation will begin with assured alkaline conditions, and if possible to seed the lower compartment with bacteria, through sludge from an operating tank.

Diffusion Between Chambers.—The sludge chamber of Imhoff tanks is intended to retain its contents in a stagnant condition, without any circulation of liquid through the slots opening into it from the sedimentation chambers above. It is therefore desirable to design and operate the tanks so that no material fluctuations of sewage level will occur in the tanks, for such fluctuations, particularly if sudden, will cause differences of hydrostatic pressure and, consequently, surges of septic sewage up through the slots. Furthermore, if currents down through the slot exist to such a degree that considerable fresh sewage enters the sludge chamber, there is danger of the evolution of hydrogen sulphide. It was a desire to prevent any septic sewage rising from the sludge chamber into the sedimentation chambers, where nothing but fresh sewage should be present in order to assure the most satisfactory operating conditions, that led the engineers under Dr. Imhoff to pay unusually careful attention to grades and other features controlling the elevation of the sewage during its flow through the tanks.

The danger of diffusion increases with the number of sedimentation chambers over a single sludge chamber. Care must be taken to have the inlets and outlets designed to give the proper proportion of flow to each sedimentation chamber. If the operating conditions are such as to require careful attention from the superintendent, it may be advisable to connect the sedimentation chambers at each end of the tank so as to maintain the same surface elevation in all of them.

Diffusion, temperature changes, and the gradual increase in the volume of sludge in the sludge chamber must result in some septic liquid passing from it into the sedimentation chambers. Investigations by Bach show that, under satisfactory operating conditions, this diffusion is very slight and without substantial effect on the sewage in the sedimentation chambers. The volume of the sludge is decreased from 25 to 50 per cent in passing through the slots, according to these experiments; this is true not only of sewage sludge but, to a somewhat smaller degree, of sediment formed of fresh, washed iron hydroxide and very finely powdered hard coal. Furthermore, as the sludge collects above the slot before passing down through it, its weight and volume hinder the formation of any currents contrary to the motion of the subsiding sludge. In addition to these influences retarding diffusion, the specific gravity of the liquid in the sludge chamber is greater than that of the sewage passing through the sedimentation chambers.

Quantity of Sludge.—The size of the sludge chamber is based primarily on the amount of solid matter which is expected to settle out of the sewage during the assumed maximum period between removals of sludge. On account of the relatively high cost of Imhoff tanks compared with plain sedimentation tanks, it is desirable to form as close estimates as possible of the volume of sludge to be expected. In the Emscher District this was not a serious matter, because it was customary there to estimate sewage treatment requirements only 5 years in advance of construction. The works were built and operated under skilled supervision and additions made to the plants as needed. In this way the capital investment was kept to a minimum and the utmost service obtained from the installation. In the United States, where it is customary to construct treatment works capable of serving without additions for a much longer future period than 5 years, this greater investment of capital in extra space for future requirements makes it necessary to be as accurate as possible in forecasting conditions.

Imhoff assumed in his designs that the average amount of sludge where combined sewers were used would be 0.007 cu. ft. per capita daily and 0.0035 cu. ft. where separate sewers were used. For reasons just explained, great accuracy in these estimates was not attempted, for the original plans were prepared so that additional tank capacity could be added whenever needed without reconstruction of the older works. Imhoff preferred to estimate the quantity of sludge on the basis of persons connected with the sewerage system rather than on the basis of quantity of sewage on account of the variation in dilution of sewage in different cities. In designing his tanks he felt the most important thing[1] to be considered was the detention period which he varied to meet local conditions. Where the sewage was fresh and strictly domestic, a detention period of 2 hr. was considered most satisfactory, about 1½ hr. was the average for all his

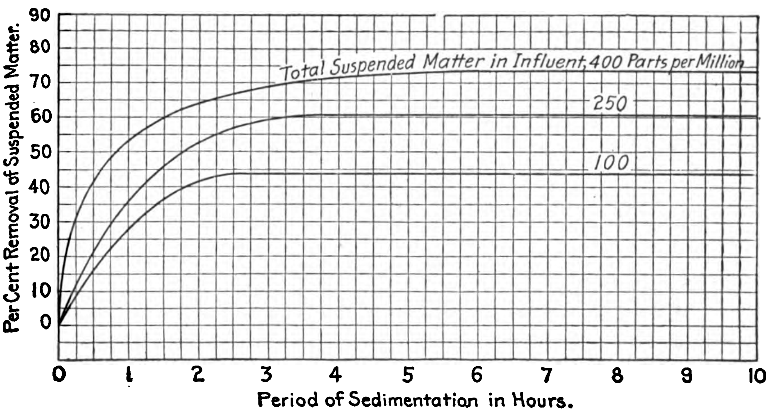


FIG. 173.—Removal of suspended matter in large settling tanks.

plants, and where a large amount of iron salts was present in the sewage 1 hr. was considered the longest period that should be adopted. Imhoff opposed a longer period than 2 hr. for German conditions on account of the danger of septic conditions being set up in the sedimentation chambers. The rate of flow through the sedimentation chambers adopted by Imhoff was 108 ft. per hour for dry-weather conditions and 324 ft. when the sewage was diluted threefold with storm water.

American conditions are so different from those in the Emscher district that it was felt by designing engineers to be unsafe to employ detention periods as low as those used by Imhoff, in

[1] The relative importance of the factors influencing sedimentation was rated by Imhoff as follows: Detention period, 100; change in temperature, 30; depth, 30; construction of inlets and outlets, 30; character of sewage, 30; velocity through tank, if less than 350 to 600 ft. per hour, 0.

laying out the early American tanks of this type, or to use rates of flow as high as he employed. These early American tanks had detention periods about 50 per cent greater than the Emscher district practice and the rates of flow in the sedimentation

TABLE 47.—VOLUME OF SLUDGE OF DIFFERENT SPECIFIC GRAVITIES AND PERCENTAGES OF MOISTURE OBTAINED FROM 100 LB. OF SETTLED DRY SOLIDS

Percentage of moisture	Cubic yards of sludge, with 90 per cent moisture, having specific gravity, given in column head					
	1.02	1.03	1.04	1.05	1.06	1.07
99	5.922	5.917	5.912	5.906	5.901	5.896
98	2.955	2.950	2.945	2.939	2.934	2.929
97	1.966	1.961	1.956	1.950	1.945	1.940
96	1.467	1.462	1.457	1.451	1.446	1.441
95	1.175	1.170	1.165	1.159	1.154	1.149
94	0.977	0.972	0.967	0.961	0.956	0.951
93	0.827	0.822	0.817	0.811	0.806	0.801
92	0.729	0.724	0.719	0.713	0.708	0.703
91	0.647	0.642	0.637	0.631	0.626	0.621
90	0.581	0.576	0.571	0.565	0.560	0.555
89	0.527	0.522	0.517	0.511	0.506	0.501
88	0.482	0.477	0.472	0.466	0.461	0.456
87	0.444	0.439	0.434	0.428	0.423	0.418
86	0.403	0.398	0.393	0.387	0.382	0.377
85	0.383	0.378	0.373	0.367	0.362	0.357
84	0.359	0.354	0.349	0.343	0.338	0.333
83	0.337	0.332	0.327	0.321	0.316	0.311
82	0.318	0.313	0.308	0.302	0.297	0.292
81	0.300	0.295	0.290	0.284	0.279	0.274
80	0.284	0.279	0.274	0.268	0.263	0.258

NOTE.—The sludge in the column headed 1.02 is of the character to be expected from small strictly domestic separate sewerage systems, where, through infiltration or high water consumption, the sewage is very weak. The sludge in the column headed 1.07 is from very strong sewage delivered by a combined system, with but little of the mineral matter removed by grit chambers. A specific gravity of 1.03 may be assumed for average sewage from strictly separate systems and 1.05 for average sewage from combined systems.

chamber ranged from about 7 to about 87 ft. per hour. Recent American practice has shown a tendency to shorten the detention period. Fig. 173 has been found useful by the authors in estimating the removal of suspended matter during different detention periods.

The quantity of sludge to be expected will depend very largely on its density, as is evident from Table 47. The Imhoff tanks in the Emscher district yielded sludge of unusual density, but how much this was influenced by the great depth of the tanks, which averaged 30 ft., and how much by prolonged storage is uncertain. What Imhoff desired was sludge which would dry readily without causing offensive conditions, and this he believed could be obtained most economically by deep tanks. The dense sludge from these tanks held more entrained gases, in his opinion, than sludge from tanks of less depth, and he believed that the greater the amount of entrained gases in the sludge the more rapidly it would dry. Experiments by Clark and Gage with the sludge collected in sedimentation tanks 17 and 30 ft. deep partly confirmed this opinion. The total reduction of suspended solids was 19 per cent in the shallow tank and 30 per cent in the deep tank. The sludge contained 91.3 per cent of moisture in each case, but that from the shallow tank contained 76.6 per cent of moisture after draining for 10 days while that from the deep tank contained only 65.9 per cent.

The sludge which is collected from sewage differs in the weight per unit volume of the solid matter as well as in the percentage of the suspended matter which has settled in a given period of time. There is a tendency among engineers to compare sludges by expressing their characteristics as those of similar sludges with 90 per cent moisture, which is readily done by means of Fig. 174, from Webster's report on the Philadelphia experiments. The specific gravities of most sludges of which analytical records have been examined by the authors lie within 1.02 and 1.07, with 90 per cent moisture. The exceptions to such a relation were too erratic to make their consideration necessary. Sludge of 1.02 specific gravity at 90 per cent moisture contains very light suspended matter, such as might be expected from weak domestic sewage. A specific gravity of 1.07 with 90 per cent moisture is indicative of very heavy solids, such as those entering combined sewers from macadam streets in times of storm. Probably 1.03 will prove a fairly accurate figure to assume for average sewage from strictly separate systems, and 1.05 for average sewage from combined systems.

An examination of American records of septic, sedimentation and Imhoff tank sludges indicates that the probable percentage of moisture is highest in sludges with solids of light weight, ranging from 91 to 98 per cent; with average weak sewage the sludge, of a specific gravity of 1.03 at 90 per cent moisture, will have

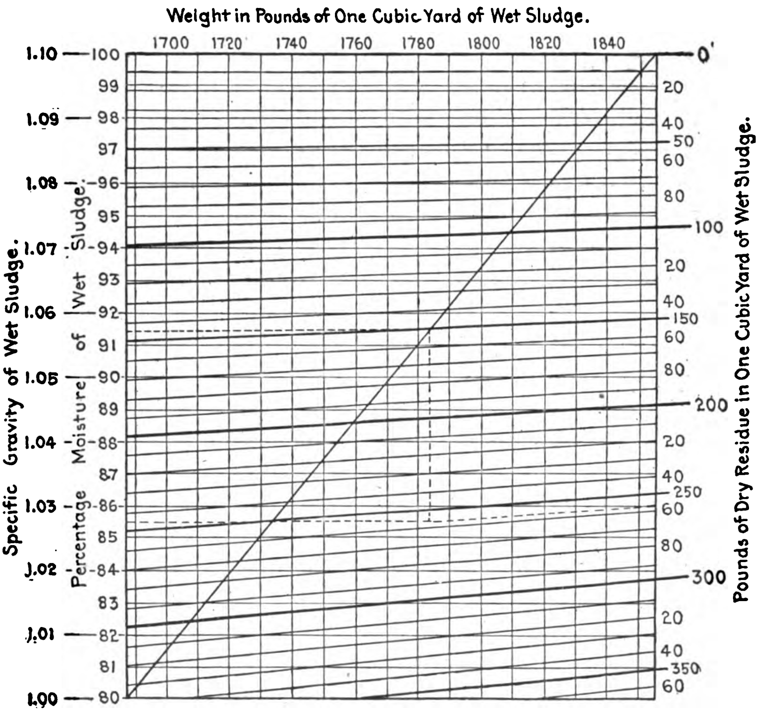


FIG. 174.—Relation between specific gravity, percentage of moisture and weight of dry solids in 1 cu. yd. of wet sludge.

Example of Use.—Assume that the sludge is found to contain 85.5 per cent of water and has a specific gravity of 1.057. From the 1.057 point on the scale of specific gravity, at the left, follow a horizontal line to the diagonal. Directly above the intersection, on the top line, will be found the weight of 1 cu. yd. of such sludge, 1,783 lb. Drop a vertical line from the same intersection point and extend to meet it a horizontal from 85.5 on the scale of percentage of moisture. From their intersection, follow the sloping lines to the right to 258 on the right-hand scale, giving the pounds of dry residue in 1 cu. yd. of sludge.

from 90 to 96 per cent of moisture; with average strong sewage, the sludge, of a specific gravity of 1.05 at 90 per cent moisture, will have from 87 to 93 per cent moisture. The following figures of the percentage of moisture in sludges from American Imhoff tanks indicate that the dense sludges of the Emscher district are not likely to be obtained here: Stock Yards Testing Station, Chicago, tank 17 ft. deep, sludge with 91.5 per cent of moisture and specific gravity of 1.02. Experimental Imhoff tank at Worcester, 22 ft. deep, sludge with average specific gravity of 1.016 and 91.86 per cent of moisture. Akron experimental Imhoff tank, 16 ft. deep, sludge with specific gravity of 1.12 and 74 per cent moisture. Proctor Creek tanks, Atlanta, sludge with specific gravity of 1.02 and 87.05 per cent of moisture; Peachtree Creek tanks, Atlanta, sludge with specific gravity of 1.02 and 90.2 per cent of moisture.

Sludge Storage Capacity Required.—This is estimated in the following manner: From analyses of the sewage or by a study of the sources of the sewage when analytical records are lacking, the amount of suspended matter is first determined. For example, take this as 175 p.p.m., which is probable for domestic sewage collected by small separate sewerage systems. This quantity is equivalent to 1,460 lb. of suspended matter per 1,000,000 gal. of sewage. It will be assumed that a detention period of 3 hr. is selected and that in this time 60 per cent of the suspended matter passes from the sedimentation chamber to the sludge chamber, a decidedly high assumption, as reference to Fig. 173 will show. On this basis 876 lb. of solids will be collected in the sludge chamber from each million gallons of sewage. Some of this solid matter will be digested by the anaerobic processes going on in the sludge chamber, and this reduction of the solids will be assumed as 25 per cent, leaving 657 lb. of solids to go into the sludge. The sludge is allowed to remain for 7 months in a deep tank, it will be assumed, so the water content of the sludge will be taken as 85 per cent. Table 47 shows that sludge of this water content measures 0.378 cu. yd. per 100 lb. of suspended matter. Hence $6.57 \times 0.378 = 2.48$ cu. yd. of sludge storage capacity must be provided for every million gallons of sewage passing through the tank during the sludge digestion and storage period. This period may be taken in this case as 7 months, with the result that about 528 cu. yd. of space is required for a daily flow of 1,000,000 gal.

In providing this amount of storage for the sludge, care must be taken that the shape of the tank will permit all the room allotted to the sludge to be actually available for it and that the sludge does not approach within 18 in. of the slot measured vertically. With some sewage the ebullition in the sludge chamber and the formation of scum are very active at times, particularly in protracted hot weather.

Another method of estimating the volume of sludge space has been developed[1] by Kenneth Allen, who has proposed the following formulas:

$$\text{Storage (combined sewage) in cubic feet} = 10.5\,PD$$
$$\text{Storage (domestic sewage) in cubic feet} = 5.25\,PD$$

where P is the population in thousands and D is the detention period in days. The formulas are based on sludge with 80 per cent moisture.

It is to be noted that in certain industrial wastes, such as those from packing houses, the total amount of suspended solids may be so much greater than in the sewages ordinarily dealt with, as to require abnormally large sludge compartments, scum spaces and gas vents which will necessitate radical change in their design and in the usual proportions of Imhoff tanks.

Radial-flow Tanks.—There are two general types of Imhoff tanks, one in which the sewage flows radially from an inlet at the center of a circular basin and the other in which the sewage flows longitudinally through a basin rectangular in general plan.

The Imhoff tanks designed under the direction of Calvin W. Hendrick by Leslie C. Frank for Baltimore are examples of radial tanks. As shown in Fig. 175, the sewage passes from the main distributing channel through a trough 12 in. wide and 12 in. deep to a central distributing ring of the same cross-section. There are eight 12 by 12-in. openings in its bottom through which the sewage passes down into the sedimentation chamber; provision was made for partially closing the openings near the inlet trough with flat plates if this seems necessary to equalize the flow of sewage. One purpose in admitting the sewage to the sedimentation chamber in this way was to give it a downward velocity so that short-path surface currents would not exist. Deep baffle plates have also been used in radial flow tanks to attain the same end.

The settled sewage is drawn off through 50 V-shaped weirs on the inner side of a circumferential effluent collector. The weirs are made of galvanized-iron plates, slotted so that they can be accurately adjusted on the side of the channel by means of thumb screws inserted through the slots. From the

[1] "Sewage Sludge," p. 228.

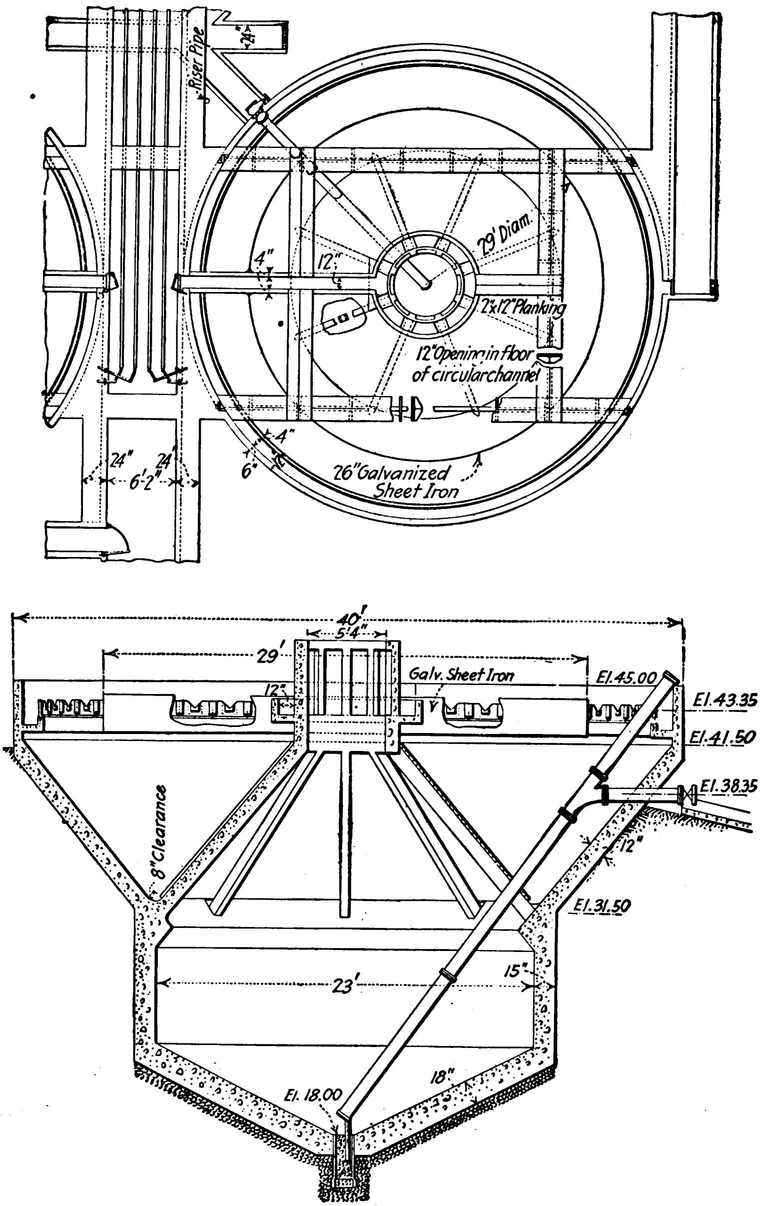


FIG. 175.—Radial-flow type of Imhoff tank, Baltimore.

collector the sewage passes into an effluent channel leading to the control house of trickling filters which give the next stage of the treatment. The tanks were designed for a 2-hr. detention period, each tank being designed to care for the sewage from a population of 4,000 persons.

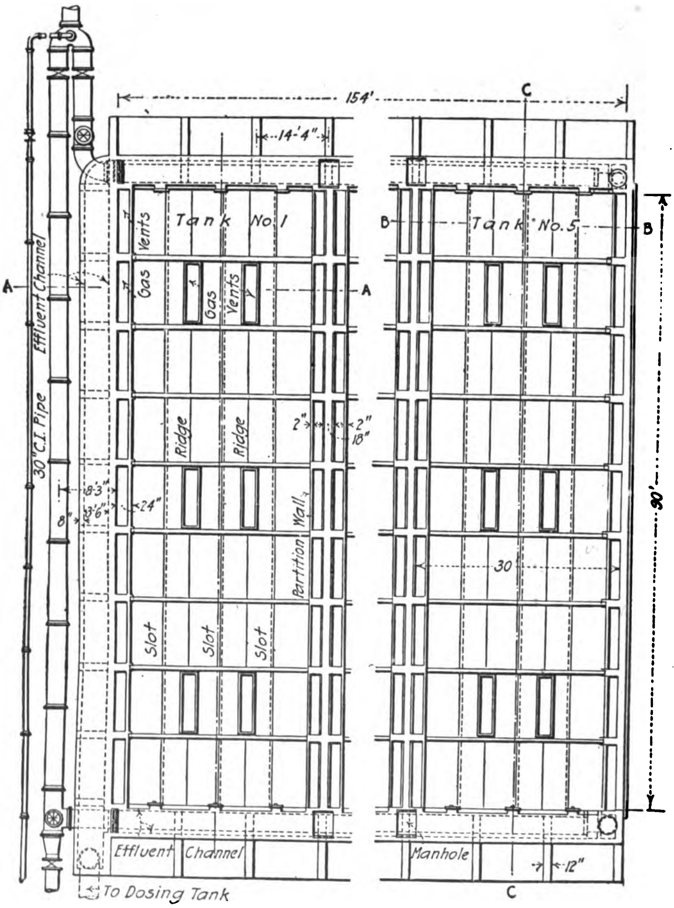


FIG. 176.—Partial plan of Imhoff tanks, Fitchburg, Mass.

The solids settle through the sewage in the sedimentation chamber and collect in the annular V-shaped space at the bottom, eventually slipping through the slot there into the sludge chamber below. It will be noticed that the slot is guarded by a projecting ring on the wall of the sludge chamber, so that bubbles of gas cannot pass through it into the sedimentation chamber.

The sludge chamber was designed to hold 1 cu. ft. of accumulated solids per capita of tributary population. The depth of the lowest part of the chamber below the surface of the sewage in the sedimentation chamber is 25.5 ft. The gases given off during the digestion of the sludge escape through the central gas vent. The conical upper part of the sludge chamber, or the part above the slot in rectangular tanks, is sometimes called the scum chamber because of the large quantities of scum often formed there in some tanks of this type. The sludge is removed through a pipe

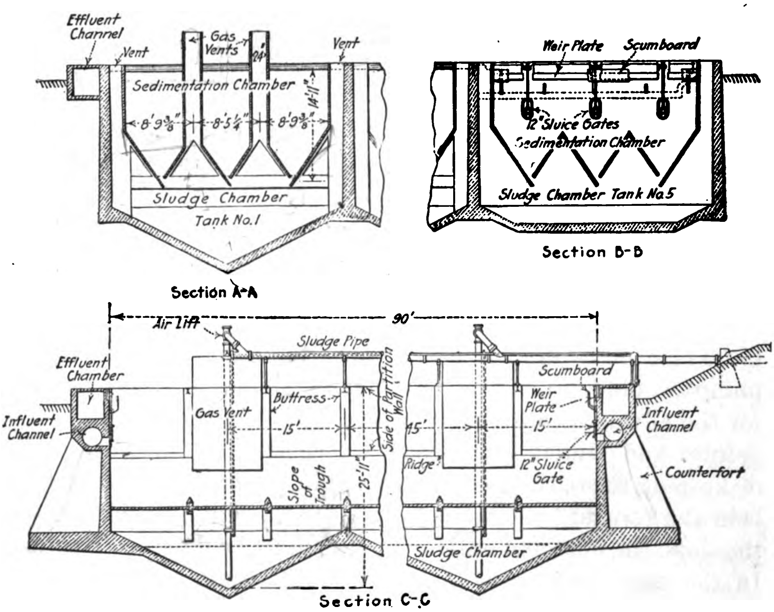


FIG. 177.—Sections of Imhoff tanks, Fitchburg, Mass. (The lines on which the sections are taken are given in Fig. 176.)

by gravity when the outlet gate is opened. The outlet is 4.85 ft. below the surface of the sewage in the sedimentation chamber.

Parallel-flow Tanks.—An example of the parallel-flow type of tank is given in Figs. 176 and 177, showing an installation built at Fitchburg, Mass., from the plans of D. A. Hartwell and H. P. Eddy. This plant consists of five 90 by 30-ft. tanks placed side by side, with an inlet conduit and effluent channel at each end, so that the direction of flow through the tanks can be reversed.

The sewage enters each tank through three 12-in. sluice gates, one for each trough in the sedimentation chamber. In most American Imhoff tanks only one or two troughs have been used over a sludge chamber. The sedimentation chambers were designed to detain the sewage for 3 hr. when flowing at the rate of 125 gal. per capita daily for a population of 55,000. The velocity in the tanks under such a condition is about 30 ft. per hour. The actual detention period in 1918 was from 5.4 hr. in March to 11.4 hr. in July, averaging 8 hr. for the year. The tanks removed in that year 64.6 per cent of the suspended matter in the sewage delivered to them, and about 95 per cent of the settling solids were removed. At the foot of the tank the settled sewage falls over adjustable metal weir plates into the effluent channel.

The solids slip through the three slots into the sludge chamber, which was built large enough to hold the sludge accumulating for 6 months at the rate of 0.007 cu. ft. per capita daily. The gas vents are arranged in four lines along each tank and their area is 17 per cent of the total area of the tank. The bottom of the sludge chamber consists of three hoppers, from which the sludge is raised by air lifts because the sludge beds are too high to be reached by gravity discharge and pumping with centrifugal pumps would tend to remove the gases which seem to be essential for the advantageous characteristic drying of the material.

Inlet and Outlet Channels.—On account of the desirability of keeping the surface of the sewage in the sedimentation chambers at a constant elevation, in order to prevent diffusion through the slots, the inlet and outlet channels must be designed carefully. In the case of the radial-flow tanks of Baltimore, Fig. 175, thin longitudinal walls were so placed in the inlet channel that one of the sub-channels does not have to serve more than four tanks. This subdivision was made because it was considered easier to distribute a given flow properly among a few units than among many, and experience had shown elsewhere that four tanks could be served by one channel. The following notes on this feature of the design were written by Frank:[1]

"In subdividing a given channel, the total difference in water level in its length is increased, but this increase can be neutralized by increasing the total width of channel. This was done in the present instance. The whole distributor system has been so computed hydraulically that the elevation of the water surface at the initial point of subdivision is practically the same

[1] *Eng. Rec.*, July 4, 1914.

in all the subchannels. This has resulted, of course, in subchannels of different widths. The distributor channels have, in addition, been provided throughout with adjustable distributor wings so that when the plant is taken into operation the whole system may be regulated to the desired nicety."

In the case of parallel-flow tanks another factor must be taken into account. Space in the sludge chamber of an Imhoff tank is expensive and it is desirable to utilize it fully. If sewage flows through the tank in one direction, there will be a greater accumulation of sludge near the inlet than the outlet, and consequently provision has sometimes been made for reversing the flow from time to time. In German practice this reversal was made at least once a month; at Fitchburg, Mass., in 1918, only three reversals were made during the year, but the usual frequency is about once each month.

The width and depth of the inlet channels must be such that sedimentation will not take place in them. It is advisable that none of the outlet weirs of the tanks be submerged by the effluent in the lower part of the effluent channels during periods of heavy discharge.

Scum boards in front of all weir outlets are desirable as in septic tanks. The practice in the Emscher district was to submerge them 12 to 16 in. and let them rise 12 in. above the surface. Special attention should be paid in placing them to avoid causing any increase in the currents in the lower part of the sedimentation chamber.

Sedimentation Chamber.—The length of the sedimentation chamber, or its radius if a radial-flow tank is used, is determined by the detention period adopted. When that is fixed the length of the tank or the radius, according to the type chosen, will depend upon the rate of flow which is considered proper for the given local conditions.

After the length has been adopted, the other dimensions are obtained by selecting a depth and width which will best suit all local conditions. In this connection it should be kept in mind that it is desirable to have the horizontal velocity gradually decrease toward the lower part of the sedimentation chamber, in order that there may be no currents to prevent the settling solids from passing through the slot into the sludge chamber. This consideration also affects the depth and position of the baffles if used in long Imhoff tanks, for deep baffles will cause a disturbance in the currents in the parts of the chamber where it is most desirable to have steady flow.

As it is very desirable for the settling solids to move steadily to and through the slots, the inclined sides of the bottom of a sedimentation chamber must have a sharp inclination and a very hard, smooth surface. Even with great care in construction, it is necessary to clean them from time to time with rubber squeegees, for soft, sticky material accumulates on them which will cause undesirable conditions in the chamber unless it is removed. For the same reason, it is desirable to have the top details of the tank such that this cleaning can be done readily, expeditiously, and thoroughly, either from permanently fixed platforms or temporary plank walks.

The slot at the bottom of the sedimentation chamber should be from 6 to 8 in. wide, depending upon the size of the chamber and the character of the sewage. The horizontal overlap to prevent gases passing through the slot should be at least 4 in. and preferably more.

Solids which collect on the surface of the sedimentation chambers are skimmed off by the attendants and thrown into the gas vents.

Gas Vents.—The gas vents should be so designed that workmen can enter the sludge chamber through them when the tank has been emptied. In the Emscher district the vents were from one-fourth to one-third of the area of the tanks, but as no difficulty had been experienced there in a few cases with vents amounting to only one-tenth of the tank area, the early American tanks of this type generally had a small ratio of vent to tank area. This has been found to be a mistake. Where the vents are small, the scum forms rapidly, and if the attendants do not break it up as soon as it collects in large quantities, it may rise over the edge of the vents. Imhoff recorded an instance of scum rising 6 ft. above the level of the sewage because the gases in it could not escape.

The vents must be easy of access to enable the attendants, if necessary, to break up the scum and liberate the gases in it, which hold up the solids of which it is composed. These solids should be in the sludge chamber undergoing digestion. It is, however, very difficult to break up and force the scum down in many cases, and after this has been accomplished the solids are usually again carried to the surface within a very short time. It has been found by some engineers that excessive scum formation may be prevented by stirring the sludge with jets of water from perforated pipes provided for the purpose at the bottom of the tank, thus liberating entrained gases. Imhoff considered such stirring was also helpful in assisting the decomposition of the organic matter by removing the toxins caused by previous energetic bacterial action. If the scum becomes too voluminous, even after stirring the sludge with jets of water, it may be removed to the sludge beds, although some operators prefer to break it up and push it down into the sludge chamber. At Fitchburg, where the scum is thick and heavy, openings have sometimes been made through it to afford a free outlet for the gas. Foaming has very seldom occurred at this plant. It has been promptly stopped after a small quantity of sludge in the hopper below the vent had been pumped out.

Sludge Removal.—The sludge chamber should be designed so that the oldest portion of the sludge can be drawn off first, which necessitates inclined bottoms sloping to sumps. In the Emscher district it was found that a slope of 1 on 2 was practicable if provision was made for flushing the sludge with water if it failed to slip down. The water was admitted through a horizontal 1½- to 2-in. lead pipe at or near the top of the slopes, and occasionally through another ring or single jet at the sump at the bottom of the hopper. The perforations were about $\frac{5}{32}$ in. in diameter and about 20 in. apart.

The sludge is usually removed through a pipe of about 8 in. diameter. In the Emscher district tanks this is always carried up straight from the sump until the top is above the surface of the sewage, so that in case it becomes clogged with sludge, which occasionally forms a mass within it, the material can be stirred by a jet of water through a cock tapped into the cap on the end of the pipe. In America a straight pipe, although desirable, has not been regarded as essential. Where a straight pipe is used, the sludge is not discharged from the end but through a curved branch with a valve, which is located, if possible, so that the outlet of the branch is from 4 to 6 ft. below the level of the sewage. This head of sewage is enough to force the sludge, after it is loosened by jets of water from the perforated pipes before mentioned, up through the sludge pipe and its branch to a drain to sludge-drying beds. Where a gravity discharge is impracticable, the sludge can be raised vertically by an air lift to a sufficient elevation to ensure a gravity flow to the drying beds.

In the Emscher district, after the sludge chamber had developed normal operating conditions, a portion of the sludge was drawn off about every 4 weeks, except when winter weather made it undesirable to place sludge on the drying beds. American tanks have been designed as a rule so that no sludge need be removed from them during the winter, provided they are operated properly. Experience shows that all the sludge should never be withdrawn, for if this is done it is necessary to repeat the tedious and sometimes difficult initial ripening process.

In drawing off sludge, the rate of withdrawal should be slow in order that the whole mass of sludge may settle and no conical depression exist in the center. In the latter case a considerable quantity of relatively fresh sludge will be withdrawn and some of the older well-rotted material left, whereas all the sludge which is still undergoing active decomposition should remain in the chamber to keep up, without interruption, the changes taking place there.

CHAPTER XIV

ACTIVATED SLUDGE

The idea from which the activated sludge process for the treatment of sewage has been developed appears to have originated from a series of experiments started at the Lawrence Experiment Station, in 1912, to determine the effect of aeration upon sewage. Sewage had been aerated experimentally and in at least one case as a method of regular treatment,[1] before this, but not in the same way. The results of the work at Lawrence, which was continued during subsequent years, were so unusual that a knowledge of them led to other experiments along similar lines at Manchester, England. This process of aerating sewage in the presence of sludge of a special character continues to produce similar sludge, which the English investigators called "activated sludge." The treatment of sewage by aeration in the presence of such sludge is now generally called activated sludge treatment.

In plain sedimentation an opportunity is given for a part of the settling solids to subside naturally. An attempt is usually made to keep the sewage in practically the same condition as when it enters the sedimentation tanks, except for this partial clarification, and therefore the sludge is removed before active anaerobic decomposition takes place. In chemical precipitation the removal of solids is greatly increased by the formation of small particles of floc[2] of a somewhat gelatinous nature in the sewage, which attract the minute particles of suspended matter and some of the colloids to themselves. This sludge is much more voluminous than that from plain sedimentation and is removed before active anaerobic decomposition takes place. In the septic process sedimentation takes place as in the process first mentioned, but the sludge is allowed to remain in the tanks for a long period of time, during which it is in a state of active anaerobic decomposition which soon makes septic the sewage entering the tank. In the Imhoff tank plain sedimentation goes on in the

[1] S. R. Lowcock, *Proc.* Inst. C. E., vol. clxiv, Part II, 1906.

[2] The word "floc" as used herein, signifies a single sponge-like mass or an aggregation of such masses.

upper story and septic decomposition in the lower story, the two processes being kept separate by preventing diffusion between the liquids in the two stories and the escape of gases and sludge from the lower to the upper story.

In activated sludge treatment a wholly different principle is utilized, the removal of suspended matter coupled with the aerobic decomposition of some of the organic matter, and in some cases the subsequent nitrification of the ammoniacal compounds. This is similar to the process which nature follows in changing the composition of sewage while it passes through a successfully operated intermittent sand filter. The septic tank treatment is merely an exaggerated and accelerated example of the processes by which Nature changes the composition of sewage discharged into bodies of water relatively so small as not to provide sufficient oxygen to carry on successful aerobic decomposition of the organic matter.

Description of Process.—When sewage containing a suitable supply of dissolved oxygen is agitated for a sufficient length of time, the fine suspended matter and the colloidal substances gather into floc. When the sewage thus coagulated is permitted to stand quiescent, this floc quickly settles and forms at the bottom of the tank a very thin mud or sludge. This sludge is then withdrawn and normally contains about 2 per cent of solids. If such sludge be introduced into raw sewage the rapidity with which the suspended and colloidal matters are coagulated may be greatly increased.

The process, as ordinarily carried out, consists of adding sludge to the sewage in proper proportion—roughly, 1 to 5—introducing sufficient air to provide enough dissolved oxygen to maintain aerobic conditions, and agitating the mixture until practically all of the suspended and colloidal matter has been flocculated or absorbed by the floc introduced into the sewage. The mixture is then conducted to tanks where the floc is removed by sedimentation and the clear supernatant water passes away as effluent. That portion of the sludge which is not required for the treatment of the incoming sewage is diverted and disposed of as described later.

Although the idea from which the early development of this process followed appears to have originated in this country, the process was carried through the first stages of development in England and certain patent rights are claimed by some of these early investigators. Much of the recent development and the perfection of the process has been in this country.

Theory of Purification.[1]—The first and perhaps most noticeable function of the process is that of coagulating or flocculating the suspended and colloidal matters in the sewage. This action is similar in effect to the well-known chemical coagulation with sulphate of alumina or sulphate of iron and lime, and the floc resembles the chemical coagulum, particularly the ferric hydrate from the ferrous iron and lime treatment.

The floc is a sponge-like mass, or, as expressed by Stein[2] "an open-mesh network" which, in the process of formation, may envelop, entrap or entrain colloidal matter and bacteria. The sponge-like structure of the floc offers a very large surface area for contact and this floc appears to be able to absorb colloidal matter, gases and coloring compounds. When the floc is driven about in the liquid it has a sweeping action[3] by which the colloidal substances may be said to be swept out of the water, or as stated by Parker[4] the "process may be regarded as passing a filter through the water in place of passing the water through a filter." Thus far the process appears to be primarily of a physical nature. It has been demonstrated, however,[5] that it cannot be carried out under sterile conditions.

Just what the action of bacteria and other organisms may be, is not fully understood. One plausible theory is that the bacteria which are contained in the cell-like structure of the floc feed upon the very finely divided matter and thus relieve the floc of its burden of such substances and restore its faculty of absorption to such an extent that, when introduced into the incoming sewage, the floc efficiently performs its function of absorbing the colloidal matter, which will again be consumed by the living organisms which thus cause its regeneration. It is because of these properties that the sludge has come to be called "activated sludge," a term suggested by Ardern and Lockett.

One of the cardinal principles of this method of treatment is that there must be dissolved in the sewage an ample supply of

[1] Adapted from article by Harrison P. Eddy in *Jour.* West. Soc. Engrs. vol. xxvi, p. 259; 1921.

[2] Stein "Water Purification Plants and their Operation," p. 143.

[3] Wm. R. Copeland, *Second Annual Report*, Milwaukee Sewage Commission, Dec. 31, 1915, p. 115.

[4] "The Control of Water," p. 556.

[5] Ardern and Lockett, *Jour.* Soc. Chem. Ind., vol. xxxiii, p. 535, "Oxidation of Sewage without Filters."

oxygen to maintain aerobic conditions. The action of the organisms in consuming the colloidal matter is one of digestion or oxidation, sometimes referred to as "moist combustion." Through this action the actual weight of suspended and colloidal matter is reduced, the products of the combustion being carried off in the form of dissolved or gaseous matter. This process, under favorable conditions, may extend to nitrification, by which substantial quantities of nitrates and nitrites are formed. Nitrification, however, is not necessary to the maintenance of sludge activity.

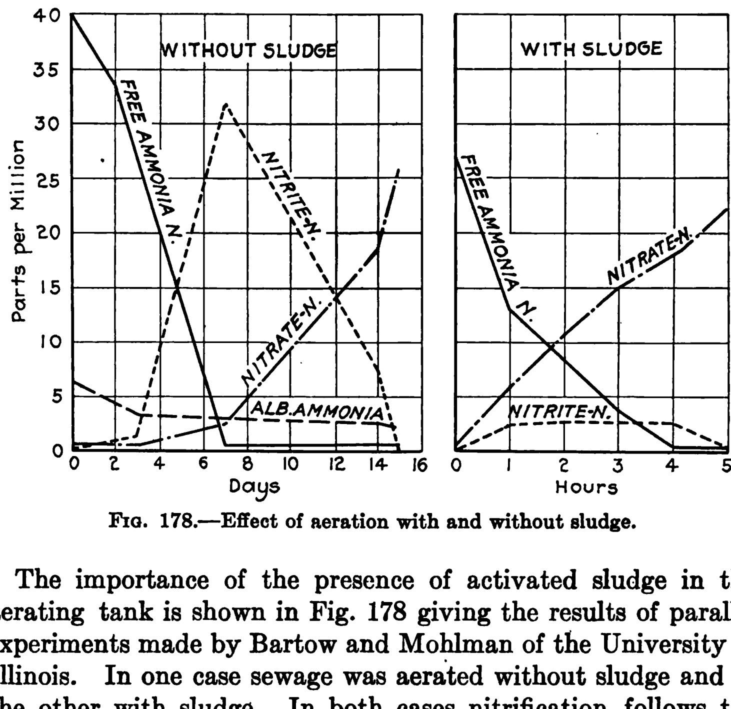


FIG. 178.—Effect of aeration with and without sludge.

The importance of the presence of activated sludge in the aerating tank is shown in Fig. 178 giving the results of parallel experiments made by Bartow and Mohlman of the University of Illinois. In one case sewage was aerated without sludge and in the other with sludge. In both cases nitrification follows the nitrogen cycle, but without sludge the completion of the cycle is a matter of days and the stages of change are distinct, while with activated sludge the cycle is completed n a few hours and the nitrite nitrogen is oxidized at once to nitrate nitrogen.

Establishment and Maintenance of Sludge Activity.—In order that the treatment of sewage by this process may proceed properly and as rapidly as possible it is necessary for the sludge to become "activated" with great numbers of aerobic and facultative bacteria, which are able to bring about the decomposition of the organic matter and the conversion of the nitrogen into nitrites and nitrates.

The production of activated sludge from the suspended matter in raw sewage was formerly a tedious operation. The most rapid method developed at Manchester, England, was to fill a tank with sewage, aerate it for 21 hr., let it settle for 3 hr., then draw off the supernatant sewage retaining the sludge, and repeat this process during a period of 12 days. Then the period of aeration was reduced to 9 hr. and the tank was filled twice a day. A few days later the aeration period was reduced so that the tank could be filled three times daily. After 48 fillings requiring 26 days, the accumulation of partially activated sludge amounted to 18.6 per cent of the contents of the tank measured after 2-hr. sedimentation. The sewage in the tank was then replaced with fresh sewage and aeration carried on continuously for 15 days, in order to nitrify the sewage completely. The clear liquid was then replaced by fresh sewage and aerated for three days until nitrification was complete. This process was continued until, at the end of 8 weeks from the beginning of the work, activated sludge was obtained which produced a nitrified effluent from raw sewage after 8-hr. aeration. In the original work at Manchester at least 6 months were needed to obtain satisfactory sludge.

The accumulation of an initial supply of well-activated sludge can be completed much more quickly when starting with the sludge from sedimentation tanks receiving effluent from trickling filters, for such sludge is similar in character to activated sludge and can be thoroughly activated more quickly than the sludge which accumulates from plain sedimentation or from the aeration of fresh sewage. For example, in starting some of his activated sludge experiments at the Milwaukee testing station, Copeland produced good sludge in this manner in a little more than a week. Under favorable conditions, with daily accretion and periodic withdrawal of the excess, the sludge will retain its activity, and treatment by this process can be continued indefinitely. It is necessary, however, not to overwork the sludge, for in that case it will gradually lose its activity and become ineffective.

Bartow and Mohlman secured a supply of activated sludge from fresh wholly domestic sewage at the University of Illinois experiment station, by drawing off the sewage and adding fresh sewage at intervals of 24 hr. for 15 days. The sewage was changed every 12 hr. during the next 8 days and then every 6 hr. for 4 days. The difference between this process and the original English method of obtaining activated sludge, is that no attempt is made to aerate each charge of sewage until the ammonia nitrogen has been removed. Later experience showed that it is best to add the fresh sewage every 6 hr. from the beginning of the run. With the kind of sewage used in the test sufficient sludge could be obtained in this way, in a week, to produce a fair effluent, and at the end of 20 days there was enough sludge to produce in 6 hr. an effluent with less than 1 p.p.m. of ammonia nitrogen.

In a city having several independent activated sludge plants, it may be desirable to start up new units that may be built, with activated sludge brought from the older operating plants, in order to reduce the period of time required to produce activated sludge.

Care must be taken that none of the floc collects in the channels between tanks, and in other places where it might become septic, for if septic decomposition takes place to a material extent, the proper operation of the activated sludge process may be seriously disturbed owing to, or at least accompanied by, reduction of the dissolved oxygen in the liquid. For the same reason it is important to avoid storing the sludge for long periods of time, unless it is being continuously supplied with air and moderately agitated.

Physical Properties of Activated Sludge.—Thoroughly activated sludge floc generally has a golden-brown color and is relatively compact. Under-aerated floc is usually light-brown in color, fluffy and relatively light in weight. Over-aerated floc may be of a muddy-brown color, probably due to unabsorbed colloidal substances, and may be considerably broken up and disintegrated into relatively fine material which settles slowly and leaves a turbid supernatent liquid.

Well-activated floc settles more readily than under-aerated floc and forms a denser sludge, leaving the supernatant liquor very clear, under favorable conditions. Under-aerated floc is so light that it forms a very thin and bulky sludge and a considerable proportion of it is apt to be carried out of sedimentation tanks.

Activated sludge contains a very small proportion of suspended solids, as compared with sludges obtained from the treatment of sewage by other processes. The activated sludge, when well-aerated and accumulated under favorable conditions, generally contains about 2 per cent solids (98 per cent water). Particularly well-activated sludge may contain as much as 3 per cent solids, or perhaps slightly more, while under-aerated sludge may contain only 0.5 per cent suspended solids, or even less. This variation in density has a very important bearing upon the design of the plant and the handling of the sludge, for a sludge containing 0.5 per cent solids will have a volume four times as great as that containing 2 per cent solids, as explained in Chapter XV.

Hatton reported that sedimentation of the Milwaukee sludge containing 2 per cent solids in a tank 26 ft. deep increased the solids to 6 per cent. In such work it is necessary to keep in mind that if the depth of sedimentation tanks is increased, the time required for the concentration of the sludge from one proportion of moisture to another is also increased. Pratt and Gascoigne found that the rate of increase and time required for concentration was about as the cube root of the depth. Copeland found that sludge acidified with sulphuric acid and heated for 2 hr. was concentrated from approximately 2 to 9 per cent solids.

Another property of activated sludge, of importance to the designer, is the manner in which it clings to sloping surfaces. It does not slide readily on slopes much flatter than 60 deg. with the horizontal. This has been observed many times and the deterioration of sludge which has collected on slopes of 45 deg., instead of sliding down them freely to the sludge removal pipes as was expected, has been reported as the cause of inferior results in some work with the activated sludge process.

Reactivating Sludge.—If the period of aeration of the sewage, containing the proper proportion of activated sludge, and all other conditions be properly regulated, the sludge can be kept continuously in good active condition. On the other hand, if the sewage be under- or over-aerated, or if the activated sludge be overworked, it will lose a portion or the whole of its activity and must be reactivated in some manner, in order to restore it to a condition in which it can exercise its proper functions efficiently. Such reactivation may be carried out in the mixture of sewage and sludge, or the sludge, after separation from the major portion of the treated sewage in sedimentation tanks, can be removed to separate tanks and there be aerated for a sufficient period of time to restore its normal activity.

In some cases it has appeared that the reactivation of the sludge was advantageous as a regular part of the process, permitting the use of a smaller quantity of air and volume of return sludge than where the process was so conducted as to maintain the sludge in prime condition throughout. The balance between the work required of the sludge and its activity, in at least one case, was so closely adjusted that the sludge showed a gradual decrease in activity from Monday to Saturday, and a rapid recovery of activity during the period of lighter load from Saturday noon until Monday morning. During the latter time the sludge regained the activity lost during the preceding days of heavy load. In this case the sludge was reactivated in tanks set apart for this purpose, but the general reactivation process during the period of light load was required in addition to the regular re-aeration.

In other cases there has appeared to be little if any advantage in re-aeration of the sludge in separate tanks. In tests at Milwaukee, where sludge containing from 2 to 3 per cent of solids was aerated, the air appeared to accumulate in large bubbles and escape from the surface at irregular intervals. This tendency appeared to render the air less effective as a means of agitation, and possibly also to interfere to some extent with the uniform distribution of the dissolved oxygen throughout the sludge.

Bacterial Digestion of Sludge.—More or less bacterial digestion of the organic matter of the sludge goes on continuously in aeration and sedimentation tanks. This action is more marked at moderate temperatures when bacterial activity is high, than at the low temperatures of winter.

In certain tests made by the authors it was found that aeration can be carried so far that the sludge will become disintegrated and lose its clarifying power. At such times the increase in the proportion of sludge accumulation in the tank becomes very small, and there may even be a decrease. This condition can be remedied by removing a portion of the sludge, by reducing the period of aeration, or both; this will result in the rapid building up of a large quantity of fresh activated sludge and, almost immediately, in a well-clarified effluent.

Winter Operation.—Very high or very low temperature of the sewage is detrimental to treatment. The unfavorable effect of low temperature has been repeatedly demonstrated.

In very cold weather at Milwaukee, Copeland found that the free ammonia was not reduced and nitrates were not formed to a material extent, but the removal of organic matter was so great and the low temperature of the sewage permitted the liquid to absorb so much dissolved oxygen that the effluent was stable in the absence of nitrates. It was also found that melted snow from the streets and the cold muddy storm water in the early spring caused a decrease in the size of the floc and in its ability to absorb colloidal matter. This condition could be limited, in part at least, by increasing the air supply about 25 per cent.

Cold weather was found by Pratt and Gascoigne to have no appreciable effect on the process at the Cleveland Testing Station, where clarification instead of stability was the operating aim. Lederer has suggested that in cold weather the turbidity of the effluent is a good index of its quality. At such times he found the stability (explained in Chapter IX on "Chemical and Biological Characteristics of Sewage") was 100 per cent with turbidities of 10 p.p.m. or less; it varied between 50 and 100 per cent with a turbidity of 15; and fell off rapidly when the turbidity exceeded 15.

Extent of Purification Secured.—An important characteristic of this method of treatment is the high degree of purification which may be accomplished.

So large a proportion of the suspended solids and colloids may be removed that under ordinary conditions the effluent will be clear and contain but very little suspended matter and comparatively few bacteria. The reduction in bacteria, however, is accomplished by physically removing them, rather than by destroying them as in the case of disinfection.

In many cases color may be removed, probably by action similar to that of mordants.

In the case of odors, some may be oxidized directly and some absorbed, the resulting effluent being comparatively free from odors noticeable to the ordinary observer.

Oxidation of organic matter in sludge occurs coincidentally with the flocculation, and there may be nitrification where the treatment is fairly complete. In the latter case, the treated sewage may be charged with a substantial quantity of available oxygen in the form of nitrites or nitrates, which will serve as a factor of safety against future putrefaction.

An interesting characteristic of this process is its adaptability to partial treatment. If the removal of colloids and bacteria and the production of a moderate degree of stability be sufficient, the process can be stopped short of material oxidation of organic matter and nitrification. On the other hand, if complete stability and oxidation of a substantial quantity of organic matter are required, the process can be so operated as to accomplish this.

TABLE 48.—PURIFICATION OF SEWAGE COMPARED WITH PERIOD OF AERATION

(T. Chalkley Hatton, *Engineering News*, vol. lxxv, 1916, page 307.)

Aeration, hours	0	1	2	3	4	5
Cubic feet of air per minute	0	160.00	160.00	160.00	160.00	160.00
Cubic feet of air per gallon	0	0.67	1.32	1.98	2.64	3.31
Appearance of settled liquor	Turbid	Clear	Clear	Clear	Clear	Clear
Stability, hours	0	2.00	33.00	120+	120+	120+
Bacteria removed, per cent	0	52.00	81+	92+	95+	98+
Free ammonia, parts per million	22.00	17.00	15.00	11.00	7.00	5.00
Nitrites, parts per million	0.08	0.00	0.95	1.75	2.20	2.50
Nitrates, parts per million	0.08	0.04	0.70	2.80	5.60	8.20
Dissolved oxygen, parts per million	0.00	0.30	1.90	4.30	5.90	6.70

The progressive nature of the changes that take place is shown in Table 48, giving the results of a continuous supply of 0.67 cu. ft. of air per minute to sewage which originally had 225 p.p.m. of suspended matter. These tests were made in 1915 at the Milwaukee Testing Station, by Copeland. The data in this table indicate that all that it is necessary to do to obtain an effluent of the desired quality is to select the proper detention period for the operating conditions. This, however, is not as simple as it appears, for these conditions, such as the design of the tanks, the rate of application of the air, the character of the sewage, and the quality of the activated sludge, will vary greatly under different conditions and some of them will vary continuously in the same plant.

As early as 1917 Hatton determined upon the following standard as a measure and limit of purification to be effected by the activated sludge process at Milwaukee:

Reduction of bacteria, per cent	90
Reduction of suspended solids, per cent	95
Stability by methylene blue, hours	72

Prolonged tests with the strong sewage containing a substantial proportion of industrial wastes have demonstrated that it is entirely practicable to meet this standard at Milwaukee.

While it may be proper, at the present state of development, to assume that, in general, purification may be carried to almost any extent desired, this assumption must be made with great caution. It is important to consider carefully the character of the sewage or industrial wastes to be treated, for there may be disturbing influences similar to those which appear to have been important at Worcester, where it is stated that it was practically impossible to operate the activated sludge plant so as to obtain, continuously, uniformly good results, and, probably because of the acid iron wastes from the wire mills, that the extent of purification was decidedly limited, notwithstanding the use of very large quantities of air, a goodly period of aeration and the reaeration of the sludge.

Preparatory Treatment of Sewage Required.—In the treatment of some sewages, particularly where industrial wastes such as those from tanneries and packing houses are present, it may be necessary, or at least advisable, to subject the sewage to preparatory treatment for the removal of those solids which are likely to form deposits upon the porous plates, to be described later, through which the air is introduced, otherwise the plates may be rendered practically ineffective. It has been found at Milwaukee that sufficient solids are removed by first passing the sewage through grit chambers and fine screens. In some cases it may be found that subjecting the sewage to a brief period of sedimentation may remove sufficient suspended matter to permit subsequent treatment with a very much smaller quantity of air than would otherwise be required.

Proportion of Return Sludge.—There is an intimate relation between the period of aeration, the volume of air required, and the proportion of activated sludge introduced into the sewage. The variation in one of these factors may be offset within limits by proper adjustment of the others. It is important to secure the most effective and economical regulation of these quantities.

The activated sludge is the workshop of the bacteria. Here they receive the raw materials of the sewage, work them over producing remarkable bio-chemical changes, and finally discharge some of the products of their processes, which pass away in the effluent. But the sludge performs another very important function, that of serving as a vehicle for the removal, from the water in which they came to the tanks, of much of the unchanged and partially changed matters, including the bacteria themselves. It is necessary to have an ample supply of this sludge, the proportion or quantity depending in a measure upon the character of the sewage and the degree of the purification desired. In general, the quantity of sludge (containing 2 per cent solids), required for the treatment of moderately strong municipal sewage will be equivalent to about 20 per cent of the volume of sewage to be treated; this proportion, measured as it is introduced into the raw sewage, will generally correspond approximately with the volume of sludge remaining in a test sample taken from the aeration tank, after standing for ½ hr.

At Milwaukee, Copeland found that there was practically no difference in the quality of effluent obtained with 18, 22, 26 and 30 per cent of activated sludge in the aeration tanks, measured in this manner. With very strong sewage, particularly that containing large quantities of colloidal matter, a greater proportion of sludge, even as high as 50 per cent of the volume of the sewage to be treated, may be required for satisfactory results.

Pearse found, with a very strong sewage from the packing houses in Chicago, that the quantity of activated sludge in the aerating tanks should be much greater than that required at Milwaukee.

The character and condition of the sewage or industrial wastes which necessitate the use of a very large proportion of activated sludge are also likely to require long periods of aeration, such as ten hours, and large volumes of air, such as from 5 to 10 cu. ft. per gallon of sewage, in order to maintain satisfactory working conditions.

The use of an unnecessarily large volume of sludge is likely to be disadvantageous because of its demand for oxygen.

The volume of sludge required for successful treatment will depend in large measure upon the density of the sludge. If for any reason the sludge is particularly voluminous as drawn from the sedimentation tanks, say such as to have a density of but 0.5 per cent solids, the volume required to produce satisfactory results will be from two to four times as great as if it contained 2 per cent solids.

Period of Aeration.—A substantial length of time is required for the fine suspended and colloidal substances to become coagulated and for the absorption of colloidal matter by the return sludge and the newly formed floc.

As indicated by Table 48, the greater the degree of oxidation of the organic matter required, the longer must be the period of aeration.

The character of the sewage has a marked influence upon the required period of contact. A dilute fresh domestic sewage may become well coagulated in from 3 or 4 hr., whereas a strong sewage containing industrial wastes may require 6 or even 8 hr., and it may be necessary to aerate for 8 or 10 hr. or even longer industrial wastes high in colloidal matter, to secure the same relative extent of purification. For American municipal sewages the economic period of aeration appears to lie between 4 and 8 hr.

Quantity of Air Required.—Air may be utilized for two purposes, for maintaining aerobic biological conditions and for agitation. Agitation, however, may be accomplished by purely mechanical means, in which case the introduction of air would be necessary simply to maintain sufficient dissolved oxygen for aerobic conditions. In some cases the supply of oxygen has been derived from contact with the open air and no artificial supply has been required.

The quantity of oxygen necessary for the maintenance of aerobic conditions is relatively small. From a comparison of analyses of the air entering and the gases escaping from aeration tanks, Crawford and Bartow estimated that only about 5 per cent of the oxygen of the air introduced during the aeration process was utilized in maintaining aerobic conditions. In the case of strong tannery wastes the authors found that only 10 per cent of the oxygen in the air introduced disappeared during the process.

Experimenting to determine the actual amount of oxygen necessary to stabilize sewage of "average strength," Clark found the average requirement to be 42.6 p.p.m., equivalent to 350 lb. of oxygen per 1,000,000 gal. This is somewhat less than 1 per cent of the quantity which would be used in the aeration process if the rate were 2 cu. ft. of air per gallon of sewage treated, or about 2 per cent in case 1 cu. ft. of air were sufficient for sewage of this character.

Where agitation is effected by blowing air into the sewage, the quantity of air required is very large. It is necessary to provide sufficient agitation to prevent the heavy suspended solids from settling down on to the air diffuser plates and thus clogging them. As already pointed out, removal of suspended matter by fine screening may be a substantial aid in preventing such clogging. Even sewage which has passed a fine screen having openings ⅛ by 2 in. may contain sufficient solids of this character to cause clogging of the plates, if the air supply be permitted to fall much below 1 cu. ft. per minute per square foot of plate area. Finer screening might make it possible to reduce this quantity slightly.

The volume of air required in any given case will depend mainly upon the following general conditions, previously discussed:

1. Quantity of sewage to be treated,
2. Quality of effluent desired,
3. Period of aeration,
4. Proportion of sludge to be returned to the sewage,
5. Temperature of aeration.

As a general statement, where the aeration is effected by blowing air into the sewage, passing continuously through tanks from 10 to 15 ft. deep, with the area of diffuser plates in the proportion of 1:6 of tank area, and a period of aeration of about 6 hr., the quantity of air may be about as follows:

Character of Sewage	Cubic Feet of Free Air per Gallon of Sewage
1. For rather weak domestic sewage	1.0
2. For strong municipal sewage containing some industrial wastes not particularly detrimental to bacterial life	1.5
3. For weak municipal sewage containing considerable industrial wastes some of which are detrimental to bacterial life, as for example acid-iron wastes	2.0
4. For strong municipal sewage containing considerable industrial wastes some of which are detrimental to bacterial life, as for example, acid-iron wastes	4.0
5. For municipal sewage containing sufficient industrial wastes, not specifically inhibitive of bacterial life, to decidedly influence the composition of the sewage, *e.g.*, packinghouse, tannery, etc.	3.0
6. For strong packinghouse wastes	4.0
7. For strong tannery wastes containing a very large proportion of colloids, but settled before aeration	6.0

As the cost incident to the use of a large quantity of air is burdensome in operating expense, as well as in the fixed charges (interest and depreciation) upon the installation, it is necessary to reduce it to the minimum practical limit.

Some English experiments indicate that 0.5 cu. ft. of air per gallon will give excellent purification, while at Milwaukee 1.5 cu. ft. is as small a quantity as gives assurance of satisfactory results. This difference may be due in part to a difference in the character of sewages, and in part to the fact that the English tests were made on a small laboratory scale while some of those at Milwaukee were made on a relatively large scale. Many experiments have been performed with a view to discovering means for reducing the quantity of air required. In some cases, instead of agitation by blowing air into the sewage, the sewage has been pumped over and over, and only a small quantity of air introduced. In other cases the sewage has been agitated for a short period, allowed to stand for a time during which the biological process was supported by the accumulated dissolved oxygen in the sewage, this period being followed by another short period during which the sewage was aerated. By continuing this intermittent aeration, it is stated that satisfactory results have been obtained with a relatively small volume of air.

From experiments with paddle agitators at Sheffield, England, Haworth concluded that given suitable agitation to obtain thorough intermixing of the activated sludge with the sewage and to prevent settlement, the necessary air required to maintain the biological activity might be obtained by surface contact. To accomplish this it was found necessary to produce constant change of surface of the liquid. Consequently agitators consisting of vertical pistons of the Root blower type, Fig. 181, were installed at the ends of relatively long channels. By rotating the agitators the sewage was circulated at a velocity sufficient to prevent settlement of the sludge. A velocity of 1½ to 2 ft. per second was found to be ample to accomplish this result. The power expended by this process was calculated to be about 20 hp. per million gallons, but there appeared to be no doubt that the agitators employed could handle a much larger volume with only a slight additional power consumption, thus reducing the power consumption per million gallons.

Measurement of Sludge.—The volume and character of sludge is such an important matter in this process of treatment that its measurement is very essential. The volume of sludge contained in the aeration tank liquor may be determined by allowing a measured portion of the liquor to settle for a definite period of time and then reading the volume of the settled sludge. If comparisons are to be made of the quantities of sludge produced and used, it is necessary that measurements be made under reasonably comparable conditions. One of the most convenient means of measurement is a 1,000-c.c. laboratory glass measuring cylinder about 2¼ by 16 in. inside dimensions, graduated in 10 c.c. units, from which the percentage of sludge can be read directly. For determining and regulating the proportion of activated sludge present in the aeration tanks and entering the sedimentation tanks, it is probable that a period of sedimentation in the measuring glass of ½ hr. will generally prove most satisfactory, particularly as the results of tests can be obtained with relative promptness. Tests for a shorter period are subject to too great error and variation.

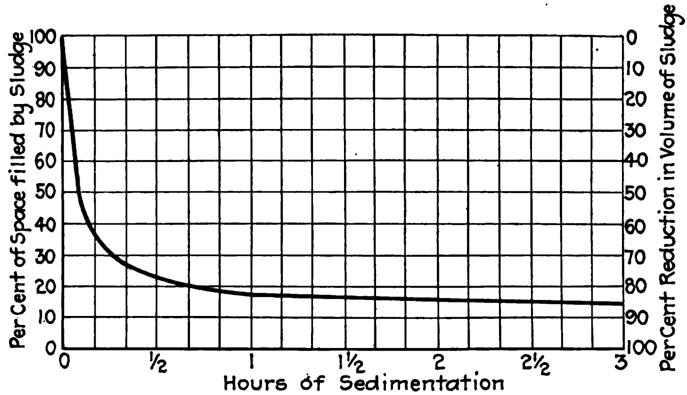


FIG. 179.—Rate of sedimentation (Milwaukee).

It has been found at Milwaukee that a test of this kind, made at the same hour each morning, serves as a valuable means of regulating and controlling the process, the adjustment of the sludge being accomplished by fixing the rate of drawing off sludge for disposal, according to the indications of the test.

An occasional series of tests for several different sedimentation periods may furnish valuable information and enable the operator to compare his operating conditions with those of other places where different test periods are adopted. The rate of sedimentation, or the volume of sludge after different periods of test at Milwaukee, is shown by Fig. 179.

Importance of Measurements of Sewage, Sludge and Air.—In order that the plant may be most advantageously operated, it is important that convenient means be provided for obtaining accurate measurements of the rate of sewage flow, the volume of sludge returned to the sewage, the volume of sludge drawn off to be disposed of, and the volume of air used in the aeration process. For these purposes the Venturi meter with the register-indicator-recorder equipment has proved satisfactory. Such measurements should serve as a guide in proportioning the return sludge and air to the volume of sewage being treated. The results of all such measurements should be entered in the records.

Aeration Tanks.—The earlier experiments were performed with fill-and-draw tanks which permitted a uniform period of detention and practically uniform contact of the sludge floc and air with all portions of the sewage. It was subsequently found, however, that the continuous flow method of operation was not only entirely practicable but much to be preferred.

While, for practical purposes, it is customary to speak of sewage being aerated in a continuous-flow tank for a period of time corresponding to the assumed detention period, as a matter of fact some of it will be aerated a much shorter time and some a much longer time. In such a tank sewage does not flow forward steadily and uniformly from inlet to outlet. The experiments by Clark at the Lawrence Experiment Station in 1917 showed that in a tank 8 ft. deep above its conical bottom, operated with a detention period of 10.5 hr., with air applied at the rate of 0.15 cu. ft. per gallon per hour, some of the sewage passed through the tank in 1 hr., half passed through in 7½ hr., and all of it did not pass out until the 35th hour. The tests were made by dosing the entering sewage with salt or ammonium chloride and determining the proportion of these substances which the effluent contained during the period of experiment. In experiments with horizontal-flow tanks at Cleveland, Pratt and Gascoigne found that instead of securing a displacement of 100 per cent of the content of the tank only 40 per cent was obtained, until under-and-over vertical baffles were installed, and thereafter only 75 per cent. Thus the detention period under the two conditions actually averaged only 40 and 75 per cent, respectively, of its computed length.

The depth of aeration tanks will depend in part upon local conditions, although it is important to remember that the amount of power required for compressing the air increases with the increase in depth of liquor into which it is introduced, as discussed later. At Milwaukee, where a depth of 15 ft. was adopted

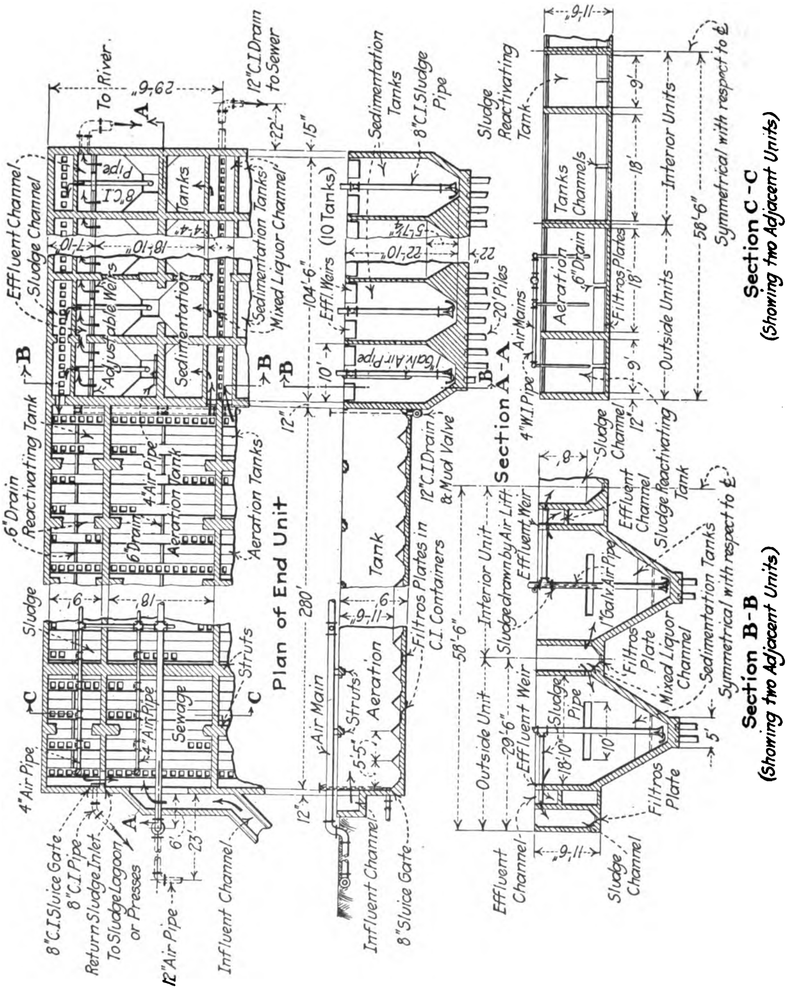


FIG. 180.—Activated sludge sewage treatment plant at Houston, Tex.

for the tanks, the controlling consideration was the impracticability of obtaining sufficient land for shallower tanks.

Such tanks are usually rectangular in shape and from 6 to 15 ft. in depth. One type of rectangular tank is illustrated by Fig. 180.

Where air is introduced through porous plates in the bottom of the tank, the plates are generally arranged crosswise of the flow of sewage, thus causing the sewage to be subjected to successive impacts of air, sometimes referred to as "air baffles." Theoretically there appears to be advantage in securing as great a number of such impacts as possible, which favors the use of long tanks. The length of the tank is also influenced by the desirable width.

The desirable width of tank may be influenced by the arrangement of the air piping and diffuser plates. Thus far it has seemed wise to limit the number of diffuser plates to from eight to ten 12-in. square porous plates per container to be fed by a single air line. By installing the air main along the center line of the tank and supplying plate containers on either side, a tank width of about 20 ft. results.

Much study has been given to the shape of aeration tank floors. The most common is the alternate ridge and valley type, sometimes called the "sawtooth" type. The porous plates are placed in the valleys. The ridges between the plates are intended to deflect the settling solids to a position over the plates, whence they will be again carried up by the upward currents induced by the air escaping from the plates.

The aeration tank dimensions are controlled by the period of aeration selected as most economical and suitable for the extent of purification required, under the conditions of the particular problem being considered. At Milwaukee, the aeration period is to be 6 hr. at the average rate of flow, resulting in tanks about 22 ft. wide, 237 ft. long and 15 ft. deep. The volume of the tanks below the level of the surface of the sewage is determined arbitrarily by dividing the average 24-hr. flow of sewage by four, and providing a tank capacity equivalent to this volume.

The most suitable proportion of the area of air diffusers to the area of the tank appears to lie between 1 to 4 and 1 to 6, although smaller ratios have been adopted in some cases.

Where perforated pipe diffusers are used, the type of tank bottom may be similar to that adopted in the case of the porous plate diffuser.

Air Diffusers.—Much study has been given to the subject of air diffusion, with a view to securing that type of diffuser which shall deliver the air to the sewage in a state of very fine division and which shall not become clogged, and which shall not offer excessive frictional resistance to the passage of the air.

The simplest method is to deliver the air to the sewage in the tank through holes in pipes laid in the bottom of the aerating tank. There has been some difficulty with this system, however, because of clogging of the orifices with rust, and with sludge after the air supply has been shut off.

Monel-metal cloth has been used experimentally for diffusing air and is apparently efficient, giving small, well-distributed bubbles; but the cloth becomes clogged in continuous operation, causing a great increase in frictional resistance and for this reason has not been used practically.

In a number of English installations, the diffusers are flat porous tile laid as covers of concrete troughs containing perforated pipes through which the compressed air is conveyed to the space below the porous tile.

In the course of the investigations at Milwaukee, tests were made of diffusers of basswood ½ in. thick and from 2½ to 4 in. square. They were developed by Nordell, who designed a concrete container for holding them in place and delivering compressed air to their lower surface. They gave good distribution of air, but were found to have objectionable features which will be mentioned later.

Satisfactory results have been obtained with diffusers made of filtros plates. These are usually 12 in. square and about 1½ in. thick, made of ground quartz sand and a silicious binding material baked until hard. There are several grades of plates having the same degree of porosity but varying in frictional resistance. Tests made at Milwaukee of one of these grades showed that the initial loss of pressure for passing 2 cu. ft. of air per minute per square foot, under 5 lb. air pressure, was ¾ lb. and for every additional cubic foot of air there was a further loss of ¼ lb.

Attempts have been made to set filtros plates in cast-iron holders but there is danger that they will become choked with iron rust if this method of support is used. In 2½ years service in aeration tanks at Houston, Tex., rust from such frames reduced the capacity of filtros plates from 7.1 cu. ft. of air per minute under a pressure of 2 in. of water to 0.7 cu. ft. The plates were set in frames with sulphur joints, and about one-tenth of them were broken in removal. Those that were uninjured were immersed in hot hydrochloric acid and after being cleaned in this way were able to pass 6.2 cu. ft. of air per minute.

The best setting for plates seems to be portland cement mortar. Of a number of grades of bituminous mortar used by the authors for this purpose none has been found to be satisfactory in service.

In a series of tests by Bartow and Mickle at the University of Illinois in 1918, with perforated pipes, wood blocks and coarse and fine filtros plates, it was concluded that the filtros plates were most efficient, giving sludge that settled readily, leaving clear, stable effluents; there was little difference in the effluents from the tanks with fine and coarse filtros plates. The wood block diffusers were difficult to install, deteriorated rapidly, and gave unstable effluents in a majority of samples, and these effluents were inferior chemically to those from filtros plate aeration. The effluents from tanks with perforated pipes were the least satisfactory.

Filtros plates become clogged if the compressed air contains oil and dust. For this reason, it is important to provide clean air and to have the air compressed by a mechanism that does not allow oil to enter the air even in very minute quantities. This trouble was experienced at Milwaukee in 1916. As a result of the clogging of the plates, the air was unevenly distributed and sand, pieces of water-logged wood and other coarse material gathered over the spots where the air was shut off, and finally covered parts of the plates through which a decreased amount of air rose. Through the plates which were porous, the air streamed in excessive volume, forming large bubbles, less effective than those ⅛ in. or less in diameter. In later experiments at Milwaukee, the air was filtered through 10- or 12-oz. duck, then through canton flannel and was finally washed by the hydroturbine blower used in compressing it.

The air is best supplied to the diffusers through pipes protected against rust, for if rust forms in these pipes the plates are liable to become clogged by it.

After the activated sludge has been introduced into the raw sewage, it is important to prevent it from settling in channels leading to aeration tanks and from aeration tanks to sedimentation tanks. In order to avoid loss of head within the plant, it may be advantageous in some cases to construct such channels with relatively large dimensions, either level or on very slight slopes. The velocity of flow will then be low. In such cases it may be advantageous to provide for aerating the mixture of sewage and activated sludge while flowing through the channels, by equipping them with air diffusers in a similar manner to that adopted for the aeration tanks.

Power Required for Compressing Air.—The pressure at which the air must be supplied depends chiefly upon the depth of the sewage in the aeration tanks, but is influenced by the frictional resistance of the distribution system, and particularly that of the diffusers in case porous plates are used for this purpose. Practical limitations will be such in most cases as to require an air pressure of between 2.5 and 10 lb. per square inch.

In selecting the type of blower, there should be borne in mind the probability that impurities in the air will gradually accumulate on the surface or in the pores of plate diffusers, resulting in partial clogging and a gradual increase in the frictional resistance. The initial operating pressure may have to be increased considerably from time to time on this account, and the blower should be selected with this in view. It is also important to provide a flexible blower plant capable of adjustment in air supply to the volume and quality of sewage and other conditions subject to frequent change. Variation in volume of air can be secured by throwing in or out of service one or more compressors. Greater flexibility, however, can be secured by the use of centrifugal compressors, Fig. 181, with which the volume of air furnished can be varied by opening or closing the blast gate in the suction pipe. The power required to drive this type of compressor varies approximately with the volume of air used, when operating at a constant speed. The possibility of varying the volume of air compressed, without wasting power, is one of the advantages of this type of machine. On the other hand, this type of compressor furnishes air at a uniform pressure, which can only be changed by changing the speed of the impeller, which with certain types of driving apparatus is not practicable. In such cases, the pressure must be high enough to meet all emergencies. Therefore at all other times there is a waste of power in compressing the air to a higher pressure than is necessary.

At a given speed a practically constant volume of air is furnished by positive pressure blowers and rotary blowers alike, Fig. 181, the pressure depending upon the resistance offered, as by the head of water against which the air is introduced into the aeration tanks, and by the friction in pipe lines and diffuser plates. One of the advantages of this type of blower is that it will build up pressure sufficient to overcome increased frictional resistance in the filtros plates and that it is not necessary to waste power

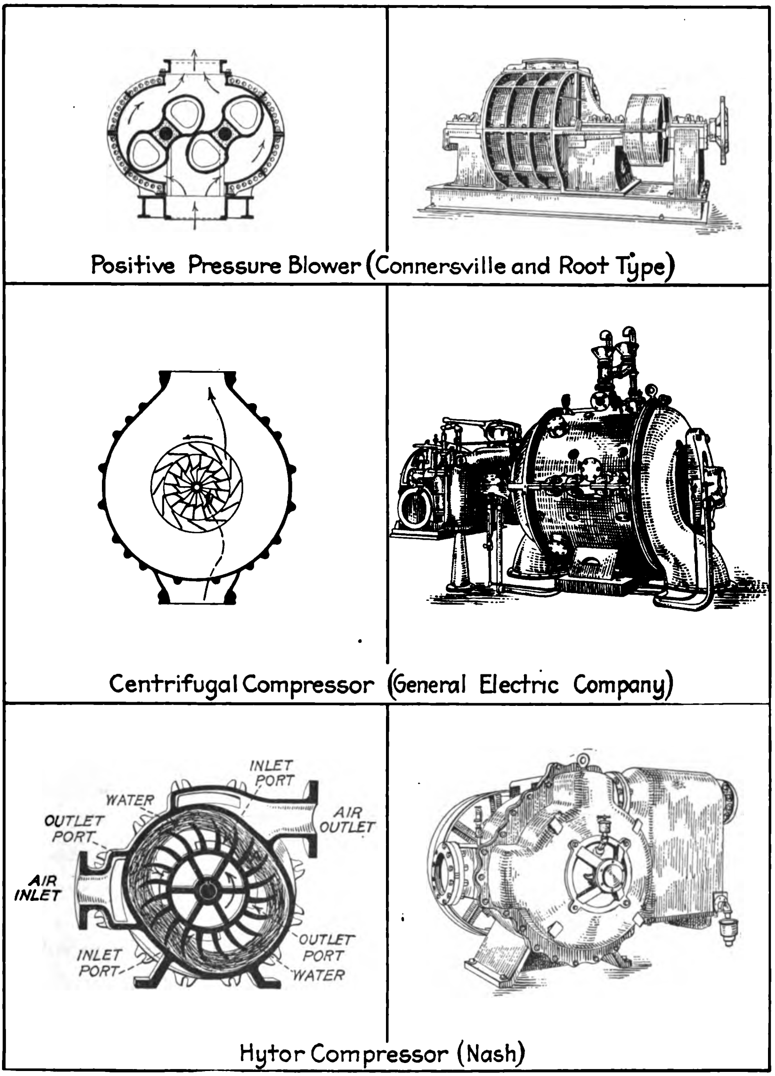


FIG. 181.—Types of air compressors.

by raising the pressure unnecessarily high, as in the case of the centrifugal blower. On the other hand, if great flexibility be required in the volume of air, there may be a waste of power due to compressing an unnecessarily large volume. Probably in most activated sludge plants sufficient flexibility can be obtained by providing a proper number of units of appropriate size and varying the number of units in use. The amount of power required and its cost can be estimated from data in Table 49, by applying the assumed efficiency and other data peculiar to the case under consideration. The efficiency which may be expected from a high-grade electric-motor-driven blower plant will vary according to the type and size of machine, but in general may be assumed to be from 60 to 70 per cent.

TABLE 49.—POWER REQUIRED FOR COMPRESSING AIR

Final pressure of air (lb. per sq. in.)	Theoretical work to compress mil. cu. ft. of free air (horsepower-hours)	Theoretical power to compress 100 cu. ft. free air per minute (horsepowers)	Theoretical cost of compressing 1 mil. cu. ft. free air with elec. power at 1 ct. per kw.-hr., dollars
1	72.3	0.43	0.539
2	144.0	0.86	1.074
3	200.8	1.20	1.498
4	265.2	1.59	1.978
5	325.8	1.95	2.430
6	384.7	2.31	2.869
7	442.5	2.66	3.301
8	490.2	2.94	3.656
9	543.4	3.26	4.053
10	596.5	3.58	4.449
12	697.1	4.18	5.199
14	785.4	4.71	5.858
16	875.9	5.26	6.533

NOTE.—This table is based on the assumption that the air is compressed under adiabatic conditions (without cooling) as is the practice in nearly all blower work, from atmospheric pressure (14.7 lb. per square inch) and an initial temperature of 60°F. Weight of 1 cu. ft. free air = 0.0764 lb. Slightly lower power consumption may be obtained with good water-jacketed reciprocating compressors.

A convenient approximate unit of power to bear in mind is that 30 hp. per hour will be required for furnishing 1.5 cu. ft. of free air per gallon of sewage for the treatment of 1,000,000 gal. of ordinary municipal sewage in 24 hr., in tanks 10 ft. deep.

Sedimentation Tanks.—The mixed liquor[1] from the aeration tanks, containing from 0.2 to 0.6 per cent solids, is run into settling tanks for clarification. The extent of treatment in the aeration tanks, therefore, has a very important effect upon the

[1] Aeration tank effluent; aerated sewage including floc.

process of sedimentation. It has been proved in some cases that aeration sufficient to produce a satisfactory effluent, if the floc could be removed, was insufficient to produce a floc which would readily settle in sedimentation tanks, or a reasonably dense sludge. In these cases, therefore, the extent of aeration may be controlled by the requirements of the sedimentation process or by the required density of the sludge.

In some of the earlier experimental work, sedimentation was secured in the same tank in which the aeration process was conducted, that is, the tank was filled with sewage and activated sludge, the mixture aerated for a sufficient length of time for the formation of floc, and thereafter the mixed liquor was allowed to stand until a clear supernatant effluent could be drawn off and the excess sludge removed. The process was then repeated. This process is too complicated for practical use in a large plant, and has been succeeded by aeration and sedimentation in separate continuous-flow tanks. Such sedimentation tanks may be constructed either according to the vertical flow or the horizontal flow principle; Pearse used the former successfully at the Packingtown (Chicago) experimental plant, and the latter has been used at Houston, Milwaukee and other places.

Copeland found that in order to secure complete removal of the floc the velocity of flow of liquor through the sedimentation tank must not be allowed to exceed 3 ft. per minute. This velocity should be computed upon the approximate cross-section of flow to which the stream of liquor may be expected to be confined.

As the area of the sedimentation tank with respect to the volume of mixed liquor which it can successfully handle is an important consideration, it has been found convenient to use the unit of horizontal area in computations relating to sedimentation tanks. As a result of experiments at Milwaukee, the proposed sedimentation tanks have been designed to provide 1 sq. ft. of tank area for each 1,600 gal. of sewage to be treated, measured at the time of assumed maximum flow of sewage, such rate being expected to occur only in times of storm. This is equivalent to 1 sq. ft. of tank for each 850 gal. more or less, based upon the average daily flow.

While sedimentation can be efficiently carried out in very shallow tanks, it is important that the tanks should have a sufficient depth to provide for temporary storage for the volume of sludge produced during peak flows, or at other times when sludge is produced at an unusually high rate or when the sludge is particularly voluminous as a result of under-aeration. Sedimentation tanks may be either rectangular or round, and may be provided with hopper bottoms or with bottoms sloping slightly toward the center, provided the tanks are equipped with mechanical squeegees. In the case of hopper-bottom tanks, the slope of the bottom should be at least 60 deg. with the horizontal, in order to assure the sliding along it to the outlet pipe situated at the apex of the hopper, of that portion of the accumulated sludge which comes in contact with the floor.

The expense and difficulty of providing hoppers with such steep slopes for very large tanks has led to the adoption, in some cases, of tanks in which the floors have only a very slight slope toward the center, as, for example, ¾ in. per foot. Such tanks, equipped with mechanical squeegees or plows for scraping toward the center of the tank any solids which may actually reach the floor, should be round at the bottom. They may be relatively large, those proposed for Milwaukee being about 98 ft. in diameter. Most of the sludge in such tanks flows toward the center without coming into contact with the bottom of the tank. The plows are necessary only to push along that portion which does lodge upon the bottom.

Volume and Character of Sludge Produced.—The first stage of successful sewage aeration with activated sludge produces a clear effluent after sedimentation, and the quantity of sludge yielded by this stage of the treatment is probably as great as, or greater than, that yielded by carrying the treatment along to the production of a thoroughly nitrified effluent. This quantity depends primarily upon the quantity of suspended and colloidal matter in the sewage, and as there is a great variation in the content of such matter there is inevitably a great variation in the quantity of sludge produced, particularly when the sewage contains certain classes of industrial wastes, such as iron salts and organic matter from packing houses and tanneries.

Experience indicates that a short period of aeration with good activated sludge will reduce the suspended matter carried by the influent about 90 per cent. A portion of the suspended matter thus removed is converted into gaseous and dissolved substances by digestion, the remainder forming sludge. The investigations of Pratt and Gascoigne, at the Cleveland Testing Station, appear to indicate that sometimes as much as 50 per cent of the suspended matter might be digested, the average being about 35 per cent. Bartow and Mohlman found that the liquefaction was about 4.5 per cent in their experiments at the University of Illinois. This change in the suspended matter makes it difficult to estimate the probable quantity of such matter in the sludge produced by this treatment. The tests of Bartow and Mohlman indicate that with strictly domestic sewage there will be from 750 to 1,200 lb. of dry sludge per million gallons of sewage treated.[1]

At Milwaukee, with sewage containing nearly 300 p.p.m. of suspended solids, the quantity of dry solids in the sludge may be as high as 2,000 to 2,500 lb. per million gallons.

If industrial wastes and storm water are added to domestic sewage, corresponding increases in suspended solids must be provided for.

The volume of activated sludge depends upon its density, which may be expected to vary according to many conditions from 0.5 to 3.0 per cent solids (99.5 to 97 per cent water). In many cases, for economical handling and dewatering, it is probable that the aeration and sedimentation processes should be so controlled as to provide a sludge having 2 per cent or more of solids.

The sludge drawn from the sedimentation tanks consists of the activated sludge introduced into the raw sewage plus that formed from the solids in the sewage thus treated. The relation of the volume of sludge to the volume of sewage treated, and the volume of waste sludge to that returned to the sewage, are illustrated by the following assumed proportions, based upon a sludge containing 2 per cent solids.

Volume of sewage treated	1,000,000 gal.
Volume of sludge returned to sewage, 20 per cent of the sewage	200,000 gal.
Excess sludge, 12 per cent of the sewage	120,000 gal.
Total sludge from sedimentation tanks, 32 per cent of the sewage	320,000 gal.

$$\text{Return sludge} = \left(\frac{200,000}{320,000}\right) = 62.5 \text{ per cent of total sludge.}$$

$$\text{Excess sludge} = \left(\frac{120,000}{320,000}\right) = 37.5 \text{ per cent of total sludge.}$$

[1] The quantity per million gallons will vary greatly, according as the sewage is strong or weak.

There has been a tendency to underestimate the volume of sludge produced by this treatment. The early investigation at Milwaukee indicated that 3,000 to 6,000 gal. of excess sludge with 2 per cent solids would be produced per million gallons of sewage treated, in addition to the quantity returned to the activating tanks. Later investigation showed that 15,000 gal. might be produced and the treatment works were finally designed on the assumption of 15,000 gal. of excess sludge containing 2 per cent solids.

Sludge from the sedimentation tanks contains so much water that it will flow readily in pipes. In fact in computing the size of pipe lines and conduits to transport sludge, but slightly lower coefficients of discharge are used than for water. Where sludge is aerated during its flow through conduits, an additional allowance may be advisable on account of interference with flow caused by cross-currents. In some computations the authors have used a coefficient as low as $C = 80$ in the Hazen and Williams formula, to allow for both of these influences.

The velocity in sludge pipes and conduits should be not less than 2 ft. per second, if deposition of solids is to be avoided, which is essential. Where the velocity must be less than this, air should be utilized to secure agitation and to maintain the sludge activity.

Activated sludge may be raised by centrifugal and other types of pumps, by the air lift, and by the compressed air ejector. It is important to avoid violent agitation which will break up the floc and reconvert it into colloidal condition. There is no serious danger of this in slow or moderate speed centrifugal pumps of suitable design, and even less in the air lift and the ejector. Where the ejector is used, as may be desirable in sludge pressing plants, provision should be made to prevent as far as possible forming a layer of sludge on the bottom and a stratum of comparatively clear supernatant liquor.

Disposal of Sludge.—Three methods of disposing of activated sludge have been studied, as follows:

1. Drying on sand beds and using the dried sludge for filling waste land;
2. Pumping the sludge into lagoons in which eventually it will dry out and become suitable for raising crops, either *in situ* or by removal to appropriate areas;
3. Conversion into commercial fertilizer.

At Worcester Lanphear dried the sludge from the experimental plant upon sand beds, but found that this method would be impracticable on a large scale because of the small depth of sludge, 4 or 5 in., which could be applied to the beds at one time, the usual dose being 2.0 to 2.5 gal. per square foot. If more than this depth was applied, the sludge settled rapidly and water remained on top, which had to evaporate before actual drying of the sludge commenced. The dose had to be of such size that most of the water would drain away during application to the bed. In good weather the partially dried sludge began to crack in about 24 hr., and it was possible to remove the sludge from the bed in about 48 hr. from the time when it was applied.

At Houston, Tex., sludge from two plants has been pumped into lagoons for a considerable period of time. When these lagoons were examined by the authors, during hot weather, no offensive odor was observed. There was, however, the characteristic odor of algæ, which appeared to be growing in considerable quantities. The supernatant water was clear. Many gas bubbles were rising to the surface, carrying with them some sludge which, however, promptly resubsided with the escape of the gas. There was very little floating sludge upon the lagoons.

Activated sludge usually contains from two to four times as much nitrogen as sludge from other processes of sewage treatment. The sources of this increase in nitrogen content are not clearly defined. Clark showed in 1916 that the colloidal matter in the Lawrence sewage contained from two to three times as much organic matter as the coarser suspended matter, and that the increase in nitrogen in the sludge, due to these colloids, is about 33 per cent, but this accounts for only about three-fourths of the total increase. It has been suggested that part of this unexplained increase may be due to the fixation of nitrogen, although Clark showed, by experiments in 1917, that part was apparently due to the more rapid oxidation of the carbonaceous matter than of the nitrogenous matter, which thus increased the proportion of the latter.

The proportion of nitrogen in some activated sludges has been sufficiently high to justify the hope that its conversion into commercial fertilizer may be economically practicable.

Preparation of Sludge for Market.—Preparation of sludge for use as a commercial fertilizer involves two major processes, dewatering and drying, aside from such minor treatments as grinding, bolting and bagging.

The most difficult step in the whole activated sludge process, including manufacture of fertilizer, has been the dewatering of the sludge. It has been extensively studied at Milwaukee, Packingtown (Chicago), Houston and Worcester. It has been demonstrated repeatedly that well-activated sludge can be dewatered more advantageously than under-aerated sludge, or stale septic sludge. It is therefore necessary to operate the plant, insofar as practicable, to avoid under-aeration, prolonged storage which tends to cause septic action, and, probably also, over-aeration. As previously stated, it is probable that the aeration process must be controlled and limited with a view to producing the sludge in the best condition for dewatering, unless a method be found for conditioning a poor sludge so that it may be successfully dewatered.

For dewatering activated sludge, machines of three types are available, the plate or standard filter press, the squeeze press commonly known as the Worthington or Berrigan press, and the Besco-ter-Meer centrifuge which is a modification of the older Shaefer-ter-Meer centrifuge. These machines are described in Chapter XV. Experiments upon a relatively large scale with each of these machines have been carried out at Milwaukee.

From investigations there and elsewhere, it appears that activated sludge is much more difficult to dewater than sludge from chemical precipitation, which has been dewatered for many years by means of filter presses. It has been found in the plate press and in the squeeze press that the sludge is much more easily dewatered under summer conditions when the biochemical agencies are probably most active in the aeration tanks, than at other seasons of the year, particularly the winter and spring. This fact suggests that the sludge may be conditioned by subjecting it to biological action at summer temperature.

The treatment of the liquid sludge with a small proportion of sulphuric acid improves its physical condition in such a manner as to greatly facilitate dewatering by pressing. This treatment also fixes some of the nitrogen compounds so that they are not driven off during the subsequent process of drying. The corresponding increase in nitrogen in the dried sludge in some cases has been sufficient to offset the cost of the acid treatment. For a sludge similar to that produced at Milwaukee, amounting to 1.5 per cent of the volume of sewage treated or 15,000 gal. per 1,000,000 gal. of sewage, the quantity of sulphuric acid required will be about 2.5 gal. per 1,000 gal., of sludge, equivalent to 270 lb. of commercial acid per 1,000,000 gal. of sewage. At 1 ct. per pound, the cost of acid will be about $2.70 per 1,000,000 gal. of sewage treated.

The nitrogen or ammonia,[1] the latter term being generally used in the trade, will vary considerably in different sludges. At Worcester, the ammonia content is very low, probably because of the comparatively high proportion of mineral matter in the sewage due to the iron wastes. In dried acidified press cake from municipal sewage not affected in this way by inorganic material, it may be assumed very roughly at from 5 to 6.5 per cent of the dry weight of the sludge.

Sludge drying in revolving machine dryers is similar to the drying of many other materials. The process has been thoroughly developed in the arts and there does not seem to be any unusual problem peculiar to the drying of activated sludge. There is, of course, the danger of burning the sludge, of creating objectionable odors and of the escape of considerable dust. The dryers should be designed with a view to reducing these dangers as much as possible and should be carefully operated with the same end in view. If the gases from the dryers are passed through suitable dust chambers and are thoroughly washed, it is probable that dust can be entirely eliminated and there does not seem to be danger of dissemination of disagreeable odors. The gases, however, should preferably be discharged through a relatively high stack.

Management.—One of the most serious inherent dangers in the application of this process lies in its management. The process is complicated by a large number of steps, each more or less dependent upon the preceding ones. Skillful, conscientious, technical management will be required to keep all features operating to best advantage with the marked variations in volume, character and temperature of the sewage and the multitude of ordinary operating difficulties. At this time (1921), it remains to be demonstrated whether such a process can be successfully operated as a commercial project under municipal conditions generally prevailing in the United States. To assure success, it will be necessary to enter the market and dispose of the commodity produced as advantageously as the law of supply and demand will permit. Prices are subject to radical fluctuations but the

[1] Ammonia = $\frac{17}{14}$ or 1.215 nitrogen.

costs under municipal control will probably not fluctuate in a commensurate manner. The one thing that is assured from the outset is the very substantial cost, but it may be hoped that in some cases enough revenue will be derived to so reduce operating expenses as to prevent them from becoming an unreasonable burden.

The process is in its infancy and marked improvements are to be expected which will greatly reduce the cost. The advantages over some other processes are so marked and there is such promise of improvements that further investigation is warranted.

CHAPTER XV

SLUDGE

The term "sludge" is generally used to designate the deposit which accumulates during one of the tank treatments of sewage and also the deposit from the final sedimentation of the effluent from trickling filters. Grit-chamber sediment and screenings are not usually classed as sludge, but when no preliminary separation of these solids is made the sludge contains both these elements. When treating similar sewages under similar conditions the volumes of sludge produced by the different processes are in the following order of magnitude: activated sludge process, chemical precipitation and sedimentation in plain, septic and Imhoff tanks.

Volume of Sludge.—The volume of sludge depends mainly on the percentage of water it contains, and it is sometimes desirable in comparing sludges to express their characteristics as those of similar sludges with 90 per cent of water, which is readily done by either of two formulas:

$$\frac{V_2}{V_1} = \frac{100 - P_1}{100 - P_2} \qquad (a)$$

where V_2 is the volume desired, V_1 is the volume under the actual conditions, P_1 is the percentage of water in the original sludge, and P_2 is the percentage in the sludge under the assumed modified conditions;

$$\frac{V_2}{V_1} = \frac{P_1}{P_2} \qquad (b)$$

Where V_2 is the volume desired, V_1 is the volume under the actual conditions, P_1 is the percentage of solids in the original sludge, and P_2 is the percentage of solids in the sludge under the assumed modified conditions.

These formulas disregard the effect of the dissolved solids, which are generally small in proportion to the total solids.

While it has hitherto been customary to consider sludges in terms of their water content, the student may find it more direct and helpful to consider them in terms of their content of solids.

Table 50 will be found helpful in making such comparisons and shows clearly the great effect of a small increase in water content upon the volume of sludge already containing a large proportion of water.

TABLE 50.—WEIGHT AND SPECIFIC GRAVITY OF 1 CU. YD. OF SLUDGE WITH DIFFERENT PERCENTAGES OF MOISTURE AND CONTAINING SOLIDS OF DIFFERENT WEIGHTS PER CUBIC FOOT

Percentage of moisture in sludge	Weight of dry residue in pounds per cubic foot														
	68			78			92			110			134		
	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent water	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent water	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent water	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent water.	Specific gravity	Weight per cu. yd., lb.	Ratio of vol. to that with 90 per cent water	Weight per cu. yd., lb.
98	1688	5.04	1.002	1692	5.08	1.004	1696	5.13	1.007	1699	5.18	1.008	1702	5.23	1.010
97	1689	3.35	1.003	1695	3.38	1.007	1702	3.41	1.010	1707	3.44	1.013	1711	3.47	1.016
96	1691	2.51	1.004	1699	2.53	1.009	1708	2.55	1.013	1715	2.57	1.018	1721	2.59	1.022
95	1692	2.01	1.004	1703	2.02	1.010	1713	2.03	1.017	1723	2.04	1.023	1731	2.06	1.027
94	1694	1.67	1.005	1706	1.68	1.012	1718	1.69	1.019	1730	1.70	1.027	1740	1.71	1.033
93	1695	1.43	1.007	1710	1.44	1.015	1724	1.44	1.023	1738	1.45	1.032	1750	1.45	1.039
92	1697	1.25	1.007	1713	1.25	1.017	1730	1.26	1.027	1745	1.26	1.035	1760	1.26	1.044
91	1698	1.11	1.008	1717	1.11	1.019	1735	1.11	1.030	1752	1.11	1.040	1770	1.11	1.050
90	1700	1.00	1.009	1720	1.00	1.020	1740	1.00	1.032	1760	1.00	1.044	1780	1.00	1.056
89	1701	0.91	1.010	1724	0.91	1.023	1748	0.91	1.037	1768	0.91	1.047	1790	0.91	1.062
88	1703	0.83	1.010	1727	0.83	1.025	1752	0.83	1.040	1776	0.83	1.054	1800	0.83	1.068
87	1704	0.77	1.011	1730	0.77	1.027	1758	0.77	1.043	1785	0.76	1.059	1810	0.76	1.074
86	1706	0.71	1.012	1734	0.71	1.029	1764	0.71	1.046	1793	0.70	1.064	1820	0.70	1.080
85	1707	0.66	1.013	1737	0.66	1.031	1770	0.66	1.050	1802	0.65	1.069	1831	0.65	1.086
84	1709	0.62	1.014	1741	0.62	1.033	1776	0.62	1.054	1810	0.61	1.074	1841	0.61	1.093
83	1710	0.59	1.015	1744	0.58	1.035	1782	0.58	1.058	1818	0.57	1.079	1852	0.56	1.099
82	1711	0.56	1.016	1748	0.55	1.037	1788	0.55	1.062	1827	0.54	1.084	1863	0.53	1.106
81	1713	0.53	1.017	1751	0.52	1.039	1794	0.52	1.065	1835	0.51	1.089	1874	0.50	1.112
80	1714	0.50	1.018	1755	0.49	1.041	1800	0.49	1.068	1844	0.48	1.094	1885	0.47	1.118
79	1715	0.47	1.019	1758	0.46	1.043	1807	0.46	1.072	1853	0.45	1.100	1896	0.44	1.125
78	1717	0.45	1.019	1762	0.44	1.045	1813	0.44	1.076	1863	0.43	1.106	1908	0.42	1.132

A satisfactory method of studying sludge problems of different plants is by means of the dry solids in the sludge, reduced to the number of pounds daily per 1,000 population. In this way the influence of differences in the water consumption and the ground water leakage is eliminated, but a chance for serious error is introduced if the population of a city is only partially served by sewers and there are large storm overflows. At Providence, R. I., chemical precipitation produced sludge containing from 202 to 266 lb. of dry solids per 1,000 population daily and plain sedimentation produced sludge with 136 to 140 lb. The chemical precipitation of weak night sewage at Worcester, Mass., gave sludge containing from 140 to 180 lb. of dry solids per 1,000 population daily, while plain sedimentation of strong sewage for 7 hr. gave from 145 to 180 lb. Chemical precipitation at London gave about 236 lb.

Another method of reporting the quantity of sludge, which can be used advantageously at times, is in terms of cubic feet per capita daily. For example, the slow sedimentation of the domestic sewage of a small Massachusetts city was found by the authors to give from 0.0146 to 0.0165 cu. ft. per capita daily, the average being 0.0162 cu. ft.

While these two methods of reporting the quantity of sludge are the best for comparing sludges of different cities, the common practice of operators of treatment plants is to report the gallons of sludge per million gallons of sewage, and its water content.

The weight of the dry solids in sludge has little influence on the weight per cubic yard of the sludge produced from most types of treatment, its influence only becoming marked in sludges with the lower percentages of moisture given in Table 50.

Measuring Sludge.—The elevation of the top of the sludge in a basin can be measured by means of a glass tube, through which a wire is strung, a hollow ball being attached to the end of the wire. The tube is lowered slowly into the tank, allowed to remain there long enough for all disturbances to quiet down, and then the bottom is closed by pulling the rubber ball against it. The elevation of the surface of the sewage is marked on the tube. When the latter is carefully raised, the elevation of the top of the sludge can be determined from its distance below the sewage level mark, and the volume of the sludge in the tank can then be computed from the dimensions of the basin. Kinnicutt used a 2½-in. glass tube 7 ft. long at Worcester. Johnson used at Columbus a ½-in. glass tube 2½ ft. long, fastened to one end of a wood rod 12 ft. long and closed at the bottom by a rubber stopper which could be pulled into place by a fine wire strung along the rod. This tube had too small a diameter to admit coarse matter into the bore, it was found.

The elevation of the upper surface of the sludge can sometimes be measured by slowly lowering through the sewage a weighted flat board or disk of sheet steel. The motion of the plate is noticeably checked when it reaches the sludge, unless the latter is very fluid.

Characteristics of Sludge.—The most voluminous sludge is that produced by the activated sludge method of treatment, amounting to 6,000 to 12,000 gal. or more of excess sludge at 97½ to 98 per cent moisture per million gallons of sewage treated, and even greater volumes from some classes of industrial wastes. It is generally of a brown flocculent appearance. When the color is very dark the sludge may be approaching septic conditions and if the color is very light there may be over-aeration with a tendency for the solids to settle slowly. If the sludge is flowed over a sand bed to a greater depth than 4 or 5 in., the suspended matter settles rapidly and clogs the pores of the sand before the water can pass through them, thus making the drying process a slow one even under the most favorable conditions, because a large part of the water can escape only through evaporation. This sludge has an inoffensive earthy odor when in good condition, but sometimes it has a tendency to become septic rather rapidly and when this occurs it has a disagreeable odor of decay. It is so voluminous that apparently the only practicable way of preparing it for final disposal is by mechanical dewatering. Its nitrogen content is generally so high as to be a noticeable characteristic.

The sludge from plain sedimentation is gray in color and possesses a very offensive odor in most cases; the sludge from disinfected sewage at Providence was not particularly offensive, being an exception to the usual condition. The sludge from plain sedimentation is slimy, usually quite dense and very difficult to dewater in filter presses. It can be dried by spreading it on porous beds, but must be spread very thin in order to enable the water to percolate away rapidly.

The large volume of sludge produced by chemical precipitation is due to the chemicals introduced into the sewage. If it is left in the tank it undergoes decomposition like the sludge from plain sedimentation, but at a slower rate. It gives off gas in substantial quantities and its density is increased by standing. If it contains considerable iron its surface may be red but below the surface the color is generally black. The odors from it may be objectionable but not so bad as those from plain sedimentation sludge. While it is somewhat slimy the hydrate of iron or aluminum in it makes it gelatinous. When spread on porous draining beds the water will gradually drain away or evaporate, leaving after several weeks a stratum of slimy sludge of about the consistence of lard, containing 70 to 80 per cent of water.

Septic tank sludge is black and usually very offensive on account of the hydrogen sulphide and other gases it gives off. That produced at Birmingham, England, is said to have relatively little odor because of the copper compounds in it. Other septic sludges are reported to possess relatively little odor, due in most cases to long digestion in the septic tanks. The coarser particles settling in the tanks are largely disintegrated by the putrefactive action going on in the sludge, so that it contains relatively small quantities of coarse material. The sludge can be dried on porous beds, if spread out in thin layers, but objectionable odors are to be expected while it is doing so.

The sludge from the Imhoff tank is characterized by an exceptionally large amount of gas contained in it. Although usually containing a larger proportion of solid matter than other sludges, it flows readily because of the fluidity imparted by the entrained gases. It is less offensive than sedimentation or septic sludge, its odor being like that of hot tar or sealing wax. When drawn off on to porous beds in layers 6 to 10 in. deep, the solids are first carried to the surface by the entrained gases, leaving a sheet of comparatively clear water below them which drains off rapidly and allows the solids to sink down slowly on the bed. As the sludge dries the gases escape, leaving it more or less spongy and with an odor resembling that of garden loam.

The sedimentation of trickling filter effluents gives a sludge quite different from any produced by the primary tank processes. It is brownish, relatively inoffensive when fresh, flocculent and usually contains a high percentage of water. It undergoes decomposition more slowly than other undigested sludges, although when it contains many worms it may become offensive quickly. In warm weather gases are liberated from it and large masses of it may at times be carried to the surface by entrained gas. It is like activated sludge in being very difficult to dry on porous beds, and at Worcester, where it contained large quantities of hydrate of iron, it could not be dried readily unless applied in a layer not more than a few inches deep.

In some cases, as at Fitchburg, Mass., the secondary tank sludge is pumped to the sewage Imhoff tanks, to take advantage of the fibrous matter and entrained gas in the Imhoff tank sludge and thus secure a sludge mixture which is much more easily drained and dried than the secondary tank sludge alone. By this method of operation, unstable organic matter in the secondary-tank sludge, such as worms, will be put through the rotting process and reduced to a stable condition not likely to cause offense when the sludge is dried on porous beds.

Table 51*a* gives analyses of dried sludges produced by all tank treatments at Worcester, Mass. Additional analyses of activated sludge produced at four periods during the investigations gave the following average percentages: Total nitrogen, 4.06; available nitrogen, 1.39; total phosphoric acid (P_2O_5), 2.26; citrate insoluble acid, 0.50; citrate soluble acid (available), 1.76.[1]

TABLE 51*a*.—PERCENTAGE COMPOSITION OF DRIED SLUDGES FROM DIFFIRENT METHODS OF SEWAGE TREATMENT AT WORCESTER, MASS.

	Chemical precipitation	Plain sedimentation	Septic tank	Imhoff tank	Secondary sedimentation tank[1]	Activated sludge treatment
Volatile solids	47.26	51.04	43.94	49.12	51.05	63.75
Fixed solids	52.74	48.96	56.06	50.88	48.95	36.25
Silica (SiO_2)	25.46	28.59	20.41			
Iron sulphide (FeS)	0.02	0.57	16.58			
Iron, not as sulphide	5.80	2.45	2.98			
Total iron	...	.....	.....	4.78	18.57	12.72
Sulphur, not as sulphide	0.44	0.60	0.64			
Aluminum oxide (Al_2O_3)	0.57	1.94	7.29			
Calcium oxide (CaO)	2.88	0.61	1.14			
Magnesium oxide (MgO)	0.74	0.29	0.97			
Phosphorus pentoxide (P_2O_5)	0.47	1.71	1.85	.....	.....	2.24
Carbon (C)	28.60	31.26	23.95			
Hydrogen (H)	4.21	4.46	3.64			
Nitrogen (N)	2.77	3.05	3.01	2.63	2.97	4.29
Fats	.....	.....	.....	.....	.....	2 to 9

[1]Treating the effluent from a trickling filter.

Removing Sludge from Tanks after Draining off the Sewage.—There are two distinct methods of removing sludge, depending upon whether the tank is thrown out of commission and the sewage withdrawn preparatory to removing the sludge or the latter is removed while the tank is in operation without withdrawing the supernatant liquid. The bottom of the tank must be designed to conform with the method of sludge removal adopted.

In case the supernatant liquid is first drawn off it is desirable to give the floor a slope which will permit it to be cleaned readily by squeegees and will cause the thin sludge to flow towards the

[1] R. S. Lanphear, *Eng. News-Rec.*, April 22, 1920.

drains without much assistance. It is often impracticable to provide such slopes in large tanks, however, for laborers are unable to move about readily on a sludge-covered slope steeper than 1 in 20. In such tanks a series of sumps with steep sides is necessary for the complete removal of sludge by gravity, and as such construction is expensive it is customary to give the bottom a slope as great as the conditions of construction and operation permit and rely upon hand cleaning for the complete removal of the sludge. In order to facilitate the work, pipes with hose connections may be run between the tanks, so that the sludge may be flushed along with the aid of water, or sewage from one tank can be used in cleaning adjoining tanks.

The cost of hand cleaning is low, amounting under pre-war conditions to 4.9 to 7.3 cts. per 1,000,000 gal. of sewage at Worcester, with labor at $2 per 8-hr. day. The lower figure is for chemical precipitation tanks and the higher for plain sedimentation tanks. A more important element than labor cost in hand cleaning is the length of time during which it is necessary to keep the basins out of service. For several hours before the cleaning is begun they must be cut out to permit the sedimentation of the solids in the supernatant sewage. After this, considerable time is required for drawing off the supernatant liquid before sludge removal is possible. At Worcester, Mass., about 10 per cent of the basin capacity is out of service all the time for cleaning. The offset to this disadvantage of intermittent removal is that the sludge is sometimes more dense than that obtained by continuous removal, and its subsequent handling may therefore be easier and less expensive under some conditions.

Labor-saving mechanical devices for removing sludge from basins after the sewage has been drained from them are used in England and on the Continent. The Ashton-Booth cleaner is a car running on a longitudinal track on the bottom of the tank and provided with a steel frame for holding a bulkhead of 1-in. planks with a rubber sealing piece along the bottom. After the tank has had its sewage drawn off, sewage from an adjoining tank is admitted behind the car until it attains sufficient height to shove the car along the track toward the lower end of the tank, pushing the sludge along in front of it. When the car reaches the lower end, it is moved back to its original position by admitting sewage against the lower face of the bulkhead. In Bremen a somewhat similar system of cars is used except that they are pulled along a track by rope and windlass at the lower end of the tank.

Sludge Removal without Interruption of Tank Service.—Where the topographic conditions permit, it is often desirable to draw off the sludge by gravity without interrupting the operation of the tanks. Fig. 182 illustrates settling tanks at North Attleboro, Mass., designed by Barbour for operation in this manner. Most vertical-flow sedimentation tanks are similarly

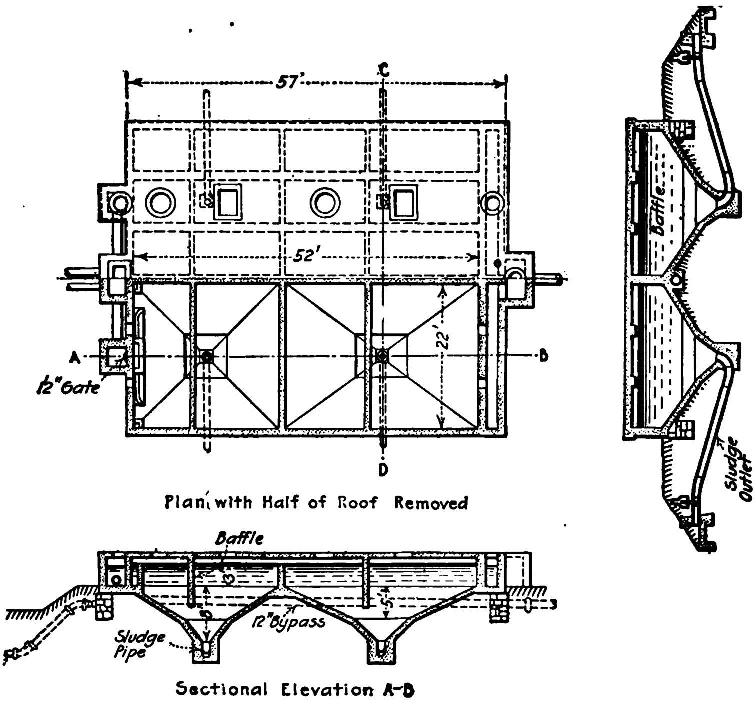


FIG. 182.—Settling tanks at North Attleboro, Mass.

operated. The Gloversville tanks illustrated in Fig. 163, for example, which yield a very heavy sludge on account of the large quantity of tannery wastes in the sewage, are operated continuously.

Some difficulty has been experienced, due to the gradual accumulation of solids in the tanks in spite of the frequent withdrawal of all sludge which would flow to the outlet. In one of these tanks the slope of the conical bottom was increased to about 60 deg. and the wooden distributing cone removed. These changes proved a material aid in getting the sludge out, but in spite of these changes it has been found necessary to draw the sludge once or twice a day, to avoid the accumulation of solids in this tank. In the other tank hosing and other means of breaking down the accumulation of heavy sludge are necessary.

Experience at many plants has shown that where gravity is the only force relied upon to make the sludge slip down slopes, the latter should be at least 2 on 1. Sludge which contains gelatinous floc, often called colloidal matter by plant operators, has a

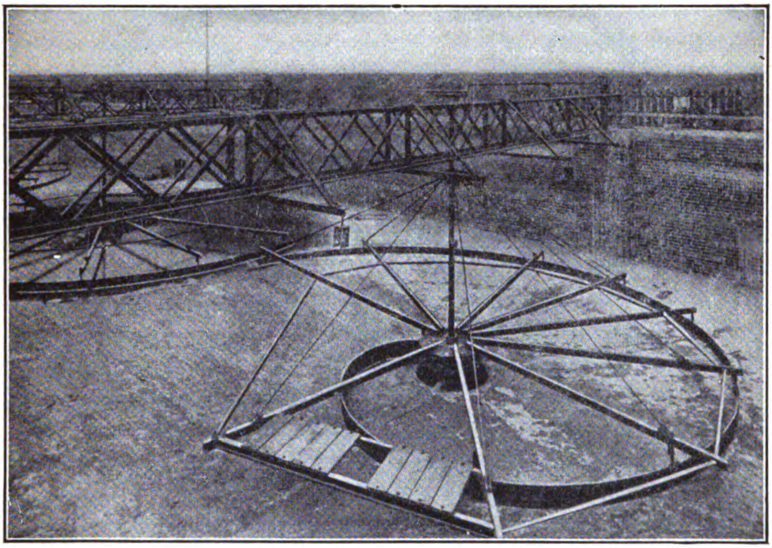


FIG. 183.—Fidler sludge remover, Bolton, England.

tendency at times to adhere to very smooth surfaces as well as those of a rougher character, and wherever two surfaces form an angle the sludge tends to remain. Slopes sufficient to cause sludge to flow freely after the supernatant sewage has been drawn off are not necessarily sufficient to cause it to flow when the sludge is submerged, for the effective weight of the latter in causing motion is reduced by the weight of the displaced sewage. As a result, in drawing off sludge from tanks operated continuously, there is a danger of forming a cone-shaped opening through the mass of sludge in the bottom of a tank and removing sewage as well as sludge, unless the bottom of the hopper is so formed by steep sides that the sludge is inevitably concentrated in it.

The construction of hopper-bottom tanks is often expensive and in order to avoid this expense, reduce labor charges and at the same time operate the tanks continuously, mechanical appliances are used to remove the sludge. The Fidler apparatus has been employed in a number of British cities for this purpose. It consists of a spiral band of sheet steel revolved by means of gears

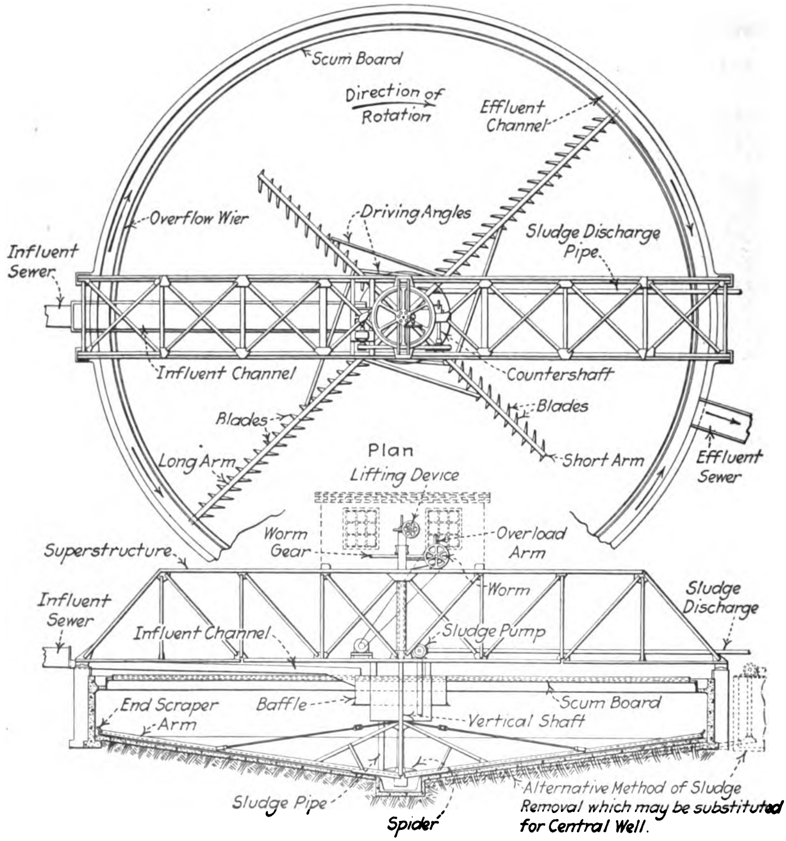


FIG. 184.—Dorr thickener or sewage clarifier.

which gradually draws the sludge toward an outlet under the center of the band, as shown in Fig. 183. Experience with this apparatus shows that it is highly desirable to construct it of materials which will resist the corrosive action of sewage.

The same problem has arisen in removing sludge formed in the cyanide metallurgical process and the Dorr thickener, employed in that process, has recently been adapted for removing the sludge from the settling tanks employed in the activated sludge plant at Milwaukee and at some industrial wastes treatment plants. The apparatus, Fig. 184, consists of radial arms at the bottom of a vertical shaft and carrying a number of plows The shaft is at the center of the tank and driven very slowly, so that the plows force the sludge gradually and without disturbance toward a sludge outlet at the center of the bottom of the tank, whence it may flow by gravity through a suitable outlet pipe or be pumped from a central cylinder carried by the revolving mechanism.

In case it is necessary to pump sludge the work can be done with various types of equipment. If the quantity is small a (rubber) diaphragm pump such as is used for removing water when excavating trenches in wet earth is often employed. Centrifugal pumps are often used in permanent installations, but care must be taken that their suctions are screened against the entrance of large objects. The air lift is employed successfully in pumping sludge from the Imhoff tanks at Fitchburg, Mass., and the Shone ejector has been used successfully for pumping sludge from sedimentation basins to sludge basins at Worcester, Mass. If sludge must be forced into filter presses for dewatering, stuff pumps, such as are used in handling pulp in paper mills, and ejectors give satisfactory service.

Disposal of Sludge.—The disposal of sludge is one of the most troublesome problems connected with the tank treatment of sewage. A method followed in some cities is to carry it to sea and dump it in deep water, Table 51*b*, and at London six steamers built for the purpose were regularly engaged in this method of disposal in 1903.

Most sludge is discharged from the tanks in which it settles to beds where it dries in the open air. Part of the water sinks into the soil and the rest evaporates. If the soil does not drain readily and the sludge is dense and greasy, the drying process may take some weeks. The beds are preferably porous and well under-drained with 4- to 6-in. open-joint tile. The effluent from the beds must be kept under observation, for it may be necessary to return it to the works for treatment as raw sewage if there is no opportunity for filtering or otherwise treating it independently of the sewage. The area required for the beds depends upon the character of the sludge to be treated and the climate in which they are located, a greater area being required where the

TABLE 51b.—COST OF DUMPING SLUDGE CONTAINING 90 PER CENT OF WATER AT SEA UNDER CONDITIONS EXISTING AT SEVERAL CITIES PRACTICING SUCH DISPOSAL

City and year	Tons per year	Moisture, per cent	Cost per ton		Miles to dump
			Dry solids	Sludge with 90 per cent moisture	
Dublin, 1906-7	128,300	90.0	$0.91	$0.09	10
Glasgow, 1 06-7	341,600	86.8	0.75	0.07	40
London, 1903-6	2,838,100	92.0	1.02	0.10	50
Manchester, 1903-5, 7	188,700	86.0	1.25	0.13	61
Salford, 1902-6	152,300	79.0	0.82	0.08	60
Southampton, 1906-7	95,600	90.0	3.04	0.30	25
Providence, R. I., 1910	....	72.4	....	0.059[1]	14

("American Sewerage Practice," vol. iii, p. 486).

[1] Cost per ton for disposal of pressed sludge cake, using a bottom-dump hopper barge.

rainfall is 48 in. annually than where it is 24 in. As a general rule, in the Northern part of this country, the range is from about 0.5 sq. ft. per person where quick-drying sludge from Imhoff tanks is to be treated to 7 sq. ft. where the voluminous sludge from chemical precipitation must be dried in this way. It is questionable if sludge from chemical precipitation or activated sludge tanks can be dried economically on open beds except in small plants or in very dry climates. Both classes must be spread in thin layers in order to dry and their volume is so great that a relatively large area of drying beds must be provided and the cost of their operation will be considerable.

Owing to the uncertainty about the effect of rainy weather on the drying of sludge in thin layers on porous beds, the method of lagooning has been employed in disposing of it in some cities. A lagoon is a tract of some size surrounded by a dike of earth, which is filled with sludge to a depth of 1 to 5 ft. The sludge is left to dry by evaporation and such small losses of water through the soil as may take place naturally. This process of drying may take from 2 to 6 months and is usually accompanied by offensive odors. When it is finished the sludge is wet and its disposal in this condition is not so easy as is that of a drier sludge.

A modification of the lagoon method of treatment is to run the sludge into a trench about 3 ft. wide and 18 in. deep and then cover it with earth. This method has been used in a number of English disposal works where it has been found to require from 1/4 to 1 acre of land per 1,000 long tons of sludge. As the land once used in this way cannot be employed again for the purpose for a long period, a very large tract must be available for trenching when sludge is disposed of in trenches.

Sludge from drying beds can be burned if mixed with coal or other fuel, but generally an attempt is made to get rid of it by dumping it on low land as a waste material without value. In a few places it is hauled away by farmers as a low-grade fertilizer, and probably a greater use of the material in this way would be advantageous if persistent endeavor were made to interest farmers in its value.

Dewatering Sludge Mechanically.—The first method of dewatering sewage sludge which was employed practically was filter pressing, used extensively in various industries. A filter press, Fig. 185, is a device by which the liquid sludge is forced under about 80 lb. per square inch pressure into cloth lined chambers in which the solids are retained between the cloths and the water is forced through the cloth and drainage ducts in the plates and escapes. When the chambers have become filled with solids the press is opened and the cakes of compressed material are readily removed because the cloth prevents the material from adhering to the plates.

In order to press sewage sludge it is often necessary to add lime to it; from 20 to 30 lb. per 1000 gal. are added in treating the sludge from the chemical precipitation process at Worcester, and 100 lb. were found necessary in preparing septic sludge for pressing.

The sludge press shown in Fig. 185 comprises a fixed head stock to which is attached the piping through which the sludge is pumped into the press, a fixed tail stock provided with the hand or power-driven apparatus for opening and closing the press, from 50 to 75 cast-iron plates, and a follower which is heavier than the plates and receives the pressure exerted by the operating apparatus carried by the tail stock. The plates and follower move along parallel steel side rods which also tie the head and tail stocks together. The plates used in pressing sludge are generally about 36 in. in diameter and 3 in. thick, with a 6-in. hole at the center. Tests at Providence show that the best cloth used with the plates, called "clothing," is 11-oz. army duck. The clothing is made by cutting strips a little more than twice the vertical length of a plate and then cutting two 6-in.

holes in each strip to correspond with the central hole in the plate. The cloth is hung over the top edge of the plate, and a clamp is put through the holes in the cloth and the opening in the plate and drawn tight, thus covering both faces of the plate with canvas.

The surfaces of the plates are corrugated and the liquid which gathers in the corrugations in the pressing process runs eventually

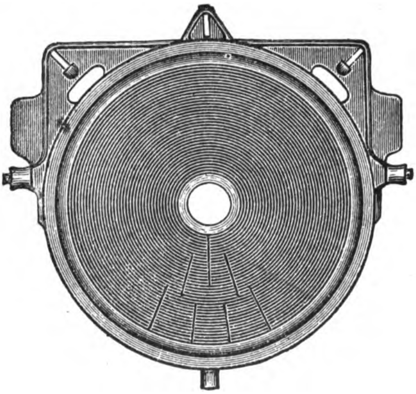


Plate used in filter press

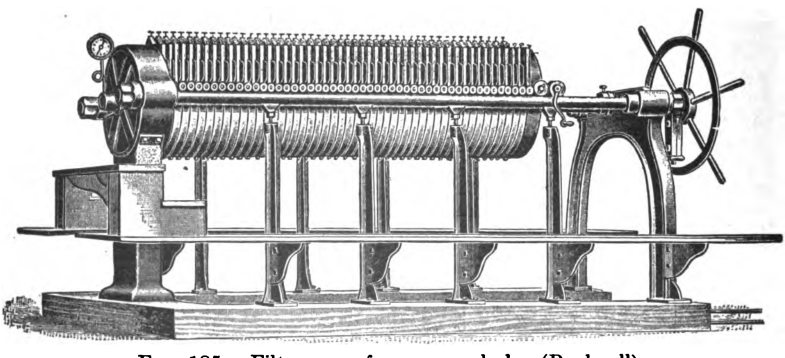


Fig. 185.—Filter press for sewage sludge (Bushnell).

into a drainage duct, usually a ½-in. hole in the bottom of the plate, through which it escapes to a drainage channel below the press. After the plates have been closed, they are forced together by the closing apparatus and the sludge is pumped into the press. The pressure under which it is pumped forces out a part of the water. After the presses are filled, the inlet valve is closed, the follower is pulled back, and the cakes of sludge are allowed to drop through hoppers into cars which remove the material from the press house. At Worcester the pressing process expels sufficient water to leave a sludge cake of 68 to 74 per cent moisture. The sludge cake is used for filling and a subtantial amount is hauled away by farmers for use as a fertilizer.

Dewatering may also be accomplished by centrifugal machines. The first practical sludge centrifuge was the Schaefer-ter-Meer machine, Fig. 186, used in several German cities. The sludge is admitted to the top from an elevated tank and passes down through a hollow shaft to the runner of the centrifuge. This runner has six radial cells, one radial side of each cell being formed of a sieve which permits the water to escape from the sludge while it is being whirled rapidly. The sludge is admitted to these cells from the hollow central shaft through openings which may be closed by gates, and it is thrown out of the cells through openings in the outer ends of the cells, which can also be closed by gates. While the cells are filling with sludge, the outer gates are shut and the inner set open. When the cells are full, the inner gates are closed and the machine kept whirling until the operator judges the dewatering has reached the desired point. The outer gates are then opened and the sludge is thrown out of the cells by centrifugal force against the walls of the chamber. The impact breaks it into pieces which fall through a hopper to a conveyor by which they are removed. The water is driven through the solids accumulated in each cell and is collected by a plate behind each sieve and guided to a vertical drain delivering it to an effluent trough. The gates are moved by oil-pressure and it is unnecessary to stop the runner in order to charge and unload the cells. The apparatus is reported by German engineers to furnish sludge with its moisture content reduced to 55 to 75 per cent.

A centrifuge developed from and said to be an improvement upon this Schaefer-ter-Meer machine has been used to some extent and is now (1921) being tried upon sludge from the activated sludge demonstration plant at Milwaukee. It has a drum or basket similar to that of the usual laundry centrifuge, but subdivided around its perimeter by radial vertical vanes projecting about 6 in. in from the drum wall. As sludge flows into the revolving drum, the solids are driven into the compartments, where a cake is gradually built up from the wall inward. In this case, none of the water is forced through the solids, but, on

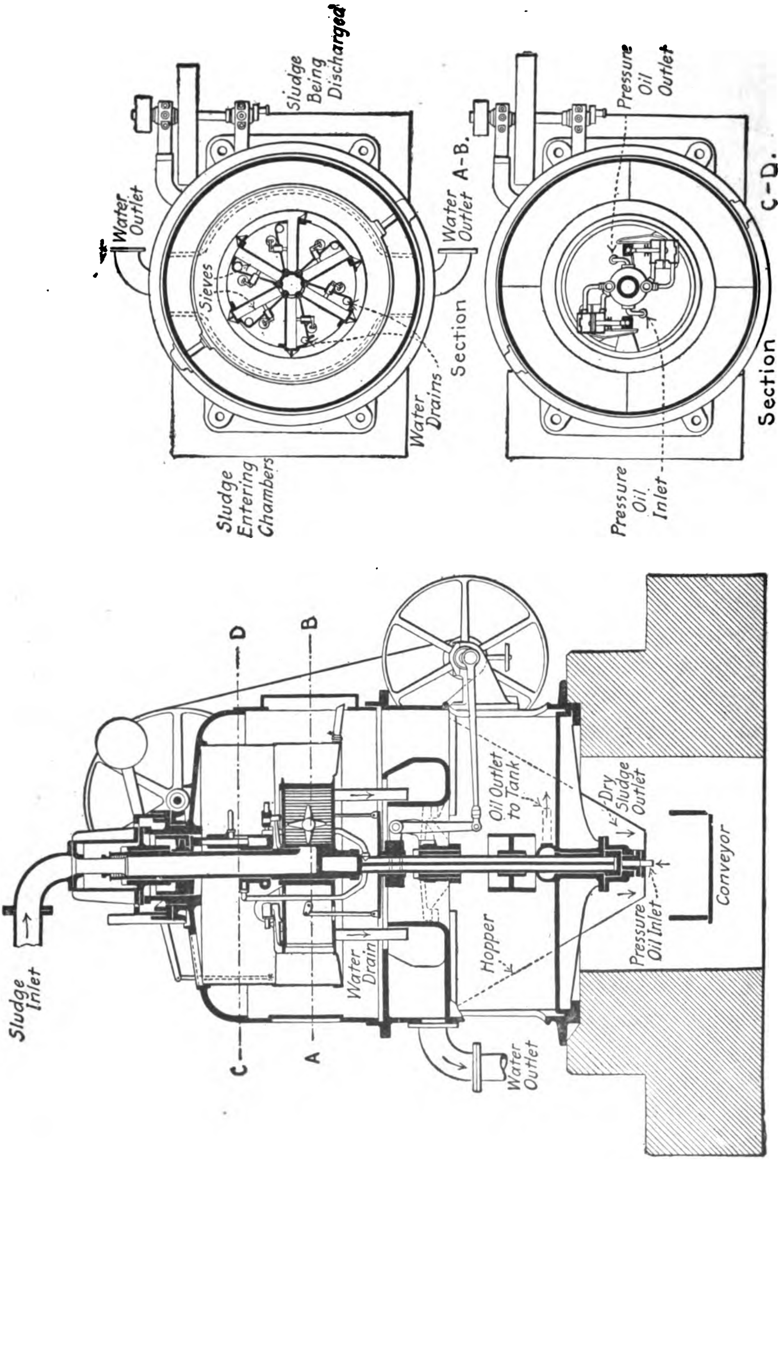


FIG. 186.—Centrifugal machine for drying sludge (Schaefer-ter-Meer).

the contrary, the solids are driven by centrifugal force outward against the drum wall leaving the water in the center to escape over the top of the drum. Like its predecessor, this centrifuge may be run continuously, the opening and closing of the valves being done automatically while the drum is revolving at full speed. Tests thus far made with the activated sludge show a tendency for the coarser and heavier solids to be formed into cake quickly and for the finer solids to be carried over with the effluent, The cake generally contains upwards of 80 per cent moisture, although a drier cake can be produced by prolonging the cycle and admitting the sludge slowly.

While numerous experiments in dewatering sludge have been conducted with centrifugal machines made in the United States for treating slimes and other industrial purposes, they have not been encouraging by comparison with the results obtainable with filter presses.

Drying Sludge by Heat.—If sludge is to be dried to serve as a fertilizer base or other purpose requiring a practically dry material, a rotary dryer such as is used in many industrial processes may be employed. At Milwaukee, where the dewatered sludge cake must be converted into fertilizer base, the preliminary experiments showed that in producing bone-dry material in such an apparatus from pressed acidified sludge containing 82 per cent moisture, no nitrogen was lost; in reducing pressed sludge containing 78 per cent moisture to material with 5.6 per cent moisture, the volume of the dried material was diminished to 18 per cent of the volume of the sludge put into the dryer. Where sludge cake is treated in this way provision must be made for preventing odors. At Milwaukee the gases from the experimental dryer passed through a dust chamber and were then washed with water by a spraying apparatus, after which they were discharged through a stack.

Fertilizing Value of Sludge.—The ingredients most needed in fertilizers are potash, phosphoric acid and nitrogen. Potash is usually obtained as muriates and sulphates, and all forms used in fertilizers are freely soluble. Phosphoric acid comes mainly from phosphates, in which it exists in combination with lime, iron or aluminum. It occurs in three forms: (1) soluble in water and readily taken up by plants; (2) slightly soluble but nevertheless readily assimilated, in which condition it is termed 'reverted;" (3) very slightly soluble and assimilated but slowly.

The first and second forms are collectively termed "available" phosphoric acid. The term "superphosphate" is applied commercially to any material containing soluble phosphoric acid as a chief constituent. Nitrogen is usually supplied as sulphate of ammonia or nitrate of soda.

These ingredients are dissolved so easily in the case of commercial fertilizers that they are not applied until just before they are needed. On the other hand, in the case of barnyard manure a portion of those elements useful to stimulate plant growth are so combined with other substances as to require breaking down into simpler and more easily soluble matter through bacterial action in order to render them available for plant food. Hence farmers sometimes apply manure to their fields in the autumn in order that its valuable ingredients may be made available and leached into the soil by the time they are needed in the spring. Sewage sludge is similar to barnyard manure in these respects.

Many practical tests of the fertilizing value of the sludge from plain and chemical precipitation tanks have been made in Great Britain. In a general way they indicate that such sludge is of little value on grass land or as a fertilizer for root crops of rapid growth, but it has increased the yield of wheat 10 to 12 per cent in some cases.

Investigations made by Lipman and Burgess at the University of California show that different soils have markedly different capacities for utilizing the nitrogen of sludge. Their experiments were made with nine sludges having the fertilizing values given in Table 52. The "availability" of the nitrogen, by which they consider such materials should be judged, was determined by a method originated by them. One gram of thoroughly dried, finely ground sludge was mixed with 100 g. of soil and enough water added to produce the best moisture conditions. The tumbler containing this sample was covered with a Petri dish cover and incubated for a month at 28 to 30°C. At the end of this period nitrate determinations were made on these soil cultures. Experiments were also made to determine the average "availability" of the nitrogen in several commercial organic fertilizers.

These investigators were much encouraged by the results of their tests. In the dry climate of parts of California it may be practicable to produce air-dried sludge more easily and certainly than where there is a greater rainfall, and in this way obtain at low cost a useful material for adding to the humus-poor soils of that State. An important fact connected with the results given in Table 52 is that the absolute amounts of nitrate produced from sludge nitrogen were often 50 to 75 per cent as high as those produced from similiar weights of dried blood and high-grade tankage.

TABLE 52.—PARTIAL COMPOSITION OF AIR-DRIED SLUDGES AND THEIR AVAILABLE NITROGEN COMPARED WITH THAT OF COMMERCIAL ORGANIC FERTILIZERS

(Experiments by C. B. Lipman and P. S. Burgess, University of California)

Material	Volatile matter, per cent	Ash, per cent	Total N., per cent	Nitrate N., per cent	Phosphoric acid, per cent	Percentage of N. in sludges and fertilizers available when inoculated with		
						Davis soil	Oakley soil	Anaheim soil
Sludge:								
Orange, Imhoff tank (city)	49.68	50.32	2.66	0.012	1.11	32.50	32.30	27.20
Fullerton Imhoff tank	25.31	74.69	1.23	0.045	0.86	43.90	43.50	40.60
Anaheim Imhoff tank	33.09	76.91	1.54	0.115	0.99	32.40	36.00	40.20
Lindsay septic tank	42.92	57.08	1.83	0.090	0.89	28.70	18.00	18.80
Pasadena Imhoff tank	29.34	70.76	1.68	0.135	1.46	38.00	28.20	35.70
Orange Imhoff tank (county)	38.41	61.59	2.38	0.060	0.77	25.70	21.40	15.70
Worcester exp. Imhoff tank	43.86	56.14	2.10	0.010	1.82	26.90	12.40	34.50
Cleveland exp. Imhoff tank	36.37	63.63	1.44	0.000	1.28	32.90	8.30	44.10
Chicago Stock Yards exp. Imhoff tank.	50.46	49.54	1.73	0.400	1.46	24.50	9.80	10.10
Fertilizers:								
Dried blood	......	....	......	....	......	12.79	0.00	4.05
High-grade tankage	......	....	......	....	......	16.21	0.00	3.95
Low-grade tankage	......	....	......	....	......	27.39	22.70	43.89
Fish guano	......	....	......	....	......	15.11	trace	4.65
Cottonseed meal	......	....	......	....	......	14.18	2.00	21.45
Goat manure	......	....	......	....	......	4.89	3.50	10.39

A possible explanation of the failure attending the use of sewage sludge as a fertilizer in some places may be that it was applied too late for its useful constituents to have time to leach out, or the soil may not have been adapted to receive such sludge. Among sewage sludges that from tanks used to settle trickling filter effluent, after drying, appears to be a relatively good fertilizer. Activated sludge has given some surprisingly good results, as at Toronto,[1] where root crops gave yields with

[1] *Engineering & Contracting*, July 10, 1918.

it from 40 to 55 per cent larger than those obtained with barnyard manure.

Many attempts have been made to sell dried sludge as a fertilizer in Europe, probably the most successful being that of Glasgow, Scotland, where all the sludge produced at the Dalmarnock works is sold in the form of pressed cake or dried fertilizer. This commercial success was ascribed to careful business organization and judicious advertising by the Royal Commission on Sewage Disposal, which estimated that the sales of sludge decreased the net cost of sewage treatment about 80 cts. per 1,000,000 gal. Investigations of the practicability of drying sludge to form a fertilizer base have generally been discouraging in the United States. Since the advent of the activated sludge process there has been a somewhat brighter prospect for sludge utilization at large treatment plants.

The advisability of preparing sludge for the market will ordinarily depend upon whether or not the fertilizer and grease which can be recovered have sufficient value to render this complicated method of sludge disposal materially less expensive than other simple methods which may be available and suitable for the local conditions. Before the sludge can be put on the market its water content must be reduced to about 10 per cent, which is considered commercially dry. This dewatering can be done by combinations of the methods already explained. The authors have estimated[1] that activated sludge can be pressed and drum-dried to 10 per cent moisture for about $4.90 per 1,000,000 gal. of sewage treated, assuming that each 1,000,000 gal. of sewage will produce 4500 gal. of activated sludge containing 97.5 per cent water. To this must be added the cost of marketing, handling and transporting the final product.

The theoretical fertilizer value of commercially dried activated sludge containing 3 per cent available nitrogen and 3.5 per cent phosphoric acid (P_2O_5) was estimated at $10.30 per ton, or $5.47 per 1,000,000 gal. of sewage treated, assuming that each 1,000,000 gal. of sewage would produce 1,063 lb. of fertilizer. The volume of activated sludge produced and weight of dried material per unit volume of sewage will vary at different places according to the composition of the sewage. The composition of the dried material will also vary at different places.

If grease is present in such proportion as to be seriously detri-

[1] "American Sewerage Practice," vol. iii, 2nd ed., p. 825 *et seq.* (1916).

mental to the fertilizer, it will be necessary to degrease the dried sludge as provided for in the Miles acid process of sewage treatment. The best way to degrease the dried sludge appears to be by naphtha percolation, a process somewhat dangerous and not likely to prove advisable unless the sludge contains a relatively large proportion of grease.

The cost of degreasing Milwaukee dried activated sludge was estimated to be about 90 cts. per 1,000,000 gal. of sewage treated, and after degreasing this material will probably have to be redried at an estimated cost of $1.70 per 1,000,000 gal. of sewage. This makes an additional cost of $2.60 per 1,000,000 gal., for the recovery of grease. It was estimated that the value of the grease in activated sludge containing 5 per cent of grease in the dry material, would be about $4.50 per ton of fertilizer, or about $2.39 per 1,000,000 gal. of sewage. This proportion of grease would not be sufficient to warrant its recovery. It is not so high as to seriously affect the value of the sludge for fertilizer.

From what has been said it will be seen that the preparation of fertilizer from activated sludge is a manufacturing process having a number of stages which can probably be conducted profitably only on a large scale. In addition to this production aspect of the work, there is a business side involving the sale of the product at the highest possible price and the management of the entire enterprise in an efficient yet economical manner. Whether it is desirable for a municipality to embark upon such a business is a debatable question and will depend upon the local conditions. Unless the enterprise is well handled there are possibilities of grave financial troubles. The overhead expense is likely in any case to be large in comparison with many municipal enterprises, and the proportionate cost of operating a small sludge fertilizer plant will probably be much greater than that with large plants.

CHAPTER XVI

CONTACT BEDS

Prior to the development of the activated sludge process, tank treatment of sewage did not remove fine suspended and colloidal matter and it was necessary to treat tank effluents in order to oxidize their organic matter before their discharge into some waters. The purpose of these treatments was essentially to transform the organic matter in the tank effluents into substances possessing so little demand for oxygen that they would not exhaust the supply of that element in the waters receiving the effluents and produce putrefactive conditions. Perhaps the

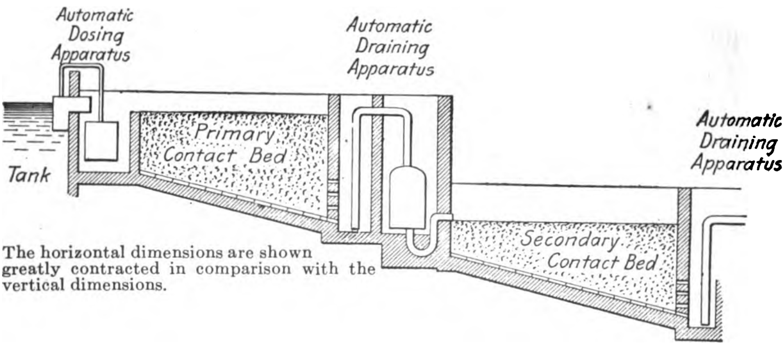


FIG. 187.—Arrangement of double contact beds.

most logical step forward from the types of tank treatment in use prior to about 1913 was treatment in the contact or bacteria bed.

The contact bed is a watertight tank, generally uncovered, filled with broken stone, cinders or other inert material, commonly ½ to 1½ in. in size and 4 to 5 ft. deep. This material is often termed "ballast." When new the bed will have from 40 to 50 per cent of voids, but these gradually become filled with sewage solids and the contact material has to be removed, cleaned and replaced after a service of perhaps 5 years. Such beds are often built in series, as shown in Fig. 187; the first bed to receive the sewage is termed the "primary" bed, and its effluent goes to the "secondary" bed; sometimes the effluent from the secondary bed is treated in a "tertiary" bed.

The contact bed is operated by filling it with sewage, allowing it to stand full for a time, then drawing off the sewage and finally allowing it to rest empty. After resting, the cycle is repeated. The changes that take place in the sewage during the cycle are physical, biological and chemical.

Although the sewage applied to a contact bed has generally been settled, it retains a part of its settling solids and much of its suspended and colloidal matter. The settling solids are largely retained in the interstices of the ballast and the colloids are withdrawn from the sewage by the physical attraction of the contact material. Certain bacteria develop in the organic matter adhering to the ballast and the pieces of stone or clinker become covered with a gelatinous film. The organisms in this film play a large part in converting the organic matter of the applied sewage into more stable organic and mineral substances by oxidation processes.

In the contact bed operating successfully, the organic nitrogen compounds are transformed, with the production of free ammonia, the free ammonia is transformed into nitrites and nitrates, and nitrogen gas is evolved. There is much uncertainty about the processes which take place, but apparently the nitrates are formed mainly while the bed is standing empty and are partly or wholly reduced during the standing-full period, so that the effluent has a relatively low nitrate content. While there is probably some mechanical straining out of suspended and colloidal matter, this mechanical action accounts for but a part of the removal effected in a contact bed, for a matured unclogged bed will withdraw more suspended matter from sewage than a bed not matured but otherwise the same.

The contact bed requires relatively less head for operation than the trickling filter does, which is a decided advantage where the available head is sufficient for a contact bed, but pumping is necessary for a trickling filter. Further, with contact beds there is a comparative absence of the objectionable odors and the moth flies more or less prevalent about trickling filters.

Filling and Draining.—As these two operations have no biological significance, it is generally held that the more quickly they take place the better, so long as the movement of the liquid is not sufficiently rapid to disturb the materials in the bed or the film adhering to them.

Standing Full.—This period should not be so long that anaerobic decomposition takes places to an appreciable extent. In some cases Clark and Gage found[1] no marked difference when the period of contact varied from 0 to 5 hr., but, on the whole, periods exceeding 2 hr. gave inferior effluents. Johnson found that at Columbus even 1 hr. was too long a period to give the best results. The Royal Commission on Sewage Disposal stated in its Fifth Report that 2 hr. contact and 4 hr. rest generally gave the best results in the practical operation of contact beds, but added that no rule of general applicability could be laid down.

Standing Empty.—It is generally considered that the most important biological purification of the organic matter takes place while the bed is standing empty, and that this period should be made as long as possible, taking into account the necessary time for filling, standing full, and draining.

It is difficult to determine the proper number of fillings per day in advance of actual experience with a plant, for this depends upon the quality and strength of the sewage, the quantity and character of its suspended matter, the size and character of the ballast, the extent to which the ballast is clogged, and the character of the effluent desired. The stronger the sewage and the greater the quantity of suspended matter in it, the smaller will be the quantity which the contact bed can convert into an effluent of good quality. A fine contact material cannot treat as great a quantity as can material of medium or coarse size, although for a time the quality of the effluent produced by the first may be decidedly better. Clogging the interstices of the ballast results in a loss of capacity, for it becomes necessary to operate the bed with small quantities of sewage and allow long periods of rest. As before stated, British experience has indicated that 4 hr. rest is enough for average conditions, but this statement applies only to matured beds and does not take into account any periods of complete rest for a day or more when the ballast has become unusually clogged.

Maturing of Beds.—In starting contact beds with either new or washed ballast, it is necessary to allow the sewage a long period of contact in order to obtain a non-putrefactive effluent. At Manchester, England, where the largest installation of contact beds has been made, it was found advisable to employ a 12-hr.

[1] *Report* Mass. State Board Health, 1908, p. 445.

contact period with beds having clean ballast. This period was gradually reduced as the beds matured until, after a service of several years, the total time for filling, standing full and draining did not exceed 2 hr. This slow maturing of the bed seems to be necessary in order that the pieces of the ballast may become coated with the gelatinous film which plays such an important role in the action of a contact bed and to afford an opportunity for the proper biological conditions to become established before the beds are worked to their highest capacity.

Loss of Capacity.—As soon as a contact bed is put in operation it begins to lose capacity. This may continue until the voids are completely clogged and the bed is inoperative, or it may stop after reaching a matured condition, depending upon circumstances of design and operation. The position of the clogging material within a bed sometimes varies from year to year; in some beds it will be near the bottom and in others near the top. The Royal Commission on Sewage Disposal found that there were eight causes of clogging, as follows:

1. When beds are constructed of friable material, it may disintegrate, so that the voids will become partly filled with small pieces of the ballast.

2. The ballast may become more consolidated than when it was placed in the bed, particularly if the material disintegrates to any extent. Consolidation apparently is more serious in beds of fine material than in those of material of medium or coarse size.

3. Deposition of colloidal matter.

4. The growth of organisms is at once the cause of increased efficiency in the bed and of loss of capacity, according to Dr. Fowler. By working beds at high speed, without long periods of rest, the effluent may remain good, but the bacterial growth increases so rapidly that the bed becomes too spongy and will not allow the liquid to drain away freely. When this condition is observed a rest of 1 to 2 weeks should be given the bed, Fowler stated.

5. The volume of liquid passed on to the bed.

6. Insufficient rest.

7. Inefficient drainage.

8. Where the greater part of the suspended matter in the incoming sewage finds its way into the body of the ballast, as is the case with coarse material, the Royal Commission on Sewage Disposal found that there was a tendency for the loss of capacity

to vary in direct proportion to the quantity of suspended solids in the applied sewage. But if the bed is composed of very fine material or a bed of material of medium size is covered with very fine material, a large proportion of the suspended solids are caught on the surface. In a few American contact beds, clogging of the ballast by suspended solids has been reduced by replacing from 6 to 12 in. depth of ballast at one corner of the bed with fine cinders and constructing a bank of the same cinders about 1 ft. high around this small area. The sewage is applied at this corner and must filter through the cinders before entering the mass of the ballast. The clogged surface cinders may be raked off from time to time.

Multiple Contact.—Under some conditions a single contact has not furnished a satisfactory effluent and another contact has been given it in order to obtain an effluent of the desired character. Investigations have generally shown that double contact will permit a somewhat greater quantity of sewage to be treated per acre of the entire area with satisfactory results than is possible with single contact beds of the same area. Triple contact generally requires so much head for operation that it is financially impracticable.

Kinds of Contact Material.—Beds have been constructed of various kinds of contact material, cinders, slag, coke, broken stone, pebbles, broken brick, slate and saggars or waste from potteries. Experience has shown that the contact material should be strong enough to withstand the strains developing within a bed without disintegrating. Cinders are generally easily broken into fine pieces. Clinker is usually hard enough to withstand such action. Coke is so light that the rising of the sewage in a bed during the filling period and the reverse action during this drainage period tend to displace it and this movement aids disintegration. The rough surface of all such materials is helpful in holding the gelatinous film which is the seat of biological activity in a bed of this type. On the other hand, friable materials may prove so much cheaper than those more durable that economy will dictate their use, with enlarged area to offset reduced unit capacity and with provision for frequent washing and replenishing of the quantity of material lost at each washing.

English experience with the various kinds of contact material has been summarized by the Royal Commission as follows:

"The experience at Manchester, with regard to contact beds, has been that hard furnace clinker is the best material, being vesicular and yet permanent. Dr. Fowler, however, expressed the opinion that broken Staffordshire bricks, saggers and gravel, if properly sized, should do equally well.

"It will be seen from the evidence that many witnesses have stated that substances like gravel, flints, or broken granite, are suitable as a filtering medium, but that, in most cases, they have expressed a decided preference for the use of those materials which present the roughest and most irregular surfaces, such as clinker, coke and saggers.

"So far as our own experience goes, we should prefer, as a general rule, to use coke or good clinker in a contact bed in preference to other materials.[1] It is true that there is some evidence to the effect that rather better effluents can be obtained from contact beds of coke than from contact beds of clinker, but we think that this advantage is balanced by the tendency of the lighter material, coke, to shift slightly every time the bed is filled, and therefore to be more liable to disintegration." (Fifth Report, p. 66.)

Johnson found at Columbus, Ohio, in 1905 but little choice between broken stone and coke. There is some evidence that hard, smooth material offers less resistance to the passage of suspended solids into the lower portions of the bed, where it is undesirable that there should be an undue accumulation of organic matter.

Size of Contact Material.—The Royal Commision on Sewage Disposal has summed up[2] the British experience as follows:

"It is important that the material in a contact bed should not be too small, especially if the liquid to be treated contains an appreciable quantity of suspended matter, as some of this suspended matter will undoubtedly find its way into the interstitial spaces and prevent proper drainage of the bed.

"In deciding upon the size of the material to be used, the amount of suspended matter in the liquid to be treated must be considered. As a general rule, the greater the amount of suspended matter in the liquid the larger the material should be.

"With a crude sewage containing 40 parts per 100,000 of suspended matter, the material will probably have to be 3 in. and upward in diameter, and even then sludge will accumulate on the top.

"With septic tank liquor containing 8 to 10 parts per 100,000 of suspended matter, material of a diameter from 3/8 to 5/8 in. may probably be used effectively; while with a good precipitation liquor containing from 1 to 3 parts of suspended matter the best results will probably be obtained with material as fine as 1/4 in. diameter.

"It is, however, impossible to make any but the most general statement as to the most suitable size of material for contact beds, as, in some cases,

[1] "Broken 'saggers' might be equally good, but we have had no experience of this material in contact beds."

[2] *Fifth Report*, p. 67.

there may be special circumstances which affect the question, such as the character of the suspended matter or the smoothness of the filtering material."

Another point to be considered in connection with the size of the contact material is the effect of the size on the clogging of the bed. Beds constructed of materials of the sizes recommended by the Royal Commission have often become so clogged that these materials had to be removed, washed and replaced after 3 to 5 years' use. This is an expensive operation. There is some evidence that if contact material not smaller than 1 in. in size is used and the management of the beds is good that serious clogging of the beds can be avoided, but it is questionable if the average results obtained with such coarse beds will be as good as those obtained with the finer beds, unless a much greater quantity of contact material is used so as to obtain adequate area of gelatinous surface to effect the desired changes in the applied liquid. Furthermore, with such coarse beds, the underdrainage must receive greater attention than is necessary with beds of finer material and may require more expensive construction, and the effluent may contain such a large quantity of suspended organic matter at times that settling basins should be provided for its removal, as is the case with the effluents from trickling filters.

Depth and Drainage of Contact Beds.—The investigations of British contact beds by the Royal Commission on Sewage Disposal and the experiments of the Lawrence Experiment Station have shown that the depth of contact beds has very little effect on their activity when they are in good operating condition. Other conditions governing the depth are the drainage and the maintenance of beds in good condition.

Ordinarily these beds have pipe underdrains laid in large stones, making an underdrainage system at least 6 in. deep. The sewage which remains in this underdrainage system is not so well treated as that in the main body of the bed, where the smaller size of the stones affords a much greater area of gelatinous film per cubic yard of material. There is little to be said in favor of such underdrainage, and better drainage can be undoubtedly afforded by a floor system similar to those now used with trickling filters. Experience with British contact beds, particularly those at Manchester, has shown that loss of capacity was often due more to imperfect drainage than to an accumulation of solids.

The objectionable feature of such a floor is its absence of purifying influence on the sewage in it, but this can be remedied by filling the spaces in the floor with effluent from an adjacent bed before dosing the bed. In any case, it seems undesirable to have this least efficient part of the bed more than one-fifth of the whole depth, and for this reason the Royal Commission suggested 2½ ft. as the minimum allowable depth.

Deep beds place a considerable weight on the material in the lower part, which may lead to its disintegration. Furthermore, when washing becomes necessary, it is much more difficult to remove and handle the material from deep than from shallow beds and the danger of breaking it up is increased. For this reason the Royal Commission suggested that 6 ft. was probably the maximum limit of depth.

Size of Beds.—The size of the units is more important with contact beds than with most other methods of treatment. The unit should be small enough so that under normal conditions of sewage flow the time required for filling shall not be unduly long. Large units are also inadvisable on account of the long time required for drainage. At Manchester, England, with a sewage flow of about 36,000,000 gal. daily, the individual beds were made ½ acre in area. At Plainfield, N. J., and Mansfield, Ohio, the individual units were made 0.22 and 0.25 acre respectively.

Construction.—Some of the first British contact beds were constructed merely in excavations in practically impervious earth, the bottoms and sides being unlined. Experience with them showed a general tendency of earth to work into the contact material and for the underdrains to become distorted by unequal settlement. Where earth embankments were used to separate beds the leakage through them was occasionally so great that adjoining beds had to be filled simultaneously. On the other hand, contact beds at Mansfield, Ohio, constructed with an earth bottom into which hard cinders were rolled to secure as great density as possible and with earth embankments, which went into service in 1902 were found in 1914 to have a total leakage of 5 per cent, mostly through a revolving valve controlling the discharge of the underdrains. In the United States concrete has, however, generally been employed for the walls and bottom of contact beds.

Fig. 188 shows the general arrangement of the underdrainage and distribution systems of a typical bed used at Manchester, England. The bottoms of the beds are concrete, in which the

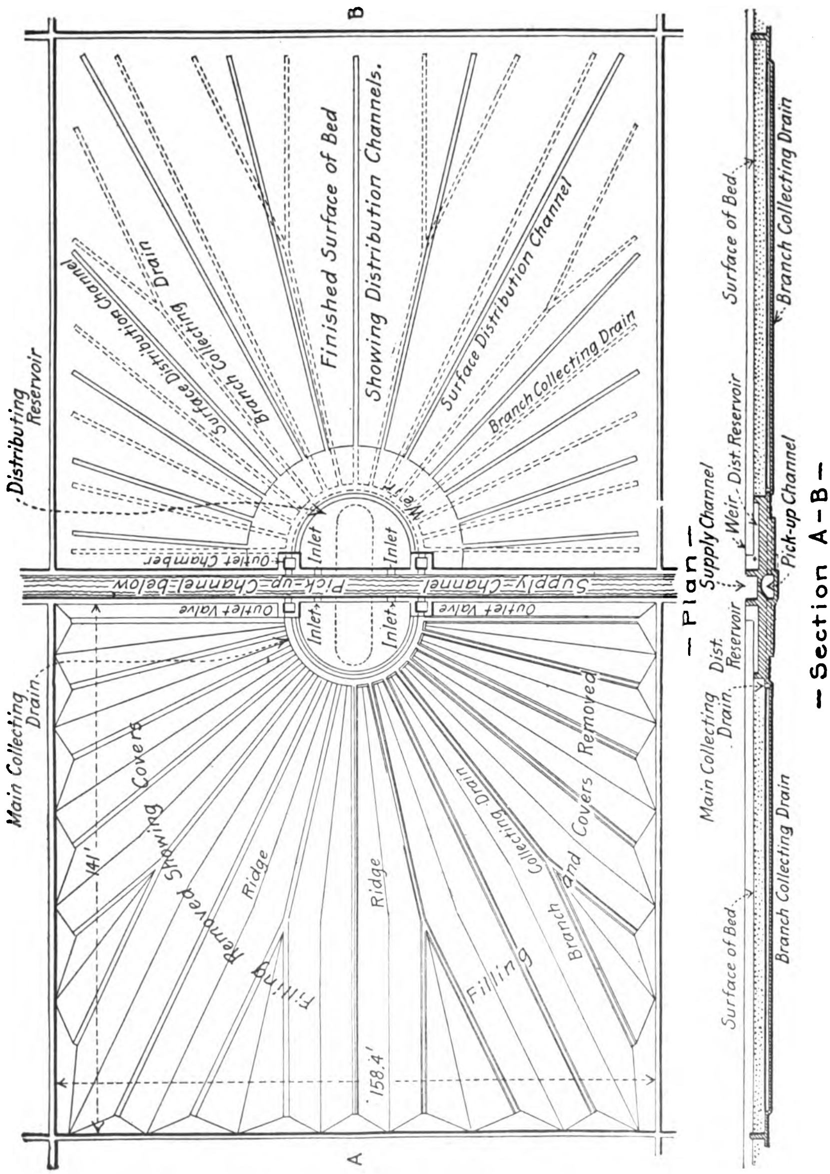


FIG. 188.—Plan of two half-acre contact beds, Manchester, England.

underdrain channels are formed. These channels are covered with perforated stoneware slabs laid flush with the surface of the bottom. The bottom was graded in ridges with flat slopes to the underdrains. The sewage from the supply channel flows into a small distributing reservoir, from which it passes over a semicircular weir into channels in the surface of the bed. The channels are lined with fine material which tends to arrest suspended matter and keep it from entering the bed. The material in the beds is washed and screened at intervals of 4 to 5.4 years; the portion from ⅛ to ¼ in. in size is used for surfacing secondary contact beds and the portion from ¼ to 2 in. in size is used in primary beds. The beds receiving sewage are 3 ft. 4 in. deep and those receiving storm water are 2½ ft. deep.

The beds at Mansfield, Ohio, designed by Barbour, are laid out in the form of a circle, each of the ¼-acre beds forming a sector separated from the next by earth embankments. The beds were underdrained by open-joint tile laid in depressions about 6 in. deep and covered to a depth of 4 to 9 in. with cinders from ½ to 1½ in. in size. The contact material was hard clinker crushed and screened at the plant to ¼ to ¾ in. in size; the total depth of the contact material was 4 ft. 9 in. Septic sewage was discharged by automatic dosing apparatus into an 18-in. iron pipe terminating in a concrete culvert at the apex of each filter. The liquid was distributed over the surface by wooden carriers 6 in. deep and from 3 ft. 4 in. wide where the sewage was received to 6 in. wide at the ends of the branches.

The largest American installation of contact beds was at Plainfield, N. J., where four pairs of primary and secondary beds were constructed in 1901 and four more in 1905. This plant was superseded in 1916 by a trickling filter installation. The underdrainage was provided by 5-in. horseshoe tile radiating from the gate chambers in the center of each set of four filters, and coarse stone spread between and over them to a depth of 6 in. This drainage system proved inadequate according to Lanphear,[1] who operated the plant for some time. It was found when beds were opened in 1909–1910 that the lower portion of the material had become a wet, black mass resembling mud. The contact material was 4 ft. deep. That originally used in the first set of primary beds was broken trap rock ¼ to 1½ in. in size and somewhat smaller material in the secondary beds, slag

[1] *Eng. Rec.*, Aug. 10, 1912.

being used in some and cinders in the others. In the beds constructed in 1905 cinders ½ to 1½ in. in size were used in both the primary and secondary beds. Lanphear recommended the use of trap rock 1 to 2 in. in size for primary beds under the local conditions, for finer material became too clogged. While the secondary filters were considerably clogged after a service of 7 and 12 years, they held one-half to two-thirds of their original capacity and about two and one-half times as much as the primary filters. Until 1909 all beds were filled from the top; in 1910 the primary filters were filled from below in order to reduce the odor from the exposed septic tank effluent which was applied. Experience showed that where bottom filling is practiced larger stone and better underdrainage are necessary than characterized the plant at that time.

Automatic Control.—The four stages of an operation cycle in the contact treatment can be controlled manually or by means of automatic devices specially designed for the purpose. In considering the advisability of automatic control, it must be remembered that both the quantity and quality of sewage vary greatly from hour to hour and from day to day, and that the capacity and efficiency of the beds change from time to time. In a small plant, with from three to five beds, it is possible that the same bed may receive day after day the strong sewage of the day time, while the other beds, receiving the weaker sewage, are called upon to do less work. In most cases, however, the fluctuations in flow from day to day will cause more or less change in the hourly cycle, and under such conditions the use of automatic apparatus for dosing will probably insure better work on the part of the beds and less expense for caretakers than the operation of the controlling gates by manual labor. In large plants the complication of the apparatus is so great and the beds vary so much in their capabilities that it is probable that better results can be obtained by the intelligent operation of gates by manual labor than by automatic dosing apparatus.

For automatic control, two sets of apparatus are required, one for filling the bed with sewage and the other for emptying the bed after a definite period of contact. Various types of apparatus have been used for the purpose[1] and among these the air-lock feed has been found particularly satisfactory where only a limited head is available. For discharging sewage from the bed after contact, timed siphons have been extensively used.

[1] "American Sewerage Practice, vol. iii, p. 714."

Air-lock Feeds.—Fig. 189 shows in outline the general features of an installation of air-lock feeds for four contact beds.

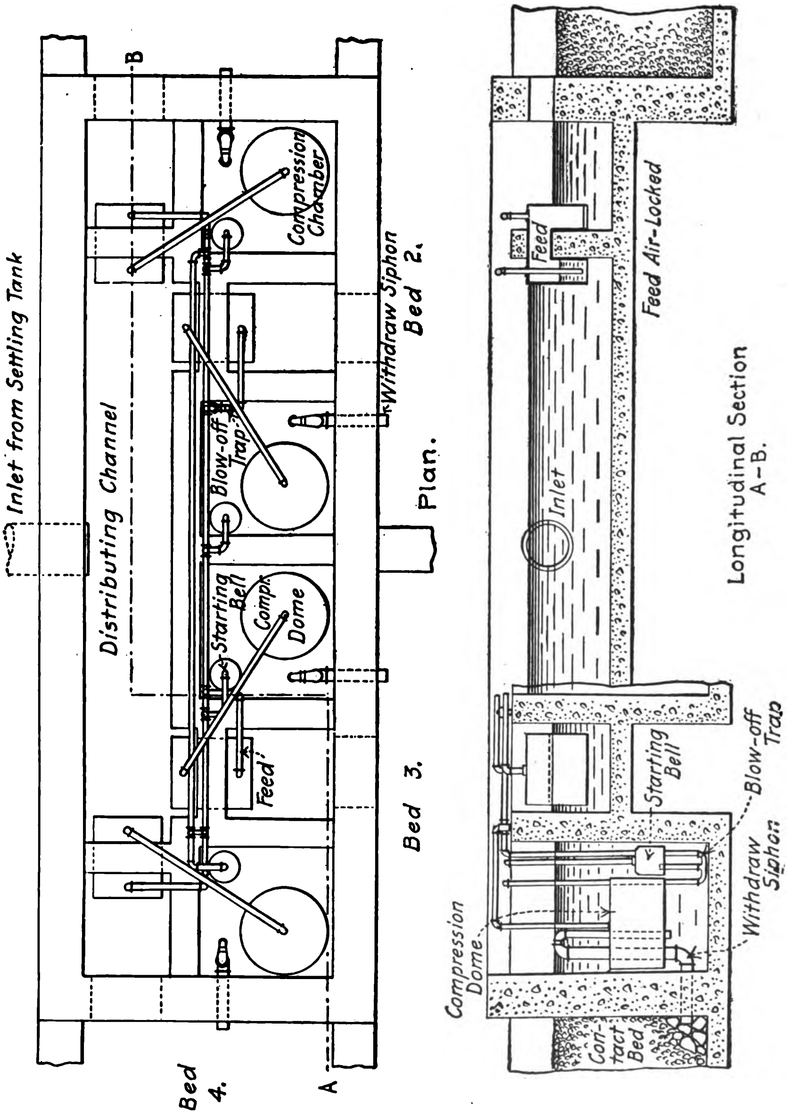


FIG. 189.—Air-lock feeds for contact beds (Miller-Adams).

At the beginning of a cycle of operation the wells in front of the air-lock feeds are filled with water, except that which is to operate first. All of the blowoff traps are filled with water.

Sewage entering the channel from the settling tanks flows through the feed (the one not sealed with water) into the bed until it is filled, or until the sewage level reaches the top of the withdraw siphon. A slight additional rise in water level causes the withdraw siphon to come into operation, and the compression chamber is filled through the withdraw siphon until the sewage level inside is the same as in the bed. As the compression chamber fills, the air in the compression dome is put under pressure and forced into the upper part of the feed, gradually displacing the sewage flowing through the feed until the sewage level is forced down below the inside crest of the feed, when the flow through the feed ceases and the feed is air-locked.

The same rise of sewage in the compression chamber also produces a pressure in the starting bell, which is transmitted to the blowoff trap of the feed next to operate. Just before feed (1) is air-locked the seal in blowoff trap (2) is forced, thus releasing the air confined in feed (2), and allowing the sewage to discharge into contact bed (2). This prevents any backing up in the distributing channel or settling tanks.

After standing for the required time, the sewage in the bed is discharged by a timed siphon, described later. As the sewage level in the bed falls, siphonic action is started in the withdraw siphon and the compression chamber is drained. By this means the compression dome and starting bell are vented, the blowoff trap is filled and the chamber is ready for the next cycle of operation.

Timed Siphons.—These siphons, one for each contact bed, are for the purpose of automatically controlling the time the sewage stands in the bed. The general details of the apparatus are shown in Fig. 190.

At the start, the main trap, the blowoff trap and tile well in the timing chamber are filled with water. The size of the timing chamber and the size of the opening in the timing valve determine the period of contact and require trial and adjustment to obtain the specified period of contact. The timed siphon is controlled by the starting bell in the timing chamber, and until there is sufficient pressure in the starting bell to force the seal of the blowoff trap, the siphon will not discharge and the bed will stand full. The timing valve is located below the full water level in the contact bed, so that when the bed is full there is a continuous discharge through the timing valve into the timing chamber. The siphon receives air through the vent when the sewage has been drawn down to the low level and the discharge then ceases. When the timed siphon is operating, the draining siphon is discharging the sewage in the timing chamber, and at the end of the discharge the starting bell is vented, the timing chamber emptied and the apparatus is ready for the next filling of the bed.

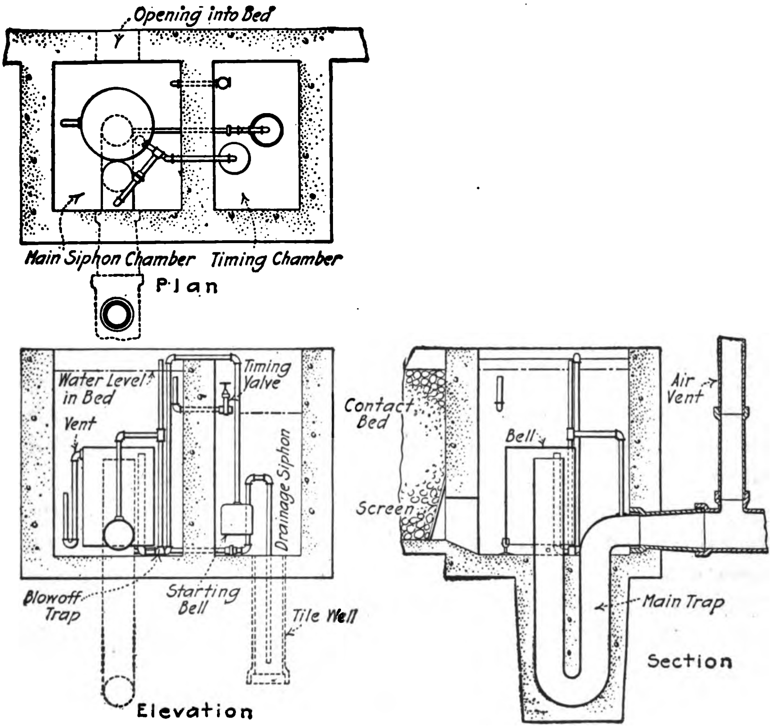


Fig. 190.—A timed siphon (Miller).

The total head required to operate contact beds equipped with air-lock feed and timed siphon ranges from about 12 in. for the smallest installations to 21 in. for the largest.

Operating Results.—If a 1-acre bed 5 ft. in depth is composed of contact material having 30 per cent of voids, this percentage being usually substantially exceeded when the bed is new, one dose would consist of 488,700 gal. of sewage, and the rate of operation would depend upon the number of fillings per day. It is difficult to determine the proper number of fillings per day, as this depends upon the quality and strength of the sewage, the quantity and character of the applied suspended matter, the size and character of the contact material, the extent to which the material is clogged, and the character of the effluent to be produced. The smaller sewage treatment plants in the United States have rarely been looked after as their designers intended; they have been continued in service after their capacity was greatly exceeded, and the information to be obtained from their operating history is usually incomplete in consequence. For example, the five 0.25-acre contact beds, 4 ft. 9 in. deep, at Mansfield, Ohio, were put in service in 1902 when the sewage was weak and amounted to 765,000 gal. per day. It was found by the designer in 1914, 12 years later, that the plant was receiving 1,899,000 gal. of much stronger sewage daily. The automatic apparatus governing the flow from the septic tanks had not been looked after properly, at least during the later years, and the filling period of the beds was found to range from 3½ to 8¼ hr. But in spite of these conditions the beds were not clogged with mud at the bottom, a frequently observed condition in overworked contact beds, and the air space in the contact material was about 30 per cent, so that Barbour recommended after the examination the continuation in service of the beds, but at a rate not exceeding 600,000 gal. per acre daily.

The Plainfield contact beds were under the control of trained operators from the outset of their service in 1901. After 1 year operation, under excessive loads during a part of this period, Lanphear reported[1] that the effluent of the secondary beds was non-putrescible from October to May inclusive, and putrescible from June to September inclusive.

The beds at Manchester, England, were dosed at the rate of about 480,000 gal. per acre per day during the first few years of their operation. Before the secondary beds were finished, the effluents from the primary beds were reported to be putrescible on account of the suspended matter in them. The rate of operation had been increased to about 649,000 gal. per acre daily, which is greater than that contemplated by the designers and was reduced a little later to 574,000. gal. By that time it was found that the beds needed washing after about 2,700 fillings. In 1909–11 a large number of secondary beds were put in operation and the rate of application of the sewage to the primary beds was

[1] *Eng. Rec.*, July 1, 1911.

increased to 713,000 gal.; later rates of these beds were 661,000, 576,000, 567,600 and 612,000 gal. By 1914 many of the beds had been renewed a second time, and it was reported that the number of fillings between the first and second renewals had been from 4,786 to 6,404. The water capacity of a ½-acre bed 3 ft. 4 in. deep ranged from 216,000 gal. when new to 108,000 gal. after 2,250 fillings. At that time part of the effluent from the primary beds was treated on secondary beds at the rate of 18 fillings of the latter per week. Although the plant had been under the operating charge of specialists from the outset, the results were officially reported in 1914 to be unsatisfactory; part of the difficulty was apparently due to insufficient available head for operation.

The effect of different operating conditions on similar contact beds was studied at the Lawrence Experiment Station by means of eight parallel experiments with beds 33 in. deep, filled with cinders ¼ to 1¼ in. in size and dosed with settled sewage. The general conditions and results of the tests, started in 1911 and ended in 1915, are given in Table 53. Clark's conclusions[1] from the results are as follows:

"The differences in the degree of purification due to the different methods of operation are relatively small. The longer period of contact tends to produce a more stable effluent but with greater loss of open space. The number of fillings daily has little effect on stability. With three fillings daily there is a greater proportional decrease of open space than with two fillings daily. Filling from the bottom instead of the top decreases the stability of the effluent but gives the least loss of open space. Briefly summed up, these results indicated that for conditions similar to those in these experiments, the best results will be obtained by filling twice daily from the top with as much aeration as practicable and a period of contact somewhat longer than an hour."

The results obtained with double-contact beds depend upon all those factors affecting single-contact beds and also upon the relative loads the primary and secondary beds are called upon to carry. Some data are furnished by a contact bed, No. 443, placed in operation at the Lawrence Experiment Station in 1912. It was dosed twice daily with the effluent from bed 425, see Table 53, and allowed to stand full 1 hr. The bed had 21 in. of broken stone, all of which passed a ½-in. screen, 43 per cent passed a ¼-in. screen and practically none passed a

[1] "Experiments upon the Purification of Sewage and Water at the Lawrence Experiment Station," 1915, p. 31.

TABLE 53.—RESULTS OF 5 YEARS' OPERATION OF CONTACT BEDS AT THE LAWRENCE EXPERIMENT STATION

Filter number	Filled from	Time of filling, minutes	Standing full time, hours	Times filled daily	Dose, gallons per acre daily	Effluent, parts per 1,000,000				Samples putresci-ble, per cent
						Ammonia		Nitrates	Oxygen consumed	
						Free	Albuminoid			
421	Top	2	8	1	383,800 in 1911	19.89	2.24	1.0	16.2	
					377,100 in 1912	15.69	2.50	0.9	15.8	
					279,300 in 1913	14.96	2.62	0.1	16.6	0.0
					260,000 in 1914	11.50	2.29	1.7	16.7	0.0
					255,500 in 1915	5.90	1.60	2.3	13.4	1.4
422	Top	2	4	1	376,500 in 1911	19.61	2.48	1.4	17.0	
					375,400 in 1912	17.06	2.66	2.6	17.0	
					277,700 in 1913	16.79	2.68	3.7	16.9	5.5
					264,000 in 1914	13.80	2.35	5.6	17.1	0.0
					272,400 in 1915	7.30	1.60	7.5	13.5	8.2
423	Top	2	2	1	391,000 in 1911	21.72	2.55	1.8	18.5	
					404,100 in 1912	18.48	2.92	2.6	19.2	
					312,300 in 1913	18.31	2.97	4.1	19.8	44.4
					284,000 in 1914	16.85	2.70	5.8	20.0	5.0
					290,700 in 1915	10.40	1.90	8.0	15.7	31.4
424	Top	2	1	1	388,400 in 1911	24.33	2.70	2.1	19.9	
					401,000 in 1912	20.33	3.23	2.9	21.3	
					304,900 in 1913	18.67	3.00	5.5	21.4	55.6
					293,000 in 1914	17.04	2.96	6.9	22.0	5.0
					291,000 in 1915	9.10	1.90	9.7	17.0	28.9

425	Top	2	1	2	671,800 in 1911	22.22	2.81	1.4	20.9	
					765,000 in 1912	21.07	3.59	3.3	22.1	
					548,700 in 1913	18.72	3.45	5.6	22.6	55.6
					527,000 in 1914	16.15	3.02	7.0	21.4	10.0
					464,000 in 1915	9.40	2.00	8.3	17.0	32.8
426	Top	2	1	3	894,000 in 1911	21.49	2.71	1.7	20.4	
					1,004,700 in 1912	20.07	3.22	1.9	21.9	
					599,000 in 1913	16.92	3.09	4.4	21.8	5.6
					507,000 in 1914	12.74	2.89	6.6	19.5	5.0
					454,300 in 1915	7.50	2.30	7.8	19.0	19.0
427	Bottom	2	1	1	396,400 in 1911	24.87	2.79	0.7	21.7	
					426,100 in 1912	23.30	3.37	1.3	22.7	
					324,200 in 1913	18.79	3.09	3.4	23.4	38.8
					311,000 in 1914	16.08	3.12	4.2	22.5	25.0
					312,400 in 1915	15.50	2.20	4.0	17.5	42.9
428	Top[1]	60	1	1	387,300 in 1911	19.95	2.41	3.2	18.3	
					397,600 in 1912	18.50	2.69	7.5	18.7	
					304,100 in 1913	17.90	2.97	8.9	19.8	33.3
					296,000 in 1914	15.90	2.68	9.2	19.3	0.0
					301,000 in 1915	11.40	2.10	9.5	17.3	20.7

[1] Dose applied by a tipping basin emptying into a perforated pan 1 ft. above the surface of the bed, the sewage dripping through the perforations to the bed.

TABLE 54.— OPERATING RATE AND CHARACTER OF EFFLUENT OF A SECONDARY BED TESTED AT THE LAWRENCE EXPERIMENT STATION

Year	Rate, gallons per acre daily	Ammonia		Nitrates, parts per million	Oxygen consumed, parts per million	Samples putrescible, per cent	Loss of open space at end of year, per cent
		Free, parts per million	Albuminoid, parts per million				
1912	686,400	14.70	2.31	11.6	15.6	0.0	27
1913	548,700	12.58	3.04	15.2	20.0	0.0	25
1914	527,000	10.26	2.40	17.1	17.8	0.0	36
1915	455,700	4.60	1.50	16.1	13.1	0.0	32
1916	522,000	3.41	1.36	12.9	12.1	0.0	36
1917	433,000	2.96	2.38	10.6	21.1	11.0	40

⅛-in. screen. The operating results with this secondary bed are given in Table 54. During 1916, 96 per cent of the samples of the effluent applied to this bed were stable, but during 1917 only 63 per cent of the samples of the applied effluent were stable; the oxygen consumed of the applied effluent was 14.5 p.p.m. in 1916 while in 1917 it averaged 24.8 p.p.m. It is evident from these data and others given by Clark that the applied liquid in 1917 threw a heavier burden on the bed than in previous years.

CHAPTER XVII

TRICKLING FILTERS

The limitations of the capacity of contact beds encouraged efforts to devise beds which would accomplish similar work without being subject to such restrictions. The result was the trickling filter,[1] similar in construction to the contact bed, except that it is not necessarily built within a water-tight tank. The filter material is placed upon a draining floor, through which the water can flow freely. The sewage is applied to the surface of the filter as uniformly as possible by sprinklers or other devices. Fig. 191 is a view of the trickling filter at Fitchburg, Mass.,

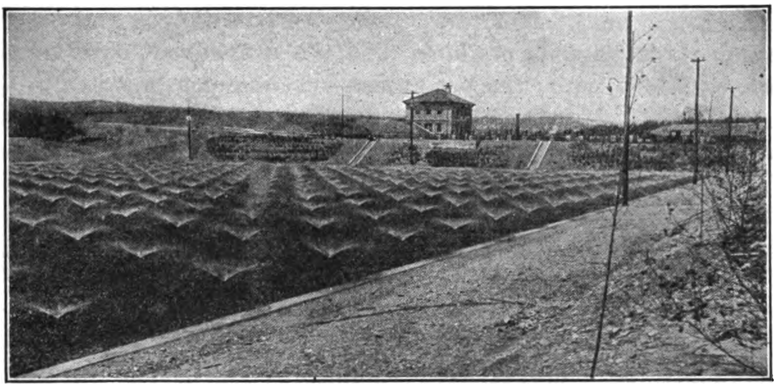


FIG. 191.—View of the trickling filter at Fitchburg, Mass.

built from the plans of D. A. Hartwell and the authors. It is usual to apply the sewage for a few minutes and then shut it off for a short time, as 5-min. dosing and 10-min. resting.

This construction offers as large an absorption surface as the contact bed. The sewage trickles very slowly over the stones and has ample opportunity to give up colloidal and dissolved substances to the bacterial jelly. The method of dosing is such that the bed may absorb oxygen continuously instead of during a brief period, as in the contact bed, and the danger of anaerobic conditions is greatly reduced. Experience has demonstrated

[1] Also called "percolating," "aerating," "sprinkling" or "sprinkler" filter.

that usually trickling filters are self-cleansing, while the material of contact beds must be removed and cleaned occasionally.

Theory.—The settling solids not removed by preliminary sedimentation are retained mechanically in the pores of the filter. The colloidal matter adheres to the surfaces through attraction and absorption, and the dissolved organic matter is absorbed by the bacterial jelly or acted upon directly, probably both processes occurring. The bacterial jelly coats the stones from top to bottom of a bed in mature condition and appears to perform its functions in relays, that at the top acting upon so much of the organic matter as its capacity will permit, that of the next stratum below attacking the substances passing by the upper one or perhaps emitted from it only partly oxidized, and so on to the bottom, where the oxidized sewage falls through the floor grating into the underdrainage system.

In the process of oxidation the organic matter is converted into humus-like material or resolved into carbon dioxide and other soluble and gaseous products, and the nitrogenous substances are oxidized with the production of ammonia, nitrites and nitrates. The processes are chiefly aerobic, for failure of the air supply is quickly reflected in the inferior quality of the effluent. The presence of vast numbers of higher forms of life, such as worms varying in size from those almost microscopic to earth worms 2 or 3 in. long, leads to the conviction that bacteria are not the only organisms laboring to transform the putrescible organic matter into substances less likely to cause offense.

Unloading of Trickling Filters.—The periodic storage and disgorging of solids by the trickling filter is one of its most important functions. Being a community of innumerable living organisms, it is not surprising that it responds quickly to changes of temperature and other conditions. In summer, in the north-eastern states, the quantity of solids in the effluent is about the same as in the applied sewage. Oxidation is then more active and nitrification will take place with doses not too great to permit efficient oxidation. A trickling filter designed and operated for that purpose will convert a large portion of the organic nitrogen into nitrates. In the fall, the efficiency of oxidation drops progressively with the lowering of the temperature, and the proportion of solids in the effluent to those in the sewage is also reduced. During the winter oxidation is low and much organic matter applied to the filter is stored in it. But with the first warm weather of spring, the filter emits great quantities of solids, far in excess of those in the sewage applied at the time. Thus the stored matter, including the bacterial jelly which has served its purpose, is ejected from the filter, which thus recovers its capacity. Likewise, the renewed activity of the organisms produces more complete oxidation, and the quality of the effluent gradually improves until it equals that of previous summers.

Results.—The trickling filter is capable of converting putrescible settled sewage into stable effluent not subject to putrefaction under the most exacting conditions. The effluent contains much settling matter, especially during the spring unloading period. This should be removed from the effluent by sedimentation and the filter should be judged by the effluent thus settled. After sedimentation the effluent is usually more or less turbid and may be somewhat colored. There may be places where its quality would not be as good as the conditions require, and further purification or a substitute for trickling filters must be provided.

A practical operating difficulty not yet overcome is irregular distribution of the sewage over the surface. The sewage passes more quickly through those portions of the filter which are overdosed than through the underdosed portion. On this account, the effluent is composed of portions of the applied sewage which have received varying degrees of oxidation rather than of sewage all of which has been oxidized to the degree apparently represented by the analysis of the effluent.

The removal of bacteria at the Lawrence Experiment Station is probably considerably greater than that accomplished in practical sewage treatment plants. The following figures[1] give the relative efficiencies of the methods of treatment at Lawrence:

	Removal of Bacteria, Per Cent
Single contact beds dosed with septic tank effluent	53–50
Single contact beds dosed with settled sewage	58–62
Double contact beds dosed with settled sewage	77–78
Trickling filter[2] dosed with settled sewage	78–79
Trickling filter[3] dosed with settled sewage	96–97
Trickling filters dosed with settled sewage; effluent settled and refiltered through intermittent sand filter	99
Intermittent sand filters	99

[1] *Report* Mass. State Board of Health, 1908, p. 513.
[2] This filter had 5 ft. of ¼- to 1-in. stone.
[3] This filter had 10 ft. of ¼- to 1-in. stone.

Neither contact beds nor trickling filters alone will remove all bacteria, although the trickling filters appear more efficient than contact beds. In spite of the turbidity and color of the effluent from trickling filters, its large bacterial content and considerable residual organic matter, this type of filter is now the most generally practicable means of artificial oxidation of sewage. Though incapable of producing as pure an effluent as some other methods, its efficiency is such as to render it suitable for adoption in many municipalities requiring artificial sewage oxidation plants.

Character of Filtering Medium.—The greater the surface afforded, the more efficient the filtering medium will be. Rough materials have the disadvantage that they tend to retard the unloading of transformed solid matters and are, therefore, more liable to become clogged than smoother materials. Clogging interferes with the natural passage of air into the filter and an abundant air supply is of the utmost importance. It may mean that a portion or all of the filtering material must be removed and washed, which adds greatly to the cost of operation. Under ordinary conditions, therefore, it will be well to avoid extremely rough materials like slag as well as very smooth materials like gravel.

Certain kinds of materials disintegrate, so that the bed as a whole may in time come to be entirely different in character. In some climates the upper portion of the bed is liable to be alternately frozen and thawed during the winter, and if the material is porous there is a decided tendency toward disintegration. Trap rock is a satisfactory material in this respect. Coke and cinders are undesirable. There may be places where it will be more economical to use cheap materials like cinders even though they must occasionally be renewed, than to build of more permanent but considerably more expensive material like broken stone.

Size of Filtering Medium.—The smaller the filtering medium the greater will be its bacterial surface and the longer will be the time afforded the sewage for contact with this surface. The time of contact or passage through filters of different depths and sizes of ballast which was observed at the Lawrence Experiment Station is given in Table 55, but this time, it should be understood, relates only to the conditions of those particular experiments. Dunbar found[1] that if a quantity of sewage equal to the water-retaining capacity of a filter 3 ft. deep is applied to such a bed, 20 to 30 min. will elapse before that sewage is discharged from the bottom.

TABLE 55.—TIME OF PASSAGE OF SEWAGE THROUGH EXPERIMENTAL TRICKLING FILTERS, LAWRENCE, MASS.

(Report of the Massachusetts State Board of Health, 1908, page 386)

Material	Size of material, inches	Depth		Ave. daily rate[1]		Time of passage of sewage, hours	Nitrates, parts per million
		ft.	in.	Gal. per acre	Gal. per cu. yd.		
Broken stone.....	¼–1	10	0	1,150,000	71.3	3	29.5
Broken stone.....	¼–1	10	0	1,938,000	120.1	2	20.5
Broken stone.....	¼–1	8	0	963,400	74.6	2	21.2
Broken stone.....	¼–1	5	0	925,500	114.7	2	5.2
Coarse clinker....	¾–1¾	5	9	885,000	95.4	6	12.4
Coarse clinker....	¾–1¾	3	10	949,000	153.5	1	7.3
Fine clinker......	¼– ¾	5	9	896,500	96.6	14	22.0
Fine clinker......	¼– ¾	3	10	908,900	147.1	6	5.0

[1] Six days a week.

Numerous experiments with trickling filters of different sizes of coal, gravel and clinker from refuse destructors were carried out by William Clifford, who concluded[2] from the results he obtained that the time of percolation through clean filtering material varies inversely as the rate of sprinkling and directly as the amount of water taking part in the general water movement through the bed, the amount of water in motion being generally represented by the interstitial water. Assuming that a given sewage will require 100 min. of contact to become changed in character to the desired extent, he found that this contact time could be obtained by using any of the combinations of depth and size of filtering medium given in Table 56. His experiments were made with clean material instead of that which has been coated with bacterial jelly, as in practice, and for this reason some of the variations in depth given in the table are probably too great. American practice shows a tendency to use stone from 1 to 2½ in. in size.

[1] "Sewage Treatment," p. 140. The water-retaining capacity of a soil is defined by Dunbar as the volume of water which remains in the soil after first drying the soil, filling it with water and allowing the excess water to drain away.

[2] *Proc.* Inst. C. E., vol. clxxii, part i, p. 289.

As small material should be used as is consistent with abundant air supply and freedom to unload from time to time the solids which have accumulated in the filter. Unloading is apparently facilitated by having the filtering material fairly uniform in size, for if the voids between the large stones are filled with finer stones the sewage solids may be retained indefinitely in the bed and cause clogging. In practice difficulty is often experienced in securing stone within the specified sizes, and unless fine material is definitely excluded at the crusher, the stone should be rescreened before it is placed in the filter.

The character of the applied sewage must be considered in selecting the size of filtering medium. The more thoroughly the suspended matter is removed by preliminary treatment, the finer the filtering medium it is safe to choose. Generally sewage is fairly well settled before it is applied to trickling filters in order to reduce the danger of clogging to a minimum and to avoid placing upon the filter a burden of work which can be performed by less expensive structures like tanks.

TABLE 56.—PERCOLATING FILTERS OF EQUIVALENT CAPACITY BASED ON A TIME OF PERCOLATION OF 100 MINUTES

(Clifford, *Proc. Inst. C. E.*, volume clxxii, part i, page 291)

Gravel			Coal			Slag			Clinker		
Grade in. in.	Depth ft.	in.	Grade in. in.	Depth ft.	in.	Grade in. in.	Depth ft.	in.	Grade in. in.	Depth ft.	in.
1 –¾	8	3	¾–⅝	7	2	1¼–¾	6	1	1–¾	6	1
¾–⅝	7	3	½–⅜	5	2	¾–⅝	5	4	⅝–⅜	3	9
⅝–½	6	7	⅜–¼	4	5	⅝–⅜	4	9	....	.....	
½–¼	4	4	¼–⅛	4	0	⅜–¼	4	0	....	.....	
....	....		....	....		¼–⅛	3	10	....	.....	

Depth of Trickling Filters.—The depth of a trickling filter should be limited by the requirements of proper ventilation, for if the oxygen of the air passing through the filter is used up before the bottom is reached the lower part of the filter will be ineffective. For treating unusually strong sewages or industrial wastes having a marked avidity for oxygen, such filters should not be so deep as when ordinary domestic sewage is to be treated. Furthermore, there will be a greater tendency toward surface clogging and consequent obstruction of the circulation of air in deep beds on account of the greater quantity of sewage applied per unit of superficial area. This condition is governed largely by the character and size of the filtering medium and the efficiency of preliminary clarification. Organic growths have appeared at times on some filters, more or less clogging the surface layer. Uneven distribution will more quickly result in clogging over-dosed areas in the case of deep beds because of the higher rate of application per unit of area. If portions of the filter area must be rested from time to time on account of surface clogging, a larger proportion of the filter will necessarily be out of use in the case of deep beds.

In most cases where sufficient head is available, the cost of trickling filters per unit of volume decreases with the increase in depth, owing principally to the reduced cost of floor, under-drainage and distribution with deeper beds. If the quantity of sewage which can be satisfactorily purified by trickling filters is proportional to the depth, a subject which will be discussed later, the deeper filters will be the more economical. Estimates made by one of the authors, based on the contract prices for the trickling filters at Fitchburg, Mass., showed that the cost per effective cubic yard of filtering medium was $4.54 in the case of 6 ft. depth, $4.20 with 7 ft. depth, $3.96 with 8 ft. and $3.60 with 10 ft. depth.

It is impossible to ascertain the most economical depth of filter from the varying degrees of purification effected by different depths of the same filtering medium dosed at a uniform rate per acre. On the other hand, if similar filters of different depths are dosed with the same quantities of sewage per cubic yard of filtering medium, it is easy to determine the relation of depth to filter efficiency; or, still better, if similar filters of different depths are dosed with portions of the same sewage in such quantities as will permit of effluents of practically the same quality in all cases the respective capacities are evident. The relation of the rate of application per acre to the rate per cubic yard of filtering material is shown in Table 57.

The danger of carrying too far the theory of equal loading per cubic yard lies in part in the greatly increased dose per unit of surface area, which may lead to surface clogging. This is illustrated by Table 58, from which it will be seen that very large doses are required for the deeper filters.

TABLE 57.—EQUIVALENT RATES OF FILTRATION PER ACRE EXPRESSED IN GALLONS PER CUBIC YARD

Gallons per acre	Gallons per cubic yard for depths given in column headings					
	5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.
500,000	62	52	44	39	34	31
1,000,000	124	103	89	78	69	62
1,500,000	186	155	133	116	103	93
2,000,000	248	207	177	155	138	124
2,500,000	310	259	221	194	172	155
3,000,000	372	310	266	233	207	186

TABLE 58.—EQUIVALENT RATES OF FILTRATION IN GALLONS PER ACRE PER DAY FOR DEPTHS GIVEN IN FIRST COLUMN

Depth, feet	Dose, gallons per cubic yard			
	100	200	300	400
5	806,600	1,613,200	2,419,800	3,226,400
6	968,000	1,936,000	2,904,000	3,872,000
7	1,129,300	2,258,600	3,387,900	4,517,200
8	1,290,600	2,581,200	3,871,800	5,162,400
9	1,452,000	2,904,000	4,356,000	5,808,000
10	1,613,300	3,226,600	4,839,900	6,453,200
Gal. per acre-foot per day..	161,330	322,660	483,990	645,320
Load, persons per acre-foot at 100 gal. per capita daily...	1,613	3,227	4,840	6,453

British Opinions on Depths.—The Royal Commission on Sewage Disposal reached the conclusion that a distinction should be made between trickling filters of coarse material and those using fine material. By coarse material the Commission meant material 3 in. in size and upward, medium-size material includes that from ½ in. to 1 in. in size, and fine material is ¼ in. in size. The Commission's conclusions[1] read:

[1] *Fifth Report*, 1908, p. 87.

1. "The deeper the filter the better the effluent. This holds both for fine and coarse material, assuming good distribution and aeration.

2. "For practical purposes and assuming good distribution, the same purification will be obtained from a given quantity of *coarse* material, whether it is arranged in the form of a deep or of a shallow percolating filter, if the volume of sewage liquor treated per cubic yard be the same in each case. Thus a filter 3 ft. deep, worked at the rate of x gal. per square yard per day, would give similar results to a filter 6 ft. deep, worked at the rate of $2x$ gal. per square yard.

"In the absence of clogging, the balance would be slightly in favor of the deep filter, because the greater the depth the more the errors of distribution would be neutralized. There must always be a limit of shallowness (say 3 ft.), beyond which it would be unwise to go, for however good the distribution might be, if a coarse filter were constructed very shallow, a considerable proportion of the sewage would pass through by 'short cuts.'

3. "With regard to filters of *fine* material, if the liquid to be purified were absolutely free from suspended and colloidal solids, and if thorough aeration could be maintained, the statement just made for filters of coarse material might possibly hold good for filters of fine material also. In practice, however, these conditions can scarcely be maintained with large rates of flow and we think that the greatest efficiency can be got out of a given quantity of *fine* material by arranging it in the form of a shallow filter rather than a deep filter. But we are not in a position to make an exact quantitative statement as to the difference in efficiency of the two forms.

"It is probable that for the treatment of a strong sewage liquor the filter should not be quite so shallow as for a weak liquor, but the point has not been fully worked out."

Investigations at Lawrence Experiment Station.—After studying 16 years' operating records of broken stone trickling filters at the Lawrence Experiment Station, Clark reported[1] that the maximum results in efficiency of purification and volume of sewage purified were obtained when the filter is of considerable depth and constructed of comparatively fine material. In order to obtain more definite information four new filters, Nos. 452 to 455 inclusive, were started in 1913. They were filled with broken stone between ¾ and 1½ in. in size, what the Royal Commission termed medium-size stone, and were 4, 6, 8 and 10 ft. deep respectively. In 1915 four more trickling filters, Nos. 472 to 475 inclusive, were started, also 4, 6, 8 and 10 ft. in depth, but filled with stone having an average volume about six times (roughly 3 in. in size) that of the stone used in the first group of filters. These 1915 filters were of the Royal Commissions' "coarse" class.

[1] "Experiments upon the Purification of Sewage and Water," 1914, p. 17.

The filters were operated at first at rates giving effluents containing 15 p.p.m. nitrates, which previous experience at the Station showed was necessary to insure stable effluents from trickling filters. Later changes in the character of the applied sewage resulted in a dropping of the nitrates to about 10 p.p.m., and still later to 7.5 to 10 p.p.m. Speaking generally, the shallow filters proved more troublesome to mature and the process took more time than was the case with the deep filters.

In estimating the relative operating performance of these filters, Clark assumed that the amount of nitrification is at least an approximate measure for this purpose. The amount of oxygen absorbed on incubating samples of effluent for 24 hr. at 70°F. in closed bottles was determined at least once each month. On incubation part of the oxygen absorbed by the oxidation of the organic matter present is taken from the dissolved oxygen and part from the nitrites and nitrates. Clark assumed that three-fourths of the nitrite oxygen and five-sixths of the nitrate oxygen were available for this purpose.

The results of these tests are given in Table 59. Clark pointed out[1] that the greater efficiency of deep filters is somewhat more pronounced with material of the size used in filters 452 to 455 than with coarse filters. If it were practicable to have every trickling filter operated under the constant skilled supervision maintained at the Lawrence Station, then for the maintenance of high rates with a good degree of purification the material used in the first series of filters would be more desirable than that used in the coarse series. Clark reported the conclusion in 1917 that even with the coarse material used in filters 472 to 475 inclusive, which averaged 25 to 29½ cu. cm. in size as against an average size of 3.7 to 5.3 cu. cm. in filters 452 to 455, a filter 10 ft. in depth can be operated at a rate per acre per foot depth approximately twice that of a filter 4 ft. deep.

The practical importance of avoiding the use of such fine stone that a filter will become unserviceable through clogging is shown by the record of filter 135 at the Lawrence Experiment Station. This was 10 ft. deep and filled with broken stone from ¼ to 1 in. in size. It was placed in service in 1899 and in spite of unusually careful management it soon showed a tendency to become clogged. During early years of operation, the clogging

[1] "Experiments upon the Purification of Sewage and Water," 1917, p. 25.

TABLE 59.—OPERATION OF TRICKLING FILTERS OF MEDIUM-SIZE AND COARSE STONE OF DIFFERENT DEPTHS AT THE LAWRENCE EXPERIMENT STATION

Filter number	Depth, feet	Year	Gallons daily		Samples stable, per cent	Total oxygen absorbed, parts per million
			Per acre	Per acre. per foot depth		
452	4	1914	309,000	77,000	88	11.5
		1915	466,400	117,000	94	11.5
		1916	410,000	103,000	78	15.1
		1917	287,000	72,000	84	31.5
453	6	1914	584,000	98,000	93	11.3
		1915	805,500	134,000	100	11.7
		1916	931,000	155,000	97	16.4
		1917	780,000	130,000	74	34.3
454	8	1914	2,047,000	256,000	90	9.7
		1915	1,767,250	221,000	100	9.9
		1916	1,471,500	184,000	97	10.9
		1917	1,400,000	175,000	89	27.3
455	10	1914	3,971,000	397,000	81	13.2
		1915	3,501,000	350,000	88	10.9
		1916	3,837,000	384,000	87	12.8
		1917	3,250,000	325,000	68	29.6
472	4	1915	464,200	116,000	14	
		1916	339,300	85,000	37	16.1
		1917	190,000	48 000	68	26.7
473	6	1915	616,200	103,000	71	
		1916	714,000	119,000	91	17.7
		1917	720,000	120,000	48	26.7
474	8	1915	674,600	84,000	86	
		1916	1,004,000	125,000	80	15.7
		1917	710,000	89,000	74	26.0
475	10	1915	738,100	74,000	100	
		1916	1,814,000	181,000	81	17.2
		1917	1,755,000	176,000	79	24.2

could be removed by spading the surface and allowing it to rest for a short time. In 1917, however, its operation was very unsatisfactory, although dosed at the low rate of 173,000 gal. per acre per foot depth. It was given a week's rest and on two occasions the top 3 in. of stone were removed, washed and replaced. The quality of the effluent continued to deteriorate during that year and the filter had evidently become clogged throughout its depth. This experience with material of the Royal Commission's "medium-size" class shows the danger which attends attempts to secure high rates of operation by using rather fine material, and teaches the advisability of using a filtering medium which will unload freely, even if it is so coarse that a rather low rate of operation per acre per foot depth becomes necessary to maintain a stable effluent.

An explanation of the greater efficiency of deep filters has been given[1] by Clark. It is based on the fact that in filters in operation there is a certain quantity of water retained in the interstices of the filtering material and held by the organic matter on the stones. He found that the deeper the trickling filter up to a certain point, the greater the tendency of the applied water to mingle with the retained water instead of merely passing by it. In other words, there was greater contact in deep filters than in shallow filters. Experiments were made with filters 452 to 455 inclusive, which gave the results recorded in Table 60. The experiments were made by adding a definite quantity of salt to the applied sewage and then determining the additional quantity of chlorine above the normal in the effluent every few

TABLE 60.—EFFECT OF DEPTH OF FILTER ON TIME OF CONTACT OF SEWAGE PASSING THROUGH THE FILTER

Rate of application, gallons per acre	Time in minutes for 50 per cent of the applied sewage to reach the underdrains			
	Number 452, 4 ft. deep	Number 453, 6 ft. deep	Number 454, 8 ft. deep	Number 455, 10 ft. deep
500,000	50	..	..	..
800,000	..	55	..	..
1,000,000	12	18	48	105
1,500,000	..	..	73	..
2,500,000	..	..	..	60

[1] "Experiments upon the Purification of Sewage and Water," 1915, p. 22.

minutes. It will be seen that the 10-ft. filter can be operated at a rate per acre per foot depth about twice that of the 4-ft. filter and maintain a contact period in each case of about an hour.

Motion of Sewage in Bed.—Experiments by Taylor with filtering material of different sizes and depths showed that the lateral diffusion of percolating liquid was greater with fine than coarse material, but was not large in any case. With 2-in. crushed gneiss in a 6-ft. bed, 90 per cent of the liquid applied at the surface in drops was not diffused beyond the circumference of a circle of 7 in. radius with its center below the point of application. With ⅛-in. stone the diffusion in a 6-ft. bed did not exceed 12 in. from the center. Most of the lateral diffusion took place in the top foot of depth. Experiments with several points of application of the sewage to the same bed showed that where the cones of diffusion became joined, the increase in the quantity of liquid produced a streaming effect, which was much more marked with high than low rates of application. Other investigations have confirmed these results.

In practice accumulations of solids in filters cause some lateral movements of the sewage and may actually improve the distribution by their retarding effect upon the passage of the sewage through the bed. On the other hand, if the clogging causes pooling at different points, the sewage will be diverted to the cleaner areas and streaming will occur, seriously interfering with uniformity of distribution. Since little may be expected in the way of lateral distribution below the surface, the importance of uniformity of surface distribution is apparent. The ideal desired may be illustrated by a rainfall of sufficient intensity to produce the desired rate of application. A rate of 1,000,000 gal. per acre daily would be equivalent to a rainfall of about 1.5 in. per hour, or 36 in. per day.

Revolving Distributors.—Traveling distributors are used extensively in Great Britain and at many of the Canadian plants to dose trickling filters. There are two general types, one consisting of two or four horizontal arms revolving about a central post and used to dose circular filters, and the other consisting of a low truss spanning a rectangular bed and moving back and forth on its side walls while distributing liquid over the bed in various ways, according to the make of the machine.

The revolving distributor used for dosing two filters 30 ft. in diameter, shown in Fig. 192, is typical of small British apparatus. There is a revolving distributor 130 ft. in diameter at Malvern and others from 70 to 120 ft. in diameter have been constructed for different places, but most installations have probably been under 70 ft. In the apparatus shown in Fig. 192 the weight is carried by a ball bearing at the top of the vertical

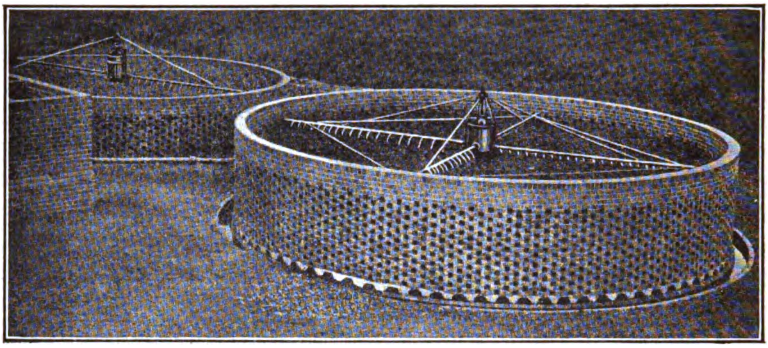


FIG. 192.—Revolving distributors of small English trickling filters.

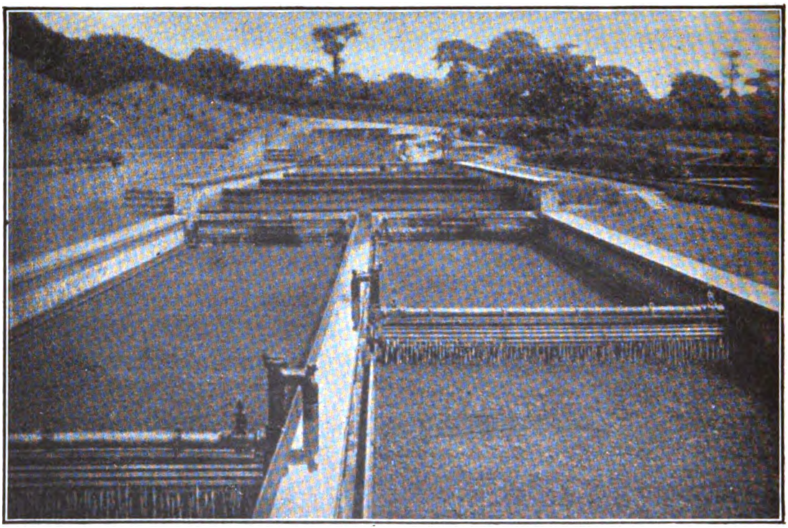


FIG. 193.—Traveling distributor of water-wheel type (Fiddian).

standard. In some other makes the weights of the moving parts are carried by floats. The head lost in these revolving distributors is very small.

Traveling Distributors.—An entirely different principle is employed in the waterwheel type of distributor, which seems to be favored in Great Britain for dosing filters of fine material where particularly uniform distribution is desired. These distributors can be constructed in a revolving form or to travel back and forth as shown in Fig. 193. Either type is driven by the fall of liquid

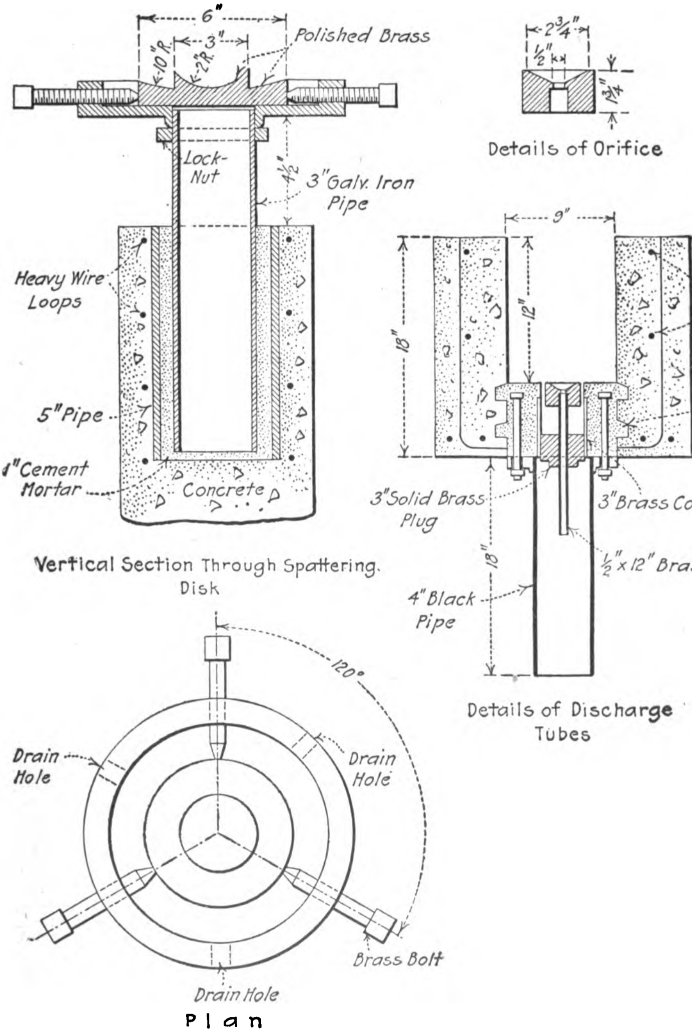


Fig. 194.—Splash plate and discharge tube, Calvert, Tex.

through a height of about 18 in. With the rotating type, the liquid is supplied through a hollow standard in the center of the bed, while in the traveling type it is siphoned out of a channel running along one side of the bed or between each pair of beds. The liquid passes through a feed tube running the whole length of the distributor This tube has apertures through which the liquid drops into the buckets of a long waterwheel. The feeding arrangements on the distributor are reversed at each end of the bed by a buffer attached to the masonry, and as soon as reversal occurs the apparatus begins its return trip. The beds shown in the illustration are 92 ft. long, 14 ft. wide and 3 ft. deep. Traveling distributors of another type are used at the trickling

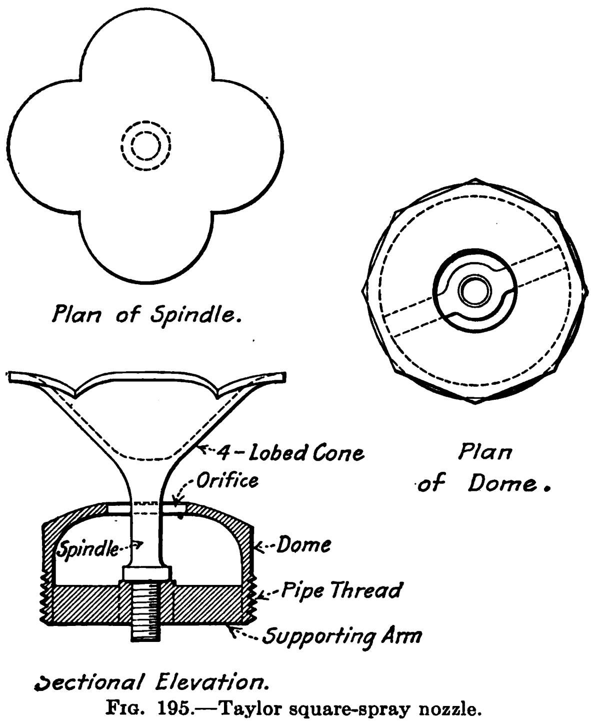


FIG. 195.—Taylor square-spray nozzle.

filters at Springfield, Mo., built from the plans of Potter. They were adopted on account of the low-operating head, 12 in., which they required.

Dash Plates.—The gravity distributor used at the experiment station of the Massachusetts Institute of Technology discharged sewage from openings in the bottoms of troughs or from pipes on to concave metal discs placed beneath, which caused the sewage to splash upward and outward in the form of spray. These dash plates are used on 1.2 acres of filters at Mt. Vernon, N. Y. Fig. 194 is an example of this type of distributor designed by Fountain for use at Calvert, Tex. This method of distribution is better suited for warm climates than for cold on account of the exposed distribution system.

Spraying Nozzles.—Another method of distribution, originated at Salford, England, and employed almost exclusively for dosing trickling filters in the United States, is to spray the liquid from a number of fixed nozzles. These nozzles are now made in several forms, some throwing a spray which covers an approximately square area and others dosing a circular area. A typical square spray nozzle, designed by Taylor, is shown in Fig. 195. It has an orifice 1 in. in diameter, through which a spindle passes, carrying a four-lobed, spreading cone, which is designed to throw the desired square-covering spray. The distributing cones of such nozzles must be kept in a definite position in order to spray contiguous square areas. These nozzles are designed to concentrate the spray in a narrow zone and their success in use depends upon rapidly varying the head of the applied liquid. Taylor accomplishes this by using a pressure-undulating valve on the main supplying the sewage to the bed. Taylor nozzles are also made to spray hexagonal and circular areas. A typical circular spraying nozzle is shown in Fig. 196, illustrating the nozzle developed at Worcester, Mass. It has at the base of the spraying cone a rim or lip, the function of which is to break up the sewage into a finer spray and spread it over a wider area. The spindle has a locking device so that it can be quickly removed when cleaning the orifice.

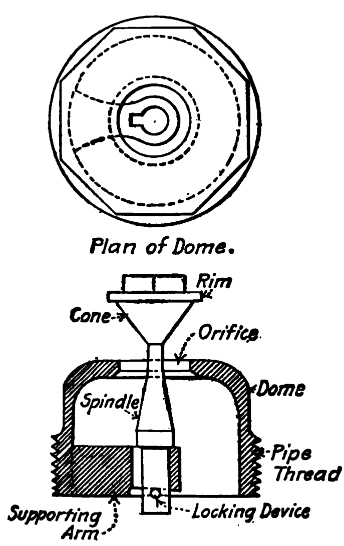


Fig. 196.—Worcester type of nozzle.

Efficiency of Fixed Nozzles.—The design of a nozzle should be such as to minimize clogging and permit rapid cleaning. Nozzles having small orifices generally produce a relatively fine spray, somewhat greater absorption by the sewage of atmospheric oxygen, and a more uniform distribution, but the cost of keeping them clear may be a large item. If the orifice is so large that the sewage is delivered to the bed in a sheet, the streaming effect may prevent efficient purification. With a fine spray more odor and a greater reduction in the temperature of the sewage during cold weather are to be expected than with a coarse spray.

Other conditions being equal, the nozzle which sprays the greatest area under a given head is the most economical. If this result is secured by a concentration of spray in a narrow ring, satisfactory distribution will depend upon a rapid variation in head during the discharge. The efficiency of practically all fixed nozzles may be greatly improved by a variation in head, because a comparatively large proportion of the discharge is concentrated upon a relatively narrow ring of the filter area if the head is constant. Methods of causing such variations are explained later in this chapter.

A long series of careful tests was made at Fitchburg, Mass., under the direction of the authors, to determine the uniformity of distribution with various types of nozzle working under a proper variation in head. A battery of nozzles was used in order to determine the actual effect of overlapping sprays. Spacings of nozzles of 13 and 15 ft. and heads between 2 and 10 ft. were studied. The spray was collected in small compartments of pans; the pans had one-eighth of the area of the sprinkled surface in the tests of square-spray nozzles and one-twelfth of the area in the case of circular-spraying nozzles. Each test was conducted for an exact period of time at a definite working head and the quantities of liquid collected in each compartment were measured. The discharge of each nozzle under the different heads was metered and the effective head was determined in each case.

The relative period of operation required at each head to produce the most even distribution was computed, and the distribution that could be effected by the nozzle with different ranges in head was determined on that basis. It was assumed that on all areas receiving between 87½ and 112½ per cent of the mean rate, the distribution was good. For the underdosed areas receiving 75 to 87½ per cent of the mean rate, the distribution was considered fair, and also for overdosed areas receiving 112½ to 125 per cent of the mean rate. The distribution on the remaining areas was called poor, whether overdosed or underdosed.

A typical example of the distribution effected by the Taylor square spray nozzle is shown in Fig. 197. The marked variation in distribution along the different radial lines from the nozzle is striking. An almost ideal distribution was given by the Wor-

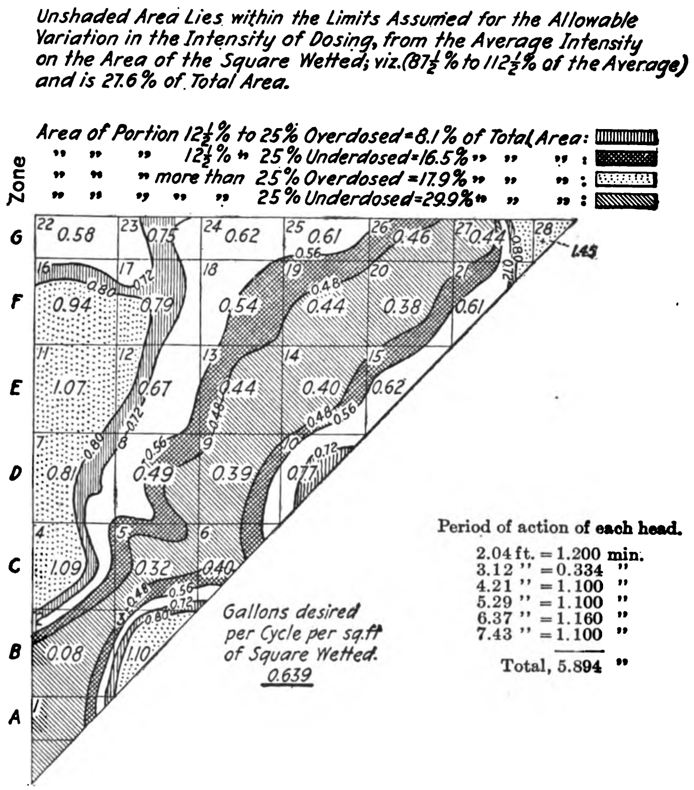


FIG. 197.—Estimated distribution in gallons per square foot per theoretical dosing cycle by Taylor square-spray nozzle, with 7/8-in. orifice and 13-ft. spacing.

The average operating heads range from 2.04 to 7.43 ft. and the period of action of each head is varied to produce an approximately uniform distribution in one cycle.

Letter of zone	A	B	C	D	E	F	G	Total
Area, sq. ft.	3.0	12.0	20.0	28.0	36.0	44.0	25.0	168.0

cester circular spray nozzle. An experimental dosing tank was constructed and further tests were made with it, with results shown in Fig. 198. While this distribution was not so good as that indicated by the theoretical dosing cycle, there was no area badly overdosed and the only area badly underdosed was adjacent to the nozzle. The latter defect would be remedied largely by a lower minimum head. In practice, there is likely to be a surplus of sewage in the distribution system before the siphon goes out of action, which will find its way on to this area. It is probable, also, that on account of the streaming effect produced

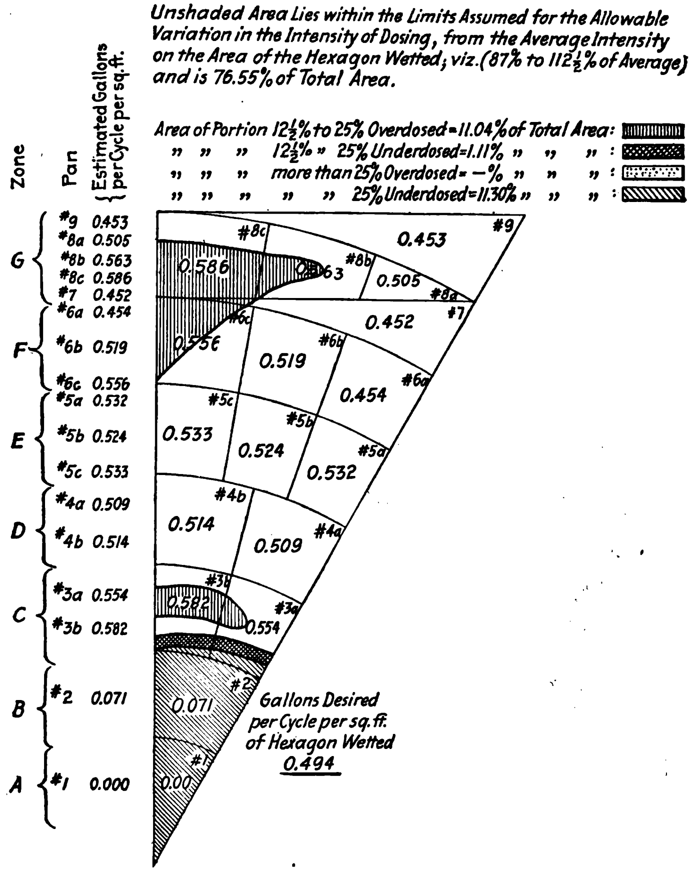


FIG. 198.—Distribution accomplished by Fitchburg experimental dosing tank using Worcester nozzle with $1\frac{3}{16}$-in. orifice and 15-ft. spacing.

The operating heads range from 2.00 to 7.96 ft. The quantities are in gallons per cycle per square foot. The time required to empty the tank was 5 min. 29 sec.

Percentage of desired dose	75	87½	100	112½	125
Gallons per square foot	0.371	0.432	0.494	0.557	0.618

by a fixed nozzle under very low head, it would be unwise to apply as great a quantity of sewage to the area close to the nozzle as to the area more remote.

Spacing of Nozzles.—Circular-spray nozzles which do not give over lapping sprays cover only 71½ per cent of a rectangular area when placed at the centers of contiguous square areas and only 90.1 per cent of the bed when arranged at the apices of equilateral triangles. Such a waste of filter area is impracticable. Accordingly the usual arrangement is to place the nozzles at the apices of equilateral triangles and provide for some overlapping of the sprays, as shown in Fig. 199. With all fixed nozzles, the maximum quantity of liquid along a radial line under a given head does not fall upon the perimeter of the wetted area. Uniform distribution along any radial line necessitates overlapping the sprays, the extent varying with different nozzles and with different heads on the same nozzle. For example, in Fig. 199, Worcester nozzles having 1⅟16-in. orifice are operating under a maximum head of 8 ft. They are 14 ft. apart at *A*, *B*, and *C*, the point *o* being 8 ft. from the nozzles. The spray is carried out 10 ft. from each nozzle, as shown by the arcs described about *A*, *B* and *C*. The extreme overlap beyond the point *E*, half way between nozzles, is 3 ft. The figure *abc* is the area of overlapping from all three nozzles.

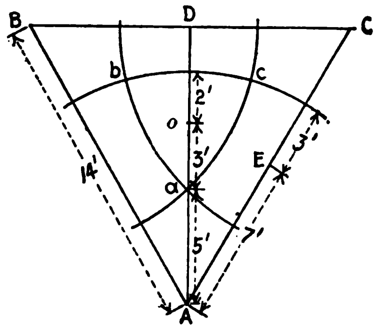


FIG. 199.—Method of providing for overlapping of sprays.

There is no overlapping of sprays around the edges of a filter dosed by fixed nozzles, and a smaller quantity of sewage per unit area reaches this portion. Assuming that the capacity of a trickling filter is approximately proportional to the depth, a smaller volume of filtering material may be provided around the edge of a filter, or, in other words, the bottom of the filter may be made smaller in area than the top, thus effecting a considerable saving in filtering material. For example, at Fitchburg the length and width of the filters at the bottom are 5 ft. less than at the top.

Relative Advantages of Intermittent and Continuous Operation.—It is necessary to vary the head on fixed nozzles in order to secure even distribution over the filter area. It follows, therefore, that no part of the area is in continuous operation, and all parts must be dosed at relatively high rates per unit of area for short intervals of time.

The period of operation of fixed nozzles, from the time they go into operation at maximum head until they are again in operation at the same head is called the dosing cycle. When a single-dosing tank is used, the nozzles are discharging nothing during the filling of the tank and this interval is known as the rest period of the cycle. At a given rate of operation, the rest period will depend upon the size of the dosing tank. The smaller the dosing tank the shorter will be the rest period and the greater

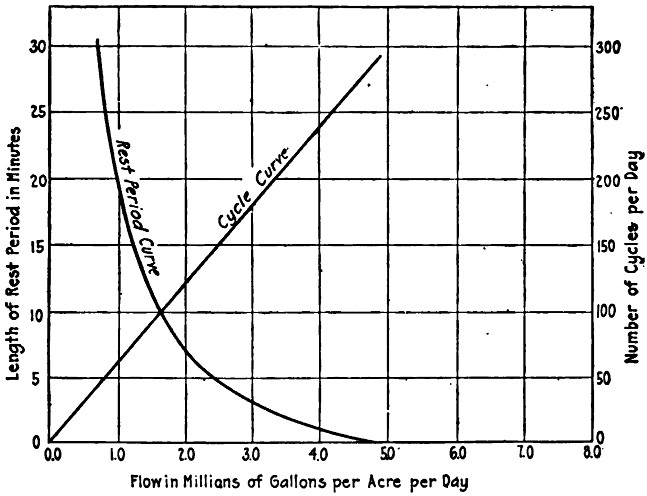


FIG. 200.—Curve of variations in length of rest period and number of cycles per day for various rates of flow of sewage.

Based on performance of Worcester nozzle with 13/16-in. orifice and 15-ft. spacing, working under heads of 2.04 to 7.43 ft. on a work cycle of 5 minutes.

the number of dosing cycles. A typical example of the relation of the rest period to the number of cycles at different rates of flow under specific conditions is shown in Fig. 200. Under these conditions there is no rest period above 4,800,000 gal. per acre per day, the nozzles going into continuous operation at that rate. The length of the rest period is limited in cold climates by the danger of freezing the sewage in the exposed portion of the distribution system.

If the sewage is allowed to flow into the dosing tank while it is discharging, the sewage flow must be limited to the rate of discharge of the nozzles under minimum head. In many cases this quantity is less than the filter is capable of satisfactorily purifying. Provision should accordingly be made in such cases for storage of the flow during the discharge of the dosing tank or for an additional dosing tank. By the use of double dosing tanks, as at Fitchburg, one tank is filling while the other is emptying, thus making possible a much higher rate of operation of the filter under varying head and a shortening of the rest period. Other conditions being equal, the nearer the application of sewage approaches continuous distribution at the mean rate, the more efficient will be the purification.

There appears to be some difference of opinion as to the value of periods of recuperation of longer duration than the rest periods occasioned by the dosing cycle. The experience of the authors has been that a recuperation period of 24 hr. or more is of great assistance at times in aiding trickling filters to unload accumulated solids. Similar resting in rotation of portions of filters affected by organic growths has sometimes proved of marked benefit. The length of the recuperation period will be governed by the extent of the clogging and the results obtained by actual operating tests.

Distribution Systems.—Mains for distributing sewage to fixed nozzles may be of concrete or vitrified pipe encased in concrete but the smaller sizes are preferably cast-iron pipe with lead joints. The pipes should be large enough to prevent undue friction and consequent loss of head at the nozzles. Economy of construction requires the lateral distributors to be relatively short to avoid the use of unnecessarily large pipe.

The distributing laterals may be laid just above the filter floor, as at Columbus; nearer the surface, as at Baltimore; or at the surface, as at Fitchburg. They may rest on the filtering material with no other foundation, but if it should become necessary to remove the filtering material, the pipes would then have to be supported or temporarily removed. Where the dash plate or gravity nozzle is used, the distributing troughs or pipes are necessarily supported at an elevation above the surface of the bed. It is advisable in all cases to provide ample facilities for flushing and draining the distribution system.

In certain cases fractures have occurred in lateral distributing pipes owing to excesssive pressures. If these pipes are laid at the bottom of the filter, repairs upon them will entail excavation of filtering material, which will add greatly to the cost of the repairs. Placing the laterals at or near the surface makes them more accessible, but large pipe placed at the surface are likely to affect the uniformity of distribution somewhat and clogging is more likely to take place about the pipes than on the remainder of the area. In this case also, the sewage in the distributing pipes is exposed to low temperatures of a cold climate to a greater extent than when these pipes are below the surface of the bed.

If the lateral distributors are at the surface of the filter, as at Fitchburg, the nozzles may be attached directly to the distributing pipes. If the lateral distributors are placed beneath the surface, riser pipes will be necessary. The most common form of riser is cast-iron pipe jointed with bitumen or some other flexible-joint material. The riser should be large enough to minimize friction, due allowance being made for a considerable deposit or growth on the inside of the pipe. Long risers are more liable to become bent or broken during construction than are shorter ones. The elevation of the top of the riser above the filter should ordinarily not exceed 6 in., although certain types of nozzle afford better distribution when the nozzle is placed 12 in. or more above the filter surface. The exposed portion of the riser is liable to cause trouble from freezing during the rest periods of the dosing cycle in cold climates.

Underdrainage Systems.—Trickling filters, built in excavation without masonry floors, are objectionable because the soil may become displaced by water flowing over it, thus causing settlement. Furthermore, the soil may become intermingled with the filtering material and impair the drainage of the filter. Such filters should be built upon floors of concrete or other masonry.

One of the earliest drainage systems consists of a concrete floor upon which is a layer of material composed of pieces preferably not less than 6 in. in diameter, the floor being sloped toward the main collectors. This coarse material affords better drainage than would the finer material of the bed, but clogging is liable to result from the solids unloaded by the filter. The coarse material accomplishes little in the purification of the sewage because of the relatively small bacterial surface per unit of filter volume. In order that the solids may be more readily carried away with the effluent or flushed through the drains, and the gases of decomposition may have a ready means of escape, false floor systems are generally adopted.

The floor system used at Columbus and some other places, Fig. 201, is the least expensive type under some conditions. The

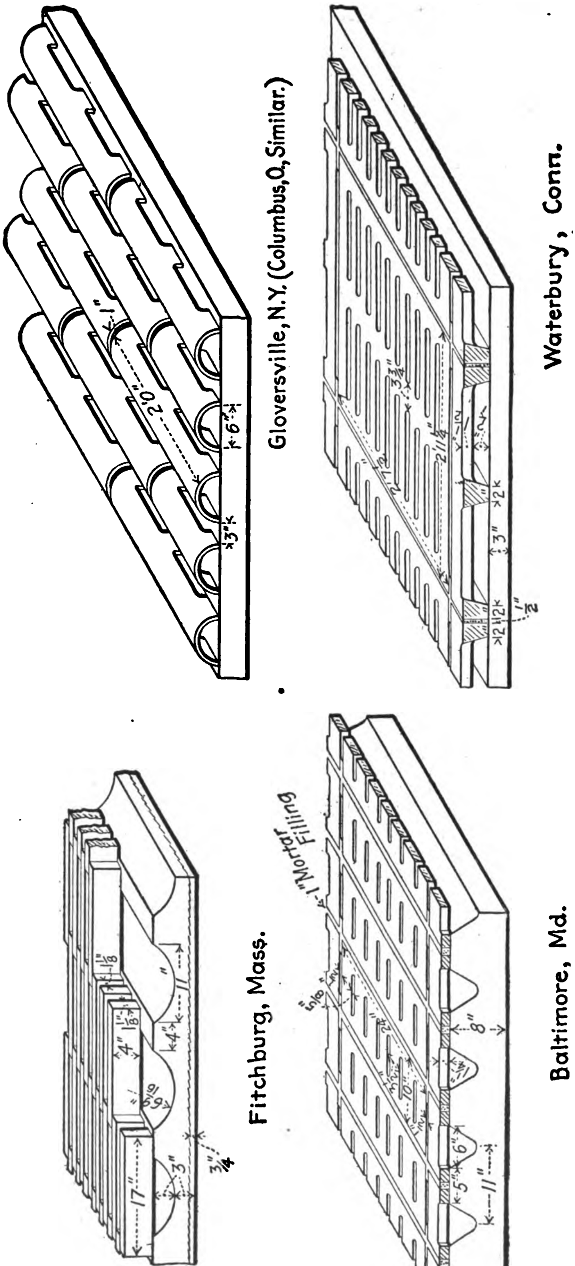


FIG. 201.—Types of floors of American trickling filters.

concrete slopes toward the main drains and the lateral drains of inverted half-round, slotted tile practically cover the entire floor. They are bedded in cement to afford an even bearing, thus enabling them to carry the load of stone placed upon them. A layer of coarse filtering material about 4 to 6 in. in diameter is usually spread over the pipes to prevent the finer material from passing through the openings into the drains.

This floor system offers a fairly good opportunity for the passage of solids from the bed into the drains. If the bed is composed of coke, cinders, or other material liable to disintegrate, the valleys between the pipes will probably become gradually filled with fine material which may clog the drains. A disadvantage of this floor is that the flow of effluent is distributed over the floor beneath the tile so that a relatively low velocity is afforded for carrying away the solids. Furthermore, it is not readily flushed because the water spreads over the bottom, rendering the stream ineffective.

Fig. 201 also shows the floor system designed for Waterbury, Conn. The concrete is flat, sloping toward the main drains. A special block of vitrified clay or reinforced concrete is set on the floor, furnishing a false bottom over the entire area. The drainage area is much greater than in the case of the Columbus type and the effluent flow is confined to the drains. The effluent is allowed to spread out over a very large part of the floor, however, so that the self-cleansing qualities and facilities for flushing are not ideal.

The Baltimore floor system Fig. 201, consists of a series of grooves and ridges in the concrete bottom. The grooves are covered with slotted vitrified clay or concrete slabs bedded in cement, concentrating the flow of effluent in narrow channels. The floor of the filter slopes toward the main drains and the inverts of the lateral drains are parallel with the floor, which necessitates a varying depth of stone in accordance with the slope of the floor. While the practical objections to such a variation in depth of filter may be questionable, it is theoretically an advantage to build the bed of uniform depth. With this type of floor system it is feasible, although possibly somewhat more expensive, to lay the floor level and give the necessary grade to the drains. In this case the filtering medium will be of uniform depth.

A type of floor designed by the authors and used at Fitchburg, Mass., is shown in Fig. 201. The lateral drains in the concrete are covered with narrow cement beams, affording a very large percentage of drainage area as well as a concentrated flow of effluent. Cobble stones, which were abundant at the site, were placed by hand over the openings before placing the crushed stone on the bed.

It is evident that the thickness of the floor has an important bearing on the cost of the filter. The percentage of open area in the false floor for drainage and the proportion of free space for carrying away the effluent and gases of decomposition should be as large as can be provided at reasonable cost. The strength of the false floor should be ample for the load coming upon it.

The possibility that the drains may become clogged with solids unloaded by the filter or by organic growths in them necessitates ample provision for flushing. In some cases the upper ends of the laterals are carried through the filter wall, affording an opportunity for flushing, and in other cases flushing galleries have been provided. The size of the lateral drains should be sufficient to carry away the effluent promptly, to make flushing possible and to afford space for the circulation of air.

The main drains may be rectangular or semicircular channels covered with slabs, or they may be circular. An open channel around the outside of the filter serving as a main drain, Fig. 192, affords an excellent means of ventilation. The grades of all parts of the drainage system should ensure freedom from backing-up of effluent in the ducts and channels.

Ventilation.—Inasmuch as the proper action of the filter depends on an abundance of oxygen within the bed throughout its depth at all times, adequate ventilation is essential. Downward ventilation is secured by the natural flow of sewage through the bed. Any interference with the free passage of water and air, due to the accumulation of clogging matters or to insufficient drainage capacity, causes a decrease in the efficiency of the filter.

Some filters have walls of large stones with openings for air to enter the beds laterally. In other filters, openings are left in the outer brick walls for the same purpose. While this may prove beneficial for small filters, like that shown in Fig. 192, its value for large units is doubtful. Any advantage gained during warm weather may be more than offset by decreased bacterial activity in cold weather, owing to lower temperatures produced in the air. At Gloversville, manholes were provided at the ends of the flushing galleries to aid in ventilating the filters. In several instances, vent pipes connected with the floor system and extending above the filter surface have been provided at regular intervals. In a few cases ventilating cowls have been placed at the tops of these pipes.

Filter Units.—The shape of the filters will be controlled in part by the topography of their site and to a large extent by the distribution scheme used. The size of circular filters with mechanical distributors is limited by the economical size of the distributing device, and is influenced by the fact that it is less expensive to build a few large beds than many small ones. For filters using mechanical distributors more than one unit should be provided, so that in case repairs become necessary the entire plant need not be shut down. The Royal Commission on Sewage Disposal recommended not less than three units for each plant. There is a decided advantage in having small units, for they permit resting portions of the filter at intervals. A filter fed by fixed nozzles may be divided into as small units as desired. At Columbus, the 10 acres of trickling filters are divided into six units, each an equilateral triangle and all arranged about a central point. This arrangement is advantageous for distribution but not for some forms of underdrainage. It has proved unsatisfactory to have the administration building in the center of the plant, owing to obnoxious odors and flies.

Preliminary Treatment.—Some form of preliminary treatment will ordinarily be required to prevent filter clogging. How far such treatment should be carried will depend upon the size of the filtering medium and the economy of construction and operation of the treatment plant. Washing clogged filtering material should be avoided if possible. Grit chambers and coarse screens alone will ordinarily be insufficient. Fine screens have been advocated in certain instances, although it is doubtful if the solids which would pass such screens but would be removed by sedimentation-tanks can be as economically handled by filters as by preliminary tank treatment. If tanks are provided they should be designed and operated to take out all the settling solids it is practicable to remove.

A disadvantage of the septic tank for preliminary treatment is that foul odors are often given off by the tank effluent. Spraying the effluent on trickling filters is particularly favorable to disseminating odors. Further there is a general belief that sewage can be purified by bacterial agencies more advantageously when fresh than when in a septic state.

The Royal Commission on Sewage Disposal reached the following conclusion:[1]

"In the absence of special circumstances favoring a particular plan, it would appear that there is very little difference in annual cost between the various methods of tank treatment followed by filtration through percolating filters, assuming that the kind of filter adopted in each case is that which is best adapted to the particular tank treatment provided."

Sedimentation of Effluent.—Notwithstanding losses due to digestion as much suspended matter may be expected in the effluent as in the influent in some cases; indeed, colloidal matter is often precipitated within the filter so that the effluent averages higher in suspended matter than the influent in such cases. This difference may be further augmented by bits of organic growths, worms, and the like carried through the filter.

In appearance, the suspended matter of trickling filter effluents is quite different from that in sewage. It is of a granular and gelatinous nature instead of the slimy, mucilaginous character of sewage sediment. The increase in stability of the suspended matter in trickling filter effluent over that in the applied sewage was shown by an experiment at the Lawrence Experiment Station in which 0.2 g. of each sediment was mixed with 4,000 c.c. of river water saturated with oxygen (1908 *Report*, Mass. State Board Health, p. 376). The results are given as follows:

	Dissolved Oxygen at End of 5 Days; Percentage of Saturation
River water	90.0
Trickling filter 135 sediment and river water	100.0
Trickling filter 136 sediment and river water	76.0
Sewage sediment and river water	1.5

It seems to be clearly established that the deposit from trickling filters is much more stable than that from sewage; yet it appears that the effluent may carry, at times of unloading, such large quantities of this matter that the water is rendered putrescible thereby. Sludge resulting from the sedimentation of trickling filter effluents has usually been found to be putrescible. In some cases it has been rendered offensive by the presence of many decaying worms or by particles of organic growths.

In the great majority of trickling filter installations it will be

[1] *Fifth Report*, p. 43.

necessary to remove suspended matter from the effluent by tanks, strainers or filters. Much of the suspended matter lends itself readily to sedimentation, but there is considerable fine flaky matter which does not settle readily. Quiescent sedimentation is required to remove a large percentage of this matter, and for complete removal, strainers or filters must be used. In the latter case it will probably be economical to pass the trickling filter effluent through sedimentation tanks first. Intermittent sand filters will not only remove this suspended matter but will afford further bacterial purification and may be operated at comparatively high rates, as at Gloversville, N. Y.

Siphonic-dosing Apparatus.—Automatic air-lock siphons used in conjunction with specially designed dosing tanks afford a satisfactory means of regulating the head on pressure nozzles. The siphons for the purpose are similar to those used in dosing intermittent sand filters. The dosing tanks are used in order to obtain the necessary fluctuation in head on the nozzles, so as to distribute the sewage as uniformly as possible over the bed. This head ranges from a maximum of 5 to 10 ft. down to a minimum of 1 or 2 ft. The usual form of dosing tank is hopper shaped or is tapered by either sloping side walls or steps, so that the capacity of the upper portion of the tank is much greater than that of the lower portion. This causes the nozzles to discharge longer under the higher heads and throws a larger proportion of the spray to the large outer areas.

The size of the siphon should be governed by the maximum rate of discharge of the nozzles and the allowable loss in head by friction in the siphon at that rate. In general the loss of head at the maximum rate of discharge should be definitely limited and for average conditions 0.5 ft. may be all that can be allowed. In making such computations, allowance should be made for losses from the following sources: (1) friction in equivalent length of straight pipe due to roughness of surface; (2) head lost at entrance; (3) head lost in changes of direction or bends; (4) head lost in sudden enlargements of the section; (5) head lost in sudden contraction of the section; (6) head lost at obstructions such as valves; (7) excessive surging due to impact of sudden discharge.

The head lost in entrance may be materially reduced by using rounded orifices and bellmouth openings. Sudden changes in sectional area should be avoided as far as possible and where a change in section is necessary it should be a gradual one.

An example of such a dosing installation is given in Fig. 202. The treatment works at Fitchburg, where this installation was made, were built to care not only for the normal daily sewage but also for excessive rates of flow during storms when considerable surface water is admitted to the sewers. The dosing apparatus is designed to shut off the inflow to one of the two dosing tanks while the siphon is discharging and allow the other tank to be filled, thus assuring the same size of dose and the same distribution of the sewage on the filter at each dose, regardless of the rate

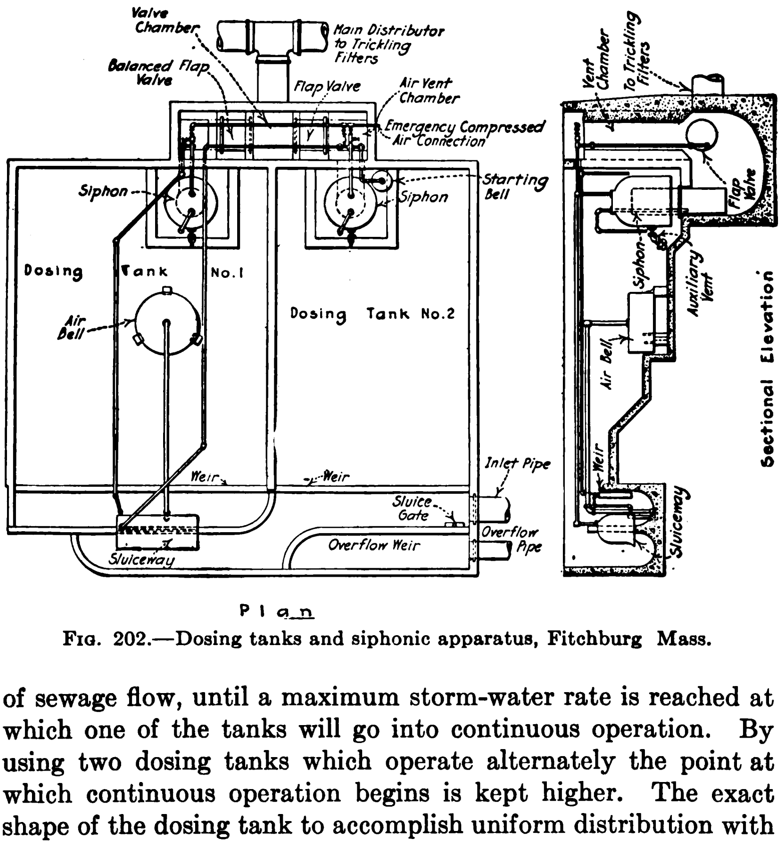


FIG. 202.—Dosing tanks and siphonic apparatus, Fitchburg Mass.

of sewage flow, until a maximum storm-water rate is reached at which one of the tanks will go into continuous operation. By using two dosing tanks which operate alternately the point at which continuous operation begins is kept higher. The exact shape of the dosing tank to accomplish uniform distribution with a given type of nozzle is largely a matter to be determined by experiment. There are a number of features entering into the design of a large tank which may not affect a small experimental tank. The following points should be considered in designing a large tank and proper allowances made for each:

1. The capacity for each foot in depth required to give uniform distribution of spray.

2. While the siphon is getting under way, the nozzles discharge under heads from zero to maximum head. The quantity thus discharged should be deducted from the respective volumes required at each foot of depth.

3. While the siphon is getting under way, the nozzles discharge under heads from zero to the maximum, thereby reducing the capacity of the dosing tank, drawing down the high-water level and resulting in a lower maximum head on the nozzles than would have been the case if the nozzle discharge did not lag behind the siphon discharge. This is largely due to the inertia of the great mass of water, which requires time to attain the maximum velocity.

4. Provide additional capacity above the maximum flow line for the capacity lost in filling the air vent or other spaces below the hydraulic grade line. The entire distribution system, so far as possible, should be below the minimum hydraulic grade line.

5. After the siphon sniffs, the nozzles will continue to discharge for a short period. This will affect the distribution at low heads.

6. Set the high-water line at the proper elevation to compensate for friction losses in the siphon, distribution system and appurtenances.

7. The maximum high-water level has only slight effect, as it is immediately reduced by the draft from the tank.

8. When the siphon is discharging at low heads, the friction losses are greatly reduced and consequently more of the head is available at the nozzles.

9. If sewage is allowed to enter the tank continuously during discharge, the capacity of the tank at different depths and the distribution are affected according to the rate of sewage flow.

Head-varying Valves.—In 1906 Stearns suggested using a mechanically operated butterfly valve in the delivery pipe to trickling filters as a means of varying the head on the nozzles, and about the same time Weand designed float-operated butterfly valves for use with the trickling filters at Reading, Pa.

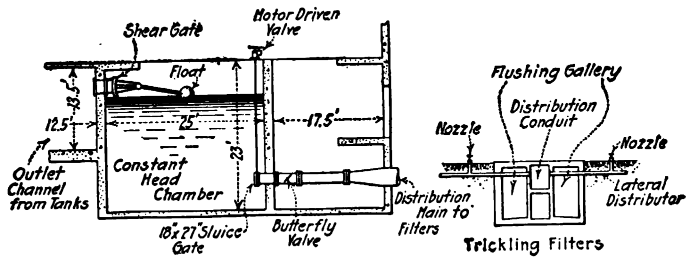


FIG. 203.—Sectional elevation of dosing apparatus, Baltimore.

One of the largest filter plants employing this means of regulating head is that at Baltimore, where the head on the nozzles runs from almost zero to 9 or 10 ft. The installation is shown diagrammatically in Figs. 203 and 204. There are ten 18 by

27-in. rectangular conduit castings, one to each filter bed, leading from the constant head chamber to the distribution system of the

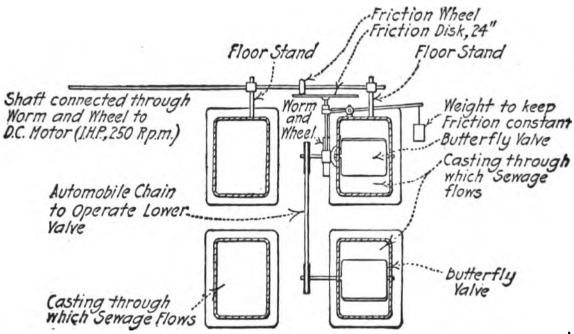


FIG. 204.—Operating device for butterfly valve, Baltimore.

beds. The opening to each of these castings is controlled by an electrically operated sluice gate. Each motor is connected to an auto-starter actuated by a trip on a float rod extending upward from a float in the outlet channel. The trips on these rods are set at four different levels so as to bring sections of the trickling filters into operation one after the other by opening the motor-operated valves. For example, if two units are in operation and sewage begins to back up in the outlet channel faster than it can be taken care of by the area of trickling filters then in service, the float under the third auto-starter is raised sufficiently to trip the latter and open up a sluice gate on the inlet line to section 3 of the trickling filters.

Inside of each rectangular casting above referred to is a flat iron plate swung on its horizontal axis by a shaft extending through a stuffing box to the outside of the casting. This plate fits the inside dimensions of the casting with a clearance of about ½ in. The end of the shaft is geared to a 1-hp. direct-current motor. The plate is turned through 1 revolution (2 cycles) approximately every 7 min., thus alternately damming and releasing the sewage flow to the nozzles. The spray on the filters under this action falls first on the stone immediately adjacent to the nozzle, gradually increasing its breadth to a distance of 7½ ft. away from the nozzle, at which moment the flat plate in the casting has turned till it is in a horizontal position and the maximum amount of sewage is flowing.

Organic Growths and Insects.—Organic growths which appeared on trickling filters at Dorking, England, were removed without impairing the efficiency of the filters by applying a 20 per cent solution of caustic soda at the rate of 1 liter per 1.6 sq. yd. A pinkish, slimy organic growth on the surface of several American trickling filters has been eliminated by applying a strong solution of copper sulphate or by breaking up the surface with a pick and then washing it with a hose. At Philadelphia, the growth was suppressed by a continuous disinfection of the influent, which aided in the purification and in no way interfered with the bacterial efficiency of the filter. In some cases the growths have disappeared after resting the beds for a few days and loosening their surface with a pick.

Great quantities of worms develop in trickling filters, and are discharged with the accumulated organic matter when the filters "unload" these solids into the effluent. The abundance of these worms indicates that they must perform some function in the general transformation of organic matter into more stable matter which goes on inside the filters.

A small, gray moth-fly is sometimes very troublesome about trickling filters, getting into the eyes, nostrils and ears of the filter attendants. At Gloversville, N. Y., they caused trouble at a tannery, 200 ft. from the filters, by lighting on skins while being tanned. Fly larvæ, which clogged the upper part of a trickling filter at Andover, Mass., were removed by applying caustic soda at the rate of 1,000 lb. per acre. Neither copper sulphate nor bleach was wholly successful in killing these larvæ. After the insects were killed, the clogging was removed by digging over the surface and allowing it to rest.

At Plainfield, N. J., the fly maggots have been killed by closing the filter outlets and flooding the beds every seven days during the flybreeding season.

The abundance of flies about these filters often leads to the development of large numbers of spiders, and many other kinds of insects have been observed to flourish at some of these plants.

At Fitchburg an equilibrium appears to have been established between the spiders and the flies, and neither gains the ascendency to such an extent as to be troublesome.

Comparison of Trickling Filters and Contact Beds.—The Royal Commission on Sewage Disposal made the following statements[1] as a result of its investigations of these two methods of treatment:

[1] *Fifth Report*, p. 231.

"The amount of sewage which can be purified per cubic yard of contact bed or of percolating filter varies, within practical limits, nearly inversely as the strength of the liquor treated. This statement is based on the assumption that the size of the material of which the filter is composed is, in each case, suitable to the character of the liquor treated, and that the material is arranged at the proper depth to secure maximum efficiency. Taking into account the gradual loss of capacity of contact beds, a cubic yard of material arranged in the form of a percolating filter will generally treat about twice as much tank liquor as a cubic yard of material in a contact bed. In the case of sewage containing substances which have an inhibitory effect upon the activity of micro-organisms, the working power per cubic yard of filter of either type may be more nearly equal. This point, however, is not clearly established.

"Percolating filters are better adapted to variations of flow than contact beds. Effluents from percolating filters are usually much better aerated than effluents from contact beds, and, apart from suspended solids, are of a more uniform character. On emptying a contact bed, the first flush is usually much more impure than the average effluent from the bed. The risk of nuisance from smell is greater with percolating filters than with contact beds."

The authors believe that with trickling filters of coarse material not subject to disintegration, the evidence seems to indicate that they will be self-cleansing if properly operated, whereas contact beds usually clog periodically. Hence the cost of treatment by trickling filters is usually much less than that by contact beds. The effluent from the trickling filters is ordinarily, more highly nitrified than the effluent from contact beds and after secondary sedimentation is more uniform in quality than contact bed effluent. The trickling filter is better adapted for variations in rates of flow than is the contact bed. The chief advantages in the use of contact beds rather than trickling filters are the relatively low head required, the somewhat simpler method of dosing, minimizing foul odors and avoiding a fly nuisance.

CHAPTER XVIII

INTERMITTENT SAND FILTERS

If sewage is applied intermittently and uniformly over a bed of well-drained sand for a considerable period, two distinct changes in its character take place. The first change is the removal of the suspended matter by the straining action of the sand, and the second change is the oxidation of the organic matter, dissolved as well as suspended, by the bacteria living in the bed. These changes are so complete in a well-built and operated intermittent sand filter that it furnishes, under the most favorable conditions, an effluent which may be said to be literally purified. It is not safe, however, for bacteriological reasons, to regard the effluent from any form of sewage treatment as suitable for drinking and generally such an effluent should be chlorinated or again filtered if it is to be discharged directly into water which may be used for domestic purposes.

Mechanical Straining.—If the suspended matter in the applied sewage contains a considerable proportion of coarse particles they will soon form a thin compact mat on the sand, which intercepts the smaller suspended matter and retards the passage of the liquids into the sand. This mat must be removed to enable the sand filter to function properly. To avoid reduction in filtering capacity due to these mats and to the delays necessary for their removal, it is customary to pass the sewage through sedimentation basins before delivering it to these filters. The settled sewage still contains the fine suspended matter, however, which penetrates the sand to a depth of 1 in. or more. Therefore, while sedimentation reduces the frequency with which the surface mats must be removed it increases somewhat the frequency with which the dirty surface sand must be removed. The advantages of using settled sewage outweigh the disadvantage in most cases.

The straining action of the sand leaves the suspended solids of the applied sewage in the upper part of the filter, where the organic matter is subject to biological action with its accompanying oxidation and liquefaction. The accumulation of solids is more rapid than the oxidation and liquefaction, and consequently the interstices of the sand gradually become filled with very small particles, some of which swell and become gelatinous when wet. This condition not only checks the passage of the sewage into the sand but also hinders the free circulation of air, a serious matter because ample oxygen must be supplied throughout the bed after each dose of sewage, in order that the desirable aerobic bacterial changes may take place.

Oxidation.—In filters of this type in good operating condition, every particle of sewage comes into contact with the sand in such a way that part of the colloidal and dissolved matters may be thrown out of solution by attraction or absorption. These substances with the bacterial growths form a gelatinous film over each grain, which is apparently able to absorb oxygen at a rapid rate. When the bacterial conditions in this film have been established properly and the filter is not overdosed, the effluent is much more mineralized and nitrified than the average effluent from a trickling filter. To bring a sand filter to this condition will take from a week in hot weather to several months in cold weather.

The formation of nitrates indicates the biological activity of sand filters. Nitrogenous matters are decomposed in the filters by the processes of either putrefaction or decay, the former occurring only when the filters are prevented by overdosing or otherwise from receiving an adequate supply of oxygen. These processes convert part of the organic nitrogen into free ammonia and part into nitrogen gas. If a filter has been yielding a well-nitrified effluent and suddenly gives one with high free ammonia, it is probably being overdosed or the supply of air is checked by clogging of its pores. The increase in free ammonia may be due to reduced nitrification, or occasionally, to a reduction of the nitrates. Nitrification increases and decreases with the rise and fall of the temperature, while the inverse change takes place in the free ammonia content of the effluent.

Filter Sand.—Where suitable sand or gravel is available in place, a filter bed can be constructed by grading its surface to receive sewage. Loam and silt have a tendency to hold water by capillarity, which prevents aerating a filter of such materials unless it is operated at a very low rate. Clay, cementitious sand and other relatively impervious materials are useless for intermittent filters.

Loam and subsoil must be removed if a filter is to be operated at more than a low rate. The extent to which subsoil is removed is usually determined by the cost of the work. Roots of trees and the fine earth about them must be removed, and the same is true of large stones. The aim should be to obtain a filter bed of uniform porosity, so that differences in the porosity will not cause overloading at places where the sewage passes through the sand most freely, or undesirable conditions in the bed due to lack of uniformity in the sand throughout its depth. An intermittent sand filter is not an inert mass, like a machine, but is the home of living organisms which do the work for which the bed is built. The better the home and the better the treatment these organisms receive, the better results they will give, as is the case with any other working colony.

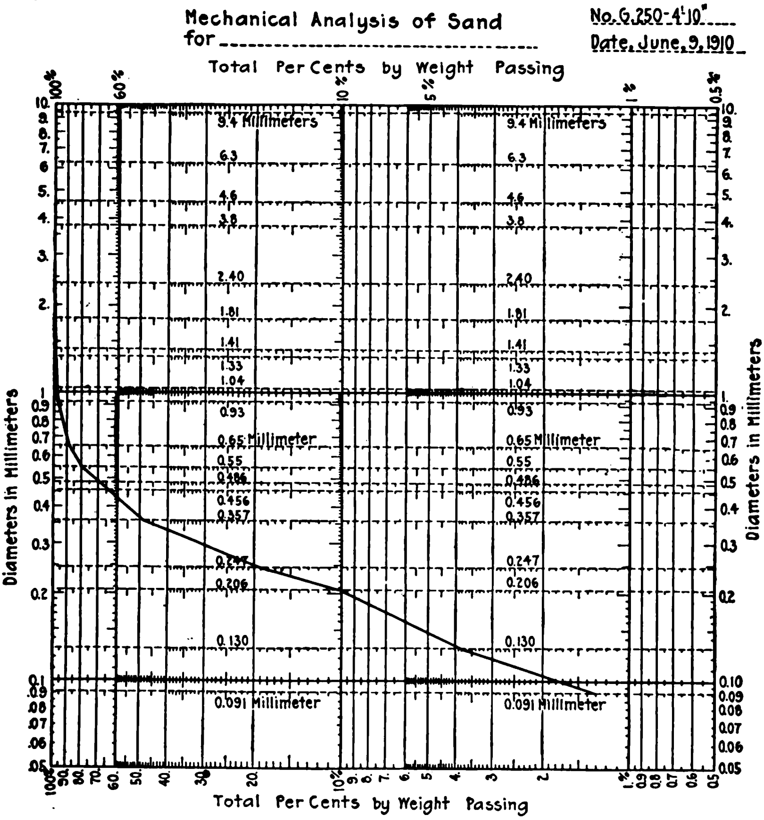
Effective Size and Uniformity Coefficient.—As a ready means of determining the comparative value of sands, the standard developed by Hazen when in the employ of the Massachusetts State Board of Health (1892 *Report*, p. 541), defining the "effective size" and the "uniformity coefficient," has been found most useful.

"As a result of experiments made at the Lawrence Experiment Station we have a standard by which we can definitely compare various sands. The size of a sand grain is uniformly taken as the diameter of a sphere of equal volume, regardless of its shape. As a result of numerous measurements of grains of Lawrence sands, it is found that when the diameter, as given above, is 1, the 3 axes of the grain, selecting the longest possible and taking the other two at right angles to it, are, on an average, 1.38, 1.05, and 0.69, respectively, and the mean diameter is equal to the cube root of their product.

"It was also found that in mixed materials containing particles of various sizes, the water is forced to go around the larger particles and through the finer portions which occupy the intervening spaces, so that it is the finest portion which mainly determines the character of the sand for filtration. As a provisional basis which best accounts for the known facts, the size of grain such that 10 per cent by weight of the particles are smaller and 90 per cent larger than itself is considered to be the effective size. The size so calculated is uniformly referred to in speaking of the size of grain in this work.

"Another important point in regard to a material is its degree of uniformity, whether the particles are mainly of the same size or whether there is a great range in their diameters. This is shown by the uniformity coefficient, a term used to designate the ratio of the size of grain which has 60 per cent of the sample finer than itself to the size which has 10 per cent finer than itself."

Mechanical Analysis of Sand.—Sets of sieves having several sizes of screens, as commercially obtainable, are usually made to nest one over the other, the coarsest on top. The nest is de-



Mechanical Analysis

Date of Collection, June 9, 1910				
Date of Analysis, June 15, 1910				
	Size of Sieve		Amt. Passing	
	Sieve	M.M.	Grams	Per Cent
Woven Brass Wire	200.	0.091	2.6	1.3
	140.	0.130	7.5	3.8
	100.	0.206	20.0	10.3
	80.	0.247	38.3	19.6
	60.	0.357	94.0	48.2
	Cu. 40.	0.456		
	40.	0.486	136.5	70.0
	35.	0.55	157.1	80.6
	30	0.65	174.5	89.5
	Cu. 26	0.66		
	25.	0.93	188.4	96.7
	20.	1.04	191.5	98.3
	16.	1.41	193.7	99.2
Punched Metal	0.057"	1.33		
	0.079"	1.81		
	0.103"	2.40	194.3	99.7
	0.156"	3.8		
	0.187"	4.6		
	0.25"	6.3		
	0.375"	9.4		
Total			194.3	99.7
Loss			0.7	0.3
Amount Sieved			195.0	100.0
60% Finer than 0.42 mm.				
10% Finer = Eff. Size 0.20 mm.				
Uniformity Coefficient 2.1				

Analyst S.A.W.

Fig. 205.—Form for recording mechanical analysis of sand.

signed to fit in a mechanical shaker. The openings in screens of the same nominal number of meshes per linear inch vary with different makes of wire cloth and even with cloth woven by the maker at different times. It is necessary, therefore, to rate each screen by placing about ¼ lb. of thoroughly dried sand in the top sieve and shaking the nest until only a little sand remains upon each sieve. Each sieve is then shaken by hand over a sheet of paper, the grains which fall through giving a measure of the screen through which they pass. Their average diameter is obtained by weighing a number of them on a laboratory balance and applying the formula

$$\text{diam. in mm.} = \sqrt{\frac{6}{\text{sp. gr.} \times \pi}}\sqrt[3]{w} = 0.9\sqrt[3]{w}$$

w = weight of one grain in milligrams.

In a general way a 200-mesh screen will have openings of about 0.10 mm.; 140-mesh, 0.13 mm.; 100-mesh, 0.17 mm.; 50-mesh, 0.33 mm.; 40-mesh, 0.48 mm.; 30-mesh, 0.63 mm.; and 20-mesh, 0.95 mm. The screens used by the authors, as shown on Fig. 205 illustrating their form for recording the results of mechanical analyses of sand, will be seen to vary somewhat from the above figures.

About ½ lb. of sand is usually sufficient for a mechanical analysis. After drying the sample is usually placed in the top sieve of the nest and shaken down through the series. If the sample contains much dust which adheres to the sand grains or to the wires of the screens, it is sometimes desirable to screen the full sample independently through the finest screen, the residue through the next coarser screen, continuing until the 80-mesh has been used in this way. Then proceed with the usual method, introducing the residue into the top of the nest of sieves, and shaking until separation into the several larger sizes is complete. It is well to adopt a definite number of shakes in a given time, to secure uniformity of results for comparison. The extent of shaking required varies with different machines and can be determined by examination, to ascertain whether the sand has been properly segregated into its component sizes.

Depth of Bed.—The greatest bacterial activity of the filter is in the upper portion of the bed, particularly in the top 6 to 12 in. Less purification is effected by each successive foot in depth. Nevertheless, it has been found advantageous practically to build filter beds 3 ft. and preferably 4 ft. deep above the underdrains, in order to prevent the sewage from breaking through the bed and reaching the underdrains in an improperly purified state. The greater depth has thus a steadying effect upon the purification effected by the bed, and the efficiency of the process.

Clark stated in the 1908 report of the Massachusetts State Board of Health, p. 304, that, other things being equal, the filters of greater depth gave effluents of higher purification than those of less depth. With coarse sands, 0.25 mm. effective size or larger, 4 or 5 ft. in depth of bed was desirable. A filter 10 ft. deep gave somewhat better effluent, but not markedly so. Shallow filters (2 ft.) of coarse sand gave fair results when operated at low rates.

Filters composed of fine sand must be deep enough to overcome capillarity which, at 0.04 mm. effective size or larger, will raise water 2 ft. more or less in the bed, so as to provide unsaturated sand layers near the surface. With filters composed of fine sand, when the matter of capillarity is taken care of by providing sufficient depth, the effluent is better than that from a filter of coarse sand of equal depth. These filters of fine sand are, however, operated with more difficulty.

Rate of Filtration.—The volume of sewage which can be purified per acre of any given filter bed depends primarily upon the amount and character of the organic matter present in sewage. In practical sewage treatment the flow of sewage fluctuates greatly and there is usually a decided fluctuation in its quality. If a filter plant is designed to handle only the average flow of sewage the excess flow received sometimes must be discharged without treatment. In Massachusetts, where intermittent filtration is carried on more extensively than elsewhere, because of the availability of suitable sand and the strong impetus given to this method of treatment by the investigations of the Massachusetts Board of Health, the high rates of flow come at the end of winter when the beds are generally in the worst condition, before the spring cleaning of the surface has been done and while the beds are not well aerated. On the other hand, experimental filtration plants are kept in good working condition all the time, and while they are dosed at different rates, these rates are not allowed to exceed those which will give satisfactory effluents, except for short periods. For these reasons the practical difficulties of operating intermittent sand filters make it necessary to use great caution in applying the results of experiments to actual municipal problems. The data given in Table 61 give an idea of practical operating conditions in Massachusetts.

TABLE 61.—INTERMITTENT FILTERS OF MASSACHUSETTS TOWNS AND CITIES
(From Report of State Board of Health, 1913, pages 367, 368)

Place	Population, 1910	Sewage, gal. per day	Filter beds: Number[1]	Filter beds: Net area, acres	Rate, gal per acre per day	Persons per acre	Underdrains: Depth, feet	Underdrains: Spacing, feet	Filtering material
Amherst	5,112	375,000	6	2.00	188,000	2,556	4-4.5	25	Rather fine sand found in place.
Andover	7,301	350,000	20	3.65	96,000	2,000	4	20	Fair sand, small quantity of gravel, practically all handled in construction.
Attleboro	16,215	250,000	26	15.50	16,000	1,046	4-7	35	Excellent sand and gravel found in place.
Brockton	56,878	2,175,000[2]	37	37.00	59,000	1,536	5.5	30	Good sand and gravel found in place.
Clinton	13,075	1,090,000[3]	27	26.23	42,000	498	8	60-70	Good sand and gravel found in place.
Concord	6,421	422,000[3]	4	3.30	128,000	1,945	none	....	Good sand underlaid with gravel found in place.
Framingham	12,948	700,000[3]	20	20.75	34,000	623	...[4]..	....	Surface material badly impregnated with organic matter; good underlying sand. Beds prepared by removal of trees and stumps and leveling where necessary.
Gardner									
Gardner } Gardner	14,699	135,000	20	2.50	54,000	1,175	5	20	Good sand, handled in construction.
Gardner } Templeton		600,000	26	10.00	60,000		3-4	20-30	Coarse sand, handled in construction.
Hopedale	2,188	150,000[3]	7	3.25	46,000	674	4	35-60	Some good sand, some rather fine sand, some ledge; material found in place.
Hudson	6,743	276,000	23	9.00	31,000	750	5-6	50-100	Good sand and gravel found in place.
Leicester	3,237	70,000	8	0.36	194,000	9,000	4	8	Hard, compact sand found in place.
Marion	1,460	110,000[3]	8	0.66	167,000	2,210	5	....	Mostly good sand, with pockets of fine sand and some ledge; mostly found in place.
Marlboro	14,579	750,000	33	20.90	36,000	698	4.5-6	30-50	Rather fine sand found in place.
Milford	13,055	513,000	15	9.30	55,000	1,405	5	40	Rather fine sand found in place.
Natick	9,866	720,000[3]	14	12.60	57,000	782	6	36	Good sand with strata of very fine sand in places; found in place.
North Attleboro	9,562	600,000	16	7.00	86,000	1,366	......	....	Very coarse sand and gravel.
Northbridge	8,807	244,000	24	6.00	41,000	1,468	4	50-75	Coarse sand and gravel found in place.
Norwood	8,014	500,000	6	6.64[5]	75,000	1,206	4-6	40	Good sand and gravel found in place.
Pittsfield	32,121	1,903,000[3]	35[6]	25.90	74,000	1,240	4	35	Good sand and gravel found in place.
Southbridge	12,592	900,000	11	8.50	106,000	1,481	4	40	Fair sand and gravel; some found in place, considerable quantity handled.
Spencer	6,740	450,000	12	9.30	48,000	725	...[7]..	....	Good sand and gravel, mostly found in place.
Westboro	5,446	425,000	12	3.80	73,000	1,435	5	30-40	Good sand and gravel handled during construction.
Worcester	145,986	3,860,000[8]	75	74.30	52,000	....	4-6	35-50	Good sand and gravel, largely found in place.

[1] Not including sludge beds. [2] Amount pumped to filters; about 800,000 gal. daily treated during October and November by half-acre trickling filter, sedimentation basin and sand filter. [3] Amount pumped. [4] Very little underdrainage; one line extends through certain beds at depth of 4 to 6 ft.; 40 ft. apart in one bed. [5] Three acres of filters under construction. [6] Includes sludge beds. [7] Only 3 beds underdrained. [8] Amount treated by intermittent filters only, day sewage.

The sewage should be applied at the rate of 1 cu. ft. per second for each 5,000 sq. ft. of area, and the authors prefer to continue the application until the depth of sewage on the filter is about 3 in., so that all parts of the surface will be covered. If a bed having a capacity of 30,000 gal. per acre per day is dosed in this way, it must be allowed to stand idle 2 days after each day when it is dosed.

Size, Shape and Grouping of Beds.—In the larger plants rectangular or square beds having an area from ¾ to 1 acre have generally proved most desirable. In smaller plants the size may be much smaller to avoid throwing a large proportion of the area out of use when cleaning beds and to facilitate dosing without storing the sewage too long.

The areas of some of the leading Massachusetts filters are given in Table 61. Probably in no case is the entire population served by the sewer system. Some of the plants are doing excellent work while others are apparently overloaded, poorly operated or otherwise prevented from attaining efficiency. Experience indicates that the settled sewage of 750 persons, excluding storm water, is a reasonable load for 1 acre of filters in Massachusetts, although when scientifically operated filters may satisfactorily carry a somewhat greater load, under favorable conditions. As a matter of fact it is possible to treat a very much greater quantity during warm dry weather but the working plant must be so designed as to care for the sewage under the most unfavorable conditions. It should have sufficient capacity to allow 1 or 2 beds at a time to be cut out for necessary cleaning or repairs.

Except in very small plants, intermittent filter beds are rectangular in shape when the topography permits. The cost of embankments, which are usually made of the material stripped from the beds, is so small that it is not often a factor to be considered in the determination of the shape.

The underdrainage system and the distribution of sewage over the bed are usually of most importance in determining the shape. With rectangular beds it is practicable to flood them from the four corners satisfactorily, even when they are of large size, but the underdrainage is likely to cost more than when long beds are used of such width that two can be drained by laterals running to a main drain laid in the embankment between the beds. The distribution of sewage over long beds is not so uniform as it is over square beds unless troughs are used, which operators dislike.

As a general thing, several arrangements of beds, distributing conduits, drains and roads are practicable, and some preliminary studies will be needed to determine which is best. The cost of the whole installation rather than that of one or two beds should be the deciding factor, since main drains or main carriers may prove unexpectedly expensive if judged by an examination of the needs of one or two beds.

Surface of Bed.—Filter beds are generally graded substantially level. There is little advantage in sloping the surface under ordinary conditions, for if the discharge of sewage on to the bed be rapid, satisfactory distribution will be obtained with a level bed.

Banks and Roads.—For convenience of access and for the removal of surface deposits it is necessary to provide roadways between successive rows of filters. For these roadway embankments a top width of about 8 ft. has been found to be sufficient. The height of the embankment over the bed will be determined by the fall required for the main sewage distributors, which can be laid with shallow cover, however, on account of the warmth of the sewage carried by them, 24 in. in depth usually being sufficient cover. The side slopes of these embankments should be made steep, as a matter of economy. Ordinarily a slope of 1:1 can be maintained but 1 vertical to 1½ horizontal is preferable. The subsoil and loam stripped from the surface of the beds is used for the embankments, which are grassed over to reduce the cost of maintenance.

A sloping driveway should be provided, leading into each filter bed. It will be found convenient to group these driveways in such a way as to lead from the roadway into four beds at their adjacent corners.

Partition banks should be low and narrow, to economize area. A height of 18 in. and a width on top of 24 in. are generally enough.

Underdrainage.—In some cases, particularly where deep deposits of sand and gravel occur, no underdrains are necessary.

Underdrainage by blind drains formed of large stones is undesirable, because they cannot be cleaned without digging down to them and because they do not afford a means for the effluent to flow off rapidly.

Vitrified sewer pipe makes the most satisfactory underdrains, because it is durable and easily cleaned. It should be laid true to line and grade. The bottom of the bed, when constructed artificially, should either be level or sloping toward the underdrains. With artificial beds, it is desirable to lay the underdrains in trenches below the bottom of the sand, so as to make the entire depth of sand effective for filtration and keep the drains well below the surface. If possible the drains should have a free outlet, but submerged outlets will operate satisfactorily and sometimes trapped outlets are advantageous. In laying the pipe, the spigot end should be about ⅜ in. from the shoulder of the socket to allow the water to flow readily into the pipe. The authors break off the socket on the upper part of the pipe, Fig. 206, leaving the lower part of the socket to keep the pipe in alignment. The drain is then surrounded with screened gravel, with

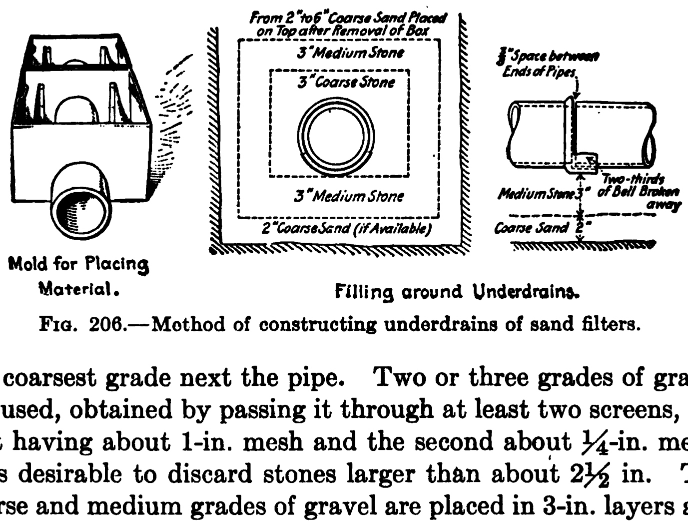


FIG. 206.—Method of constructing underdrains of sand filters.

the coarsest grade next the pipe. Two or three grades of gravel are used, obtained by passing it through at least two screens, the first having about 1-in. mesh and the second about ¼-in. mesh. It is desirable to discard stones larger than about 2½ in. The coarse and medium grades of gravel are placed in 3-in. layers and unless the sand of the bed is very coarse, a third layer of the finest size is required. The layers of gravel should surround the pipe and not merely cover it. The box shown in Fig. 206 is helpful in placing the gravel rapidly and accurately.

Underdrains are usually laid from 3 to 4 ft. deep at the upper end, on flat grades with about 6 in. in 100 ft. as a minimum. The usual spacing of the underdrains is about 40 ft. but if the sand has an effective size of 0.08 to 0.12 mm., the spacing should be about 30 ft. The minimum size for an underdrain is 4 in.

The underdrainage pipe should be connected with a main arterial system of underdrains, which may be laid either with tight or with open joints, the size of the pipe depending upon the quantity of water to be handled, the minimum diameter usually being 8 in. The outfall into the stream or open channel may advantageously be provided with a headwall of masonry. If the lines of the underdrains themselves are long, manholes or

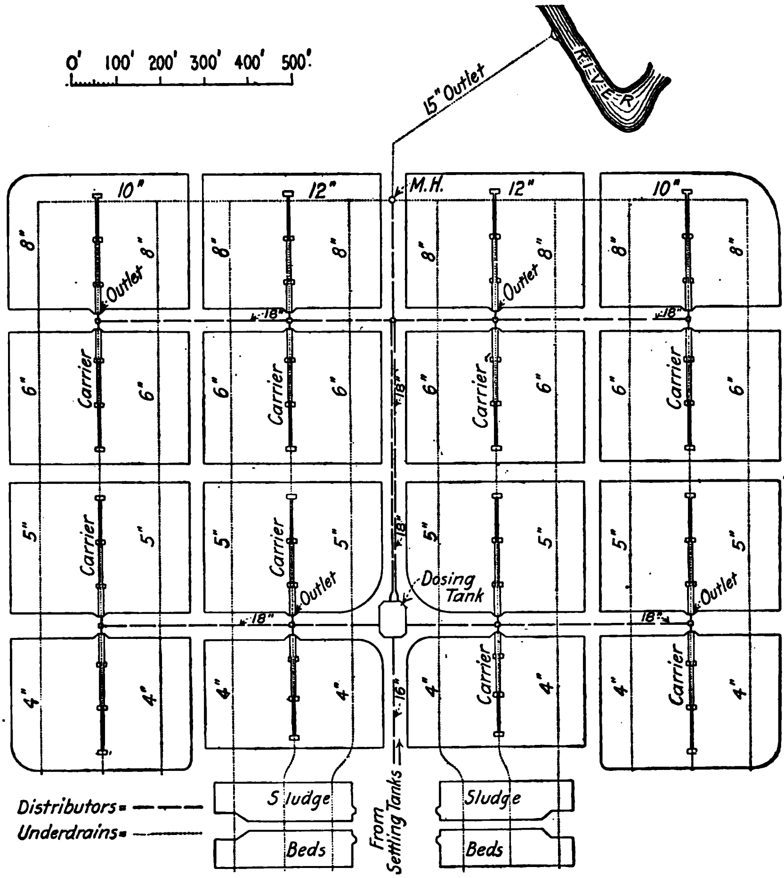


Fig. 207.—Arrangement of sand filter beds, North Attleboro, Mass.

lampholes may be desirable at convenient intervals and provision should be made for sampling the effluent from each bed in small plants and each group of beds in larger ones.

Distribution of Sewage.—The best method of controlling the distribution of the sewage will depend upon the size of the works and the topographical condition. In large works sewage may be applied simultaneously to the beds of a group, by master valves at central points, each outlet for the sewage on a bed being controlled by a gate on the lateral distribution pipe. In small filter plants, the distribution is usually controlled by gates at the individual beds. The sewage is usually distributed through vitrified-pipe lines, but if it is pumped to the beds cast-iron distributing pipe is used, in order to with-

Fig. 208.—Intermittent filter with sewage distributor, Marlboro, Mass.

stand the pressure. As the doses of sewage must be applied quickly, 1 cu. ft. per second for each 5,000 sq. ft. of area, the pipe sizes must be liberal.

The sewage is distributed over the beds by the following methods:

1. Graduated troughs running nearly the full length of the bed, as shown in Figs. 207 and 208.

2. Fan-shaped or arterial troughs, used particularly for irregular-shaped beds.

3. Quarter-point distribution, which consists of the discharge of sewage at the two quarter points on the long sides of the bed.

4. Corner distribution, in which the sewage is discharged on to the bed at or near its four corners.

5. One or two middle distributors, located upon the long side of the bed. This method, which was used for a time at Clinton, Mass., has been abandoned there as unsatisfactory.

The discharge of sewage at these distribution points should be controlled in a manhole, from which the distributor branches, by means of a shear or sluice gate. Of these the simple shear gate is the cheaper and has been found on the whole to give reasonably satisfactory service.

It is desirable to provide headwalls and a paved area at the point of discharge of the sewage to prevent erosion of the surface of the bed.

Automatic Dosing Apparatus.—There are three classes of such apparatus used with intermittent sand filters: air-controlled siphonic apparatus, mechanically-controlled siphonic apparatus, and mechanical devices.

Air-controlled Siphonic Apparatus.—The main feature is the siphon. Its principal parts, Fig. 209, are the main trap, a pipe casting with the long leg extending above the bottom of the dosing tank and the short leg connected to the discharge pipe to the filter; the bell, a cylindrical casting set over the long leg of the main trap and supported on legs or piers above the tank floor; the vent pipe, and the blowoff trap, made up of small galvanized wrought-iron pipe.

The main trap, immediately after the siphon has ceased discharging, stands full of water to the level B_1. The blowoff trap is also full to the level D_1. The vent pipe is empty. Sewage flows into the dosing tank and the water level rises until the open end of the vent pipe is reached at A. The vent pipe then becomes full of water and the siphon is sealed against the escape of air confined in the bell and upper part of the long leg of the main trap. As the water in the dosing tank continues to rise, it exerts a pressure upon the air confined in the siphon and forces the water in the long leg of the main trap down toward the lower bend. The water in that portion of the blowoff trap under the bell is likewise forced down. At the same time the water level inside the bell rises. Just before the discharge level in the dosing tank is reached, the water level in the blowoff trap is at D_2, in the main trap at B_2, and in the bell at C. A slight further rise of the water in the dosing tank forces the seal in the blowoff trap, thus releasing the air confined in the bell and causing a sudden inrush of water from the dosing tank into the bell, which sets the siphon into full operation. The sewage in the dosing tank is discharged through the siphon until the level is at the low-water line at the lower bend of the vent pipe, when air is drawn into the bell through the vent pipe, the siphonic action is broken, the bell is filled with air, the discharge ceases, and the main trap and blowoff trap are refilled with water. The dosing tank then fills again and the siphon is ready for another discharge.

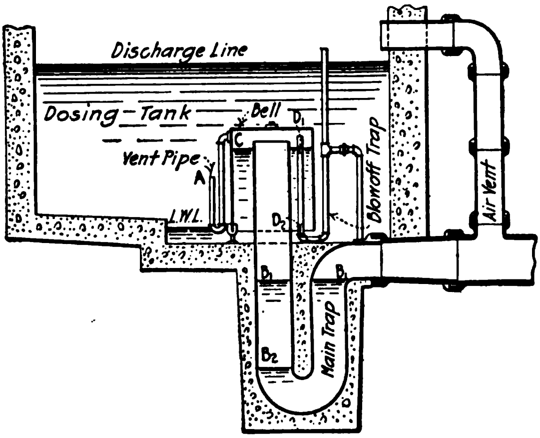


FIG. 209.—Siphon used in dosing intermittent filters (Miller).

The air vent in the discharge pipe line, although not necessary for the working of the siphon, allows the escape of air previously confined in the bell and prevents trouble from air in the pipe line. The small siphons, 3 to 8 in. in diameter, in some cases do not require blowoff traps to ensure their working.

Where it is desirable to dose two or more filter beds in rotation, this can be done by installing several siphons, each connected to a filter bed and arranged to discharge in rotation automatically. Two siphons of the type illustrated by Fig. 209, set side by side in a dosing tank, will operate alternately without special piping. For three or more alternating siphons, a special system of piping with starting bells or other controlling devices, is required.

E. G. Bradbury gives[1] the following formula as furnishing

[1] *Report* Ohio, Eng. Soc., 1910, p. 79.

a close estimate of the discharge of siphons under varying heads:

$$Q = 0.4A\sqrt{2gh}$$

where Q = discharge in cubic feet per second.
A = area of discharge pipe in square feet.
h = average head in feet or half the vertical height from high water in tank to center of outlet.

Mechanically-controlled Siphonic Apparatus.—A variety of mechanical devices are used for directing the discharge of

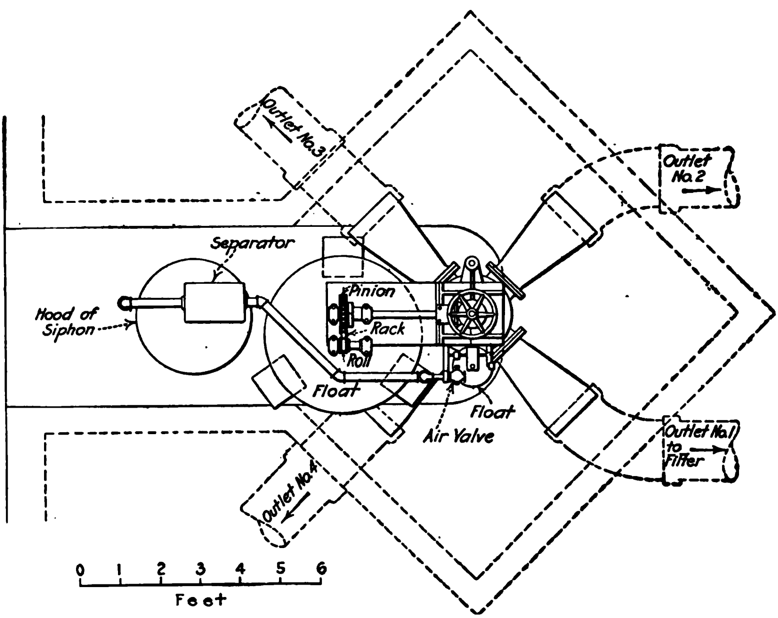


FIG. 210.—Plan of dosing apparatus at North Attleboro, Mass.

siphons to one pipe after another. Figs. 210 and 211 illustrate one used at a number of sewage treatment works by Barbour.

The main features are an air-lock siphon controlled by an air valve and float, and a revolving cylindrical gate valve, also actuated by a float, the whole apparatus being set in the dosing tank. As the level of the sewage in the dosing tank rises, it lifts the large float A to which is attached a rack and pinion, causing the cylindrical gate valve to turn until the opening in the gate is opposite one of the outlet pipes to a filter bed or group of filter beds. At the same time, float B rises until the high-water level in the tank is reached, when the trip *E* on the float rod opens the air valve *D*, suddenly reducing the air pressure in the siphon bell and causing the siphon to come into full discharge. In case the trip failed to open the air valve, an additional rise in the water level would bring the siphon into full discharge by the aid of the pilot pipe, in a manner similar to that previously described. The siphon continues to discharge until the low-water level in the tank is reached, when air is drawn in under the edge of the siphon bell, the siphonic action is broken, and the dis-

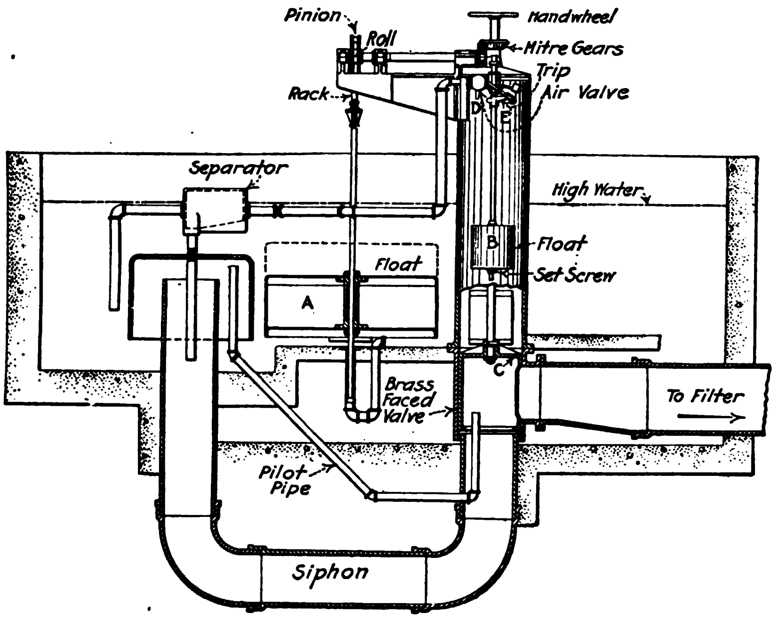


FIG. 211.—Sectional elevation of Barbour dosing apparatus at North Attleboro, Mass.

charge ceases. When float *A* falls with the sewage level, the cylindrical gate valve is not turned back, but is held in place by a pawl and ratchet on the pinion. The tank then fills again, raising float *A*, which turns the cylindrical gate valve around to the next outlet pipe, and the process is repeated as described. The advantage of the small float *B* and air valve is that the size of the dose can be regulated by setting the float *B* higher or lower on the float rod. The action is positive and productive of more satisfactory results than when the pilot pipe is relied upon to start the siphon. The "separator" is necessary to prevent suspended matter from getting into the air valve.

Mechanical-dosing Apparatus.—An example of mechanical-dosing apparatus is shown in Fig. 212. Sewage flows through the settling tank into a dosing chamber containing a float. A chain attached to the float passes over a sprocket wheel, on the shaft

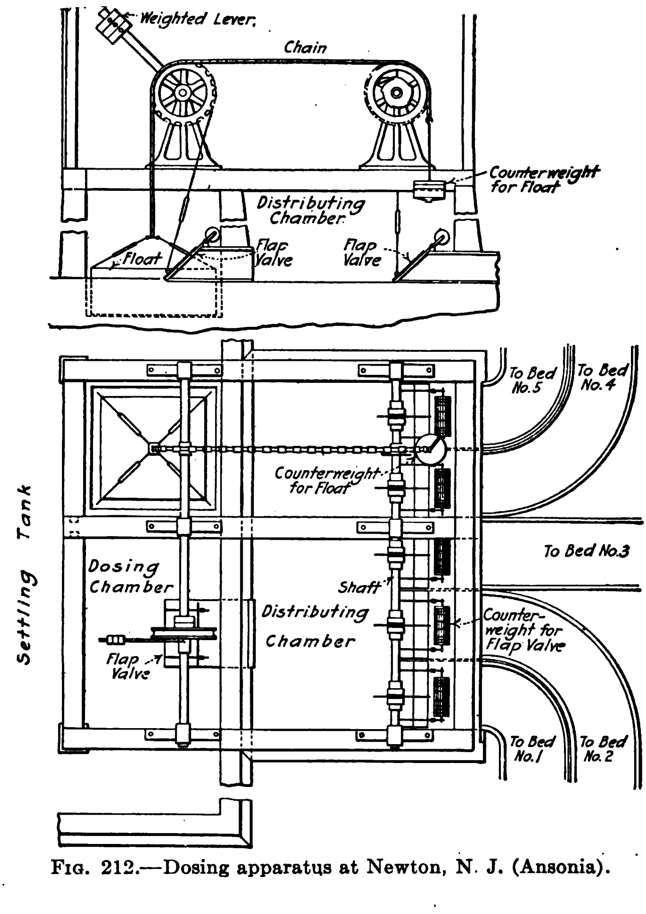


Fig. 212.—Dosing apparatus at Newton, N. J. (Ansonia).

of which is a weighted lever. As the sewage in the dosing tank rises, the float rises and the shaft revolves, bringing the weighted lever to a vertical position. As soon as the shaft turns so as to bring the center of gravity past the vertical, the weighted lever falls on the opposite side and the flap valve between the dosing chamber and the distributing chamber is suddenly opened.

Each rise of the float revolves the countershaft to which the distributing gates are attached, a fifth of a revolution, the fall of the float failing to revolve the shaft in the opposite direction on account of the pawl and ratchet. The five flap valves leading to the five filter beds are each attached to the shaft at points equally spaced around the circumference, and thus at each one-fifth turn of the shaft a new gate is opened, the other four remaining closed. In this way the doses are distributed to each filter bed in succession.

This apparatus is readily adjustable and in general has been found to work satisfactorily. Care should be exercised in designing apparatus of this type to provide parts of sufficient strength to withstand the shock due to the suddenly applied force of the falling weight and the resulting jerk on the flap valves.

Efficiency of Intermittent Sand Filters.—When sand filters are doing their best work the effluent from the underdrains will

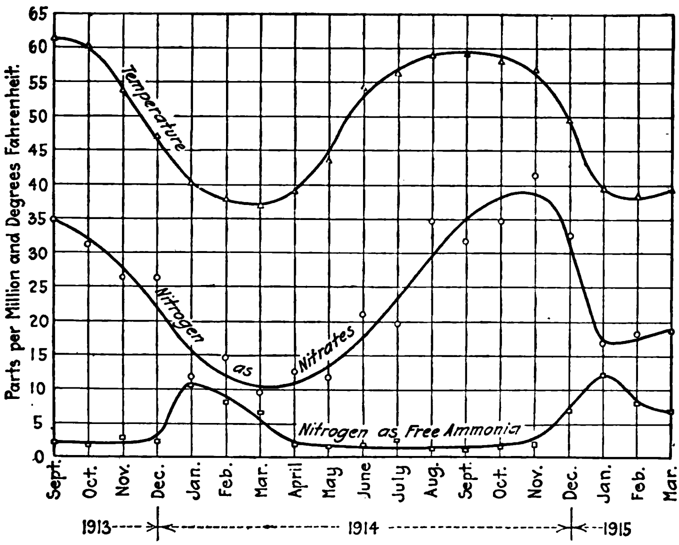


FIG. 213.—Effect of temperature on results of intermittent filtration.

be clear, substantially free from suspended matter and practically odorless. It will remain stable indefinitely, even when allowed to stand in a tightly stoppered bottle in a warm room.

The efficiency of various sand filters in Massachusetts is shown in Table 62. These results are for the most part yearly averages of monthly samples. It will be seen that there is

TABLE 62.—EFFICIENCY OF SAND FILTERS IN MASSACHUSETTS*

Results of analyses in parts per 1,000,000

City or town	Free ammonia			Total albuminoid ammonia			Oxygen consumed			Nitrogen in eff. as		Chlorine	
	Applied sewage	Effluent	Per cent reduced	Applied sewage	Effluent	Per cent reduced	Applied sewage	Effluent	Per cent reduced	Nitrates	Nitrites	Applied sewage	Effluent
Amherst	25.6	7.3	71	8.8	0.55	94	53.3	6.2	88	6.10	0.11	234.1	86.0
Andover	35.0	19.9	64	8.4	1.11	87	61.8	10.4	83	1.20	0.07	61.8	68.5
Brockton	67.7	31.8	53	13.3	1.12	92	128.9	15.3	88	4.07	0.12	122.3	120.3
Clinton	38.4	6.6	83	6.9	0.66	90	69.5	8.0	89	8.91	0.07	59.1	52.7
Concord	21.4	2.5	88	5.6	0.12	98	43.2	1.9	96	5.86	0.02	38.8	34.8
Framingham	61.1	17.4	72	15.0	0.70	95	111.3	9.1	92	2.48	0.12	94.9	84.0
Gardner[1]	54.0	14.6	73	17.6	1.01	94	136.1	8.0	94	19.04	0.19	61.6	63.2
Gardner[2]	43.8	21.6	51	7.4	1.82	76	46.2	13.2	71	9.19	0.23	74.8	67.6
Hopedale	49.0	13.1	73	7 1	1.16	84	44.0	9.2	79	21.30	0.06	58.3	50.0
Hudson	45.2	12.1	73	9.1	1.02	89	95.9	9.9	89	8.32	0.20	460.8	450.0
Leicester	35.5	6.7	81	7.1	0.98	86	56.7	9.5	83	6.07	0.13	54.5	40.6
Marion	12.2	2.0	84	3.3	0.37	89	21.8	3.9	82	4.58	0.02	37.6	36.5
Marlboro	51.8	5.0	90	5.7	0.36	94	44.5	3.8	91	18.47	0.04	98.6	78.6
Milford	39.2	6.9	82	6.1	0.39	92	51.8	5.1	90	15.04	0.09	98.7	97.9
Natick	36.7	11.6	68	8.5	0.57	93	57.8	5.9	90	4.76	0.21	89.0	65.7
Northbridge	37.7	6.5	83	6.5	1.07	84	43.1	7.5	83	9.23	0.08	54.2	43.8
Norwood	40.7	12.3	70	8.4	0.75	91	121.7	11.8	90	1.91	1.56	34.7	298.0
North Attleboro	11.5	0.6	95	1.8	0.12	94	12.8	2.0	84	4.94	0.02	34.1	29.9
Pittsfield	20.4	3.9	81	5.1	0.44	91	36.5	5.2	86	10.31	0.10	52.1	46.9
Southbridge	52.2	19.1	63	7.4	0.87	88	55.5	9.6	83	1.96	0.12	68.2	59.5
Spencer	56.8	5.4	91	12.6	0.38	97	72.3	5.1	93	4.77	0.06	88.3	55.0
Stockbridge	13.3	1.7	87	2.9	0.34	88	18.9	3.6	81	3.02	0.03	20.3	27.9
Westboro	31.4	4.1	87	8.4	0.72	91	49.7	7.0	86	11.52	0.28	82.9	48.8
Worcester	36.4	21.7	40	12.9	1.50	88	126.8	18.2	86	2.20	0.15	132.4	138.3

* Data taken from Massachusetts State Board of Health Report for 1912, pages 371–379. [1] Gardner area. [2] Templeton area.

considerable difference in the quality of the sewage applied as well as in the effluents. It is probable that a majority of the analyses of applied sewages represent more accurately the stronger day sewage. Filtration does not affect the quantity of chlorine, so that the chlorine results may be used to determine whether the effluent probably corresponds to the sewage analyzed. It will be seen that there are wide discrepancies, and these should be taken into account when studying the results. In some cases the effluent has doubtless been diluted with ground water. These results are not given with a view to showing the relative amount of work done at different plants, as other conditions must be taken into consideration, particularly the character and quantity of sewage applied. These results do show, however, in a general way, the purification accomplished in practice.

The efficiency of sewage filters in a northern climate will not be so great during the winter months as during the remainder of the year, Fig. 213. Bacterial activity is reduced by fall in temperature. The large and irregular doses applied to the filters in order to keep them thawed out and to obtain the best result during the cold weather are not productive of a high degree of purification.

Odors.—Opinions differ widely as to the odor coming from intermittent sand filters. A characteristic sewage odor can be detected in their vicinity, particularly on wet days. The distance to which this odor is carried varies with conditions of wind and weather, as well as of the filter beds themselves. In some cases no objection has been raised from residents within 600 ft., more or less, of the filtering area, but it is probably desirable to keep the beds upward of a quarter of a mile away from any substantial settlement, and the larger the area the more desirable it is that the beds should lie at a considerable distance. The odor is not believed to have any serious effect upon health, for the men employed at the various sewage filter and broad irrigation plants are usually strong and healthy.

OPERATION OF INTERMITTENT SAND FILTERS

Clogging of Beds.—On beginning the use of a sewage filter bed the surface should be smooth and level. For the first few days of use uniform distribution of the sewage over the entire bed will not be obtained, for so much sewage will penetrate the sand near the distributor outlets that the dose will be exhausted before it has an opportunity to cover the bed. Gradually the surface will become partially clogged, with marked improvement in uniformity of distribution.

The degree and nature of the clogging vary with the character of the suspended matter in the sewage applied.[1] If it contains a large quantity of coarse material, such substances will settle and accumulate on the surface of the sand, the coarsest in the immediate vicinity of the distributor outlet and the finest at the farthest points from such outlet. With the lapse of time a surface mat of fibrous material will form, if raw sewage is applied. When allowed to dry out between doses, the mat separates from the surface of the sand and usually cracks and curls up, the rapidity of drying depending upon wind and weather conditions. Such a mat is beneficial in preventing the entrance of very fine substances into the body of the bed. On the other hand, it must be removed at frequent intervals to enable the bed to receive its proper quantity of sewage and the requisite air.

The suspended matter in settled sewage, although somewhat fibrous, is so finely divided that it penetrates the sand more deeply than the coarser matters present in raw sewage. The mat formed by the finer particles is not strong and heavy and is not effective in preventing more finely divided matter from entering the bed. The suspended matter in the effluents from the septic tank and from chemical precipitation is still more finely divided and less fibrous than that from plain sedimentation. Consequently, such matters penetrate the sand to an even greater depth than the suspended matter from sedimentation. The suspended matter in these effluents does not form a mat and, therefore, it is necessary, in order to remove it, to scrape off a substantial portion of the surface sand.

The very finely divided silt washed from street surfaces in times of storm, if applied to sand filters, will cause serious surface clogging. Filters which are in good condition may be so clogged by the storm water from a single brief shower as to require scraping before they can be put into operation again.

Rain has the effect of beating down the surface of the filters, which may require raking before they will resume their normal capacity. On the other hand, frost has a tendency to open the pores of the bed, enabling it to receive sewage much more freely than it did just prior to freezing. Beds which are considerably

[1] Eddy and Fales, *Jour.* Assoc. Eng. Soc., vol. xxxvii, p. 67.

clogged may be made so porous by freezing that their capacity will be temporarily much increased, providing the frost does not go deeper than an inch or two.

Objections to Disturbing Surface of Filters.—Harrowing and plowing may, under some circumstances, be absolutely necessary. There are, however, serious objections to this treatment of the filter, for the disturbing of the upper layers of the sand mixes the organic matter with the cleaner sand below. This organic matter is decomposed more or less, but it does not entirely disappear; on the contrary it changes to a sort of humus, which remains in the sand and has the effect of reducing its capacity. It is obvious, therefore, that a time will come in the life of the filter when this material will reach such an amount as to practically clog the upper layer of sand and require its removal. It is undesirable to cause the organic matter to penetrate the filter any deeper than is absolutely necessary, because this necessitates the removal of a greater depth of sand.

It is perhaps impossible to fix any definite rule for the manipulation of the surface of the filter, as there are doubtless cases where it would be as unwise to disturb the sand as it would be impossible to avoid it in others. The general principle, however, seems to be that care should be taken to avoid mixing the solids deposited by the sewage with the sand and, therefore, the beds should be thoroughly cleaned before harrowing or plowing.

Removal of Dirty Sand.—The surface mat should be removed at frequent intervals and as the proportion of organic matter to sand in the upper layers increases, it is also necessary to remove a certain amount of this layer of sand, humus and other clogging material. The amount of this material to be removed depends upon the character of sewage which has been applied to the filter, and probably also upon the manipulation of the surface of the filter. But the chief factor appears to be the rate at which the filter is operated. If the load is light the natural biological processes will oxidize the organic matter nearly as fast as it is applied, but if the filter is forced such substances, more or less changed, will accumulate rapidly.

The mat which is peeled from the surface of the filter and the material which is scraped in a moist condition from the sand contain a large amount of organic matter. The amount of nitrogen runs, as already shown, as high as 1.3 per cent of the dried sample. This material has some value as a fertilizer, and may in some cases be entirely removed by local farmers, and if the quantity is not too great it is possible that a small revenue may be obtained from this source.

The dirty sand which is removed contains a comparatively small proportion of nitrogenous matter, and seems to have no practical use. It must, therefore, be wasted at the least possible expense. Under some conditions it may be wise to wash this material, and replace it on the surface of the filters. This, however, is open to the objection that the finer particles will be removed, leaving a coarser-grain sand for the surface, which will allow the finely divided matter to pass through it, producing the effect of stratification.

Resting.—When the bed is overtaxed or the aeration becomes inadequate, fouling begins at the bottom of the bed, on account of the lack of air. Such a condition should be avoided by careful attention to dosing and care of surface, but if it exists the bed must be thrown out of commission, and allowed to dry out so as to overcome the effect of capillarity and become charged with air. Bacterial activity will still continue for a considerable period of time under such conditions, so that the period of resting may vary within wide limits without injury. If the bed is in very bad condition a period of 4 to 8 weeks may be required for its recuperation, while 1 or 2 weeks may suffice to cure less serious cases.

Replacing Sand.—So far as can be learned but few intermittent sewage filtration plants have been in operation a sufficient length of time to make necessary the replacing of material scraped off during the process of cleaning, although some industrial plants have required such renewal. It is quite probable that filters which are used well up toward their maximum capacity must have so much material removed that ultimate replacement will be absolutely necessary. Thus at the end of an 11-year period of use, the removal of 6 in. of material proved necessary at Brockton, Mass., at a cost of $4,314 for 19 acres of beds. At Worcester it was found that on the beds taking effluent from chemical precipitation tanks, clogging due to change in the character of the surface material, caused by the depositing of lime, iron, and organic matter, necessitated the removal of about 4 in. of material 3 years after putting these filters into service. After that time an average of about 3.3 cu. yd. of material per 1,000,000 gal. of chemical effluent filtered were removed. In the case of the effluent from the septic tanks, the quantity of material removed from the beds was substantially the same as when the chemical effluent was applied, but the clogging material had to be removed at more frequent intervals. With the effluent from settling tanks the amount of material requiring removal was somewhat less, though the total amount, including the mat, was about the same. The quantity of mat and dirty sand removed from Worcester beds receiving raw sewage has been as high as 14 cu. yd. per 1,000,000 gal. sewage filtered.

At Concord, Mass., where the sand is relatively fine and the sewage dilute, 9 in. of sand were removed from a 1-acre bed, out of a total of 3.3 acres of beds, at the end of the 20th year of service.

Maintenance in Winter.—The warm, dry weather of the summer is by far the best period of the year for the operation of intermittent filters. The spring is wet, and the filters are then generally overcoming the ill effects of the previous cold weather, so that the best work is not done by them until the late spring or early summer. The fall is apt to be cold and rainy, and such weather is not advantageous. The winter months are the most difficult in which to get good results, as shown by Fig. 213.

The success of the filter during winter weather depends somewhat upon chance. If the filters are in good condition, and a heavy snow storm occurs while the filter is flooded, the snow will be melted, and the water will readily find its way into the sand. On the other hand, if the filter is seriously clogged with suspended matter from sewage previously applied, and is very slow in filtering, the water will be chilled by the snow which falls into it, and if there should be a sharp decline in temperature it is quite probable that the snow and water will be frozen to a solid field of ice, which adheres tenaciously to the sand on to which it is frozen. If a considerable depth of snow falls when the filter is not in use, it has been found that when water is turned on it is immediately chilled, and in extreme weather it is quite likely to freeze and make the filter substantially impermeable. On the other hand, if the filter is in fairly good condition, and paths are shoveled through the snow giving the warm sewage a chance to get to the center or extreme points of the bed without becoming too cold, it has been found possible to thaw a large fall of snow and still keep the filter in good condition.

More care is required in dosing filters during the winter than at other seasons. Large doses of sewage must be applied to thaw the frost in the sand or to melt the snow on the surface of the filter. On the other hand, it is essential that the dose be not large enough to freeze to the sand before it can find its way into the filter.

Various methods have been devised for preventing the ice from freezing to the sand. One of the most successful of these is to furrow the filters for use in winter. The furrows may consist of ridges and depressions about 3 ft. on centers and about 10 in. deep. When ice has formed on the sewage, the ridges hold it up as the sewage recedes, leaving a space between the bottom of the furrow and the ice, through which future applications of sewage can run. This covering of ice also protects the surface of the bed from the extreme cold, preventing serious freezing, and serves to keep succeeding snowfalls out of the sewage.

Winter Cleaning.—When sewage is applied to furrowed filters the suspended matter tends to settle in the bottom of the furrows, which afford a much smaller area for the distribution of such matter than the level surface of the bed at other seasons. Where beds are operated at nearly their maximum capacity, especially with sewage containing large quantities of suspended matter, the winter clogging may be serious. It is important, therefore, to take advantage of any opportunity afforded for winter cleaning. In the course of nearly every winter in Central Massachusetts there is at least one period lasting from one to several days, during which the temperature conditions are favorable for cleaning. If at such time the accumulation of ice does not prevent, it is desirable to remove as much of the clogging accumulation as possible. The work must be done expeditiously, however, as such periods are usually of short duration. It has been found that the mat can be readily raised from the sand if it is barely frozen. If, however, it is frozen hard, the crust will be so thick and the quantity of sand clinging to the mat so great, that cleaning will be impracticable.

CHAPTER XIX

DISINFECTION OF SEWAGE AND SEWAGE EFFLUENTS

The term "disinfection" is used here to designate the treatment of sewage or contaminated water so as to reduce greatly the number of bacteria in it. Incidentally this treatment may deodorize the liquid more or less and prevent or retard putrefaction. By "sterilization" is meant the destruction of all organisms. In the methods of treating sewage previously described, removal of suspended matter and change in the character of the organic matter originally present are the main objects sought, whereas in disinfection and sterilization the main object is to kill the bacteria in the liquids treated. It is unusual for a water receiving sewage or sewage effluents to be used for domestic consumption without previous treatment, and with modern facilities for purifying and disinfecting a water supply, it is generally wiser to apply the corrective measures to the water supply than to the sewage. There are conditions, however, under which it is desirable to disinfect the sewage even though the water supply be effectively treated. The danger of contracting typhoid fever on the bathing beaches through polluted water and by eating sewage-contaminated oysters, also justify the disinfection of the sewage of some communities.

Disinfection by Acids.—Most acids act as germicides under certain conditions. Where sewage contains spent acids from industries, the number of bacteria are usually relatively small. Mine drainage often contains large quantities of acid and iron, which keep down the bacteria as shown in the following figures of conditions in the Monongahela River, furnished by Hazen and Whipple:

Acidity, parts per million.....	35	30	25	20	15	10	5	0	0
Alkalinity, parts per million..........									5
Bacteria per centimeter.......	150	280	360	500	750	1,400	2,800	7,500	30,000

The cost of disinfection by acids has prevented the consideration of this treatment except for emergency use and where it is incidental to treatment with acid for recovery of grease.

Disinfection by Copper Sulphate.—This treatment is employed mainly in killing algæ in water supplies, but has been used occasionally in killing troublesome growths of organisms in sewage filters. The quantity of copper sulphate needed ranges from 0.07 to 10 p.p.m., according to the species to be killed, and it is accordingly necessary to determine the organism which must be removed[1] before the dose can be chosen with certainty. It is important to keep in mind that this chemical affects fish, some species being more resistant than others. Kellerman reports that the following amounts of copper sulphate, in parts per million, must not be exceeded if it is desired to save the species of fish mentioned: black bass, 2.1; carp, 0.3; catfish, 0.4; goldfish 0.5; perch, 0.75; pickerel, 0.4; sucker 0.3; sunfish, 1.2; trout, 0.14. The copper sulphate is placed in a gunny sack which is dragged slowly by a rowboat back and forth over the pond, keeping the boat on each trip within 10 to 20 ft. of the previous path. In this way about 100 lb. of the chemical can be distributed in an hour.

Good results have been obtained with copper sulphate treatment of settled sewage and the effluents from septic tanks and intermittent sand filters, but the cost has been greater than that of disinfection by chlorine and its action is not so rapid as chlorination and more affected by temperature.

Disinfection by Chlorine and its Compounds.—Investigations by Phelps[2] indicate that gaseous chlorine and hypochlorites are good germicides, but chlorates and perchlorates have little value as disinfectants. These substances, like ozone, are disinfectants because of their oxidizing effect. They release oxygen in a nascent state, which instantly combines with the most susceptible substances at hand, among which are the organic structures of the bacteria. The reactions taking place result in the production of hypochlorous acid, HClO, which is very unstable and is at once dissociated into hydrochloric acid and oxygen.

Each molecule of hypochlorous acid contains one atom of chlorine and one of oxygen, and as an atom of oxygen has twice the combining power of the chlorine atom, it follows that the oxidizing power of the compound is twice as great as its chlorine content. Thus, if a sample of bleaching powder actually contains 20 per cent of chlorine in the form of calcium hypochlorite, its potency is double this and it is said to contain 40 per cent "available chlorine," though in reality it does not. The words "available chlorine" simply constitute a term by which is expressed

[1] "American Sewerage Practice," vol iii, p 745
[2] *Water Supply Paper* 229, U. S. Geological Survey, p. 60

the power of the compound to oxidize or disinfect. The quantity of chlorine used is expressed in three different ways, the equivalents being shown in Table 63.

TABLE 63.—PARTS OF AVAILABLE CHLORINE, EQUIVALENT POUNDS OF LIQUID CHLORINE AND EQUIVALENT POUNDS OF CALCIUM HYPOCHLORITE

(Calcium hypochlorite assumed to contain 33⅓ per cent available chlorine)

Parts available chlorine per 1,000,000 gal.	Pounds liquid chlorine per 1,000,000 gal.	Pounds calcium hypochlorite per 1,000,000 gal.
0.04	0.33	1.0
0.12	1.0	3.3
0.5	4.2	12.5
1.0	8.3	25.0
5.0	41.5	125.0
10.0	83.0	250.0
25.0	207.5	625.0

The effect of a given quantity of these disinfectants upon sewage depends to some extent upon the size of particles and condition of the organic matter, for apparently the time of contact necessary for the full effect of the disinfectant to be developed varies with the character and quantity of the organic matter. The germicidal effect of the disinfectants is apparently the same where an opportunity for complete action is afforded.

The quantity of disinfectant required to treat sewage and the effluents from sewage treatment works is many times greater than that required to disinfect water supplies. The effluent from a well-designed, built and operated intermittent sand filter, not overloaded, requires the least quantity of disinfectant and septic tank effluents generally require the largest quantity. Table 64 gives the results of tests by Clark and Gage to ascertain the quantities of effective chlorine required to reduce the bacterial content of sewage and effluents to different standards. Water suitable for drinking in Massachusetts usually contains less than 100 bacteria per cubic centimeter, determined at room temperature. The total bacteria developing on litmus lactose agar at body temperature is usually less than 10 per cubic centimeter, and the red colonies usually less than 5 per cubic centimeter. This they called the 100–10–5 or drinking water standard. Two

TABLE 64.—AMOUNT OF EFFECTIVE CHLORINE REQUIRED TO REDUCE BACTERIAL CONTENT TO PRESCRIBED STANDARDS

(Parts per 1,000,000; 1 part per 1,000,000 = 8.3 lb. per 1,000,000 gal. Abridged from 1911 Report of the Massachusetts State Board of Health, pages 345–350)

Sample	Sterile	100–10–5 Standard			1000–100–50 Standard			10,000–1,000–500 Standard		
		20°C.	40°C.		20°C.	40°C.		20°C.	40°C.	
		100	Total 10	Red 5	1000	Total 100	Red 50	10,000	Total 1000	Red 500
Sewage:										
Maximum	+37.5	18.8	+37.5	+37.5	11.3	11.3	11.3	11.3	7.5	7.5
Minimum	26.3	3.8	7.5	3.8	−3.8	3.8	−3.8	−3.8	−3.8	−3.8
Average	34.7	9.4	22.5	21.3	6.6	7.5	7.6	6.6	4.7	4.7
Settled sewage:										
Maximum	+37.5	18.8	+37.5	+37.5	15.0	26.3	26.3	15.0	11.3	11.3
Minimum	+37.5	3.8	15.0	7.5	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Average	+37.5	9.0	29.3	21.0	8.3	15.0	11.3	7.5	6.8	6.8
Septic tank effluent:										
Maximum	+37.5	11.3	+37.5	+37.5	7.5	11.3	11.3	7.5	7.5	7.5
Minimum	+37.5	3.8	+37.5	+37.5	−3.8	7.5	3.8	−3.8	3.8	−3.8
Average	+37.5	8.8	+37.5	+37.5	6.3	8.8	7.5	6.3	6.3	6.3
Contact filter effluent:										
Maximum	+37.5	18.8	+37.5	+37.5	11.3	22.5	11.3	7.5	11.3	11.3
Minimum	37.5	3.8	30.0	11.3	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Average	+37.5	6.8	36.8	25.5	5.7	7.2	6.0	4.9	5.3	4.9
Trickling filter effluent:										
Maximum	+37.5	33.8	+37.5	33.8	7.5	18.8	7.5	3.8	7.5	7.5
Minimum	37.5	−3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	+37.5	6.9	26.9	20.4	4.4	5.7	4.1	3.8	4.1	4.1
Sand filter effluent:										
Maximum	+37.5	3.8	18.8	18.8	−3.8	3.8	3.8	[1]	−3.8	−3.8
Minimum	3.8	[1]	[1]	[1]	[1]	[1]	[1]	[1]	[1]	[1]
Average	17.2	3.8[2]	7.5[2]	8.5[2]	3.8[2]	3.8[2]	3.8[2]	[1]	−3.8[2]	−3.8[2]

− Less than stated amount. + More than stated amount. [2] Based on figures given; some of samples being marked +.

[1] Initial number of bacteria less than standard.

other standards containing respectively 10 and 100 times as many bacteria, called 1,000–100–50 and 10,000–1000–500 standards, were also assumed to correspond approximately to the upper and lower limits of bacterial counts on river water receiving more or less pollution. The tabulated results are all based on a period of contact of 2 to 4 hr.

TABLE 65.—AMOUNT OF EFFECTIVE CHLORINE REQUIRED TO PRODUCE STATED REDUCTIONS IN NUMBERS OF BACTERIA

(Parts per 1,000,000; 1 part per 1,000,000 = 8.3 lb. per 1,000,000 gal. Abridged from Report of the Massachusetts State Board of Health, 1911, pages 345–350)

Sample	99 per cent reduction			90 per cent reduction			75 per cent reduction		
	20° C.	40°C.		20° C.	40°C.		20° C.	40°C.	
		Total	Red		Total	Red		Total	Red
Sewage:									
Maximum	11.3	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
Minimum	3.8	−3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	6.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
Settled sewage:									
Maximum	11.3	11.3	11.3	7.5	7.5	7.5	7.5	7.5	7.5
Minimum	3.8	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	6.8	5.3	6.8	4.5	4.5	4.5	4.5	4.5	4.5
Septic tank effluent:									
Maximum	7.5	7.5	7.5	−7.5	7.5	7.5	3.8	7.5	7.5
Minimum	−3.8	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	6.3	6.3	6.3	5.0	6.3	6.3	3.8	5.0	5.0
Contact filter effluent:									
Maximum	11.3	11.3	7.5	7.5	7.5	7.5	7.5	3.8	7.5
Minimum	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	5.3	4.9	4.5	4.5	4.2	4.2	4.5	3.8	4.2
Trickling filter effluent:									
Maximum	7.5	7.5	7.5	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Minimum	3.8	3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	4.1	4.7	4.1	−3.8	3.8	3.8	−3.8	−3.8	−3.8
Sand filter effluent:									
Maximum	37.5	11.3	7.5	11.3	7.5	3.8	−3.8	7.5	−3.8
Minimum	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8	−3.8
Average	8.6	7.0	4.9	4.9	4.9	3.8	−3.8	4.9	−3.8

−Less than stated amount.

A general average of all the test results given out by Clark and Gage on the quantity of chlorine required, indicated the effect of time of contact to be about as follows:

Hours of contact	1	2	4	6	24
Relative amounts of hypochlorite required.	100	84	82	77	61

At Providence, R. I., the use of calcium hypochlorite, without previous treatment of the sewage by lime, did not prove so efficient as the application of bleach to the sewage after it had been treated with lime and had passed through the roughing tanks. Less than 4 p.p.m. of chlorine gave erratic results, but there was a remarkable uniformity in efficiency with from 4 to 6 p.p.m. The period of stability of the chemical precipitation effluent was materially increased by the addition of bleach, as shown by methylene blue tests.

An increase in the bacterial count after disinfection with hypochlorites is sometimes observed. Clark and Gage explain this by the fact that sewages and effluents from contact beds and trickling filters contain substances which combine with some of the oxygen liberated by hypochlorites, so that not all of it is available for destroying the bacteria. If the oxygen is used up before all the bacteria are killed, those that remain alive can multiply without restraint. In fact, by eliminating a large proportion of the bacteria, including probably all of certain species, the conditions become peculiarly favorable for rapid multiplication of the remaining bacteria, since bacterial equilibrium has been destroyed.

Sewage dosed with hypochlorites can be filtered successfully if the dose is not too large. Clark dosed raw sewage in this way with 25 p.p.m. of available chlorine without causing any decrease in the nitrification of this sewage when passed through a sand filter for 3 months. Similar experiments were made by dosing the sewage applied to a trickling filter, at first at the rate of 5 p.p.m. available chlorine which was gradually increased to 50 p.p.m. The last-named rate was continued for 3 months, decreasing the nitrates in the effluent from 20 to 1.5 p.p.m. but not checking nitrification wholly. The addition of disinfectants to sewage before filtration may be desirable during epidemics, and to prevent odors and kill insect pests about trickling filters, but where disinfection is practiced, it is usually the final effluent of the treatment plant which is dosed.

The size of the dose of germicide required to disinfect sewage properly is much larger than that needed to disinfect water supplies, and the disinfection is generally needed only in cases where some other form of treatment precedes it, requiring the continuous presence of labor at the plant. These conditions favor the choice of hypochlorite of lime as a disinfectant, as being cheaper than liquid chlorine. If reliable labor is not constantly available to see that the hypochlorite apparatus is in working condition or the relative price advantage of hypochlorite over liquid chlorine materially decreases, then liquid chlorine will probably be the most advantageous disinfectant. Comparative estimates based upon local conditions and current prices are necessary to an accurate decision as to the relative merits of the two chemicals in each case coming up for consideration.

Disinfection with Hypochlorites.—The quantity of hypochlorite to be used in a given case is best determined practically by observing the effect of different doses of disinfectant upon the bacterial content of the liquid to be disinfected. The sample of disinfected liquid should not be taken until the hypochlorite has had time to attack and be used up by the organic matter. Ordinarily it is practicable to select a definite period of contact, after which the test should be made, and it is to be expected that a definite quantity of free chlorine present at the end of this period will be a reasonably accurate indication of the effect of the disinfectant upon the sewage bacteria. The test period need not be as long as the working period of contact, which should be sufficiently long to allow the utilization of all disinfectant added to the sewage.

Where the sewage occasionally receives large quantities of industrial wastes inimical to bacterial life, there may be times when there will be few bacteria and other times when the numbers may be large. Except in the rare cases when these wastes are discharged in accordance with a time-schedule, the operator of the disinfecting plant has no guide to help him adjust his dose of hypochlorite to the fluctuating requirements, and the best he can do is to test the efficiency of disinfection frequently and to regulate the quantity of applied disinfectant by the tests on previous days. Under this plan it will always be necessary to apply an excess of hypochlorite, because of the danger that the sewage may need a larger dose than it did the previous day. The bacterial count is subject to hourly variations and there are also perceptible variations on Sundays and holidays from the normal daily curve of bacterial content. Consequently some economy in the use of chemicals can be introduced in a large plant by varying the dose at intervals during the day in accordance with the curves of average bacterial content determined by tests covering a sufficient period to make their average a reasonably reliable guide for disinfection.

In using hypochlorite of lime, there are two steps in the process which must be taken properly or there will be failure. In the first place the hypochlorite solution must be made up correctly, irrespective of variations in the quality of the hypochlorite of lime. Some of the serious failures in the chlorination of water supplies have been due to the use of inferior hypochlorite, resulting in the preparation of a solution so weak that its disinfecting value was very small and practically worthless. The second essential step in this method of disinfection is to feed the solution into the sewage at a definite rate. This is done in most cases by using a constant-level box from which the solution flows under a constant head through an adjustable orifice. Experience has shown that if this orifice is not inspected frequently and cleaned when clogged the disinfection will not be thorough.

The bleaching powder is dissolved in a mixing tank, large enough to hold about 4 gal. of water for every pound of the powder to be dissolved in one batch. Where the mixing is done by hand, fairly strong solutions require somewhat less labor and involve fewer difficulties in mixing than the more dilute solutions. The solution should be allowed to stand several hours, preferably 12 to 24 hr., before it is diluted, to permit the lumps of bleaching powder to become soft. The fine material dissolves readily, but the lumps must be broken up in order to utilize the full value of the powder. If not enough water is used, a gelatinizing action takes place which retards the settling of the undissolved sludge of hydrated lime and silica. Clarification is important because suspended matter tends to clog the orifice of the feeding apparatus. Cast-iron tanks are made for mixing the bleaching powder in small installations, but in plants for preparing large quantities of solution the mixing is usually done in tanks made of large vitrified pipe with wood covers and concrete bottoms, wood tanks with or without concrete linings, and concrete tanks. Care must be taken on opening the packages of bleaching powder to prevent dust and fumes escaping into the room, as they affect the nose and throat. In some cases the package, as soon as it is opened, is placed in water and the powder turned out without coming into contact with the air. The mixing tank should be provided with a drain for drawing off the sludge.

From the mixing tank the strong solution flows by gravity or is pumped to a solution tank or tanks where it is diluted to a strength of 1 to 2 per cent. The solution must be stirred to maintain a uniform strength. A 1 or 2 per cent solution is better than a stronger one because a greater quantity can be applied to the sewage, making it easier to adjust the measuring device for the feed and to obtain thorough mixing of the solution with the sewage. The solution tanks are made of the same materials as the mixing tanks. If the strong solution is discharged into them at the bottom under considerable head, the mixing of the solution with the diluting water is assisted.

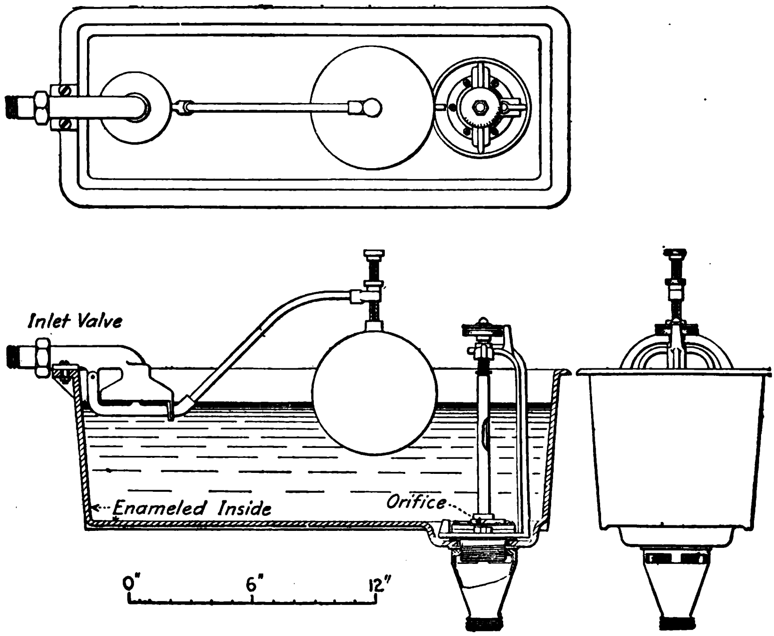


FIG. 214.—Orifice feed tank (Pittsburgh).

From the solution tank the solution passes to a chemical feed tank, by which the dose is discharged at a predetermined uniform rate into the pipe leading to the sewer or tank. This feed tank has a ball cock on the inlet pipe, which maintains the solution at a uniform height, and an adjustable orifice through which the solution passes to the outlet pipe. The orifice is adjusted by hand to discharge the solution at the desired rate. An apparatus of this kind, made by the Pittsburgh Filter & Engineering Co., is shown in Fig. 214. The orifice must be inspected frequently, as it is liable to become clogged with suspended matter carried over from the mixing tank.

Providence Hypochlorite Plant.—One of the largest installations of hypochlorite apparatus for treating sewage was made at Providence, R. I., under the direction of Bugbee. After it had been in service for some time, a lime-bleach solution plant was built near the sewage treatment works. On account of the unusual facilities for obtaining this form of chemical from this plant its use was adopted, although the experience with hypochlorite had been satisfactory. The chemical plant makes lime-bleach solution by passing chlorine into milk of lime. This is allowed to settle, and the clear liquid is decanted into tank trucks and delivered to bleacheries. The lower portion of each tank

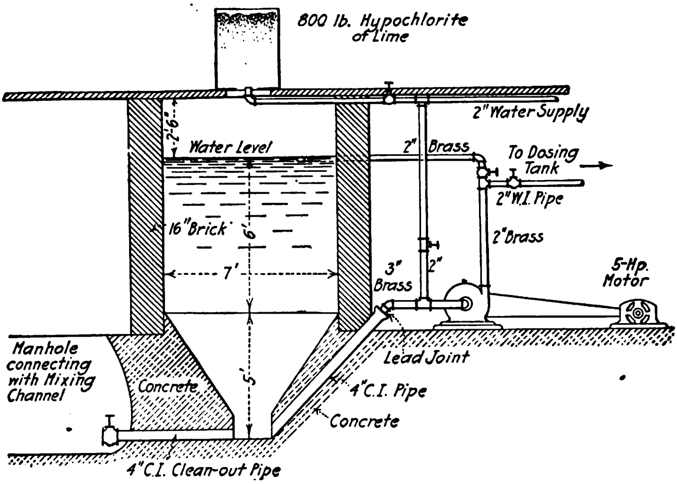


FIG. 215.—Apparatus for preparing disinfectant at Providence.

contains lime sludge, which is not suitable for bleachery purposes but can be used for sewage treatment, and it is this portion of the product which is furnished to the city at a considerably lower rate than that for the same amount of chlorine as liquid chlorine or dry bleach. Deliveries are made directly into the tanks in the chemical house at the works, and each delivery is tested and diluted to the proper strength for discharge into the sewage.

The dissolving tank used at Providence, Fig. 215, was cylindrical with a conical hopper bottom. It was built of brick and covered by the floor above, which was placed directly on top of the tank, with a tight joint between the two. From the extreme bottom a suction pipe led to a 3-in. horizontal centrifugal pump driven by a 5-hp. motor. This pump had a cast-iron shell and brass shaft and runner. The discharge pipe connected tangentially with the mixing tank at a point near the surface of the solution and with the dosing tanks. By closing the valve leading to the dosing tanks and opening the valve leading to the mixing tank, it was possible to pump the solution over and over through the tank, thus thoroughly mixing the bleaching powder with the water. When the mixing was completed, the valve on the pipe leading to the mixing tank was closed and the valve on the pipe leading to the dosing tank was opened, after which the solution was pumped to the dosing tank. There was an opening 18 in. in diameter in the floor, directly over the center of the mixing tank. Pointing upward into the center of this opening was a 2-in. wrought-iron water pipe.

The bleaching powder was furnished in metal drums holding about 800 lb. each. When it was desired to make up a tank of solution, a hole about 8 in. in diameter was cut in the bottom of a drum, thus avoiding the escape of chlorine gas which always collects at the top end. The drum was then inverted over the hole in the floor and so placed that the 2-in. water pipe pointed into the opening in the center of the drum. When the drum was in the proper position, water was turned on and the bleaching powder washed into the tank below, without the escape of any dust into the room and consequent discomfort to the operatives. The tank, which had a capacity of 2,300 gal., was then filled to the usual water level and the pump started. The liquid was kept in circulation for 1 hr.. after which it was pumped to a dosing tank, where it was diluted to a strength of about 25 lb. of bleach per 100 gal. of water, approximately equivalent to a 3 per cent solution of bleaching powder or a 1 per cent solution of available chlorine.

One mixing tank only was provided, the bleaching powder solution being pumped from it to two dosing tanks, 8 ft. wide by 20 ft. long by 3½ ft. in depth, holding about 100 gal. per inch in depth, equivalent to 25 lb. of bleaching powder. This made it easy for the operatives to gage the rate at which the solution was introduced into the sewage, by observing the drop in the level of the liquid in the tank in a given time.

Emergency Plant.—An example of a very simple apparatus for temporary, small-scale use is shown in Fig. 216.[1] It was con-

[1] *Engineering and Contracting*, July 17, 1912.

structed by the Indiana State Board of Health from three barrels, a commercial constant-level regulating box, a small amount of piping and a geared mixing contrivance much like that used on some types of ice-cream freezers.

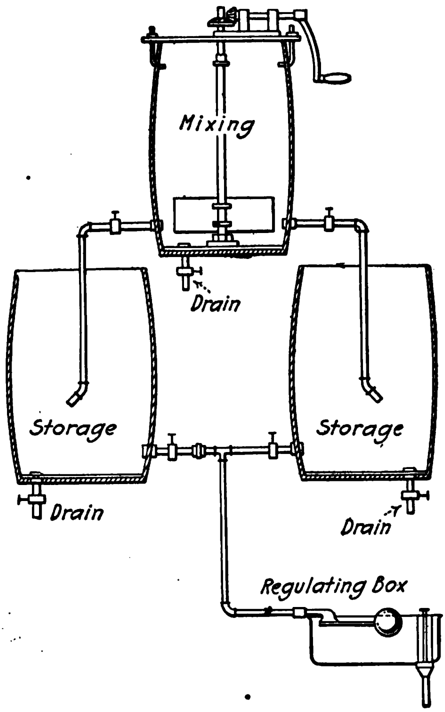


FIG. 216.—Emergency disinfecting apparatus, Indiana State Board of Health.

Liquid Chlorine.—In practice, the preparation of a disinfecting solution from bleaching powder is disagreeable work on account of the dust and fumes, the irregularity in the composition of the powder and consequent care needed to maintain a solution of proper strength, and the liability of the measuring orifice to become clogged. Liquid chlorine, which can now be obtained in steel tanks or cylinders from numerous producers, has come into extensive use for disinfecting purposes because these objections do not arise when it is used.

A manually controlled apparatus made by the Wallace & Tiernan Co. for treating sewage with chlorine gas or a so-called solution of chlorine made from liquid chlorine is shown in Fig.

217. The tanks of liquid chlorine, *A*, *A*, are generally placed on scales so that the variation in their weight as their contents are removed can be measured and in this way a check obtained on the rate of flow indicated by the chlorinating apparatus Each tank has a tank valve, *B*, *B*, to which the operator attaches an auxiliary tank valve, *C*, *C*, which is the main shut-off valve

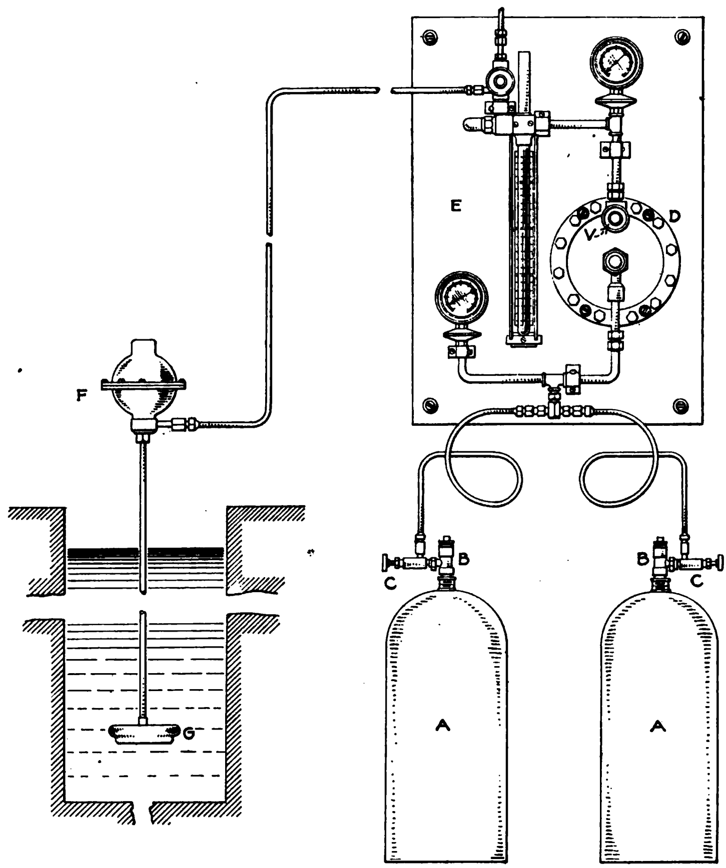


FIG. 217.—Manual control direct feed chlorinator (Wallace & Tiernan Co.).

during operation. When the valves are opened the liquid chlorine in the tank gradually becomes a gas and passes from the tank to a compensator, *D*, which maintains a constant flow of the gas irrespective of the pressure in the tank, so long as the setting of the control valve, *V*, forming part of the compensator is not changed. The rate of flow of the chlorine is changed by altering the setting of this control valve.

From the compensator, the gas passes to a meter, E, which is a manometer having a scale calibrated to show the flow of chlorine gas in pounds per 24 hr. or other unit desired by the purchaser. From the meter the gas flows to the place of application through a copper or galvanized iron pipe which should not be over 500 ft. long. The gas is delivered through a chlorine check valve, F, which is necessary to prevent moisture from the sewage passing back to the chlorinator and interfering with its operation. From the check valve, the gas passes through a silver tube to the diffuser, G, submerged at least 4 ft. in the sewage. This diffuser is composed of a composition sponge of fine porosity held in a non-corrodible casing. The sponge is kept saturated with water by capillary action of the material of which it is composed and the chlorine in passing through the sponge becomes thoroughly mingled with the water and is discharged in this condition into the sewage. The longer the period of contact of the solution with the sewage before the latter is discharged into a water course, the more complete will be the disinfection. Fifteen minutes is the minimum contact time advised by the makers of the apparatus.

This chlorinator is the simplest form used. It is known as a direct-feed apparatus because the gas is delivered directly to the sewage. In many cases it is desirable to use a solution of chlorine instead of the dry gas and for such cases the apparatus is provided with a solution jar, to which the gas passes from the manometer and in which the chlorine is mixed with an auxiliary water supply. The mixture passes from this jar to the sewage. This is known as a solution-feed apparatus. It is also very desirable when the flow of sewage fluctuates widely to alter the rate of application of the chlorine as the flow varies, and this is done by an additional regulating device converting the apparatus into an automatic control type.

The liquid chlorinating apparatus should be placed in a well-ventilated room, so that in case there is any escape of gas into the room it can be removed immediately. This room should open out of doors and not into another room. The treatment of a person affected by the gas is merely to give fresh air and ether, inhaled to relieve the pain.

CHAPTER XX

IRRIGATION WITH SEWAGE

Sewage contains chemical constituents which, when in suitable form, possess fertilizing value and for this reason there has been a popular belief that the agricultural utilization of sewage will be profitable. The most important plant food is water, which is the main constituent of sewage (99.9 per cent). Probably the chief agricultural value derived from the sewage when used for irrigation in most cases is due to its water content. Only a portion of the nitrogen, phosphorus and potassium compounds in the sewage is in a form suitable for direct utilization by plants. The nitrogen should be present as nitrates to be of most value, whereas usually only a very small part of it, or none at all, has been brought to that state and the remainder must be nitrified by biological agencies which require oxygen, a mild temperature and the presence of lime or some other base. The phosphates and potash, especially the latter, are present in such small quantities that their fertilizing value is usually small compared with that of the nitrogen compounds when converted into available form.

The fertilizing constituents of sewage are associated with fat and soap, which are injurious to land. The fat and soap clog the pores of the soil and interfere with the absorption of the sewage by the soil and the subsequent aeration of the pores. Land which has become clogged and rank in this way is called "sewage sick," and one of the most important duties of the manager of a sewage farm is to prevent such a condition from becoming chronic at any part of his fields. A precautionary measure against it is the removal of the coarse suspended matter of the raw sewage by screening and sedimentation.

The sewage is applied to the land in various ways. If it is flowed over sloping fields from an upper level to a lower one, the process is called surface or broad irrigation. In this method the sewage does not penetrate more than a few inches into the soil, except where the latter is unusually porous. When the sewage is applied by any method to soil of such a nature that the sewage percolates downward in considerable quantities it is called filtration. Filtration must be carefully distinguished from intermittent filtration, which is practiced on specially prepared areas without any agricultural use of the land, when this method of treatment is carried on most efficiently. Ridge irrigation is practiced by flooding sewage over land from channels on ridges between long, gentle slopes: the sewage collecting in the low places between the ridges is received by a cross ditch at the foot of the field and conducted to ridges in a field lying somewhat lower than the first. Where the soil is very heavy a third field may be laid out. In Europe, bed irrigation and land filtration are terms used to designate the distribution of the sewage in numerous ditches cutting up the land into beds, which are kept moist by horizontal seepage from the ditches. This method is called ridge-and-furrow irrigation in England. In flood irrigation a plot of land surrounded by a bank is periodically covered with sewage to a depth of 1 or 2 ft. In sub-surface irrigation the sewage is distributed through open-jointed pipes laid about 1 ft. below the surface.

In agriculture, fertilizers should be applied at certain stages in the rotation and growth of crops, and the proper fertilizers to use depend upon the nature of the soil, the climate, the crops to be grown and the rotation of crops. In sewage disposal, all these considerations must be waived in favor of the production of an effluent of suitable character. The crops should be regarded merely as a by-product. All evidence furnished by many years experience in many countries under many conditions does not reveal, however, that it is practicable to obtain much fertilizing value from city sewage as it must be used to make this method of treatment successful, but indicates that where irrigation has been successful agriculturally the same results would have been produced with water.

English Experience.—Irrigation with sewage has been carried on more carefully and extensively in England than elsewhere. This is not because the municipal authorities of that country have been particularly pleased with this method of treatment but because the Local Government Board has required some sort of land treatment of sewage and sewage effluents as a condition for granting its sanction to raising money for sewage treatment works. The area of land suitable for irrigation in England is limited, and consequently the Royal Commission on Sewage Disposal gave detailed attention to sewage farming. Eight farms typical of different conditions were kept under close observation for two years, and observations were made less regularly at other farms.

At these eight farms from 12 to 23 per cent of the area was not irrigable, either because it was unsuited for the purpose, or had not been prepared to receive sewage, or was required for roads, buildings, sludge beds and other purposes.

The Royal Commission's conclusions from its investigations were stated substantially as follows in its fifth report:

1. There is no substantial distinction between effluents from land and effluents from artificially constructed filters. Effluents from those soils which are particularly well adapted for the purification of sewage contain only a very small quantity of unoxidized organic matter, and are usually of a higher class than effluents from artificial filters, as constructed and used at the date of the report. Effluents from soils not well adapted to purify sewage may often be very impure.

2. Generally speaking the evidence points to a maximum rate of 36,000 U. S. gal. per acre, or 1,000 persons per acre, with the best land, after preliminary treatment of the sewage, although some witnesses who testified before the Commission put the rate as high as 72,000 gal., or 2,000 persons, per acre, under similar conditions. With unsuitable land, such as clay, not more than 3,600 gal. per acre can be efficiently treated, even after settlement of the sewage.

3. The total acreage of a farm must be relatively much greater when the sewage is purified by surface irrigation than when the method of filtration is employed, and a larger percentage of surplus area is also desirable in the former case. The larger the total irrigable area the greater is likely to be the working cost. On the other hand, with good management, the larger the surplus irrigable area, the better is the purification likely to be and, within certain limits, the prospect of profit.

4. In the case of sewage which is to be treated upon artificial filters, it is generally desirable to settle out as much as possible of the suspended solids before filtration. This is of less moment in the case of sand, but where the soil is heavy the sewage should first be efficiently screened and settled. Porous sandy soil, worked as filtration farms, may be able to treat crude unsettled sewage without detriment, but, even in those cases, there is the possibility of nuisance arising from the decomposition of sewage

solids on the surface of the soil, and such solids may cause damage to crops.

5. It is impossible to lay too much stress on the importance of sewage farms being well managed. Farm managers have a most difficult part to play, and no amount of care and attention will ever enable land of any kind to deal with a volume of sewage in excess of the effective purifying capacity of the soil. As a rule, sewage farms should not be let.

6. English soils and subsoils may be divided into the following three broad classes: Class I, all kinds of good soil and subsoil, such as sandy loam overlying gravel and sand; Class II, heavy soil overlying clay subsoil; Class III, stiff clayey soil overlying dense clay. Since variations exist in practice, both as regards the method of purification employed and the extent of cropping, the first of these three classes may be divided into three subclasses,

TABLE 66.—AVERAGE AMOUNTS OF LAND REQUIRED FOR TREATING A DRY-WEATHER FLOW OF 1,000,000 U. S. GALLONS
(Royal Commission on Sewage Disposal. Fifth Report, page 153)

Class of soil	Method of working	Volume of settled sewage which can be treated per acre per 24 hours	Total area of land required to treat a dry-weather flow of 1,000,000 U.S. gal.[1]
I. Good soil and and subsoil	Filtration with cropping	14,400 U. S. gal.	70 acres
I. Good soil and subsoil	Filtration with little cropping	30,000 U. S. gal.	33 acres
I. Good soil and subsoil	Surface irrigation with cropping	8,400 U. S. gal.	121 acres
II. Heavy soil on clay	Surface irrigation with cropping	6,000 U. S. gal.	167 acres
III. Stiff clayey soil on dense clay	Surface irrigation with cropping	3,600 U. S. gal.	278 acres

[1] These areas are sufficient for the treatment in times of storm of three times the mean dry-weather flow.

as follows; (1) filtration with cropping; (2) filtration with little cropping; (3) surface irrigation with cropping. The method of purification assumed to be suitable for the other classes of soil is surface irrigation with cropping. With a heavy soil and clay subsoil, by far the greater part of the purification is effected by surface irrigation, although in exceptional circumstances a good deal is also done by filtration.

Having regard to the volume of sewage dealt with and the purification effected at the farms kept under the Commission's observation, and the testimony given by various expert witnesses called by the Commission, it estimated that the several classses of soil and subsoil could deal effectively with the volumes of settled sewage given in Table 66, under the climatic conditions of England.

French Sewage Farming.—The utilization of sewage in one way or another has long been practiced in France. The native thrift of the French people led them many years ago to attempt quite extensive use of the wastes collected in cesspools and Victor Hugo's fervid plea[1] for employing the sewage of Paris in irrigating farms about that city was long ago realized by works described later in this chapter. In some cities the night soil was collected and delivered to farmers in more or less unsatisfactory ways. An attempt was made at Paris to render it less offensive by evaporating it and mixing it with gypsum, charcoal and other materials into a fertilizing material called "poudrette." There were 24 plants about Paris at one time where poudrette was made and they became so offensive that stringent regulations were prepared to govern their design and operation. Experience with the agricultural utilization of night-soil on a large scale finally proved that it was expensive and unsatisfactory, both as a method of disposal and as a means of fertilizing fields. It is practiced locally on a small scale in some places where sewerage systems have not been built and the local conditions are favorable.

Irrigation with sewage was carried on in about 30 French cities in 1910. The land irrigated was generally used for pastur-

[1] "Paris casts twenty-five millions of francs annually into the sea; and we assert this without any metaphor. . . Twenty-five millions are the most moderate of the approximate amounts given by the estimates of modern science. Science, after groping for a long time, knows now that the most fertilizing and effective of manures is human manure . . . There is no guano comparable in fertility to the detritus of a capital, and a large city is the strongest of stercoraries. To employ the town in manuring the plain would be certain success; for if gold be dung, on the other hand our dung is gold. . . Do you know what those piles of ordure are, collected at the corners of streets, those carts of mud carried off at night from the streets, the frightful barrels of the night-man, and the fetid streams of subterranean mud which the pavement conceals from you? All this is a flowering field, it is green grass, it is mint and thyme and sage, it is game, it is cattle, it is the satisfied lowing of heavy kine, at night it is perfumed hay, it is gilded wheat, it is bread on your table, it is warm blood in your veins, it is health, it is joy, it is life." ("Les Miserables," Book II, Chap. 1, Jean Valjean.) While the scientists of Hugo's acquaintance may have been assured of the financial figures he gives, it is an unfortunate fact that neither science, engineering nor agriculture has yet been able to obtain these millions represented by sewage, because the expense of building works to utilize sewage and the cost of operating the works exceeds the value of the sewage, and because the requirements of public health are paramount to those of sewage utilization.

age or growing grass and little attention was paid to the quality of the effluent. At that date Rheims was delivering about 12,000,000 gal. of sewage daily to a sewage farm of about 1,472 acres, which is equivalent to a rate of about 8,000 gal. per acre daily.

The sewage farms about Paris are the chief French examples of sewage irrigation. In 1910, the volume of the city's sewage, from a population of 2,800,000 in the districts connected with the farms, was 160,000,000 to 185,000,000 gal. daily, and 160,000,000 gal. was used for irrigation. The area of the farms is given in Table 67.

TABLE 67.—SEWAGE FARMS OF PARIS IN 1910

	Area of farms, acres		
	Privately owned	Owned by city	Total
Gennevilliers	1,996	15	2,011
Achères	386	2,965	3,351
Méry-Pierrelaye	3,731	1,235	4,966
Carrières-Triel	2,137	210	2,347
	8,250	4,425	12,675

The farms owned by the city had cost $7,220,000 in 1910 and the annual cost of operating them and distributing the sewage was about $1,000,000. Financial reports of the farming operations are not obtainable so it is impossible to ascertain the net cost of this method of treatment at Paris. The city's land is leased to tenants at rates ranging from about $5 per acre, where the tenant agrees to apply sewage to his land only as directed by the city's supervisors, to about $40 per acre where he uses the sewage when and how he desires. The owners of land within the irrigation districts take the sewage as they desire it.

The sewage is screened and settled. It is distributed through the farms in reinforced concrete conduits 1 to 4 ft. in diameter, lined with sheet steel where the pressures are heavy. These conduits have risers 1 ft. in diameter with outlets for the sewage into open carriers which distribute it over the fields. The land is partially underdrained at a depth of 13 ft., by drains of plain or reinforced concrete pipe. They discharge into open ditches with concrete lining.

German Sewage Farming.—In 1910 there were about 50 German cities having sewage farms. As a rule the sewage was distributed through the fields in furrows; separating the land into beds about 3 ft. wide and up to about 125 ft. long. Before sandy soil is laid out in this way it is sometimes flooded with sewage to a depth of 10 to 20 in. during the winter, as often as seems possible. This results in a deposit of sludge on the top of the soil, and this sludge is subsequently worked into the soil with a view to improving its fitness for farming.

At Magdeburg the sewage was sprayed over grass meadows by hose connected with portable 3-in. cast-iron pressure pipes laid on the surface of the ground. In a few cases surface flooding by the ridge method was followed. The general preference was to keep the sewage away from plants except as it could reach their roots by horizontal percolation through the soil.

The sewage farms of Berlin afford the largest opportunity for this method of treatment yet (1921) undertaken in any country. At the close of 1910 the area of these farms was as given in Table 68 and they were treating an average of 77,000,000 gal. daily, coming from a population estimated at 2,064,000. Of the prepared land 7,994 acres were used for broad irrigation, 12,250 acres for filtration, 502 acres for settling basins, and 2,105 acres for roads and buildings. The rate of filtration was about 3,700 gal. per acre of prepared land per day. The principal crops were rye-grass, turnips, cabbages, potatoes and grain. About one-fourth of the area was used for pasturage and there were about 40 acres of fish ponds which yielded fish worth about $3,200 annually.

TABLE 68.—BERLIN SEWAGE FARMS IN 1910

	Farmed by city	Leased to farmers	Unproductive	Total
Area irrigated and farmed, acres	16,657	3,956	395	21,008
Area farmed without irrigation, acres	10,647	2,486	8,868	22,001
	27,304	6,442	9,263	43,009

The cost of the farms to March 31, 1910, was about $17,470,000. The expenses for the year ending on that date were

$1,300,000 for maintenance and $742,000 for interest and loans. The receipts were $1,241,000 and there was an estimated increase of $122,600 in the value of live stock and other property.

According to Hazen[1] the soil is a light-brown sand with an effective size of 0.13 mm. and a uniformity coefficient of 2.5. The top soil is thin and differs so little from the lower material that it has usually been considered unnecessary to separate the two in grading operations. The thin growth of low pine which covers most of the land was cut off the tract to be prepared for farming, the ground was usually graded to a level surface with the help of light portable railways, and 2- to 3-in. underdrains were put in at depths of 4 to 6 ft. These drains were on grades as low as 1:250 in some places and were not more than 30 ft. apart. They discharged into main drains 4 to 7 in. in diameter with minimum grades of 1:500. When the effluent reached a quantity beyond the capacity of a 7-in. drain, it was carried away in open ditches. These had 1:1½ side slopes protected with brush and longitudinal rows of small stakes until willows planted along the edges of the banks developed enough to hold them.

The sewage reaches the farms through force mains with which small standpipes are connected. These are merely a few lengths of pipe set vertically to contain floats. Each float has a rod rising from it, to the top of which a large target is attached during the day and a lantern at night. The standpipe has an overflow which discharges on to the beds in the vicinity, but is not expected to come into action under ordinary conditions. The watermen are expected to watch the target or light and when it rises, to open enough gates on the pressure pipes connected with the force mains to keep the sewage from reaching the overflow. These gates discharge the sewage into small brush-lined basins with a screen of brush to intercept large floating material. These basins give off more odor than other parts of the farm, but are considered as satisfactory as any substitute that has been proposed besides being very cheap to construct and maintain. From these basins the sewage flows through open carriers formed in embankments around the beds into which the land is subdivided. The bottoms of the carriers are above the beds, so that the channels can be drained when not in use. A thick growth of grass soon covers them and holds their shape satisfactorily. Wooden boxes through the sides of the carriers, and

[1] *Eng. News*, Sept. 16, 1897.

wooden gates for closing the boxes and carriers, enable the watermen to control the discharge of the sewage over the beds.

Although sewage irrigation has had strong advocates among German engineers, largely because of the considerable areas of light sandy soil available for carrying it on, it also has its critics who predict its gradual abandonment. Dr. Dunbar states[1] the opinion of these critics as follows:

"I am convinced that it would be cheaper for many towns to abandon irrigation and replace it by artificial biological processes. It appears fairly certain that this will be the course of affairs as soon as the growth of the towns exceeds a certain limit. I do not doubt, for example, that many of us will live to see the day when Berlin will sell its irrigation farms for building purposes and construct artificial biological works in their place."

American Sewage Farming.—Sewage irrigation has not been tried in the United States on a large scale but it has been practised in a small way at a number of places, particularly in Southern California, for many years. The cities of Danbury, Conn., Pasadena, Fresno and Pomona, Cal., and San Antonio, Tex., are among the places where it has been given long trials with more or less success. Health officers report from time to time that sewage is treated by irrigation when what is done is to raise crops on intermittent sand filters, thereby running the risk of clogging the filters by roots and necessitating an early and expensive reconstruction of the beds. At one time irrigation was practiced extensively by market gardeners near Los Angeles, but it was unsatisfactory both to the health officers and to the farmers Irrigation has been carried on in a desultory way for many years at Hastings, Neb. At Salt Lake City the sewage is sold to a farmer for $1, but he makes very little use of it. Cheyenne, frequently mentioned as a city using sewage for irrigation, has not done so since 1890. Pullman, Ill., now a part of Chicago, the first large American city to irrigate farming land with sewage, gave up the practice many years ago.

At the State Farm at Howard, R. I., where an irrigation system was constructed in 1885 from the plans of Samuel M. Gray, the history has been different from that of most early American sewage farms, for the fields laid out then have been in continuous use down to the present time. One field has an area of about 2.5 acres and the other an area of 5.7 acres. Both had originally

[1] "Principles of Sewage Treatment," p. 266.

about 10 in. of fine light loam on a subsoil of sand and fine gravel, growing coarser as the depth increases. The surface was leveled and 3- and 4-in. tile drains were laid about 40 ft. apart at a depth of 5 to 6 ft. The sewage was screened and carried in channel pipe bedded in concrete on the top of embankments running around the sides of the land. This channel pipe had gates at intervals of about 100 ft. through which the sewage was discharged on the sloping fields. Dr. F. B. Jewett, superintendent, informed the authors in 1914 that it was the custom to turn the day sewage on the large lot and the night sewage on the other. Very little effluent appears at the outlets of the drains. Up to 1912 the land was cropped, vegetables being raised in the earlier years and corn later. The sewage was distributed in furrows. Since 1912 no crops have been raised and intermittent filtration has been practiced.

The more extended use of irrigation in Southern California than elsewhere is probably due to the suitable character of the soil for sewage farming and to the scarcity of water. The former condition materially reduces the cost of this method of treatment, while the latter condition greatly helps using the sewage in a fairly satisfactory agricultural way which does not have decided sanitary defects.

The sewage of San Antonio, Tex., is conveyed 12 miles from the city to a 6,700-acre privately-owned tract, where it was used in irrigating 1,500 acres in 1911, when the farms were visited by one of the authors. Ultimately about 4,000 acres will be irrigated, it is expected. The sewage amounted to about 12,000,000 gal. daily, so that the land was dosed at the rate of 8,000 gal. per acre daily. The irrigating season was from the middle of February to the middle of November and during the remaining 3 months of the year the sewage was stored in a lake having an area of about 1,000 acres.

CHAPTER XXI

COST ESTIMATING

The study of comparative costs of different types of structures and methods of building them is as important to the engineer as are considerations of strength, utility and beauty. The estimation of cost or value is essentially a problem in equitable comparison. It must be founded upon past experience. The most valuable data to the engineer will be found those acquired by himself upon work of the construction of which he has intimate knowledge.

The ability to distinguish and differentiate between the fundamental conditions materially effecting cost, to discard the unessential and study closely the important elements, must be cultivated. Every man must develop his own yardsticks. They will of necessity be founded largely upon his personal experience; the wider his experience and the simpler and better his methods, the sounder will be his ultimate judgment of costs. Clearness of conception and accuracy of information are essential. Therefore, it is important to the young engineer to give some study to this subject, to post himself as to methods which others have found from experience to be most useful, and to keep track of fluctuations in market prices of labor, machinery, materials and supplies, and of the influences affecting efficiency and output.

Bookkeeping vs. Cost-keeping.—There is an essential difference between bookkeeping and cost-keeping. The bookkeeper is interested in a correct financial accounting of the total cost of the final work and of such portions of it as are of interest to his principals. The engineer, on the other hand, is interested not only in final cost, but unit costs of different classes of work. Therefore, his accounting must serve to connect efficiency and output with cost.

Purposes of Cost-keeping.—The methods of cost-keeping will be governed very largely by the purposes for which it is undertaken. Thus the contractor desires not only a correct financial statement, such as would be furnished by a competent bookkeeper, which will enable him to keep track of his costs, month by month, to the completion of the work, but, what is far more important, an approximate daily record which will enable him to keep close track of efficiency and to do those things which may serve to lessen cost and increase output as well as excellence of work.

The engineer is interested chiefly in developing final costs of works of different character and unit costs of the major items entering into them, which he may use thereafter in the making of approximate estimates or comparative studies of cost of different works and methods of building them. His estimates are in general of three kinds; approximate or preliminary estimates which must be sufficiently accurate to guide him in design, and to aid his clients in making decisions as to extent, character and method of undertaking the execution of their work; monthly or progress estimates to govern periodic payments to the contractors; and final estimates, which must be sufficiently accurate to be equitable to the parties in interest in the case of active construction work, or which may serve as evidence of cost, or fair value, of engineering works for use in valuation or rate determination proceedings. They must be able to stand the test of rigid proof, cross-examination and court review.

The approximate current estimates of cost will also serve to disclose evidence of inefficiency and waste due to bad coordination or methods of work, dishonesty, and expensive delays in getting materials, supplies, equipment and labor.

Practical Advantages of Cost-keeping as Developed by a Successful Building Concern.—An admirable paper upon construction costs was presented before the Boston Society of Civil Engineers[1] by W. N. Connor of the Abertthaw Construction Co., with forms illustrating its up-to-date practice, which illustrates the practical necessity of cost-keeping. The student will find this article helpful. Connor classifies the functions of any cost accounting system in the order of their importance as follows:

"1. To aid in insuring the economical handling of the work while it is in progress;

"2. To obtain figures to compare with the estimate;

"3. To supply data for use in estimating future work." He calls attention to the fact that "costs must be available at once to be effective in aiding economical handling of the work. Delayed reports of costs are practically useless in effecting any savings while the work is in progress. Proper cost reports have been compared to a fire-alarm system in that they should not alone be able to give notice of some unusual condition, but equally lead the investigation to the seat of the trouble."

[1] *Jour.* Boston Soc. C. E., 1921.

"Each morning the labor costs of the previous day should be known. This is not necessary or feasible on all items of work, but on such as concrete, excavation, brickwork, etc., it is possible and extremely important that the superintendent of a job should know the daily labor-unit costs. If he has them, he can take up with the foreman at once any variations between estimates and actual performance and endeavor to equalize disproportionate costs of any unit before it is too late."

The president of the company, Leonard C. Wason, in commenting upon this paper, referred to the system as being the outgrowth of experience, and as being designed "to check the work when you can act on what you find instead of when it has become ancient history." He brought out the significant point that "8 to 12 items, out of the 50 to 70 total into which our records are subdivided, include nearly 80 per cent of the total cost of all operations. These are analyzed and on the superintendent's desk at 10 o'clock on the morning following the execution and he has all the rest of the day to act upon them, if necessary. So our costs enable him to keep the work under absolute control and thereby savings are made."

The Methods of Construction should be recorded, as these and the size of the work have a marked bearing upon cost. The work may be well balanced or it may be overmanned or overequipped.

Day Work vs. Contract.—The method of executing the work and paying for it is also an important factor, as the psychological conditions have a marked bearing upon cost. So does the extent to which the element of competition can be introduced into the work.

Various methods of payment for contract work have been used, the most common being the so-called lump-sum and unit-price methods. Other methods have also been used such as cost plus percentage profit, cost plus fixed profit, cost plus profit with maximum and minimum limits of total cost or with a variable division of the difference between the actual cost and the nominal cost fixed by the contract.

Simplicity.—It is important that the cost-keeping system should be as simple, inexpensive and flexible as possible. The greater the simplicity within the limits of adaptability, the more useful will the system prove. Complexity in detail is likely to lead to incomparability.

While it may be desirable and often is to subdivide units in some detail on specific classes of work, it is important that the total cost should also be kept divided into unit items of simple character, such as excavation, concrete masonry, etc.

Local conditions must be studied; the transportation facilities, the nationality, character and condition of available labor; legal limitations; the general business conditions as to supply and demand, and standards of efficiency.

Two Methods of Estimating.—Cost-estimating methods may be grouped under two general classes; the first, based upon the comparative final costs of completed works with judicial estimate of the influence upon cost of the difference in existing conditions and environment; the second, the determination of costs by a synthetic building up of the items entering into or making up the unit structures.

Under the first method, for instance, the total costs of intermittent sand-filter plants may be expressed per 1,000,000 gal. of rated capacity or per acre of area; the cost of pumping stations equipped with pumps of certain types, such for instance as high duty fly-wheel, or duplex, or centrifugal, per 1,000,000 gal. of daily capacity or per water horsepower: the cost of trickling filters may be based upon capacity in millions of gallons daily, acre-feet of area, or number of persons served. It is essential that structures of the same general type only be compared and that the estimator familiarize himself as far as possible with the character of the individual plants from which the cost figures were adduced.

Under the first method, it is also possible to segregate the entire costs of the structure in question, such as a trickling filter, under a few important items, such as crushed stone in place, per cubic yard; underdrainage system in place, per square foot or per acre; distributing system, including nozzles and appurtenances, per acre of bed; engineering and overhead costs; and interest during construction. The average unit costs of each of these groups may then be found, to aid the memory, and all of the figures covering different records available should be classified and tabulated under these groups and the engineer should post himself as far as possible as to the wages, conditions and character of the work and the locality in which it was done. In using the results thereafter for estimating the cost of his own project, with these individual figures and their average before him, the engineer would select the unit cost which seemed most reasonable to him in the light of his data and knowledge for each of the similar items of his job, and thus obtain an estimate of final cost.

Under the second or synthetic method, the records of various works would be tabulated according to the items of the contracts under which they were paid for with perhaps some consolidation of items, but the number of items which would be carried might well be from three to five times as many as under the method first mentioned. These records, as in the first case, would be classified, tabulated and perhaps averaged under different kinds of work, such as cement, sand, stone, mixing concrete, forms, placing, etc. From the facts thus marshalled, the engineer would make estimate of the cost of his own project by applying to the items, into which he had divided it, figures which appeared reasonable in the light of his assembled data.

If the synthetic method is based upon unit costs developed in actual works fairly interpreted, the difference between these two methods becomes one of degree of segregation of items rather than of principle.

It is important that in assembling the facts, the elements of cost, resulting from hazard, extra work or litigation, as well as engineering, administration, and overhead items should be included, either as separate items or be distributed amongst the other items into which the entire cost is subdivided, so that the figures may represent full cost and not partial cost.

Both methods are serviceable. They are based upon past records of actual cost, found in the construction of plants more or less similar in character. The latter involves the attempt to segregate the elements of cost in somewhat greater detail. The former is open to the danger that some of the influences upon cost may be overlooked in the broad review of these past costs, but it has the advantage that with the small number of classified items, the figures and conditions are more easily visualized and that the final full cost of the other works is always kept clearly in mind.

Under the synthetic method, there is danger that in the more detailed consideration and segregation of the elements and items entering into cost, the mind may be centered upon the details of the segregation, rather than upon a comparison between the resulting final estimate and the past records of cost, with the usual result that the synthetic estimate will be too low, because of the omission of unforeseen and unforeseeable items, or, in other words, of the hazard element. For the latter reason, it is desirable, after completion of the estimate by the synthetic method, to compare the final results with the records of cost based upon experience interpreted by simple overall unit costs or by the ratio method hereafter referred to.

While it rarely happens that the hazard elements, or the causes of unexpectedly high items of cost, are the same in different engineering works, they are always present in greater or less degree. It has come to be common practice, therefore, to introduce an item of "contingencies" to cover them, in the form of a percentage added to the general estimate of cost.

Contract Prices to Be Used with Caution.—In a brief but admirable article in *Engineering and Contracting*[1] entitled, "The Art of Making Rapid and Reliable Estimates of Cost," Hazen has pointed out some of the dangers in the use of contract prices and outlined two desirable methods of estimate, "The Weighted Price Method" and "The Ratio Method."

Contract prices or quotations for work with which the engineer is not personally familiar, should be used with caution if taken as a guide in making estimates of projected work. Usually such available prices or quotations do not apply exactly to the conditions under which the projected work is to be done and may not cover just the same items; the items of extra work, growing out of conditions that either were not anticipated or were intentionally excluded from the contract, may be overlooked or ignored; different items or classes of items may readily be omitted or overlooked in a comparison of contract prices; the schedule of quantities prepared in advance of any work may fail to agree closely with the final estimate unless the engineering work from which the contract was drawn was thoroughly done; usually contract quotations do not cover certain items, such as cost of rights-of-way and engineering expense; finally there are often wide differences among the quotations upon the same price of work and sometimes the lowest bidders underestimate the difficulties of the work, suffer serious loss in its execution and augment the cost to the owner by litigation. These are elements of cost not directly disclosed by the contract prices. For these and other reasons, estimates predicated *wholly* upon contract prices are likely to be materially lower than the final cost of any work.

Weighted-price Method of Estimate.—Under this method, the effort is made to segregate into a very simple classification (such as that used by Hazen in his filter-construction practice, to wit, excavation and earthwork, masonry, filtering materials, piping

[1] March 18, 1913.

and auxiliaries, the available final costs, records of which are available and which should be as comparable as possible with the class of structure the estimated cost of which is to be determined.

All of the items of cost making up the final cost of the works are then segregated into these collected groups. In this way, unit figures are finally derived from which may be selected those on the whole most closely resembling the anticipated conditions, and these costs may be applied with less liability of overlooking the contingent or hazard element always present in one form or another in engineering construction.

This is an excellent method where the engineer has accumulated a sufficient quantity of reliable data. It is necessary to use such data with caution because of changes in prices of labor and materials and in methods and machinery which may have taken place from time to time as the data have been accumulated.

The Ratio Method.—Under the application of this method, which is generally used where subdivision or segregation of individual classes of items cannot readily be made for want of full information, a schedule is made of a limited number of items of work which may reasonably cover the greater part of the construction in the several cases, the records of which are available, and a simple schedule of unit prices is then formed corresponding to these units. Then one *fixed price* is finally assumed for each kind of work, and it becomes the standard of comparison for such units of work. In assuming this fixed price, the young engineer will do well to benefit by the experience of older practising engineers. The price assumed should, of course, be a reasonable one and as nearly as known *an average one,* but precision is not to be expected and round figures are always used. The amount of work under each item in each job is then ascertained and the assumed prices are applied to them. The sum of the amounts for each job represents what that job would have cost *at the assumed price* and comparison between the *actual cost* and the *cost at the assumed price* gives a coefficient or an idea of the relative economy or difficulty of the work on a ratio or percentage basis. When the records of a number of known pieces of work are compared in this way, the assumed fixed-unit prices and coefficients found on different jobs, furnish a criterion for making preliminary estimates of cost of work in the same class.

If the young engineer will get into the habit of comparing the costs adduced in the works with which he may be connected in some such simple manner as this, he will much more rapidly acquire a sense of proportion than he could possibly do by simply comparing, in memory or by actual record, the final costs of the works or the contract prices paid for them.

In using this yardstick (ratio method) in making new estimates, the fixed prices of the basic schedule will be applied to the various classes of items involved and the final cost thus built up will then be modified by the coefficient or the ratio or percentage item which judgment may dictate as the most probable under the conditions prevailing, as compared with all of the past records known to the estimator which have been analyzed theretofore.

"In arriving at the ratio to be used, the engineer will compare, perhaps in his mind and without written schedules, the ease or difficulty of the proposed work, as compared with the ease or difficulty of the various works which served as a base; will take into account differences in labor conditions, in freights, in deliveries; will take into account the known or assumed efficiency, or lack of efficiency, in the execution of the several pieces of work from which his basic data were derived; and will reach an estimate of the addition or subtraction to be made to or from the base price in each case" (Hazen).

Effect upon Cost Estimates of Length of Construction Period.—The longer the period of time involved in building the works, the greater the hazard of estimate, particularly under such conditions as those of the present time (1921), following the Great War. This will be clear from the diagram in the Preface to this book. Even under more stabilized conditions, such as the pre-war conditions, the element of chance is an important one. This was interestingly illustrated in a large contract for public utility works, built in New York City shortly before the war, at a cost of approximately $12,000,000 and requiring a construction period of about 8 years. As to the estimate of cost upon which the bidder based his proposal for the work and upon which contract was awarded, one of the successful contractors stated after the completion of his work, that one of his greatest difficulties in bidding had been in attempting to correctly forecast the average price fluctuation in labor, materials, supplies, and money, which might prevail during the construction period of 8 years. He gave point to his remarks by stating that during the execution of his contract, two distinct minimum and maximum scales of prices were encountered which chanced to average up fairly closely with his forecast. Had either one of the minimum periods been eliminated he and his associates would have lost heavily: whereas if either of the maximum-cost periods had been eliminated they would have enjoyed a very substantial increment in profit.

Extra Cost Involved by Complexity and Novelty of Design.—An interesting example of the effect upon the contractor of the difficult labor conditions induced by the war, was shown in the comparison of bids upon two different types of sewer construction at Lynn, Mass., in 1920, the one involving simple mass concrete construction, the other substantially lighter reinforced concrete structure. The heavier type of structure commanded the lower bid, although there can be little doubt that under normal conditions with reasonably skilled contractors bidding, the reinforced type of structure would have proved the cheaper.[1]

Similarly in the letting of a contract for a 10,000,000-gal. covered concrete masonry reservoir at Dayton, Ohio, in 1917–1918, a more attractive bid was obtained upon a reinforced. flat-slab roof supported by cylindrical columns than upon a groined-arch type of structure which would normally have yielded the lower bid. The reason for this condition was found in the fact that the local contractors had not had experience in groined-arch construction and were adversely influenced by the failure of such a type of structure built in Cleveland sometime prior thereto, whereas they were skilled in building reinforced flat-slab concrete floors and had available forms adapted to the proposed structure, which had been used on other work.

In each of the above cases, therefore, abnormal local conditions governed the type of structure chosen and the minimum price.

Classification of Items.—In general, costs may be segregated under the items of labor, materials, supplies, plant, general expense and overhead costs, and profit.

Under general expense are usually included engineering and legal costs, bonding and liability insurance, bookkeeping, timekeeping and storekeeping, rentals, office and miscellaneous expense (telephone and telegraph, stationery, supplies, etc.), interest and discount.

The cost of the major items may for convenience be further subdivided into different items. Thus, the item of excavation may be subdivided into excavation in trench, hoisting excavated material out of trench, hauling or transportation, wasting and

[1] *Eng. News*-Rec. vol. *lxxxv*, No. 4; July 22, 1920, p. 148.

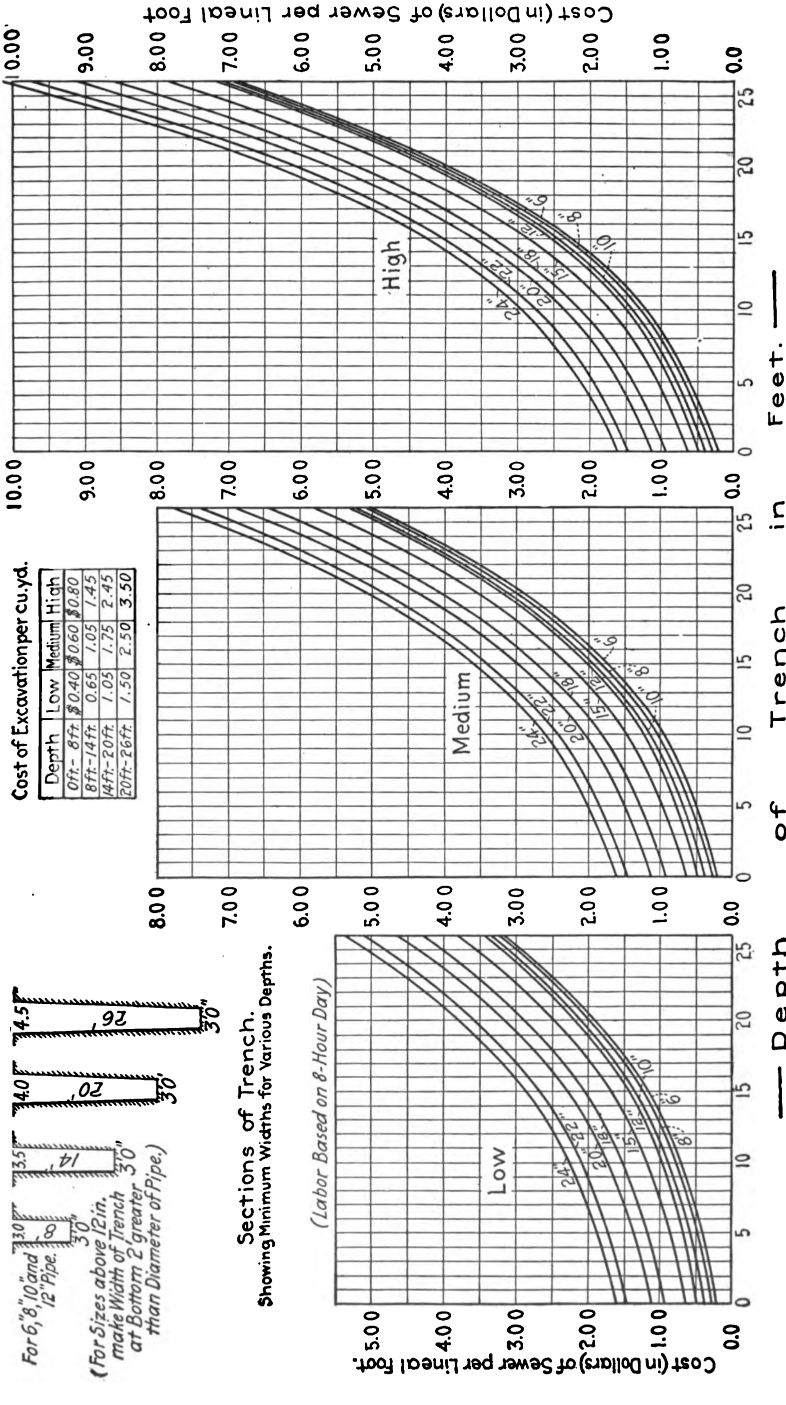


FIG. 218.—Curves giving the cost of sewers under different conditions. Trenches excavated by hand, with labor at 25 cents per hour.

spoiling, backfilling, compacting, ramming or puddling, shoring, sheeting, etc. It may involve the use of machinery, and fair allowances for its operation, repair, rental and depreciation. The operation of the machinery may in turn involve supplies, coal, oil, water, miscellaneous items, etc. While the keeping of these subdivided elements of cost may be very helpful to the young engineer, the ascertainment of the larger elements of cost is far more important, because of their comparability with the records of others.

Methods of Applying Data on Cost.—Special methods of applying data on cost must be developed for different pieces of work. Different men may accomplish the purpose in a thoroughly satisfactory but totally different manner.

Fig. 218 and Table 69 will serve to indicate methods for preparing estimates which have been used in certain cases, but it is to be clearly borne in mind that they are not generally applicable and apply directly only to similar conditions.

In Chapter VI, Figs. 111, 112 and 113 give diagramatic methods of showing the theoretical quantity of excavation for trenches of different depths and widths and in Chapter VII, Table 29, methods for showing quantities of lumber required for shoring sewer trenches in the manner assumed.

In Table 69 is indicated a method of assembling data relating to the cost per linear foot of laying sewer pipe of different diameters and in Fig. 218 are shown curves giving the cost of pipe sewers under the different specific conditions indicated. In Figs. 219 and 220 are shown diagrams of cost of circular and semi-elliptical sewers and of vitrified pipe sewers made by the authors for use in estimating cost of sewer construction for a report in May, 1919 to the City of Springfield, Mass. These diagrams, too, are applicable only to such conditions as were assumed to prevail in Springfield at that time. They are shown merely for the purpose of indicating methods and not for the submission of specific data on cost.

Progress Reports.—Progress reports, recording the number of men employed, the wages paid and the amount of work accomplished along simple lines, are also desirable. They should be prepared to meet the specific character of work, method of construction and form of contract, and be so simple in form as to have wide applicability and convenience in use. Table 70 is an example of such a simple form prepared for insertion in the

TABLE 69. —COST OF PIPE LAYING PER LINEAR FOOT (Excluding Earthwork)

Item	Diameter of pipe in inches								
	6	8	10	12	15	18	20	22	24
Pipe:									
Deep and wide socket	$0.108	$0.148	$0.216	$0.270	$0.364	$0.608	$0.743	$0.990	$1.073
Standard	0.104	0.143	0.208	0.260	0.351	......	......	......	......
Branches and stoppers:[1]									
Deep and wide socket	0.016	0.020	0.027	0.032	0.042	0.064	0.077	0.100	0.108
Standard	0.015	0.019	0.025	0.030	0.038	......	......	......	......
Loss, shipping and inspection	0.009	0.012	0.018	0.022	0.030	0.042	0.050	0.066	0.072
Hauling (1 mile) unloading, inspection and piling	0.009	0.012	0.018	0.024	0.039	0.047	0.059	0.072	0.095
Cement and sand:									
Deep and wide socket	0.005	0.007	0.009	0.010	0.016	0.017	0.022	0.027	0.030
Standard	0.002	0.004	0.004	0.005	0.006	......	......	......	......
Laying and joining	0.045	0.060	0.075	0.090	0.112	0.135	0.150	0.165	0.180
Lights and watchman	0.009	0.009	0.010	0.011	0.013	0.015	0.018	0.023	0.023
Liability insurance—8 per cent. of labor	0.004	0.006	0.007	0.008	0.010	0.012	0.013	0.015	0.016
Total (excl. of excav.)—deep and wide socket pipe	$0.205	$0.274	$0.380	$0.467	$0.626	$0.940	$1.132	$1.458	$1.597
Standard pipe	0.197	0.265	0.365	0.450	0.599	......	......	......	......

[1] Includes setting.

TABLE 70.—DAILY CONSTRUCTION REPORT

	Wages per hour, (cents)	Total hours on work	Force account: Item 1a	Item 4a	Item 6	Item 7		Item 11a		
Superintendent										
Foremen	95	16	8	4		4				
Foremen										
Mechanics	85	8	8							
Engineers	90	8		8						
Chauffeur	80	5		5		(mix	er en	gine)		
Carpenters										
Bracers	75	16	16							
Masons	90	4			4					
Mason helpers	70	12			12					
Laborers	70	16	16			(brac	ing)			
Laborers	70	61	61		(dig	ging	and	back	fill	(5)
Laborers	70	38		20		8		10		
Laborers	75	8		4		4				
Teams (double)	100	8		8						
Teams (single										
Auto truck										
Waterboy	50	8	7	1						

Job	From station	To station
Date July 20, 1920		
Pavement removed	11+05	11+30
Cut 7 ft. deep, machine	10+58	11+05
Cut 7 ft. deep, machine		
Cut 7 ft. deep, machine		
Trench sheeted both sides, 7 ft.	10+52	10+70
Pipe laid and jointed		
Pipe laid and jointed		
Brick invert laid	10+05	10+35
Steel placed { Invert	9+12	9+33
Steel placed { Walls		
Steel placed { Roof		
Concrete poured { Invert	9+12	9+32
Concrete poured { Walls		
Concrete poured { Roof		
Number of bags of cement used	18	
Forms removed { Invert		
Forms removed { Walls		
Forms removed { Roof	8+40	8+70
Backfilling completed		
3 ft. below surface	9+36	9+60
3 ft. below surface		
Pavement replaced		
Sheeting left in place by contractor { Both sides		
Sheeting left in place by contractor { Both sides	9+32	
Cut off 3 ft. from surface		9+97
Rock measured		
	A.M.	P.M.
Weather	Fair	Fair
Temperature	80°	85°
	JOHN DOE	*Inspector.*

field notebook for the use of the inspectors upon a certain sewer job, the final estimate for which follows this form. The back of the form was left blank for notes and sketches.

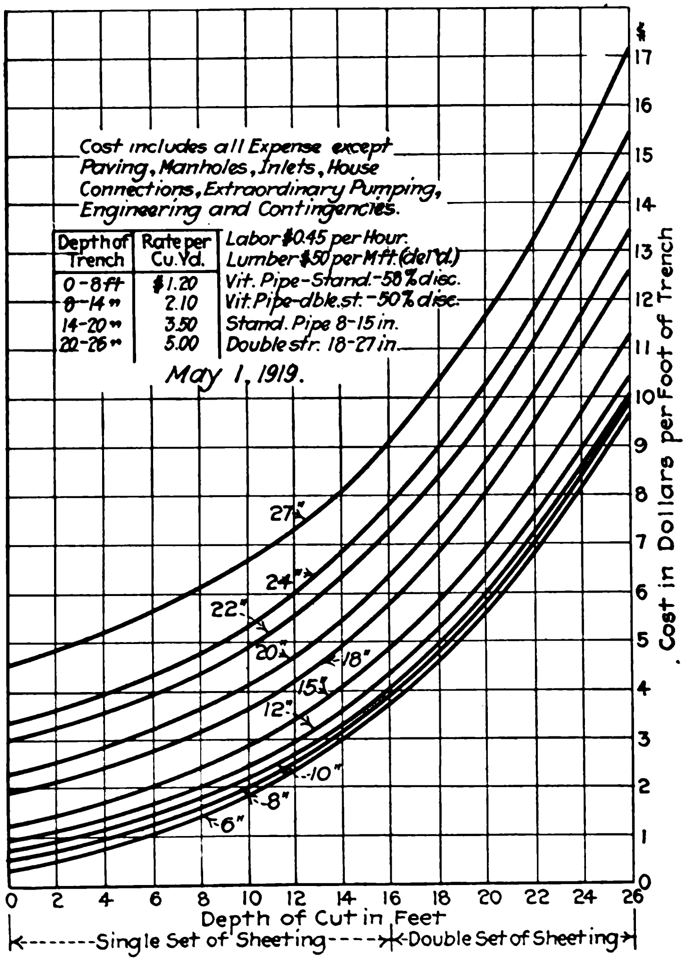


FIG. 219.—Cost of vitrified pipe sewers.

Final Estimate.—An example of a form of final estimate, upon a piece of sewer construction done by contract, is appended. The unit prices involved were abnormally high, as the work had to be let at the peak of war prices: therefore they are of no significance and are not of general applicability.

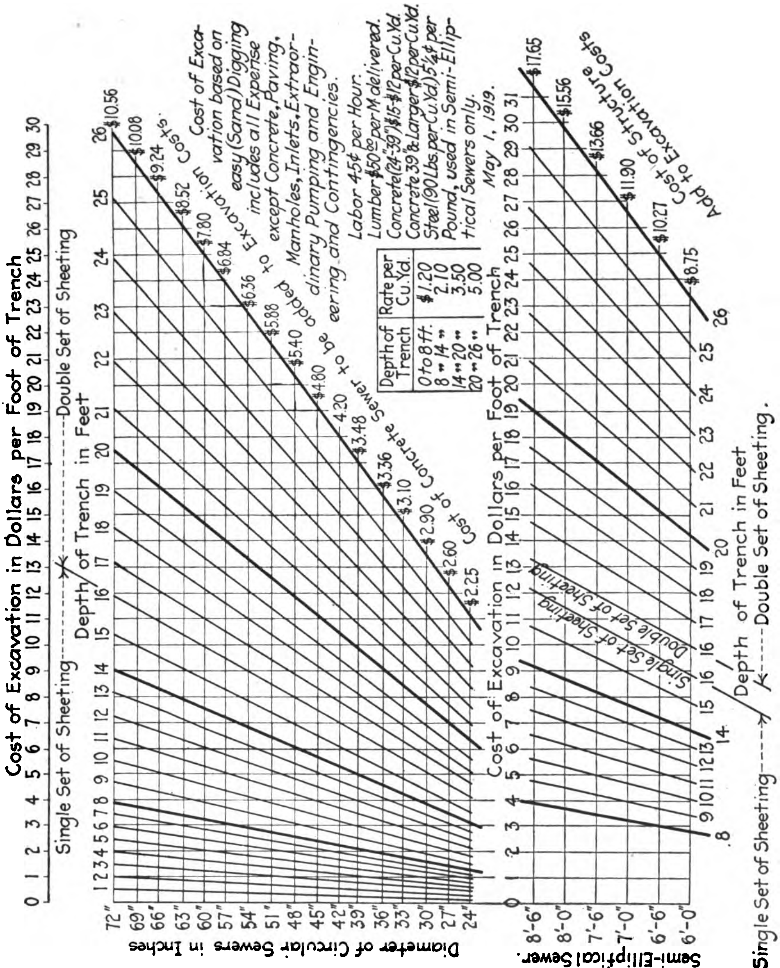


FIG. 220.—Cost of concrete sewers.

Form for final estimate. CITY OF......................

SEWER COMMISSION

Final estimate of work done by...............................contractor,.........................
on contract with the City of........................ for the construction of sewers, drains and appurtenant structures
in.............Street, from...................Square to................Street. Work completed............1920.

Item number	Item	Quantity previously estimated	Quantity to date	Unit price	Amount
1*a*	Earth excavation and refill	1,760 cu. yd.	2,625 cu. yd.	$9.50	$24,937.50
1*b*	Earth excavation below masonry	none	none		
2	Gravel refilling	none	none		
3	Rock excavation	none	none		
4*a*	Class *A* concrete	310 cu. yd.	375 cu. yd.	25.00	9,375.00
4*c*	Class *A* concrete	5 cu. yd.	10 cu. yd.	40.00	400.00
4*d*	Class *B* concrete	none	0.5 cu. yd.	10.00	5.00
4*e*	Concrete treated	23 cu. yd.	33 cu. yd.	0.50	16.50
5*a*	Brick masonry	6.5 cu. yd.	22 cu. yd.	30.00	660.00
6	Vitrified-brick lining	180 sq. yd.	234 sq. yd.	4.00	936.00
7	Steel reinforcing bars	3.0 tons	3.0 tons	150.00	450.00
8	Furnishing vitrified pipe:				
	12 in. standard pipe	none	18 ft.	1.35 less 40 per cent	14.58
	10 in. standard pipe	8 ft.	6 ft.	1.05 less 40 per cent	3.78
	8 in. standard pipe	20 ft.	118 ft.	0.70 less 40 per cent	49.56
	6 in. standard pipe	2 ft.	160 ft.	0.45 less 40 per cent	43.20
	8 in. curves	3	5	2.80 less 40 per cent	8.40
	6 in. curves	none	2	1.80 less 40 per cent	2.16
	8 in. elbows	1	1	2.80 less 40 per cent	1.68
	12 in. slants	none	1	5.40 less 40 per cent	3.24
	10 in. slants	none	1	4.20 less 40 per cent	2.52
	8 in. slants	5	8	2.80 less 40 per cent	13.44
	6 in. slants	13	20	1.80 less 40 per cent	21.60
	12 in. stoppers	none	2	0.675 less 40 per cent	0.81
	8 in. stoppers	5	4	0.35 less 40 per cent	0.84
	6 in. stoppers	13	11	0.225 less 40 per cent	1.49

9	Laying vitrified pipe:				
	12 in. pipe	none	20 ft.	0.10	2.00
	10 in. pipe	8 ft.	8 ft.	0.10	0.80
	8 in. pipe	33 ft.	122 ft.	0.10	12.20
	6 in. pipe	28 ft.	190 ft.	0.10	19.00
10	Underdrains	none	none		
11a	Granite-block pavement, sand base	110 sq. yd.	712 sq. yd.	1.00	712.00
11b	Grouted granite-block pavement, on concrete base	50 sq. yd.	98 sq. yd.	1.00	98.00
12a	Granolithic sidewalk	none	15.6 sq. yd.	2.00	31.20
12b	Brick sidewalk	none	none		
13a	Manhole frames and covers	2 sets	3 sets	25.00	75.00
13b	Cast-iron cover	none	1	15.00	15.00
13c	Manhole steps	13	20	0.50	10.00
13d	D-grates and frames	none	3	50.00	150.00
13e	Catch-basin traps	1	4	20.00	80.00
13f	Inlet curbs	none	3	20.00	60.00
13g	Catch-basin top and gutter stone	none	1 set	100.0	100.00
14	Handling sewage flow			lump sum	1,000.00
15	Building and catch-basin connections	6	18	10.00	180.00
16	Cleaning up			lump sum	100.00
17	Lumber left in place	none	none		
18a	Removing electric-railway tracks			lump sum	100.00
18b	Relaying electric-railway tracks			lump sum	100.00
18c	Removing conduit manhole	1	1	100.00	100.00
19	**Extra work**	none	none		
20	Connections between old and new work—square			lump sum	5,000.00
21	Connections between old and new work—street			lump sum	1,000.00
22a	Maintaining traffic			lump sum	100.00
	Total value of work				$45,992.50
	Premium for completion of work before contract time—75 days at $25.00				1,875.00
	Total				$47.867.50
	Less payment due on estimate No. 1, $11,319.00				
	Less payment due on estimate No. 2, 14,357.68				
	Less payment due on estimate No. 3, 12,992.32				38,669.00
	Amount due under this estimate				$9,198.50

This is to certify that the above work has been done and performed to the satisfaction of the Engineer and in accordance with the provisions of the contract.

METCALF AND EDDY, *Consulting Engineers*,

by...................... ...., 1920...................City Engineer,................, 1920.

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