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AMERICAN SEWERAGE PRACTICE

VOLUME III

DISPOSAL OF SEWAGE

BOOKS BY

METCALF AND EDDY

AMERICAN SEWERAGE PRACTICE

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AMERICAN SEWERAGE PRACTICE

VOLUME III

DISPOSAL OF SEWAGE

BY

LEONARD METCALF

AND

HARRISON P. EDDY

REVISED BY

HARRISON P. EDDY

THIRD EDITION

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PREFACE TO THE THIRD EDITION

The developments in the art of sewage treatment during the nineteen years since the publication of the second edition of this volume have been so numerous and have spread out in so many directions that it has been necessary to rewrite completely a large proportion of the text. As the fundamental principles involved in the older methods of treatment in large measure underlie the newer processes and as many of the more recent installations consist of combinations of the older and the newer features, an effort has been made to devote adequate space to both groups of processes and thus to maintain a balance that would be most helpful to readers in general, rather than to devote an excessively large proportion of space to the new processes, which naturally are of absorbing interest to many investigators. A few cost data have been included, for the purpose of giving readers a general idea of the costs of constructing and operating certain features of treatment plants.

The death of Mr. Metcalf in January, 1926, left upon the surviving author the responsibility for this revised edition. In the preparation of this volume, he has been assisted greatly by many engineers and others who have furnished information and data. Particular acknowledgment is made to Professor Gordon M. Fair of Harvard University, who suggested the adopted rearrangement of chapters and the order of treatment of various subjects and who also rendered valuable assistance in the preparation of the text for several chapters; to Professor Melville C. Whipple, also of Harvard University, who assisted in the preparation of other parts of the text; to the author's partners for helpful suggestions; and to various members of the staff of Metcalf & Eddy who have aided in the preparation of the book, especially to Frank L. Flood, Guy E. Griffin, Stuart E. Coburn and Wallace W. Sanderson, and to George C. Houser who has edited the entire volume for publication.

HARRISON P. EDDY.

BOSTON, MASS.,
October, 1935.

PREFACE TO THE FIRST EDITION

The purpose of this volume is twofold: first, to explain in simple, non-technical language the nature of sewage and the changes that take place in it when it is subject to different conditions, and second, to describe the structures designed to produce these various conditions, in order that the character of sewage may be changed to the desired extent before it finds its way into some body of water. The first purpose is carried out in Chapters I to VI inclusive.[1] The remainder of the volume is intended primarily for designing engineers and operators of such plants.

So far as the authors are aware, they have treated the subject in a new way, which has been adopted as a result of their conviction that in the present state of knowledge concerning sewage disposal, highly technical discussions of disputed theories were undesirable in a book intended to be helpful not only to engineers but also to sewer commissioners, lawyers and under-graduate students. Some of the chapters have been rewritten several times in order to avoid repetition and needless technicalities. Other chapters would be materially changed if the practical experience of today but furnished adequate precise data, rather than a certain amount of general information from which have been drawn the inferences here given. This is particularly true of the subjects of screening, the American use of contact beds, the oxidation of sewage directly or indirectly by aeration, and the disposal of sludge.

For the opportunity to present the large amount of information regarding American practice in sewage disposal contained in this volume the authors here gratefully acknowledge the valuable and generous assistance given to them by their engineering colleagues and friends, whose co-operation in this undertaking has been typical of the spirit of mutual professional helpfulness that has been responsible for a large part of the recent progress in this field. In the present state of the subject, the experience in other countries affords much valuable information which has been employed liberally, although this treatise is primarily intended to be a survey of American Sewerage Practice. For a considerable part of this information the authors are indebted to engineers in England, France and Germany, whose cordial aid in their task has proved one of the most pleasant features of the preparation of the volume.

The authors have kept constantly in mind the fact that while there is potential danger to public health in sewage, the disposal of this class of

[1] These chapter numbers refer to the first edition and are not applicable to the present edition.

municipal wastes is not alone a technical problem but one which calls for heavy, continuing expense. It is something in which the sanitarian and the civic economist are as interested as the engineer, and the authors have endeavored to treat the subject in a way to recognize this fact. Skimping funds may often lead to danger to the public health, and the enforcement of requirements for an unnecessarily high degree of purification of sewage to useless waste of money. It is the engineer's duty to safeguard the public health and to advise wise limits of expenditure. This can be done most effectually by insisting that each undertaking shall be considered upon its own conditions and that the trained specialist in this branch of engineering shall be the judge of the significance and applicability of experience gained with disposal works elsewhere. The danger of failure resulting from copying plans of one plant for use in another locality is very real in the field of sewage disposal.

The extent of the sewage treatment works in this country is indicated by statistics compiled under the direction of George M. Wisner, Chief Engineer of the Sanitary District of Chicago. These are based on the census of 1910 and indicate that out of a total population of about 91,600,000 in the United States, about 34,700,000, or 38 per cent, lived in places provided with sewerage systems. Of these systems, those serving a population of 3,900,000, or 11 per cent, were provided with sewage treatment works. The sewage of 89 per cent of this population in sewered places was discharged untreated into water. This shows the importance of dilution as a means of disposal. About 10 per cent of the population served by sewers lived in places having basins or tanks for treatment, 3 per cent where intermittent filters were used, 1 per cent contact beds, and 4 per cent trickling filters. Some of these places have two of these methods of treatment in use, so that the total of the figures just given is greater than the 11 per cent previously mentioned as served by systems with some form of treatment works.

The art of sewage treatment has made radical and important advances during the last 25 years, and it is to be expected that this progress will continue. The number of persons engaged in the study of sewage disposal problems is increasing rapidly, which, with free interchange of ideas, must stimulate more rapid future progress in the perfection of methods. The prospect of such improvements rarely justifies delay in the installation of needed treatment plants, however, for there are now available methods of economically accomplishing any degree of purification which may be required, nor is the discovery of a better method often cause for just criticism of those responsible for one already in use. Improvements in every field follow careful investigation and change in conditions.

BOSTON, MASS.,
October 6, 1915.

LEONARD METCALF.
HARRISON P. EDDY.

CONTENTS

AMERICAN SEWERAGE PRACTICE

CHAPTER I

DEVELOPMENT OF SEWAGE TREATMENT AND DISPOSAL

Various authors have defined *sewage* in different ways, generally with the idea of indicating that it is largely the water supply of a community after it has been fouled by a variety of uses. From a consideration of the sources of sewage, it may be defined as a combination of the liquid, or water-carried, 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.

Earliest Methods of Sewage Disposal.—Although the treatment of sewage is a relatively new art, which has been developed within the past 60 years, the removal of sewage from human habitations is an ancient problem. This fact is demonstrated by certain archaeological discoveries made in Asia a few years ago, when the city of Mohenjo-daro was excavated. Situated on the west bank of the Indus River, 1500 miles from Babylon, Mohenjo-daro is believed to be more than 5000 years old. The buildings had bathrooms with well-laid floors and latrines occupying recesses in the walls. Vertical pipes led the effluents from the latrines to drains laid underneath the house floors. Water chutes were cut in the outer walls of the houses and a large sewer was laid along the street to carry away the sewage.

In America the earliest sewers were introduced mainly to remove storm water. This storm runoff was often foul and contaminated the river or other body of water receiving it. Sometimes the contamination was increased by the night soil gathered by cesspool-cleaning contractors, who in many cases threw it into the water course most convenient. The introduction of water closets rendered impracticable the discharge of household wastes into tight cesspools, such as were generally used in England, and led to the use of sewers for the removal of such wastes. The provision of sewers soon served to eliminate filthy privy vaults and overflowing cesspools in populous districts. These common sewers discharged generally into the nearest stream or other body of water. By this practice, the annoyance at the individual premises was transferred to the point of discharge of the sewer, resulting in the creation of offensive conditions in the stream or along the waterfront.

As long ago as 1842, the Poor Law Commissioners of England reported that outfalls should be constructed to remove the sewage of cities to tracts of land where it could be disposed of without polluting the streams. Although a public health act was passed in 1848, it had little effect, and it was not until the Nuisance Removal Act of 1855 was passed—at the close of a severe cholera epidemic—that effective legislation began. The evil condition which was fought was understood only vaguely at that time. The legislative purpose was to prevent rivers and other receivers of sewage from becoming offensive to the eye and nose. If neither of these organs could detect anything unpleasant, it was believed that there was no ground for serious complaint against the method of disposal.

Dilution.—Although sewage disposal by dilution was the earliest method adopted by municipalities, it was not until about 1887 that investigation of the principles entering into it was made by the late Rudolph Hering in connection with the problem of sewage disposal at Chicago. His studies resulted in the construction of the Main Drainage Canal of the Sanitary District of Chicago, completed in 1900. In Massachusetts studies undertaken at about the same time established the limiting dilution ratios which served as a basis of general practice for many years. More recently much information bearing on the disposal of sewage by dilution has been accumulated. Particularly noteworthy are the studies made by the United States Public Health Service in connection with the Ohio and Illinois rivers.

Although seacoast municipalities have seldom adopted disposal methods other than that of dilution, it has often proved necessary, even in their case, to take precautions with respect to the location of outlets, in order to prevent the pollution of shores and bathing beaches.

Broad Irrigation.—Beginning about the middle of the sixteenth century the sewage of Bunzlau, a German town, was used for irrigating cultivated ground, and the same practice was instituted at Ashburton and other towns in Devonshire, England, at the beginning of the eighteenth century. A famous example of sewage irrigation existed at Edinburgh for about a century. Garner states (1).[1]

By 1876 about sixty towns in England had tried irrigation, and many schemes had proved failures both financially and otherwise, chiefly because there was a clashing of economic and sanitary interests, which are often diametrically opposed, frequently because the land was quite unsuitable for the purpose and rapidly clogged and became coated with a layer of

[1] Figures in parentheses refer to bibliography at end of chapter.

putrefying filth. In some cases, the sewage contained trade wastes, harmful to crops, and more often than not, the management was below standard.

In the United States, the State Insane Asylum at Augusta, Me., irrigated hay fields and vegetable gardens with sewage prior to 1875. Among other early attempts at broad irrigation in this country may be mentioned the farm established at Lenox, Mass., in 1876, where both subsurface irrigation and broad irrigation were used, and the broad irrigation areas at the State Insane Asylums in Worcester, Mass., in 1876, at Amherst, Mass., and at Pullman, Ill., in 1881. However, owing to the large tracts of land required, and also for aesthetic reasons, this method of treatment has never been widely adopted in the United States.

With the development of arid and semiarid lands in the West, however, there is an increasing tendency to apply settled and even more highly purified, in some cases disinfected, sewage to the land, for the watering of crops. In this connection Gillespie (2) has stated:

> With the development of inland communities and the discouragement of stream pollution, land disposal is markedly increasing. . . . Except in spots the development of revenue has been unsystematic and has not at all measured up to the possibilities in this regard. . . . I look to see more high-grade treatment added as homes encroach and with them we shall see intensive cropping.

Intermittent Filtration.—The treatment of sewage by intermittent sand filtration was a natural outgrowth of the disposal of sewage by irrigation. As such it was an attempt to intensify the activity of the forces of purification operative in natural soils by more careful selection and preparation of the disposal areas and better supervision of their operation. The thought of agricultural utilization of the filtration areas was not immediately relinquished. Cultivation or cropping, however, was made secondary to the securing of adequate treatment. Ultimately it was abandoned entirely. Development of the process of intermittent filtration is associated with the work of Frankland for the Rivers Pollution Commission of Great Britain, reported in 1870, and with the early investigations of the Massachusetts State Board of Health at Lawrence, begun in 1887.

Experiments by Frankland (3) led him to conclude that the filtration process was superior to irrigation and chemical processes in the removal of polluting organic matter in suspension. He attributed the changes in the sewage as it passed through the filter to chemical causes solely. The bacterial aspect of treatment by filtration was first pointed out by Schloesing and Müntz in a report published in 1877. Subsequent investigations at the State Experiment Station in Lawrence, Mass., reported in 1890, clearly demonstrated the biological nature of filtration and resulted in an improvement in the design of intermittent filters in the United States. Sand filtration on scientific lines has never been extensively employed in Great Britain, because of the scarcity of suitable material for filter beds.

Intermittent filtration as developed in New England consists of applying crude or settled sewage evenly to the surface of prepared areas of sand or other fine material a few feet in depth, underdrained by lines of tile with open joints. During its passage through the bed, the sewage is purified as a result of the removal and changing of the organic matter into more stable substances, by physical and biological agencies in conjunction with atmospheric oxygen present in the interstices of the sand. The process derives its name in part from the necessity of intermittent application of the sewage, in order that the air required for the oxidation of the organic matter may enter the voids of the sand. Because of the favorable soil conditions, this process was adopted by many municipalities in New England. The area required is much less than for broad irrigation, but greater than for later types of filters. Owing to the large area required per capita and the cost of operation, this method is adopted now only by small municipalities where conditions are favorable.

Sedimentation.—The partial clarification of sewage and trade wastes by plain sedimentation was an early development in sewage treatment, provided to reduce objectionable conditions along polluted streams. The results were often unsatisfactory, doubtless due in some cases to negligence in removing from the tanks the settled solid matter, which gradually occupied so much room that the sewage passed quickly through with little opportunity for sedimentation. In suitably designed and operated tanks a substantial proportion of the suspended solids in sewage can be removed by plain sedimentation. With the development of mechanical equipment now commonly employed to facilitate the removal of sludge either continuously or intermittently, as desired, without interruption of the sedimentation process, the use of sedimentation tanks as a means of partial treatment of sewage has increased. Such tanks are used quite generally as preliminary or final units preceding or following other methods of sewage treatment.

Chemical Precipitation.—Chemical precipitation was a well-established method of sewage treatment in Great Britain in 1870. Many patented processes had been devised, beginning with one proposed in 1762 by de Boisseau. Lime was used as a precipitant in most cases, sometimes alone, but more often in combination with chloride of lime, chloride of magnesium, sulfate of alumina, phosphate of alumina, green copperas, black-ash waste, charcoal, herring brine, or any one of many other substances. Occasionally several substances were used in order to obtain special results. This was the case with the A.B.C. process, patented in 1868, which was strongly advocated because its sludge would have unusual merits as a fertilizer, owing to the use in it of alum, blood, charcoal and clay. Ferrozone was brought out as a precipitant and was used somewhat. The effluent from sedimentation aided by ferrozone was sometimes filtered through a patented material called "polarite." An artificial material called alumino-ferric, of low price, gradually won its way into considerable favor in England, on account of the ease with which it can be used in treating sewage of fluctuating quantity and quality.

The first chemical precipitation plant in the United States was placed in operation at Coney Island, N. Y., about 1887, to be followed in 1888 by a plant at East Orange, N. J., and in 1890 by a plant at Worcester, Mass. The existence of the Worcester treatment plant led to an investigation of the nature and results of chemical precipitation by the Massachusetts State Board of Health, reviewed in their report in 1890, which had much influence in checking a contemporary tendency toward overestimating the degree of purification which this process was capable of producing. Owing to the incomplete purification secured, the high cost of operation and the difficulty of disposing of the large volume of sludge produced, the use of the older methods of chemical precipitation as a means for treating ordinary municipal sewage has been curtailed, although these methods are still used in some industrial-wastes treatment plants.

A revival of interest in the chemical precipitation of sewage has recently been in evidence. Numerous experiments have been carried on and several processes of sewage treatment, all employing chemical precipitation, have been given prominence in the technical press. In 1932 a sewage treatment plant using a chemical process developed by Laughlin was put into operation at Dearborn, Mich. Here the substances added to the screened sewage have varied from time to time but, in general, they have been pulped waste paper, lime and ferric chloride.

Sedimentation and Digestion of Solids in Septic Tanks.—The cost and difficulty of satisfactory disposal of sludge from plain sedimentation tanks were so great that methods of reducing them were sought. This led to the development of *septic tanks* in which the sewage solids were retained in such manner as to foster the decomposition of the settled solids in the absence of oxygen. It was originally believed that nearly complete liquefaction of the suspended solids could be obtained in this manner. Although a considerable portion of the organic matter was destroyed, the hopes for complete liquefaction proved unjustified and the system did not contribute much toward the solution of the sludge-disposal problem. Such tanks, unless covered, frequently are offensive to the sense of smell, the treated sewage is black and foul, and difficulties in operation result from the accumulation of solids in the form of a floating mass of scum on the surface of the sewage.

Similar tanks were first used in America at the State Insane Asylum at Worcester, Mass., in 1876 and were later adopted at other places. An installation at Saratoga Springs, N. Y., in 1903, led to a lawsuit brought by the owners of certain septic-tank patents, which resulted in judgment for the plaintiffs. From the beginning of septic-tank-patent litigation, the consensus among American sanitary engineers has been that the patents covered principles and practices which had been known and publicly used for a long time. The litigation served to prevent in the United States the development of as favorable an opinion of septic tanks as existed in Great Britain.

Sedimentation and Digestion of Solids in Imhoff Tanks.—The operation of septic tanks has usually given considerable cause for dissatisfaction. The idea of separating the sedimentation process from that of digesting the sludge—in order to avoid the objectionable features of septic tanks while taking advantage of the desirable ones—was first expressed in the 1899 report of the Lawrence Experiment Station. Travis of Hampton, England, during 1904, devised a two-story, so-called hydrolytic, or septic tank, having an upper sedimentation chamber with steeply sloping bottom terminating in slots, through which the deposited solids passed into a lower or sludge-digestion chamber. Through this latter chamber a predetermined part of the sewage was allowed to flow, for the purpose of seeding and maintaining bacterial life in the sludge and carrying away products of decomposition.

The Travis design was not widely adopted. A modification in the design was developed by Imhoff and was widely adopted in Germany and, on the initiative of Hering, in the United States. In the Imhoff design there is no provision for sewage flow through the sludge-digestion compartment, although experience has demonstrated that there is an interchange of liquids between compartments, owing to differences in temperature and other causes. The solids drop through a slot in the bottom of the sedimentation chamber into the sludge chamber below. This slot is constructed in such a fashion that no gases can escape through it from the sludge-digestion chamber into the compartment through which the sewage is flowing. The sludge is allowed to remain in the digestion chamber until a large part of the organic matter has been gasified or liquefied. When in suitable condition for removal, it is run on to sludge beds. After draining and drying there for a number of days, it is in a condition resembling humus and may be deposited without offense on dumps or on agricultural land for utilization as a low-grade fertilizer.

The earliest plant of this type in America was that built at Madison-Chatham, N. J., in 1911, and the first large Imhoff tanks were constructed at Atlanta, Ga., in 1912. Since then there have been many important installations, among them those at Chicago Ill., Fitchburg, Mass., Trenton, N. J., Rochester, N. Y., Akron, Cleveland and Dayton, Ohio, and Allentown and Philadelphia, Pa.

In some places difficulties have been experienced, owing to unfavorable sludge digestion, foaming, or excessive scum formation. Some measure of control to ensure favorable digestion has been secured by regulating the reaction, pH, of the sludge and scum.

Imhoff tanks have ordinarily proved to be much more satisfactory than septic tanks and, where Imhoff tanks have been designed appropriately and operated suitably, they have been satisfactory in general, although for one reason or another in a few cases they have not behaved as desired. For new plants at present, 1935, Imhoff tanks are in competition with plain sedimentation and separate sludge-digestion tanks, in conformity with the suggestion of the Massachusetts State Board of Health of 1899.

Digestion of Solids in Separate Tanks.—Although the idea of carrying on sedimentation of sewage and digestion of sludge in different tanks appears to have originated at the Lawrence Experiment Station, some precedent was provided by the sludge lagoons in Essen, Germany, as early as 1891. Other early tank installations were made at Birmingham, England, and Baltimore, Md., in 1912. Failure to appreciate the importance of seeding, reaction and temperature was largely responsible for the difficulties experienced with early separate digestion tanks and retarded their development. Research at various places since 1925 and the operation of plants since 1929 have emphasized the importance of these factors and have contributed to a rapid increase in the number of separate digestion tanks constructed in this country.

Sludge from primary sedimentation tanks is digested in separate tanks, notably at Erie, Pa., Grand Rapids, Mich., Toledo, Ohio, and Kitchener, Ont. Separate tanks are employed for the digestion of primary sludge plus excess activated sludge at a large number of plants, including Peoria and Springfield, Ill., the Easterly plant at Cleveland, as well as Elyria and Lima, Ohio, the North Toronto plant at Toronto, Ont., and Lancaster, Pa.

Mechanical equipment has been provided to facilitate the withdrawal of sludge and the breaking-up of scum and to afford slight stirring action. Among the tanks so equipped are those at Erie, Grand Rapids, Peoria and Toledo. Floating covers are provided at the Easterly plant in Cleveland and at Elyria and Springfield.

In all these recent installations provision is made for collection and utilization, usually for heating purposes, of the gases evolved during digestion. At Charlotte, N. C., Rockville Center, N. Y., and Springfield, Ill., gas collected from sludge-digestion tanks is used as fuel in gas engines driving air compressors.

At Lancaster, Pa., and at Peoria and Springfield, Ill., circular tanks without roofs are provided for the storage of digested sludge during times when dewatering of the sludge on sand beds is affected by adverse climatic conditions.

Treatment in Contact Beds and Trickling Filters.—From 1860 to 1880 great advances were made in bacteriology. Pasteur discovered that some bacteria, which he called aerobic, could live and exercise their functions only when air was present; that others, called anaerobic, could operate only where all oxygen was absent; and that a third class, called facultative, could operate under either aerobic or anaerobic conditions, although not always with equal vigor. The aerobic and facultative bacteria are of many kinds and accomplish different results.

These discoveries led to a better understanding of sewage-treatment processes and to experiments from which contact beds and trickling filters developed. The experiments at the Lawrence Experiment Station with intermittent filtration were conducted with various grades of material ranging from fine sand to coarse gravel. In the latter case, the liquid passed rapidly through the large pores without being purified. In England, where land was not suitable for intermittent filtration, renewed attempts were made to filter sewage with coarse material such as stone, gravel, ballast, coke and clinker.

To overcome the too rapid passage of sewage through the coarser materials, Dibdin (1) modified the usual procedure of allowing the sewage to pass through the filter continuously, by placing the material in a watertight tank and allowing the sewage to fill the tank, afterward holding it in contact with the filter for some time, in order to allow the oxygen in the pores and the bacteria coating the media to oxidize the organic matter. Next, the liquid was drawn off from the bottom of the filter and the latter was allowed to stand empty of sewage but charged with air. Dibdin's modification of the intermittent-sand-filter process gave rise to the "contact-bed" system.[1]

Comparatively few contact beds serving more than 10,000 persons have been built in America, and several of those have now been replaced by other forms of treatment. This was due to the almost coincident development of trickling filters, which are superior to contact beds in some respects, chief among them being the much larger volume of sewage which can be treated on a unit area of trickling filter.

Clark and Gage (4) report as follows:

[1] Among the earliest places in the United States to install contact beds were the Glenview Golf Club, near Chicago, in 1899, and Depew, N. Y., and Glencoe, Ill., in 1901. A list of the sewage-treatment plants in the United States at the beginning of 1902 was published in *Eng. News*, Apr. 3, 1902.

Sewage was filtered intermittently through gravel stones at the Lawrence Experiment Station during 1888, 1889 and 1890. Filters were constructed of stones, so large that even the coarser suspended particles of the sewage were not removed; yet the slow movement of the sewage in thin films over the surface of the stones, with air in contact, caused a removal for some months of 92 per cent of the organic nitrogenous matter as well as 99 per cent of the bacteria. These filters were the forerunners of the modern trickling filters. Early in 1891, a gravel filter was operated at a rate of 220,000 gal. per acre daily, the sewage being applied in sixty or seventy doses per day. Good nitrification results were obtained.

In 1893 and 1894 Corbett (5) developed more efficient means for distribution of the sewage by revolving pipes and sprays and succeeded in securing excellent results at comparatively high rates of flow.

From these experiments was evolved the trickling filter, which is primarily a bed of stone or other coarse filtering medium several feet deep laid on a system of underdrains. Settled sewage is applied intermittently, at short intervals, to the surface of the filter in the form of fine drops, by using symmetrically spaced spray nozzles or other devices. Comparatively few mechanical distributors, such as are commonly used in England, have been utilized in America. As the sewage is sprayed through the air and passes through the porous bed, atmospheric oxygen is absorbed and is utilized in the biological oxidation and mineralization of the organic matter. The effluent is usually settled in order to remove humus-like solids which, from time to time, escape from the filter and thus prevent it from becoming clogged.

The first municipal filter of this type to be put in service in America was that at Reading, Pa., in 1908. At present, trickling filters preceded by Imhoff tanks constitute one of the most common types of treatment in use in America.

Among the installations of trickling filters may be mentioned those at San Bernardino, Cal., Aurora, Bloomington and Decatur, Ill., Baltimore, Md., Fitchburg and Worcester, Mass., Springfield, Mo., Akron, Canton and Cleveland (Southerly plant), Ohio, Allentown and Philadelphia, Pa., and Madison, Wis.

Activated-sludge Treatment.—The aeration of sewage in tanks, to hasten oxidation of the organic matter, was investigated in England as early as 1882 by Dr. Angus Smith, who reported on it to the Local Government Board. It was studied subsequently by a number of investigators, and in 1910 Black and Phelps reported that a considerable reduction in putrescibility could be secured by forcing air into sewage in basins. Following this, experiments by Clark and Gage (6) at the Lawrence Experiment Station, conducted during 1912 and 1913 on sewage in bottles and in tanks partially filled with roofing slate, spaced about 1 in. apart, showed that in aerated sewage growths of organisms could be cultivated which would greatly increase the degree of purification obtained.

The results of the work at Lawrence were so striking that a knowledge of them led Fowler to suggest experiments along similar lines at Manchester, England, where Ardern and Lockett carried out valuable researches upon this subject. During the course of their experiments Ardern and Lockett found that the sludge played an important part in the results obtained by aeration, as announced in their paper of May 3, 1914, before the Manchester Section of the Society of Chemical Industry. At the outset it was necessary to aerate the sewage samples continuously for five weeks before complete nitrification was obtained. By repeatedly drawing off the clarified sewage and adding fresh raw sewage to the old sludge left in the experimental tank, the time for oxidation, however, was reduced to 24 hr. and eventually to a few hours only. The sludge accumulating in this manner and inducing such active nitrification was called "activated sludge." Hatton at Milwaukee, Sands at Houston, Frank at Baltimore, and others finally showed that the process could be operated on a practical, continuous basis by running sewage through aeration tanks, the activated sludge being mingled with the entering sewage and later separated from it after its passage through the tanks. More recent developments in the activated-sludge process have dealt with the perfection of the control of aeration and the introduction of mechanical devices for aerating the sewage in place of compressed air. This variation of the process is called "bio-aeration."

Many sewage-treatment plants employing the activated-sludge process have been constructed, including those at Pasadena, Cal., Chicago, Peoria and Springfield, Ill., Indianapolis, Ind., Charlotte, N. C., Lima, Ohio, Toronto, Ont., Lancaster, Pa., Houston and San Antonio, Tex., and Milwaukee, Wis. Large plants are under construction at Cleveland and New York.

In suits at Chicago and Milwaukee the United States courts recently have upheld the validity of certain patents on the activated-sludge process, held by Activated Sludge, Inc.

Closely related to the activated-sludge process and originating from the same early investigations, but differing in many respects from the process as now employed, is "contact aeration," developed by Buswell, Bach and Imhoff in 1925 and 1926. In this process aerated contact surfaces for the support of growths of organisms are provided in the flowing-through chamber of single- or two-story tanks. These growths and surfaces appear in some measure to take the place of the activated sludge in suspension in aeration tanks.

Treatment by Racks, Grit Chambers and Screens.—In order to protect pumps and other mechanical equipment from clogging and injury and to simplify and improve the efficiency of various processes of treatment it has become general practice to provide preliminary treatment for the removal of trash, mineral matter and the coarser suspended organic substances. Provision also has been made in some plants for the removal of floating oil and grease, either in separate, aerated skimming chambers, as at Springfield, Ill., in skimming-detritus tanks, as at Akron, Ohio, or from the surface of preliminary sedimentation tanks, as at Peoria, Ill.

Cage racks for the interception of trash were constructed in the main drainage pumping station in Boston and first operated in 1884. Grit chambers for the removal of the heavier mineral solids were built at Worcester in 1904. Stationary bar racks, manually and mechanically raked, have been in use for a long time, but only during the last 15 or 20 years has there been a marked development in fine, moving screens, cleaned by brushing or flushing. The first of these in America was the Weand screen built at Reading, Pa., in 1908. This was followed by the Riensch-Wurl screens at Rochester, N. Y., in 1916 and later by others, such as the Dorrco, Rex and Link-Belt screens in various places. During the past few years such screens have been used for treating sewage preparatory to more complete treatment by other processes and for removing the larger and more easily visible floating and suspended matter where more complete treatment is not necessary.

Treatment and Disposal of Sludge.—One of the troublesome features of sewage treatment has been the problem of disposal of the solids removed by screening or by sedimentation. In general, screenings have been used for filling low land. Recently provision has been made for their incineration, as at Long Beach, Cal., Dayton and Lima, Ohio, Allentown, Erie and Lancaster, Pa., and Milwaukee, Wis. Partial dewatering by pressing, as at Dayton, or in centrifuges, as at Milwaukee, is sometimes practiced prior to incineration.

Sludge from primary sedimentation tanks is generally offensive in character and difficult to dewater satisfactorily on beds of sand or similar material. At Syracuse, N. Y., the sludge from primary-sedimentation tanks is disposed of by pumping into industrial-waste lagoons of the Solvay Process Co. The sludge from the primary-sedimentation tanks of the Passaic Valley Sewerage Commission is discharged into barges and towed to a disposal area at sea, where it is dumped.

Sludge from the direct-oxidation process at Winston-Salem, N. C., is dewatered by mechanical filters and disposed of to farmers for use as fertilizer. At Dearborn, Mich., sludge from the chemical-precipitation process is dewatered by mechanical filters and has been disposed of on land, but recently provision has been made for incinerating it.

In the case of undigested activated sludge, the proportion of fertilizing ingredients, particularly nitrogenous compounds, is materially greater than in sludges from other processes and therefore the opportunity for its profitable utilization as a fertilizer has seemed somewhat attractive. At Milwaukee, Wis., a large plant has been built for dewatering and drying activated sludge, thus converting it into a marketable fertilizer which is being sold under the name of "Milorganite." Similar plants on a smaller scale have been built at Houston, Tex., Indianapolis, Ind., and Pasadena, Cal. At Charlotte and High Point, N. C., the excess activated sludge is dewatered by mechanical filters and spread on agricultural land. At Tenafly, N. J., the excess activated sludge is dewatered on sand beds and disposed of by filling low land.

The digestion of sludge in septic, Imhoff and separate tanks has been described. Well-digested sludge is inoffensive and, in many cases, drains and drys readily on beds of sand or similar material. Extensive areas of sand beds for sludge dewatering have been provided at Chicago, Decatur and Peoria, Ill., Baltimore, Md., Flint, Mich., Rochester and Schenectady, N. Y., Akron, Ohio, Allentown, Erie, Lancaster and Philadelphia, Pa., and many other places. As rain and freezing weather interfere with sludge drying in the open, beds have been built with glass covers, like greenhouse structures, notably at Alliance, Cleveland, Dayton, Lima and Marion, Ohio, and Toronto, Ont. Imhoff-tank sludge from the covered drying beds at Dayton is reported to have been further reduced in moisture content by heat drying and sold for fertilizer at a substantial price. Similar air-dried sludge has been sold to farmers for fertilizer at Marion, Rochester and Schenectady. At Canton, Ohio, wet digested sludge is spread upon land for fertilizing purposes. In a few instances digested sludge has been discharged into rivers in flood or into lake waters.

Experiments upon the mechanical dewatering of digested sludge after suitable preliminary treatment and conditioning have been so successful as to warrant the recommendation of this method of sludge treatment for several large plants. Mechanical dewatering is not affected by climatic conditions and with it a substantial saving in the cost of sludge-storage facilities required in conjunction with drying beds should be possible. The dewatered sludge cake may be disposed of either to farmers for use as fertilizer or for filling waste land. An installation for the spray-drying of sludge has been made at Plainfield, N. J.

The incineration of sludge after dewatering by mechanical filters and heat drying has been studied on a large scale by the Sanitary District of Chicago. As a result, incineration has been adopted in the plan for sludge disposal at the Calumet plant now under construction.

Chlorination.—Disinfection of sewage and sewage effluents has been developed for the protection of public water supplies, bathing beaches and shellfish areas. The first plant for this purpose was at Brewster, N. Y., on the Croton watershed, where an electrolytic plant for the production of chlorine from salt solution was built in 1892. The economical and practical utilization of chlorine, chiefly in the form of chloride of lime, as a disinfectant of sewage was demonstrated by Phelps (7) at Boston, Mass., Red Bank, N. J., and Baltimore, Md., in 1906 and 1907. Liquid chlorine is now generally used for the disinfection of sewage and sewage effluents, for the purposes mentioned above.

Chlorination has also been employed for controlling odors at treatment works, for combating filter flies and for relief of filter clogging. Experiments indicate that chlorination will effect an apparent reduction in the biochemical oxygen demand of sewage amounting to at least 2 parts per million for each part of chlorine absorbed. Chlorination supplements sedimentation during low-stream flows at Chapel Hill, N. C.

Present Status of Sewage Disposal in the United States.—In the United States the extent of sewage treatment in 1930 by cities having a population of more than 100,000 is indicated in Table 1.

TABLE 1.—EXTENT OF SEWAGE TREATMENT IN 1930 BY CITIES IN THE UNITED STATES HAVING A POPULATION IN EXCESS OF 100,000

Treatment provided	Population served	Percentage of total
Dilution alone	16,900,000	46.4
Fine screening, dilution	8,500,000	23.3
Sedimentation, dilution	5,700,000	15.6
Filtration, dilution	2,500,000	6.9
Activated-sludge process, dilution	2,600,000	7.1
Total with ultimate dilution	36,200,000	99.3
Irrigation	250,000	0.7
Total	36,450,000	100.0

This table shows that with a total population in large cities of 36,450,000, the sewage from 16,900,000 persons, or more than 45 per cent, is disposed of by dilution alone and that 25,400,000 persons dispose of their sewage with no more treatment than is provided by fine screens. Complete treatment, including some form of oxidation process, is provided for the sewage of only 14 per cent of the total population of these cities.

Of these populations, it is estimated that 14,200,000 depend ultimately upon the dilution afforded in rivers, 17,000,000 in ocean waters and 5,000,000 in lake waters.[1]

There is a marked tendency toward the installation of mechanically equipped, plain sedimentation tanks and separate tanks for sludge

[1] For tabulation giving summary data for nearly all sewage treatment works in the United States, see *Eng. News-Rec.*, 1935; **115**, 224.

digestion. The activated-sludge plants constructed in recent years have outnumbered the installations of trickling filters.

The recent developments in the application of chemical precipitation as a method of treatment are still in the experimental stage. Their status cannot be established until adequate data have been accumulated relative to the efficiency and operating cost of the several processes from the operation of plants of substantial size, which have been treating reasonably normal sewage.

Sewage disposal in the United States has made great strides during the past 20 years. Considerable credit for this advance must be given to the various agencies sponsoring investigational work and to the experimenters themselves. The researches at the Lawrence Experiment Station have furnished valuable information. Particularly noteworthy is the work of Rudolfs and his collaborators at the New Jersey Agricultural Experiment Station, of Fair and others under sponsorship of the Rockefeller Foundation at Harvard University, and of Buswell and those associated with him at the Engineering Experiment Station of the University of Illinois. The development of various means of treating sewage has been aided materially by investigations at Chicago, Milwaukee, Houston, Indianapolis and several other places. The investigational work still being carried on, particularly at Chicago, will undoubtedly result in the development of improvements in the activated-sludge process and reduction in its operating costs.

With the development of the various methods of sewage treatment and the adoption of combinations of them in the design and construction of treatment plants, the operation of such plants has become more and more complicated, necessitating the employment of skilled personnel in order to be successful. In this country the recent trend has been toward the installation of mechanical equipment in order to reduce the hand labor necessary for maintaining and operating various treatment units. There has been an increasing inclination also to replace biological processes, in part, at least, with chemical and mechanical ones, as evidenced by the numerous experiments with chemical precipitation and the plants at Chicago and Dearborn for the mechanical dewatering, drying and incineration of undigested sludge. The trend seems to be toward the use of mechanical filters instead of drying beds for dewatering digested sludge, particularly in large plants.

Sewage Disposal in England.—Sewage treatment in England differs somewhat from either American or German practice. The problem in England is often a matter of building over, relieving or adding to existing works. It may happen, therefore, that the sewage at one plant may be treated by both broad irrigation and the activated-sludge process. Sedimentation tanks may be employed to relieve sewage farms, as has been done at Nottingham, and partial activated-sludge treatment may be provided to reduce the organic load of the sewage and permit greater rates of application to trickling filters, as at Birmingham.

Racks are commonly employed. Recently the tendency has been to install coarse racks at the influent end of the plant and then, either preceding or following detritus tanks, to provide mechanically raked, fine racks of ½-in. clear spacing. Fine screens have not been widely adopted.

Practice in the design of grit chambers, according to Martin (8), is governed by the requirements of the Ministry of Health that there shall be two or more detritus tanks below the rack chamber and that the capacity of each tank shall be 1 per cent of the daily dry-weather flow. The detention period on this basis is 14.4 min. at the average rate of dry-weather flow, as compared with the American practice of one minute or less. In the larger plants grit is commonly removed from the tanks by traveling grab dredges and is disposed of on land.

Stand-by tanks for the treatment of storm flows in excess of three times the average dry-weather flow are commonly employed. The practice is largely governed by recommendations of the Royal Commission on Sewage Disposal (9):

> That special stand-by tanks should be provided at the works and kept empty for the purpose of receiving the excess of storm water.
>
> That any overflow at the works should be made from these special tanks and that their overflow should be arranged so that it will not come into operation until the tanks are full.

In regard to the capacity of stand-by tanks, the Commission thought that tanks capable of holding a quarter of the mean daily dry-weather flow usually would be sufficient.

Sedimentation tanks are used, both with and without chemical treatment, to reduce the load on subsequent treatment units. Mechanical equipment to facilitate sludge removal is not so commonly used as in America. Practically the only Imhoff-tank plant in England is a small installation at Manchester, built in 1924. According to Whitehead and O'Shaughnessy (10), the septic tank is still considered advantageous for small installations. Chemical precipitation is not employed extensively, except for the use of acids to "crack" industrial soaps. Chemicals, however, such as chloride of lime and chlorine, are being utilized for the prevention of objectionable conditions.

Trickling filters have been widely adopted in England, the common method of applying sewage to the filters being by revolving or traveling distributors.

Many plants have been built employing the activated-sludge process. Mechanical aerators are more commonly used than in America, particularly the Simplex type, as developed by Bolton. The mechanical paddle-wheel agitators, as developed by Haworth, have found some use.

The diffused-air system of aeration also is commonly employed. Present practice in new installations appears to favor the activated-sludge process over trickling filters. While Leicester and Bradford have recently installed large areas of trickling filters, the majority of the new installations have been of the activated-sludge type, notably the Mogden works in Middlesex, the Wood End works at Burnley, and the plants at Exeter and Dagenham.

Separate sludge digestion is practiced on a large scale at Bath and Birmingham. In the Saltley works at Birmingham, gas is collected under floating concrete covers and utilized in engines having a total rating of 950 brake hp. Waste heat from the engines is used to heat the digestion tanks. At Manchester, separate digestion tanks are heated by means of the gas collected. The separate digestion process has been adopted for Burnley, Dagenham and the large Middlesex County project, which serves a population of 1,000,000.

Sludge is commonly disposed of on land, except where it is practicable to barge it to sea. Lagooning and drying on beds of slag are among the common methods of preliminary treatment of sludge prior to ultimate disposal on land.

Sewage Disposal in Germany.—Imhoff (11) has summarized German practice in sewage disposal up to the end of 1932 and the following description has been prepared from his summary. Almost all municipalities in Germany are equipped with sewage-treatment plants.

Screening plants, installed in 59 large and medium-size towns, treat sewage from a total of 5,500,000 persons. New screening plants are built infrequently, as they accomplish little in comparison to their cost.

Grit chambers are provided in nearly all sewage-treatment plants to which storm water gains access. They are constructed and operated to yield clean grit as far as possible, by control of velocity, by air agitation or by washing.

Septic tanks preceding bacterial beds have been established in 25 towns serving a population of 400,000. Some 1,500,000 inhabitants of Germany are served by small septic tanks for individual houses.

Two-story sedimentation and sludge-digestion tanks of the Imhoff type have been adopted in 240 towns to serve a population of 5,500,000. Plain sedimentation with separate sludge digestion has been adopted in 70 places with a population of 2,000,000. The sedimentation tanks are mostly of the hopper-bottom or Dortmund type. More recently, tanks equipped with special sludge scrapers of many types have been favored, especially at the larger works. New installations, particularly in large towns, are being furnished with separate digestion tanks, because it is not practicable to heat two-story tanks. The temperature of separate tanks is maintained at 70 to 80°F. In large digestion tanks provision is generally made for gas collection.

The sewage from 45 towns with a population of 6,500,000 is treated on sewage farms. The degree of purification is good, although most irrigation farms are overloaded and in many places complaints of odor are common. In many towns works are being constructed to give preliminary treatment and to relieve the irrigation areas.

Trickling filters have been adopted by 90 towns with a population of 1,300,000. The effluents are good in some cases but in others complaints with regard to objectionable odors and flies are being made. Contact beds, which are used in 17 towns and serve 200,000 persons, are no longer being installed.

Plants of the activated-sludge type are in use in 13 towns with a total population of 900,000. In most cases compressed air is used for aeration, either alone or in combination with mechanical equipment. The excess activated sludge is usually pumped back to the preliminary sedimentation tanks for digestion with the primary sludge.

As a final stage in the treatment process fish ponds are used in 12 cases, serving a population of 1,000,000. The effluents are excellent in character and the area required is not more than 10 per cent of that required for sewage farms dealing with the same quantity of sewage.

Impounding reservoirs have been constructed in the Ruhr district with the object of improving polluted river water by storage. They contribute as much to the purification of the Ruhr river water as the 70 sewage-treatment plants within the area.

Sludge is commonly dried on land. Mechanical sludge filters and other methods of artificially drying sludge are awakening interest.

Bibliography

1. Garner, J. H.: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 204.
2. Gillespie, C. G.: *Sewage Works Jour.*, 1929; **1**, 460.
3. First Rept., Royal Comm. on Rivers Pollution, 1870.
4. Ann. Rept., Mass. State Board of Health, 1909.
5. *Eng. News*, 1903; **49**, 191.
6. Ann. Rept., Mass. State Board of Health, 1913.
7. *Water Supply Paper* 229, U. S. Geological Survey, 1909.
8. *Trans.*, A. S. C. E., 1927; **91**, 496.
9. Fifth Rept., Royal Comm. on Sewage Disposal, 1908.
10. Whitehead, H. C., and F. R. O'Shaughnessy: *Sewage Works Jour.*, 1933; **5**, 714.
11. Imhoff, Karl: *Surveyor*, 1933; **83**, 161.

CHAPTER II

GENERAL CONSIDERATIONS IN TREATMENT AND DISPOSAL OF SEWAGE

American practice in sewage treatment has been determined by public opinion and the development of science. Considerations of the public health had some influence in the earlier periods, but in later times public comfort has come to be the more potent incentive, except where protection of municipal water supplies has been paramount. The increase in outdoor recreation has recently focused attention upon the value of clean streams, lakes and tidal waters, with the result that a better control of their pollution has been forced upon the offending communities and industries. This tendency has resulted in the passage of state laws and judicial action under the common law in an effort to compel a more cleanly practice in disposing of sewage and industrial wastes. While the treatment of these liquids requires large capital and annual expenditures, the effect of these financial considerations generally is in the direction of delay rather than ultimate neglect to provide treatment.

Reasons for Sewage Treatment and Controlled Disposal.—Streams, lakes and tidal estuaries into which sewage is discharged are limited in the *pollutional load* they are able to receive without giving rise to objectionable conditions, just as artificial treatment plants are limited in the quantities of sewage they can handle effectively. The limiting loads depend upon the physical, chemical and biological conditions of the waters into which the sewage is discharged, conditions whose relative importance is being determined by studies of the so-called self-purification or natural purification of streams and bodies of water. In addition, however, sewage disposal by dilution is circumscribed by the wide use of water courses for purposes other than the reception, transmission and ultimate disposal of sewage. Lakes and streams are employed as sources of water supply for household and factory; they, as well as coastal waters, are centers of recreation in the form of boating, bathing and fishing; like the fields, they yield their crop of foodstuffs to support useful animal life, such as fish, mollusks and crustaceans; finally they serve as waterways of commerce and for power development. The disposal of sewage by discharge into natural waters, therefore, must be studied with regard to the varied purposes they may have to serve. Thus sewage treatment prior to disposal may become necessary not only to prevent the waters from being overworked as natural purifying agencies, but also to permit their economic employment for other essential purposes.

Prior to 1890, little attention was given to objectionable conditions produced by the discharge of sewage into American rivers. With the subsequent rapid growth of cities and industries and consequent increase in water requirements as well as in sewage pollution, there has been a growing tendency toward greater restriction upon the discharge of untreated sewage into bodies of water and toward raising the degree of purification required of treatment plants. While there is little doubt that in certain cases progress in these directions has exceeded logical needs, it is true that over the greater part of the country the requirements are not sufficiently exacting. More care in disposing of sewage will be necessary in the future, if the water resources are to be conserved for development for the greatest good. It is desirable, therefore, when adopting a plan for the treatment and disposal of sewage, to select such means as meet the needs of the present and near future and are capable of development to satisfy more exacting requirements of the future.

The discharge of sewage into natural waters is subject to the following considerations of hygiene, aesthetics, economics and law:

- I. Hygienic considerations
  - *A*. Contamination[1] of
    1. Private and public water supplies
    2. Natural ice
    3. Shellfish
    4. Waters at bathing beaches
  - *B*. Pollution[1] resulting in
    1. Offensive conditions affecting the public comfort or health
    2. Impairment of recreational facilities.
- II. Aesthetic considerations, creation of conditions offensive to
  - *A*. The eye
  - *B*. The sense of smell
- III. Economic considerations, damage to
  - *A*. Industrial water supplies

[1] In 1917 the American Public Health Association defined *contamination* of water as the introduction into it of bacteria or other substances that tend to render it unsuitable for domestic use. The degree of contamination may increase from merely nominal, according to the quantity and nature of the contaminating substances, until it reaches a maximum, when the water is unsuitable for domestic consumption in its present condition and cannot be made so by practicable methods of treatment. *Pollution* of water is defined as the introduction into it of substances of such character and in such quantity as to render the body of water objectionable in appearance or to cause it to give off objectionable odors. The degree of pollution may increase from merely nominal, when a water has just reached a degree of contamination which renders it unfit for use, or for preparation for use, as a domestic supply, until it becomes obviously filthy and offensive. It is not possible to define, according to contents as determined by analysis, the line on the one side of which a water may be said to be contaminated and on the other polluted, and local conditions will have an effect in determining whether a water should be designated as contaminated or polluted.

B. Live stock
C. Fish and other useful aquatic life
D. Property, with resulting depreciation of values
E. Private and public river and harbor improvements and navigation, such as silting due to sludge deposits.

IV. Legal considerations, interference with the rights of riparian owners.

Sewage and Disease.—Sewage may contain the organisms of any of the infectious diseases of man in which the causative organism passes out of his body with his wastes. Most important of these diseases are the so-called gastrointestinal infections, notably typhoid fever, dysentery, the diarrheas and, in oriental countries, cholera. Persons suffering from these diseases and so-called chronic carriers, *i.e.*, persons who carry the disease germs in their bodies—sometimes for life—but themselves are immune, discharge enormous numbers of pathogenic organisms through their stools or urine. Sewage contamination of water supplies, bathing waters and shellfish beds, therefore, may result in epidemics of typhoid fever or other intestinal diseases. From the hygienic standpoint, the fate of pathogenic organisms in sewage is the most important problem in sewage disposal. It is assumed, therefore, for the sake of safety, that germs of disease are always present in sewage.

Fortunately pathogenic bacteria die away rapidly in sewage and sewage-contaminated water. Accustomed to the abundant food supply and the favorable environment of the human body, they cannot survive for a long time in sewage. The forces active in their destruction are many. Being attached to suspended matter, some are removed from the sewage by sedimentation; others are ingested by predatory protozoa and other animals. They are more quickly destroyed in sewage than in clean water, where such animals are less abundant. Lack of suitable food for the bacteria causes a gradual disintegration of their cells and, as cell activity is greater at higher temperatures, survival is of less duration in warm sewage than in cold water. Cleaner water, however, permits farther penetration of the sun's rays, the ultraviolet portion of which tends to bring about disinfection. Owing to factors such as these, pathogenic organisms die away, the various forces combining to reduce their numbers rather rapidly. The death rate under a given set of conditions is fairly constant and the number destroyed in a unit of time is, therefore, proportional to the number surviving. The longer the time, the fewer survive.

Sewage Treatment and Water Purification.—In the early development of sewage and water purification it was only natural for differences of opinion to arise concerning the treatment of sewage prior to its discharge into bodies of water from which municipal water supplies were drawn. Health officers were inclined to insist that the sewage should be rendered substantially innocuous before its discharge into such waters, on the ground that any less thorough treatment would endanger the health of consumers of the water. On the other hand, many engineers believed that such requirements were too severe in most cases, because the destruction of bacteria necessitated thorough treatment, which was expensive and took no advantage of the beneficial effect of dilution of the sewage. Furthermore, the water was liable to contamination by the rain water running off cultivated lands and by the sewage from hamlets and isolated farms and the water could be rendered safe for drinking purposes by purification. The general answer made to this was that the constantly efficient operation of water-purification works was rarely attained, except in large plants, and that the correct procedure was to require both sewage treatment and water purification.

The tendency in these discussions has been to base broad general arguments on rather limited data. The differences in local conditions render such arguments of little value as a guide for the policy any city should adopt. At Columbus, Ohio, for instance, the object of the construction of sewage-treatment works was to prevent offensive river conditions within the city limits. The destruction of disease germs was unimportant. The sewage-treatment works of Worcester, Mass., were built to prevent offensive conditions in the Blackstone River, which was so polluted by industrial wastes and sewage from other communities that it was everywhere unsuitable for domestic water supply. At Fitchburg, Mass., elaborate treatment works have been constructed for reasons similar to those that governed the Columbus case. Such conditions are different from those arising when sewage is discharged into a stream or lake from which drinking water is obtained within such a distance from the sewer outlet that its contamination is probable unless careful treatment of the sewage is employed to reduce the number of pathogenic bacteria.

The consensus has come to be that protection of the public health is secured more economically and effectively by purification of the water than by treatment of the sewage prior to its discharge into the watercourse. There are limits to the degree of contamination of the water supply beyond which it is unsafe to load a water-purification plant. Sewage treatment should be supplied to reduce the burden of contamination to that which can be handled with safety by the water-purification plant.

The desirable degree of treatment of sewage which must be discharged into a body of water constituting the source of a municipal water supply depends upon a number of considerations. Where the water supply is taken from a small stream or where a larger river is entirely diverted for purposes of water supply, the treatment generally must be relatively complete, unless the volume of sewage be small in relation to the size of the stream. In such cases the treatment must generally include thorough oxidation of the organic matter and chlorination of the effluent. At Barrington, N. J., for example, the sewage is treated by screening, sedimentation, activated sludge, rapid sand filtration and chlorination, prior to discharge into a small stream which contains little pollution.

In cases where the sewage is discharged into large rivers or lakes from which the water supply is taken at a considerable distance from the point of such discharge, the degree of treatment may be considerably less, and where such rivers as the Ohio and Mississippi are under consideration, it may not be necessary to treat the sewage in any manner. For example, Cincinnati and Louisville take their raw water from the Ohio River, notwithstanding that the sewage from several large cities and many smaller ones is discharged without treatment into the river at upstream points. In all such cases, however, care must be exercised to prevent such contamination as will place too serious a burden upon the water-purification plant.

Sewage and Offensive Conditions.—If not suitably disposed of, sewage has always been the cause of annoyance. That created by overflowing cesspools and neglected privy vaults may be eliminated by provision of a suitably designed and operated sewerage system, but this system must discharge somewhere, so that the appropriate disposal of the sewage ultimately must be provided for.

The natural purifying agencies employed in disposal by dilution in watercourses or by broad irrigation on land have limitations. If these natural purifying agencies are overloaded, offensive conditions will result.

In 1924 the Massachusetts Department of Public Health, in reporting upon sewerage and sewage disposal for cities and towns in the Merrimack River Valley, described conditions in one section of the river as follows:

Observations of the flow of the river and movement of the tides in the stretch of the river below a point about two miles above the Haverhill bridge show that on the flood tide water flows upstream past the city of Haverhill to a point some two miles or more above Haverhill bridge, thus carrying some of the pollution discharged at Haverhill upstream to meet that from Lawrence and Lowell, while on the outgoing tide the polluted waters are not carried far enough downstream to prevent their return on the following flood. The result of this condition is that the sewage and pollution matter discharged into the river are not carried quickly away, but there is a tendency to concentration in the neighborhood of Haverhill and for several miles below. The condition of the river in this part of its course, as shown by the investigation of the past year, is very objectionable. The bottom and banks of the stream are covered with foul organic matter and the odor is offensive, especially in warm weather when a considerable portion of the river bed is exposed at low tide. The numerous pleasure boats in this part of the river are badly fouled by the water, while floating on the surface of the river itself during the past year were masses of sludge carried up from the bottom by decomposition.

Conditions in the vicinity of New York City are described by Donaldson (1) as follows:

Unfortunately the population adjacent to the water front has grown much faster than the provision for adequately caring for the sewage of these communities. This has resulted in many instances in local nuisances and actual danger to the health of those patronizing the beaches. This statement applies to portions of the shores of northern New Jersey, Staten Island, the westerly end of Long Island and Long Island Sound.

Sewage treatment may be provided to relieve natural purifying agencies of an overburden and thus avoid offensive conditions.

In the selection of the type of sewage treatment, consideration must be given not only to the degree of treatment required but to the character and limitations of the treatment-plant site. Some methods of treatment are more likely to produce offensive odors than others and care must be exercised to prevent the creation of offensive conditions at the treatment plant while attempting to eliminate them at some other point. The gases from sludge dryers at Pasadena and Milwaukee have been odorous and have been noticeable over large areas. Odors from sludge lagooned at Houston were so objectionable that another method of sludge disposal was required. The moth fly prevalent about trickling filters may cause complaint at some locations. Odors from septic sewage, exposed to the air in open tanks or sprinkled onto trickling filters, from screenings and sludge dumps and from other sources about the treatment plant must be guarded against.

Legal Aspects of Sewage Disposal.—The pollution of water has given rise to considerable litigation. In the case of unnavigable running streams, the law as interpreted in most states is based on the old common law principles that every riparian owner is entitled to have the waters of the stream reach his property in their natural condition except for reasonable use of them by upper riparian owners, and that every riparian owner is entitled to make such reasonable use of the water flowing past his property as he sees fit. Certain states[1] in the semiarid regions have abrogated the doctrine of riparian rights and substituted the doctrine of prior appropriation, under which an appropriator may take an amount of water necessary for his purpose when used in an economical manner, whenever such amount of water exists in a stream over and above the rights superior to his, and may continue such diversion and use as long as needed. Such an appropriator must take water in the condition in which it existed at the time of the inception of his right, but he is entitled to have the water maintained in such condition

[1] Arizona, Colorado, Idaho, Nevada, New Mexico, Utah and Wyoming.

as long as he uses it, and any increase in pollution may be enjoined.

Under the common law the water rights of a riparian owner depend upon the interpretation of the meaning of a "reasonable" use of the water. For many years the courts have been engaged in settling suits involving that question, and today it is established in many states that a riparian owner can use the water freely for watering stock, household purposes and irrigation of land, provided these uses make no appreciable reduction in the volume of the stream and result in no material pollution of its water. It is evident, however, that what might be considered reasonable use of the stream for fishing, drainage of farm lands, removal of sawmill refuse and other purposes in sparsely settled regions would be prejudicial to public welfare in more populous districts. Hence the courts have adopted no rigid rules for interpreting the law but have decided what was reasonable upon the merits of each case.

The principles of the common law are applied also to the waters of privately owned, natural ponds. If there are several owners, each has the same rights as the others, just as though the pond were a stream and the owner a riparian owner.

The invasion of public and private rights in ponds and non-navigable streams by turning the contents of sewers into them is not due to the collection of storm water in drains and sewers and its discharge into the natural drainage courses of the catchment areas. Storm water, even though increased in volume by a reduction in the area of permeable ground and changed in character by the inclusion of street refuse, is legitimately discharged into the streams naturally carrying away the run off of the valley but, according to Montgomery and Phelps (2), such discharge may not legally cause the capacity of the stream to be exceeded so as to overflow to the injury of a lower riparian proprietor.

Where the public health is endangered, the common law, which is concerned with property rights, is often supplemented by statutes, as the extent to which the former will apply to infractions of sanitary principles is uncertain. In some states the statutes are passed directly by the legislatures, while in others commissions are clothed with power to make regulations which become, for all practical purposes, the same as statutes.

Regarding the rights and duties of municipal corporations in stream pollution, Goodell (3) reached substantially the following conclusions. Considered as corporations, municipalities have only such rights and powers as are conferred on them by statute, either expressly or by necessary implication. When, under due authority, they become the owners of lakes, reservoirs and natural streams, they have the same rights to pure water and are charged with the same duties as other riparian proprietors. If authorized to construct sewerage systems draining into streams, such authority generally does not exempt them from the duty not to pollute the stream to the detriment of lower proprietors. The common-law water rights of property owners cannot be taken from them for public use except upon payment of an amount determined by condemnation proceedings authorized by statute. Until municipal corporations have acquired by such proceedings the rights of all lower proprietors and paid for them, they are required to refrain from the pollution of streams, to the same extent as private owners.

The Problem of Industrial Wastes.—Stream pollution by industrial wastes has assumed increasing importance during the last fifty years, coincident with rapid industrial development. Conditions vary greatly, owing to diversity in volume and quality of wastes and to differences in relative volumes of available diluting water.

The effect of many wastes upon waters into which they are discharged is similar to that of sewage, causing discoloration, turbidity, stimulation of fungal and algal growths, depletion of dissolved oxygen, extermination of fish life, putrefaction accompanied by objectionable odors, and deposits affecting navigation.

Unlike sewage, certain industrial wastes contain constituents sufficiently toxic directly to destroy fish life, while others, like acid mine waters, phenol wastes from gas plants and saline wastes from salt works and oil wells, render purification of waters to make them suitable for domestic use difficult and in some cases impracticable. Most wastes, if discharged into bodies of water subsequently used for water supply, tend to increase the burden placed on plants for the purification of water for domestic uses.

In many places industrial wastes discharged into municipal sewers have an important effect upon the volume and quality of the sewage and therefore upon the type and size of treatment plant which the municipality is required to build. Generally the cost of such a plant will be considerably greater than one for treating the sewage without the wastes and occasionally the quantity and volume of wastes are such as to make suitable treatment of the sewage impracticable or prohibitively expensive.

The magnitude of the problem of industrial wastes disposal is realized by few engineers, manufacturers and legislators. There are many single industrial plants the polluting properties of whose wastes are equivalent to those of the domestic sewage from a city of 100,000 persons, and some of the wastes are far more expensive to treat than the sewage from an equivalent population. It is, therefore, of the utmost importance that stream pollution laws be administered with due regard for the "rule of reason," that treatment be regulated according to the actual needs in each case, and that treatment plants be designed and operated with a keen appreciation of cost on the one hand and the accomplishment of the requisite results on the other.

Agencies for Control of Stream Pollution.—In most states stream pollution is regulated to some extent by state boards of health or other similar bodies, which exercise a limited degree of control over the discharge of sewage and industrial wastes into streams and lakes within their respective borders. Their power consists commonly of authority to review plans of sewerage systems and sewage-disposal projects, which municipalities or industries propose to construct, as well as authority to recommend the building of such works when the necessity has been shown. As a result of the fact that these boards are largely without power to enforce recommendations, they cannot abate pollution any more rapidly than public opinion dictates.

In recent years certain local and national associations have been active in molding public opinion toward maintaining the purity of streams through the treatment of sewage and industrial wastes before disposal into inland waters. As an example of such organizations may be mentioned the Izaak Walton League, a national association of sportsmen, who are interested in preventing pollution which may be detrimental to aquatic life. Through publications, meetings and correspondence, this organization has aroused public opinion and aided in the passage of laws to abate stream pollution.

Until recently there has been no effective method of controlling the pollution of watercourses which lie within the boundaries of two or more states. In many cases the contamination of interstate streams is a serious problem, especially where such streams are used as sources of domestic water supply. The problem has been attacked in recent years by drawing up cooperative agreements between health departments of various states, for the purpose of conserving the purity of interstate streams and lakes.

Such an agreement has been in force among states in the Ohio River drainage area since 1924, when it was adopted particularly for the purpose of eliminating the discharge of phenolic wastes into the river and its tributaries. Phenolic substances, even when present in minute quantities, impart to water a taste and odor that may be decidedly objectionable. In the Ohio River basin the original problem was the control of discharges from by-product-coke plants. The results obtained under the Ohio River Basin Interstate Stream Conservation Agreement have demonstrated the usefulness and wisdom of this plan, which forms the basis of cooperation between states and with the industrial interests of these states, without detracting from legal means for compelling action which may be warranted in certain cases. A similar plan for abating the pollution of the Great Lakes has recently been adopted by the state health departments in that drainage basin and is known as the Great Lakes Drainage Basin Sanitation Agreement.

Sewage Disposal and the Sewerage Plan.—The selection of the best method of treating the sewage of a community depends not only on the requirements arising from the characteristics of the water into which the effluent will be discharged, but also on the practicability of obtaining sites for the plants required by different suitable plans and the relative costs of constructing and operating these plants. In a large city two or more treatment plants may be more advantageous than one. Pratt recommended three plants for Cleveland in 1915, although their cost and that of the necessary sewers to serve them were about the same as that of the original plan which contemplated one outlet. At present one plant has grit chambers, Imhoff tanks and trickling filters; another has grit chambers, Imhoff tanks and disinfecting apparatus, and the third has grit chambers and disinfecting apparatus.[1] This arrangement permits the treatment of the sewage while it is in a relatively fresh condition, thus reducing the danger of disseminating objectionable odors.

In studying such problems, dilution as a means of treatment and disposal should be considered. The joint trunk sewers serving many of the cities and towns about Boston, Mass., and Newark, N. J., are examples of works constructed cooperatively, which relieve inland communities of treatment problems that would be quite serious in some cases. At Los Angeles, after an unsuccessful experience with sewage irrigation, it was considered best to construct an outfall sewer, 12.4 miles long, to the ocean.

Cost of Sewage Treatment.—In compiling figures for judging the relative merits of different methods of treatment, the cost of one or more pumping stations, a trunk or intercepting sewer to convey sewage to the plant, and an outfall sewer from the plant to the point of discharge must be considered a part of the total expense, and sometimes these items are a large part of the entire amount. Operating, maintenance, interest and amortization charges on such structures must be combined with similar charges on the treatment plant in order to reach a true estimate of the annual cost of the method of treatment.

It is obvious upon reflection that the cost of treating and disposing of sewage will vary according to local conditions which influence cost in many ways, including the determination of the necessary degree of purification.

A rough approximation of the costs of construction and operation of different types of treatment plants, based on their designed capacity, is given in Table 2.

A substantial amount of construction of intercepting sewers or extensions of trunk or outfall sewers may be required to intercept the

[1] An activated-sludge plant with preliminary sedimentation had been constructed but not completely equipped in 1934, at the site of the third plant.

sewage and deliver it to the treatment plant. The costs involved vary widely with local conditions but in many cases may range between $5 and $15 per capita.

TABLE 2.—APPROXIMATE PER CAPITA COSTS OF CONSTRUCTION AND OPERATION OF VARIOUS TYPES OF SEWAGE-TREATMENT PLANTS, BASED ON DESIGNED CAPACITY

Type of plant	Cost of construction	Annual cost of operation
Fine screening	$2.50–$10.00	$0.20–$0.40
Sedimentation and sludge digestion	5.00– 10.00	0.20– 0.40
Sedimentation, sludge digestion and trickling filter treatment	9.00– 16.00	0.30– 0.65
Sedimentation, sludge digestion and activated-sludge treatment	6.00– 25.00	0.60– 1.20

Economic Utilization of Sewage or By-products of Sewage Treatment. The utilization of the fertilizing or irrigating value of sewage for the raising of crops is not economically feasible in this country, except possibly in isolated cases in the semiarid regions of the Southwest.

The possibility of precipitating sewage solids and separating the grease, thereby producing a nearly greaseless fertilizer and recovering the grease, has been studied but has never been developed on a profitable working scale. With present prices of grease and fertilizer there is little probability of realizing a profit from the by-products of such a process.

Many attempts have been made to sell dried sludge as a fertilizer in Europe, probably the most successful being that at 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 by the Royal Commission on Sewage Disposal to careful business organization and judicious advertising; the commission estimated that the sales of sludge decreased the net cost of sewage treatment about 80 cents a mil. gal. Investigations of the practicability of drying sludge from the older treatment processes to form a fertilizer base have been discouraging in the United States, but since the advent of the activated-sludge process, there has been a somewhat brighter prospect for utilizing the solids removed from sewages at the large treatment plants.

The preparation of fertilizer from activated sludge is a manufacturing process having a number of stages which probably can be conducted profitably only on a large scale, if at all. 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 and economical manner. Whether it is desirable for a municipality to embark upon such a business venture will depend upon local conditions. Unless the enterprise is handled efficiently, there are possibilities of grave financial troubles. The overhead expense in any case is likely to be large and the proportionate cost of operating a small sludge-fertilizer plant will probably be much greater than for large plants.

Excess activated sludge is dewatered, dried and sold as a fertilizer or fertilizer base at several plants, notably at Milwaukee and Pasadena. The Milwaukee production was about 100 tons a day in 1930. The cost of operating and maintaining the sludge disposal plant about equaled the net return from the sale of sludge. Interest and depreciation must be charged to the cost of sewage disposal, which includes the cost of operating the sewage-treating units of the plant and the fixed charges. At the present state of the art there does not seem to be any way in which the sludge can be sold for more than enough to pay operating and maintenance charges upon the sludge dewatering, drying and handling plant.

Value of Experimentation.—The great advances in sewage treatment which have been made since the growth of cities and industries forced attention to this problem upon engineers are associated, apart from laboratory research, with the studies at experimental plants of large communities. These were undertaken chiefly in order to select the most economical method of treatment under the local conditions. Particularly noteworthy in this country are the studies made at Baltimore, Chicago, Cleveland, Milwaukee, New Haven, Philadelphia and Worcester. There can be no doubt that these studies resulted in savings which justified their cost, while advancing at the same time the art of sewage treatment and disposal.

In spite of the progress made in this field during the past 60 years, sewage treatment remains a young art. The developments of science are so rapid and far reaching and local conditions are so varied that new methods or combinations of methods will continue to be brought to light.

Bibliography

1. DONALDSON, WELLINGTON: *Nation's Health*, January, 1926.
2. *Pub. Health Bull.* 87, U.S. Pub. Health Service, 1917.
3. *Water Supply Paper* 152, U. S. Geol. Survey, 1905.

CHAPTER III

CHARACTERISTICS AND BEHAVIOR OF SEWAGE

Sewage possesses properties that render it both offensive and dangerous. Design and operation of sewage-treatment works and control of the pollution of water courses into which sewage is discharged presuppose a thorough appreciation of the physical, chemical and biological characteristics and behavior of sewage and sewage matters.

General Characteristics of Sewage.—In appearance, sewage resembles dirty dish water or bath water, largely owing to the soaps it contains, to which has been added such floating matter as fecal solids, bits of paper, matches, grease and oils, vegetable and animal refuse and fruit skins. These visible solids give rise to an exaggerated idea of the solid matter conveyed in the sewage. The organic and mineral matter constitute about 0.1 per cent of the sewage. In other words, it is mostly water, about 99.9 per cent. It is the organic portion of the relatively small quantity of solids that gives to sewage its offensive characteristics and makes its disposal a problem of far-reaching importance.

Physically, sewage contains matter in suspension and matter in solution. Of the suspended solids, some will settle when the transporting power of the water is decreased by a reduction in its velocity and some will remain in suspension even during protracted periods of quiescence. The quantity of material carried can be realized from computations of the bulk of the substances, other than water, discharged through sewerage systems. In 1910 the Metropolitan Sewerage Commission of New York, for example, estimated that the city's sewage contained, on a dry basis, per thousand population annually, 14 tons of feces, 8 tons of toilet paper and newspaper, 11 tons of soap and washings, 8 tons of street wastes, and 4 tons of miscellaneous substances; a total of 45 tons (1).

Chemically, sewage contains substances of animal, vegetable and mineral origin. The animal and vegetable substances, called collectively *organic matter,* are in large part offensive in character or behavior. They constitute about 50 per cent of the sewage solids and are made up of complex chemical substances which are readily broken down by biological and, to a lesser degree, chemical action into other, usually simpler, compounds.

Biologically, sewage contains vast numbers of living organisms among which the bacteria predominate. One gallon of sewage may harbor from

20 to 250 billion bacteria. Most of these organisms are harmless to man and are largely engaged in the beneficent activity of converting the complex organic constituents of sewage into simpler, more stable, organic and mineral compounds. Sewage, however, may contain bacteria or other organisms that have come from persons sick with typhoid fever, dysentery or other so-called water-borne diseases. Some disease-producing, or pathogenic, organisms are commonly present and it is these that constitute the real danger of sewage to the public health.

Concentration, Composition and Condition of Sewage.—Phelps (2) has pointed out that there are three distinct general characteristics of sewage concerning which analytical data may be acquired, *viz.*, concentration, composition and condition. These are physical, chemical and biological, respectively, in their manifestations and depend upon the quantity, nature and freshness of the sewage matter.

Concentration is a term commonly used to designate the proportion of sewage matter to water. A strong or concentrated sewage contains a relatively larger proportion of sewage matter, while a weak or dilute sewage contains a relatively smaller one. Strong sewage may be made weak by the addition of water, but this does not change the relative proportions of the constituents. Weak sewage may be made stronger only by the addition of certain substances, for example, by the admission of stronger industrial wastes to a sewage that is normally dilute. There is no recognized standard by which a sewage may be classed as strong or weak. Judgment of this matter is principally based upon the content of organic matter, the organic nitrogen, carbon and fats, quantity as well as kind of organic matter being given consideration. Mineral matter is of less consequence in determining the strength of sewage.

Composition is the chemical characteristic of sewage and has to do with the solid material, its various constituents, their quantities and their relations apart from the degree of concentration. Composition must be determined before concentration can be judged. If the quantities of certain constituents are high as compared with average experience, the sewage is called strong, if low the sewage is called weak. A sewage containing tannery wastes may be high in fatty substances, one containing waters from iron works may be high in iron sulfate and sulfuric acid.

Condition is a characteristic that is governed by the changes that have taken place in sewage during its time of passage through the sewerage system. Condition is not a function of time alone. In addition to the time element, or age, there are to be considered the temperature of the sewage, the number and kind of microorganisms present, the degree of mechanical abrasion that breaks up solid matters, the opportunity for absorption of oxygen and the effect of germicidal substances. The longer the sewage flows or stands and the higher the temperature, the more are its constituents changed; fecal matter and other suspended solids are comminuted; bacterial activity in the absence of inhibiting agents increases enormously and assists in the breaking down of complex organic compounds; the oxygen originally present in the water is reduced and may even disappear, so that the sewage passes from its *fresh* condition, becomes *stale* in the absence of oxygen and finally *septic*.

SEWAGE ANALYSIS

Sewage analysis is not a thoroughly standardized procedure. With the publication of "Standard Methods of Water and Sewage Analysis" and the subsequent revisions by the American Public Health Association, much was accomplished in this direction. These methods have been adopted quite generally by sewage works laboratories in the United States, but procedures are not uniform even in laboratories that make full use of the methods. There are two reasons for this condition. In the first place, "Standard Methods" in some instances does not recommend a single procedure for a given determination. A choice is sometimes allowed. Ammonia nitrogen, for example, may be determined either by direct nesslerization or by distillation, and biochemical oxygen demand may be determined by the dilution method or the sodium nitrate method. The different procedures do not always give equivalent values. Secondly, there is the question of a choice of determinations. One set of tests answers the purposes of one works or laboratory but not necessarily those of another. A study of nitrification may be thought of paramount importance at one works. At another it may be that the removal of solids or the oxygen demand is the subject for chief consideration.

Analytical data lose some of their value because tests and procedures are not uniform. Under such circumstances it becomes difficult or impossible to compare accomplishments in one place with those in another, a fact that militates against the advance of the science and art of sewage disposal and sewage treatment. Further progress in the direction of standardization can be made with this advance in view and without seriously imperiling individual initiative and research or the usefulness of data within the works producing them.

Scope of Sewage Analysis.—Sewage analysis deals with a liquid that is highly complex in the nature and number of its constituents and one that is greatly variable in its concentration and composition. The kinds of matter contributed to the sewers change from time to time and there are fluctuating increments of surface, ground and storm waters.

In general, a sewage analysis is designed to give information upon the presence, quantities and properties of sewage matters. More specifically, it may be intended to throw light upon the condition of the sewage or effluent, upon the degree of purification to which it has been subjected, or upon the probable effects when it is discharged into a stream or other body of water. To these ends the laboratory analysis may comprehend the results of many separate determinations, that in one way or another measure or reflect the physical, chemical and biological qualities of the sewage. In some cases, as for instance in certain studies of sewage-treatment processes, a few determinations may suffice to give the required information. For routine purposes in treatment plants Wagenhals, Theriault and Hommon (3) recommended as a minimum a very small number of tests.

Practically the aim of all methods of sewage treatment is the oxidation of the soluble or colloidal portion and the removal of the suspended or readily settleable portion as an inoffensive or even marketable sludge. The aim of sewage analysis, therefore, should be to determine both the degree to which the removal of suspended matter has been effected and the state of oxidation of the soluble portion which is to be discharged into a stream. . . These two tests, the oxygen demand and the suspended matter, seem to be capable of furnishing about all the analytical data that are required for the proper operation of the ordinary sewage treatment plant, and are suggested as an irreducible minimum of laboratory work. At activated-sludge plants the settleable solids and the ammonia nitrogen should be determined in addition. Other tests are of value as additional circumstantial evidence. .

It must not be inferred from the foregoing that sewage analysis is concerned only with a narrow field of tests. Where experimental and research work are involved, a wide range of determinations is available for use and this range is continually broadening, as the horizon of the sciences is pushed forward to make available new knowledge and new procedures. The study of sewage problems stands in need of advances in the field of analysis.

Outside the immediate scope of analysis, but closely concerned with it, is the task of collecting supplementary data that will make more useful the results of analysis. It is of the utmost importance, when interpreting analyses of sewage, to procure as complete information as possible regarding conditions affecting the composition of the sewage. This will often lead to discarding analyses that otherwise would have been accepted. Sometimes such information renders the results of analyses of unexpected value. Engineers and chemists therefore should be careful, when reporting analyses, to give in detail all available data relating to the production, collection, sampling and analysis of sewage and effluents.

Classification of Tests.—The examination of sewage is unlike the chemical analysis of many common substances, as, for example, that of ores and metals, in which the exact quantities of each constituent element or group of elements are determined. Sewage matter is too complex and varied to permit this by any available procedures, except in the case of a few substances, such as chlorides, nitrogen, potassium and phosphorus, that may be determined directly. It is, therefore, necessary to depend largely upon tests that measure under the conditions of the test properties of the sewage matters rather than their exact constitution. Thus, the determination of biochemical oxygen demand measures the power to absorb oxygen. Whipple (4) classes such tests, for purposes of sanitary water analysis, as *indirect*, or inferential, in contrast to those having a *direct* bearing upon quality, and gives as further examples the determinations of oxygen consumed and albuminoid nitrogen. He states that

> they supposedly measure the organic matter present, but they do so imperfectly, inaccurately and only by inference. Neither test gives a true idea of the character of the organic matter. . . The indirect tests owe their usefulness to their interpretative value. They are used to judge past, present and future conditions in the absence of suitable direct tests, or when direct tests need substantiating evidence.

Because sewage is so complex and variable in quality, it is also necessary to employ a variety of more or less interrelated tests which may be interpreted together rather than separately. The variety and interrelation of the tests commonly performed are best shown by grouping them in accordance with such a scheme as that shown in Table 3. This

TABLE 3.—APPROXIMATE CLASSIFICATION OF TESTS COMMONLY USED IN SEWAGE ANALYSIS

Gases and volatile constituents	Mineral and organic matter			Living organisms	
	Mineral	Organic	Mineral and organic	Bacteria	Plankton
Odor Dissolved oxygen Hydrogen sulfide	Fixed solids: Total Suspended Settling Nonsettling Dissolved	Volatile[1] solids: Total Suspended Settling Nonsettling Dissolved	Total solids Suspended Settling Nonsettling Dissolved	At 20°C. At 37°C. *B. Coli.*	Animal forms Plant forms
	Ammonia[1] *N* Nitrite *N* Nitrate *N* Chlorides Alkalinity Hardness Iron	Organic *N* Albuminoid *N* Oxygen consumed Biochemical oxygen demand Relative stability Ether-soluble matter	H ion concentration		

[1] Includes both organic and inorganic substances.

scheme takes into account the chemical or biological nature of the constituents or properties that are listed.

An important test that is not incorporated in this schedule is the determination of temperature. The recording of temperature is important in modern sewage-treatment methods, because most of them are dependent upon biological activity which is stimulated or retarded in accordance with the prevailing temperature. Other tests, too, are not shown, such as those for sulfates, phenols, poisonous metals, or other substances that may be significant under certain circumstances.

A suitable choice of tests must be made, for obviously not all will be useful in every analysis. The choice depends upon the purpose of the analysis. Sometimes a single simple test may be all that is required; at other times the analysis must be quite complete. To tell what tests are necessary and sufficient, calls for the exercise of good judgment. Otherwise, there will be either a wasteful expenditure of time and effort or the required data will not be forthcoming. The correct combination of tests is a matter deserving of careful attention, for there are numerous supplementary relationships among them that give assistance to the analyst in his interpretation.

Expression of Analytical Results.—Laboratory methods of expressing the results of analytical work will be found in Chap. IV. The usual term is *parts per million* (p.p.m.), ordinarily equivalent to *milligrams per liter*. It is often desirable to convert this term to some equivalent, for the varying quantities of water in different sewages make it difficult to compare the average analysis of the sewage of one community, or even of one sewerage district, with that of another. If, however, in addition to accurate analyses 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. The accepted method of stating composition is then in terms of *grams per capita daily*. It should be noted, however, that ordinarily even by this method the prediction of the composition is uncertain, because of lack of accurate data. There is greater accuracy in predicting the character of sewage from a residential city than from an industrial community, for in the latter the effect of industrial wastes is extremely difficult to estimate without studies in detail of the quantity and quality of such wastes. Another element of uncertainty is the influence of solids in water supplies and in ground water, for they may have an important influence upon the analyses of the sewage.

Parts per million can be converted to grams per capita or the converse by means of the following formulas, in which A = parts per million of any constituent and C = U. S. or Imperial gallons or liters per capita daily:

For United States gallons:
$$\text{Grams per capita daily} = 3.785AC/1000$$
For Imperial gallons:
$$\text{Grams per capita daily} = 4.543AC/1000$$
For liters:
$$\text{Grams per capita daily} = AC/1000$$

For a sewage flow of 100 gal. per capita daily 1 p.p.m. is equivalent to 0.378 gm. per capita daily.

For a sewage flow of 264.2 gal. per capita daily the number of parts per million is equivalent to grams per capita daily.

Other methods of statement are sometimes found useful, such as pounds per capita daily, tons per 1000 population yearly and pounds per million gallons, or lb. per mil. gal. One gram per capita daily equals 0.00221 lb. per capita daily and 0.403 ton per 1000 population yearly. One part per million equals 8⅓ lb. per mil. gal.

COMMON CONSTITUENTS AND PROPERTIES OF SEWAGE

Solid Matters in Sewage.—It has been stated at the beginning of this chapter that sewage contains approximately 0.1 perc ent, *i.e.*, 1 part in 1000, of solid matters. These make up what is termed the total solids, or residue on evaporation. Part of the total solids in sewage comes from the water that is its principal constituent. This water is not necessarily representative in its solids content of the water supply of the community. The whole of the water supply does not reach the sewers. The portion that does is supplemented often by water furnished from private supplies and usually by ground water and storm water. If the community supply is low in solids and that from other sources is hard or highly mineralized, the latter will contribute a considerable proportion of the solids in the sewage. On the other hand, if the water from supplementary sources is low in residue, as is storm water, it will contribute to the sewage an insignificant proportion of its solid matter.

Other solid matters are contributed by the soap, grease and food of kitchen wastes, by garbage that is sometimes thrown into water-closets, by laundry and bath waters, by wash water from other parts of households, hotels, office buildings and institutions, and by drainage from stables and automobile washstands. Feces, urine and paper from water-closets add greatly to the content of solids. Industrial wastes may contribute even larger quantities. In combined systems, street washings and storm waters impose their load of solid matter on the sewage. Table 4 presents an estimate of the average quantities of solid matters in sewage with respect to their origin.

The quantities of solids in sewage depend, therefore, upon the character of the water in the sewers, the habits of the population, the nature of the industries and the use of a combined or separate system. As a result there is generally considerable variation in the quantity of solids in sewage of different cities, even when they have about the same population.

TABLE 4.—ROUGH ESTIMATE OF TOTAL SOLIDS IN THE SEVERAL CONSTITUENTS OF SEWAGE

Constituents	Grams per capita daily	
	Items	Total
Water supplies and ground water, assumed to be soft	12.7	
Feces	20.5	
Urine	43.3	
Toilet paper and newspaper, suspended	20.0	
Solids from 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, Colloidal and Dissolved Matter.—On a physical basis there are three states in which solids mixed with water or with other liquids may manifest themselves, the suspended state, the colloidal state and the dissolved state. The solid matters of sewage are found in all three. The size of particles when placed in water will largely determine whether there will be obtained, according to the above classification, a suspension, a sol or a solution. The differences between these three are given below. Thus, the same substance in different degrees of fineness may give either a suspension or a sol. Coarse sand in water provides a suspension from which the sand soon settles out, while powdered sand gives a sol from which little material settles out after a considerable period of time.

A *suspension* is a coarse mechanical mixture, the solid or semisolid parts of which can be separated from the liquid by the processes of sedimentation or filtration, that is, the solid material is so coarse that it readily separates by gravity. Suspensions are heterogeneous, non-homogeneous, in their make-up.

The suspended matter of sewage is represented by grit and other detritus, such as paper, rags and heavy organic solids. Suspended matter varies greatly in size and specific gravity. Much of it is coarse, like fruit skins, matches, corks and paper. The finer portions are similar in nature to the coarser and come in part from their breaking up during passage through sewers, screens and pumps, and in part from disintegration of the larger masses by organisms and their enzymes. As a rule, the older the sewage, the more finely divided is the suspended matter.

Some matters are thrown out of solution by chemical changes due to combinations of soaps, carbonic acid, ammonia and, where industrial wastes are discharged into the sewers, lime, iron salts and many other spent chemicals.

A *solution* is defined by Buswell (5) as follows:

> True solutions of pure chemical substances are regarded as being composed of particles of uniform composition and of the same general size as the molecules of the solvent and it is for such examples that the name 'solution' should be reserved.

They are relatively homogeneous in make-up, cannot be separated by mechanical means into their constituents and are clear when viewed by ordinary light. The solids contained in solutions are sometimes called crystalloids, because many of them crystallize readily. They also diffuse rapidly.

Solids contained in solution in sewage are such substances as sodium chloride, the nitrates and the hardness constituents.

A *sol*, or colloidal solution, may be defined as a very fine subdivision of matter suspended in a liquid. The finely divided material is a colloid. This term, taken from the French word for glue, was first suggested by Graham, to distinguish certain amorphous substances of low diffusibility from those of marked diffusing power and crystallizing tendencies that were called crystalloids.

Colloidal particles can be precipitated by acidification, by the addition of solutions of various salts or by the introduction of other colloidal mixtures, the last being illustrated by the precipitation of organic colloidal material by gelatinous mineral hydroxides, such as aluminum or iron hydroxide, an action utilized in the clarification of sewage by the method of chemical precipitation. Chemical changes like oxidation at times precipitate colloidal matter. Alkalies in general tend to increase the stability of sols.

In the course of a long flow in sewers some coarse suspended matter disintegrates and is dispersed in a colloidal state. Conversely, contact in relatively quiescent condition with solid surfaces will throw matter out of the colloidal state into suspension. Thus colloidal matter is removed from sewage by sand filtration and accumulates upon the filtering medium.

Practically speaking, the term "colloids" is often used in the discussion of problems relating to sewage treatment to include both finely divided matter in suspension that will not settle readily in sedimentation tanks and matter in true colloidal condition. Considerable importance attaches to these finely divided particles that do not settle. For the greater part they are highly putrescible and in their decomposition reduce available oxygen resources and produce bad odors.

The colloidal matter in sewage is contributed by precipitated soaps, by portions of the fecal matter, by finely divided organic matter from the soil and from refuse, by grease and oil, by clay and fine sand particles and by industrial wastes. The last are often the source of large increments of colloidal material.

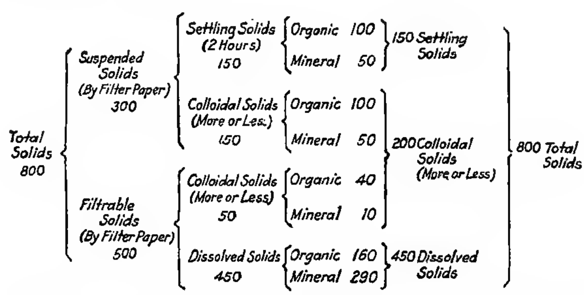


FIG. 1.—Physical condition of principal constituents of sewage of medium strength. (Numbers are parts per million.)

The physical condition of the solids in sewage of medium strength, suspended, dissolved and colloidal, and the average quantities present in each condition are shown in Fig. 1. The terms *suspended* and *filtrable* solids, as used, are based upon the separation obtained by means of filter paper. It will be noted that the colloidal solids are a portion of both the suspended and the filtrable solids. This is because some colloidal particles are removed by filter paper while others pass through it. About 38 per cent of the total solids are removed by filter paper and are classed as suspended; the other 62 per cent pass through and constitute the filtrable solids. The colloidal solids make up about 25 per cent of the total. On the basis of 2 hours' sedimentation, the settling solids amount to approximately 19 per cent of the total. Solids in true solution constitute about 56 per cent of the total solids.

Settling Solids.—The suspended matter of sewage is made up of solids in all degrees of fineness, from pieces of paper, rags and fecal matter to very fine colloidal particles. The great bulk of it, the coarser part, is capable of sedimentation and goes to make up the *settling solids*.

Settling solids are commonly understood to be the quantity of sludge or sediment that will be deposited from a given quantity of sewage in a certain interval of time. These solids are of importance, for they can cause deposits in slips, streams and other bodies of water into which sewage is discharged, there to reduce channel depths and to give rise to offensive conditions from putrefaction. They are caught on racks and screens, tend to clog filters, settle and are retained in sedimentation tanks.

Table 5.—Suspended Solids in Chicago Sewage and Stock Yards Sewage Capable of Settling in Periods Stated[1]

Kind of sewage	No. of tests	Total susp. solids, p.p.m.	1 hr.		2 hr.		4 hr.		12 hr.	
			P.p.m.	Per cent	P.p.m.	Per cent	P.p.m.	Per cent	P.p.m.	Per cent
Total settling solids										
Stock yards........	3	1620–1930								
	Av.	1775	1508	85	1580	89	1597	90	1633	92
Stock yards.	6	760–1070								
	Av.	915	650	71	686	75	705	77	714	78
Stock yards........	1	560	297	53	314	56	314	56	319	57
39th St............	3	206–266								
	Av.	236	130	55	146	62	165	70	186	79
39th St............	5	118–194								
	Av.	156	70	45	84	54	98	63	109	70
39th St............	2	68–76								
	Av.	72	17	23	22	31	31	43	42	59
Volatile settling solids										
Stock yards...... ..	1	1140	889	78	935	82	980	86	1003	88
Stock yards........	4	620–840								
	Av.	730	526	72	569	78	591	81	620	85
Stock yards........	5	420–560								
	Av.	490	299	61	304	62	314	64	353	72
39th St............	8	84–127								
	Av.	106	43	41	52	49	61	58	67	63
39th St............	2	50–52								
	Av.	51	9	18	12	24	14	27	18	35

[1] From "Report on Industrial Wastes from the Stock Yards and Packingtown in Chicago," 1914, by G. M. Wisner and Langdon Pearse. Tests made in can 2 ft. diameter and 9 ft. deep. Samples withdrawn from center of can 18 in. below surface of sewage. Can was filled from bottom to depth of 8 ft. 6 in. and allowed to stand quiescent for period stated.

The settling solids are made up of sand, grit, paper, rags, fruit skins, fecal matter and other coarse organic detritus, together with some finer particles that are carried down mechanically on the surfaces of the coarser material. Industrial wastes often add a heavy increment of settling solids, as do combinations of these wastes when mixed in the sewers or the treatment works. At Worcester, Mass., the mixture of iron pickling liquors and lime wastes forms large volumes of sludge that settle in the Imhoff tanks. The quantity of settling solids that will be deposited from a given sewage depends upon the time allowed for settling and the velocity of flow of the sewage. The longer the time and the lower the velocity, the greater will be the deposition of solids. It is generally the case, too, that the ratio of settling solids to total suspended solids increases with the strength of the sewage. This is shown by the figures given in Table 5.

It is usual to consider the time for settling to be 2 hr and in the laboratory determination of settling solids this period is used more than any other, as mentioned in Chap. IV. In practice a period of 2 hours' detention of sewage in tanks throws down the bulk of settling solids without initiating putrefactive processes in the sewage. The velocity of flow through tanks designed to remove settling solids is low enough to permit subsidence of some fine particles. In the laboratory perfect quiescence accompanies the determination of settling solids. This permits discrepancy between the results of laboratory and plant, the laboratory tests usually giving somewhat higher results than those attained in practice. The difference is due not only to the velocity maintained in tanks but also to wind action and to convection currents induced by temperature changes.

Organic Matters in Sewage.—In a sewage of medium strength, as shown in Fig. 1, about 67 per cent of the suspended solids and 40 per cent of the filtrable solids are organic in nature. They are derived from both the animal and plant kingdoms and represent a combination of the elements carbon, hydrogen and oxygen, together with nitrogen in some cases. Other important elements, such as sulfur, phosphorus and iron, may also be present. Both living and inert forms of matter go to make up this organic content. The principal groups of organic substances represented in sewage are the proteins, carbohydrates and fats and the products of their decomposition. All of these groups break down more or less readily through the activity of bacteria and other living organisms. Urea, the chief constituent of urine and another important organic compound that is contributed to sewage, decomposes so rapidly into ammonium carbonate that undecomposed urea is found in quantity only in very fresh sewage.

Proteins are the principal constituents of the animal organism. They occur to a lesser extent also in plants. All raw animal and plant foodstuffs contain proteins. The amount present varies from small percentages in watery fruits, such as tomatoes, or the fatty tissues of meat to quite high percentages in beans or lean meats (6). Proteins are complex in chemical structure and unstable, being subject to many forms of decomposition. Some are soluble in water, others insoluble. They all contain carbon, which is common to all organic substances, as well as hydrogen and oxygen. In addition they contain, as their distinguishing characteristic, a fairly high and constant proportion of nitrogen, about 16 per cent. In many cases sulfur, phosphorus or iron is a constituent. Chemical analysis shows little difference in their percentage composition, as illustrated by three important proteins (7):

	Albumin	Fibrin	Casein
C	53.5	52.7	53.8
H	7.0	6.9	7.2
N	15.5	15.4	15.6
O	22.4	23.8	22.5
S	1.9	1.2	0.9

The chemistry of the formation of proteins appears to be a combination, or linking together, of amino acids with formation of water. A large number of amino acids may be involved in the process. The number combined to form casein, for example, has been estimated to be from 60 to 130. Urea and proteins are the chief sources of nitrogen in sewage. When the latter are present in large quantities, extremely foul odors are apt to be produced by their decomposition. They are concentrated to a greater degree in sludge than in the supernatant sewage. According to Buswell (5), "It appears that ammonia, carbon dioxide, fatty acids, alcohols, amines and hydrocarbons may result from the bacterial decomposition of proteins."

The *carbohydrates* are widely distributed in nature and include sugars, starches, cellulose and wood fiber. All are found in sewage. They contain carbon, hydrogen and oxygen; the common carbohydrates contain six, or a multiple of six, carbon atoms in a molecule and hydrogen and oxygen in the proportion in which these elements are found in water. Some carbohydrates are soluble in water, notably the sugars; others are insoluble, such as the starches. The sugars are prone to decomposition, the enzymes of certain bacteria and yeasts setting up fermentation with the production of alcohol and carbon dioxide. The starches, on the other hand, are more stable and are not readily attacked, although under certain circumstances they may be converted into sugars by microbial ferments as well as by dilute mineral acids.

From the standpoint of bulk and resistance to decomposition cellulose is the most important carbohydrate found in sewage. The destruction of cellulose in the soil goes on readily. Here it is largely due to the activity of various fungi, particularly when acid conditions prevail. When the medium is slightly alkaline, certain aerobic bacteria are the more active agents, although the number of species is few. No yeast is known to ferment cellulose. According to Heuser (8), when bacteria bring about the destruction of cellulose, there are four ways in which the action may proceed.

1. Methane fermentation, produced by sewage bacteria, giving methane, carbon dioxide and the lower fatty acids, from formic to butyric acid.
2. Hydrogen fermentation, produced by *Bacterium fermentationis celluosae* (Omeliansky), giving hydrogen, carbon dioxide and the same fatty acids as in (1).
3. Methane hydrogen fermentation, produced by thermophilic bacteria of different kinds, leading to methane, hydrogen, carbon dioxide and formic and acetic acids.
4. Nitrogen fermentation, produced by denitrifying bacteria, *i.e.*, those assimilating nitrogen from the air or other sources and yielding nitrogen and carbon dioxide.

The commercial fermentation of cellulose has also been accomplished and Boruff and Buswell (9) report experiments upon the digestion of mixtures of cornstalks and domestic wastes in which the cornstalks were decomposed to the extent of 35 to 50 per cent. In sludge tanks conditions do not appear to be altogether suitable for cellulose digestion and there is considerable doubt as to whether the process goes on to any great extent. Much cellulose always remains unchanged in sludge after the usual periods of digestion.

Fats are compounds of the alcohol glycerol, commonly called glycerin, with the fatty acids, such as oleic, palmitic and stearic, the resulting glycerides being designated as olein, palmitin and stearin. Other glycerides are also known. The three mentioned are found in animal fats, the first being a liquid and the latter two solids. Fats are commonly found in meats, in the germinal area of cereals, in seeds, in particular fruits and in nuts. They contain carbon, hydrogen and oxygen in varying proportions, the chemical structure of the molecules being relatively simple. Fats are among the more stable of organic compounds and apparently they are not easily decomposed by the bacteria. The mineral acids attack them, resulting in the formation of glycerin and fatty acid. In the presence of alkalies, such as sodium hydroxide, glycerin is liberated and alkali salts of the fatty acids are formed. The latter are known as soaps and, like the fats, they are of stable character.

Common soaps are made in the manner indicated, by saponification of fats with sodium hydroxide. They are soluble in water, but in the presence of hardness constituents the sodium salts are changed to calcium and magnesium salts of the fatty acids, or so-called mineral soaps. These are insoluble and are precipitated. Soaps in sewage are largely of this kind.

Little is known about the decomposition of fats and soaps in sewage-treatment processes, except that apparently they are not greatly changed. If the sewage becomes acid, fatty acids may be liberated from both, together with glycerin from the fats and soluble salts of sodium, calcium or magnesium from the soaps. Fats and soaps are found both in sludge and floating on the supernatant sewage. They come from kitchen and laundry wastes, from packing-house, wool-scouring, tannery and other industrial wastes and from other sources. They are a deterrent to biological processes, in that they coat with a stable film surfaces that would otherwise be subject to microbial attack. Fats depreciate the commercial value of sludge as a fertilizer and tend to clog sewers when discharged into them in large quantities, as sometimes is done from packing houses.

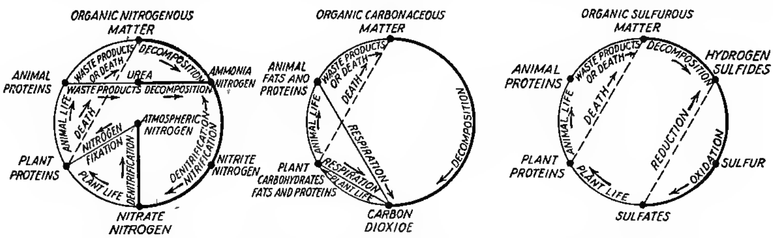
Fats are usually expressed in analysis as ether-soluble matter, for they are extracted by this solvent. There is another group of substances that forms a part of the ether-soluble matter. This embraces the mineral oils, like kerosene and lubricating and road oils, which are derived from petroleum and coal tar and which contain essentially carbon and hydrogen. These oils sometimes reach the sewers in considerable volume from shops, garages and streets. For the most part they float on the sewage, although a portion is carried into the sludge on settling solids. To an even greater extent than fats and soaps, the mineral oils tend to coat surfaces and particles and interfere with biological action.

Decomposition of the various organic constituents of sewage discharged into natural waters results in rapid depletion of the oxygen normally present in the water, after which foul-smelling compounds are formed if the volume of diluting water is inadequate. For this reason it is the organic content of sewage, and particularly its avidity for oxygen, that creates one of the principal problems of sewage treatment and disposal.

The Cycles of Nitrogen, Carbon and Sulfur in Nature.—Organic matter possesses the interesting capability of passing from one condition through others in a cycle back to its original condition. This process, for want of a better name, may be called the "cycle of life and death." In its living form organic matter has organized structure and it may or may not have the power of locomotion. Upon its death it is resolved, through the agency of living organisms, into simpler compounds, some of which are organic and some inorganic or mineral in nature. Some of these in turn serve as food for the living organisms, which use them for building up their structures, thus again incorporating them into the living body.

In order to understand the choice and significance of a number of the tests employed in sewage analysis, especially in relation to the determinations of organic matter, it is necessary to become acquainted with the cycles of nitrogen, carbon and sulfur in nature. Appreciation of these cycles, too, will be found of assistance in interpreting the behavior of sewage in treatment works and in streams or other bodies of water into which sewage is discharged.

In its simplest form the *nitrogen cycle* may be idealized diagrammatically as in Fig. 2*a*. As shown here, organic nitrogenous matter is decomposed by bacterial activity and the nitrogen it contained appears first as ammonia. By *oxidation* or *nitrification* this ammonia is converted through the medium of two distinct groups of nitrifying bacteria first into nitrites and then into nitrates. Nitrates serve as plant food and the nitrogen taken up by plants is built into plant tissue, or plant proteins. By death the plant proteins become organic nitrogenous



(a)-THE NITROGEN CYCLE (b)-THE CARBON CYCLE (c)-THE SULFUR CYCLE

FIG. 2.—The nitrogen, carbon and sulfur cycles in nature.

matter once again. If the plants are eaten by animals, part of the nitrogen is converted into animal proteins and part is wasted. The nitrogenous waste products of animal life are organic nitrogenous matter and urea. The latter is broken down into ammonia by another special group of bacteria.

As discharged into the sewers, some of the organic nitrogen compounds are relatively stable and others are bound together only loosely. Most of the compounds are easily broken up, directly or indirectly, by bacterial action into other substances and the nitrogen, some of which was at first in highly complex form, becomes part of less complex matters, while that which at first was loosely bound is liberated from its organic combination and may then be present in some inorganic form, as ammonia, or it may be liberated as nitrogen gas.

A few deviations from the complete cycle have already been mentioned; there are others. Certain bacteria, for example, reduce nitrates to gaseous nitrogen. This is known as *reduction* or *denitrification*. Other forms of denitrification are the reduction of nitrates to nitrites and ammonia and the liberation of gaseous nitrogen from either of these.

By bacterial activity connected with the growth of leguminous plants, furthermore, free nitrogen is utilized in forming plant tissue. This is known as nitrogen *fixation*. Finally, bacteria may convert ammonia into the living matter of their own cells.

It will be noted that the left half of the circle in Fig. 2*a* is associated with living matter, the right with dead or waste matter. The right half is particularly significant in sewage disposal.

The *carbon cycle* is illustrated in Fig. 2*b*. Decomposition of organic carbonaceous matter produces carbon dioxide gas or carbonates. With the aid of chlorophyll—the green coloring matter of plants—and the stimulus of sunlight, green plants are able to convert carbon dioxide into carbohydrates, which later may be changed into fats and proteins. This use of carbon dioxide is known as *photosynthesis*, the building up of complex substances with the aid of light. As will appear later, plants, while absorbing carbon dioxide, liberate oxygen. In the dark the reverse is true and oxygen is taken in, while carbon dioxide is given off. This is called *respiration*. When plants die, the elements of which they are composed recommence the cycle. Animals feeding on plants convert the plant matter into animal tissue and waste products. By respiration, furthermore, animals absorb oxygen while giving off carbon dioxide.

The third cycle that illustrates the rotation in Nature of the elements composing organic matter is the *sulfur cycle*. Sulfur is always present in sewage and ready to enter into any biological process, although it varies greatly in quantity according to its source. Sulfur in sewage may be derived from any one or all of the following:

1. Vegetable or animal organic matter. Many of the proteins contain sulfur.
2. Sulfates of calcium and magnesium, ordinarily found in water supplies and often present in larger quantities in ground water that leaks into sewers.
3. Sulfates, sulfites and sulfides of the alkalies, alkaline earths and heavy metals that are present in some industrial wastes.

Decomposition of sulfur-bearing organic matter in the absence of free oxygen, as shown in Fig. 2*c*, results in the production of sulfides and of hydrogen sulfide, an ill-smelling gas. In the presence of oxygen the so-called *sulfur bacteria* oxidize the hydrogen sulfide to produce sulfur and sulfates. The activities of plant and animal life then complete the cycle of this element in nature. By reduction sulfates can be broken down under conditions analogous to the reduction of nitrates and nitrites, hydrogen sulfide being produced.

Not shown, but associated with practically all the changes just described, is the cycle of water. Bacteria and other primitive forms of life cannot perform their functions in the absence of water. Water, however, does not play a merely passive part by preventing desiccation or acting as the conveying medium for food substances; it enters actively into both the synthesis and analysis of organic matter. The addition of water to the molecule is known as *hydrolysis* and commonly results in converting the organic substance, to which water is added, into a simpler, more soluble compound.

It should be noted that in all these changes there is no loss of matter, only a change in its form and characteristics. The fact that all the elements composing organic matter are found at one time or another, alone or in combination, as gases or volatile substances, however, does at times result in their loss from sewage to the atmosphere.

Mineral Matters in Sewage.—The mineral, or inorganic, matter of sewage is usually of much less consequence than the organic matter. Particularly is this true for domestic sewage. The fertilizing qualities, however, are due to the former class of substances, such as nitrogen, potash and phosphates, although these are combined to a greater or less extent with other elements in organic compounds. Sand, gravel and other mineral matters washed by storm water into sewers and thence into ponds, rivers or harbors, may form deposits tending to obstruct navigation. Lime and certain other chemical substances from industrial wastes are inimical to fish and acid iron liquors from galvanizing plants may cause the water into which sewage containing them is turned to become discolored and unsuitable for many industrial purposes. These conditions on the whole are exceptional. The chief dangers and troubles caused by sewage are due to its organic content.

Common salt, or sodium chloride, together with smaller quantities of the chlorides of potassium, calcium and magnesium, are present in practically all natural waters, usually being more plentiful in waters near the seacoast than in those inland. These chlorides are also constituents of food and are present in large quantities in kitchen refuse, wash waters, urine and feces. Chlorides, therefore, are found in sewage in much larger quantities than in most natural waters and their presence in the latter in excess of that normal for unpolluted waters of the locality indicates probable contamination by sewage.

Chlorides are not decomposed by the changes occurring in sewage, either natural or artificial, and for this reason their determination is often important, as indicating whether or not samples of effluent and sewage correspond; if they do, the quantity of chlorine should be substantially the same in both.

In a general way, the quantity of chlorine per inhabitant reaching the sewers in a given unit of time is uniform; therefore the chlorine content of the sewage may be used to determine its strength or concentration, by which is meant the proportion of sewage matter in a given volume of water. For this determination 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. In a similar way the quantity of infiltration into the sewers may sometimes be ascertained by the degree of dilution afforded the sewage, the chlorine content of the ground water being known. Effluents from irrigation fields and intermittent sand filters often contain large quantities of ground or surface water, the proportion of which to sewage effluent may be ascertained by a study of the relation of the quantity of chlorine in the effluent to that in the sewage applied.

Some sewage contains alkalies such as ammonium, sodium and potassium carbonates and bicarbonates and similar salts of the alkaline earths, calcium and magnesium. The authors have seen samples of sewage so strong that fumes of ammonia rose from them into the air and samples in a barrel evolved so much ammonia that, when hydrochloric acid was exposed near the surface of the sewage, the upper portion of the barrel was filled with the characteristic white fumes of ammonium chloride.

Sewage is normally alkaline, owing to the alkalinity of the water supply, the ground-water infiltration and much of the sewage matter itself. Excessive alkalinity is inimical to both plant and animal life, so that this consideration may be an important one in connection with the discharge of certain industrial wastes into natural waters.

Sewage which contains mine drainage, wastes from wire-drawing plants and some other industrial wastes will often be found acid. The degree of acidity has an important bearing upon the inauguration and maintenance of bacterial activity within a sewage filter. Acid exerts an inhibiting action upon bacteria and it is possible to carry the acidity to the point where it will kill all bacterial life.

Iron in sewage may originate in the water supply, in the ground water filtering into the sewers and in industrial wastes, particularly the acid iron wastes of certain iron works. The effluents of sand filters sometimes contain large quantities of the compounds of this metal, a condition which indicates that the beds have tendencies toward putrefaction. Under such circumstances a reducing action occurs, rendering the ferruginous compounds in the sand soluble in water.

Nitrites and nitrates, usually those of the alkaline earth metals, occur in small quantities in fresh sewage, having their origin in the water supply. Septic sewage contains neither. Effluents from biological treatment processes contain these salts, the concentration varying with the strength of the sewage and the degree of oxidation. Nitrites and nitrates owe their formation to the oxidation of nitrogenous organic matter, in accordance with the operation of the nitrogen cycle.

Small quantities of phosphorus compounds are found in sewage and they constitute one of its elements of fertilizing value. Other fertilizing constituents, mineral in character, are the potassium salts, which are readily soluble in water and, for this reason, do not ordinarily exist in appreciable quantities in sludges. In connection with the disposal of some industrial wastes, such as wool-scouring liquors, potash may be of importance.

Certain inorganic substances may increase greatly the danger of odors. If sea water, which contains sulfates, finds its way into a sewer, particularly with bad ventilation and stale sewage, concrete surfaces may be injured by the formation of hydrogen sulfide. Thus for New Bedford, Mass., the authors recommended adding 2 in. to the thickness of the walls of a concrete intercepting sewer which was subject to tidal flooding, as a precaution in case disintegration should take place on the inner surface.

The mineral matter from industrial wastes may have a much more serious effect upon the treatment of the sewage than the mineral matter from the water supply and ground water, by interfering with bacterial activities. The diluting and neutralizing effect of municipal sewage upon acid and other industrial wastes, however, often makes it practicable to treat sewage containing large quantities of them by the usual bacterial methods. Sulfate of iron may cause large and troublesome sludge deposits, where chemical treatment is necessary, or it may furnish the necessary coagulant, thus proving advantageous. It may have an unfavorable effect upon filtration through sand beds. Tannery wastes may react with other constituents of the sewage, causing troublesome sludge deposits. Industrial wastes also frequently contain the objectionable sulfides already mentioned. Occasionally industrial wastes have contained such large quantities of arsenic, copper, chlorine or other germicide that the disinfecting action upon the organisms in filters has been unfavorable.

Gases and Volatile Matters in Sewage.—The common gases of the atmosphere, nitrogen, oxygen and carbon dioxide, are found in all waters that are exposed to the air. The quantities taken into solution are limited by the following factors: the coefficient of solubility of the gas in water; the partial pressure of the gas in the atmosphere; the temperature of the water; and the purity of the water.

Of these the first three are the most important, as far as fresh water is concerned. The *coefficient of solubility* is the quantity of gas that is absorbed by a unit quantity of water at a given temperature, when the water is exposed to a pure atmosphere of the gas under a barometric pressure of 760 mm. The *partial pressure* of a gas in the atmosphere is a proportion of atmospheric pressure equal to the percentage volume of that gas in the atmosphere. The quantity dissolved in the water varies directly with the partial pressure. The higher the temperature, however, the smaller is the quantity. The purer the water, the greater is its solution power. In sea water with a salinity of 18,000 p.p.m., as measured by the chloride content, the solubility of oxygen, for example, is reduced by about 20 per cent as against fresh water. This is of importance in the disposal of sewage into tidal estuaries.

Whipple (4) comments on these matters as follows:

> The solubility of oxygen in water at a temperature of 0°C. when the water is exposed to an atmosphere of the dry gas under a pressure of 760 mm., is 49.29 cc. per liter, of carbon dioxide 1713 cc. per liter, of nitrogen 23.00 cc. per liter. The atmosphere consists of 20.94 per cent by volume of oxygen, 0.03 per cent of carbon dioxide and 78.09 per cent of nitrogen, but in water saturated with air at 0°C. oxygen constitutes 34.91 per cent by volume of the total dissolved air and nitrogen only 65.09 per cent. Carbon dioxide disappears from the gaseous phase owing to its union with water to form carbonic acid. The saturation value for water at 0°C. in contact with air is about 1.4 parts CO_2 per million, or 0.7 cc. per liter. These volumes vary relatively from those which might be expected with the respective partial pressures by reason of the different *solubility constants* of the respective gases. If the air or gas in contact with the water is not dry, the volume of the gas dissolved will be reduced in proportion to the partial pressure of the water vapor present in the supernatant atmosphere, *i.e.*, the vapor tension of water.
>
> The *total solubility* of gases also varies in general with the temperature of the water, being greater in cold than in warm water. At 20°C. and with water exposed to a pressure of the dry gas equal to 760 mm. the total solubility of oxygen is 31.44 cc. per liter, of carbon dioxide 878 cc. and of nitrogen 15.54 cc. Compare these values with those given above for 0°C. When water is warmed, there is a tendency for gases to be driven from it due to lessened solubility. It is well to point out that although a rise in temperature decreases *total solubility*, it increases the *rate of solution coefficient*, *i.e.*, the rate of solution per unit area exposed when the difference between saturation and the actual concentration is 1 cc. per liter of water.
>
> Increasing *concentration of dissolved salts* decreases the total solubility from that in distilled water. For this reason brackish water and sea water contain, when saturated, smaller volumes of atmospheric gases than does fresh water under the same conditions.

The solubility of oxygen in water is given in Table 6, calculated from measurements made by Fox (10). Table 7, compiled by Whipple, (4) shows the solubility of CO_2 at different temperatures and illustrates the fact that the quantity of gas dissolved is dependent upon the partial pressure of CO_2 in the atmosphere above the solution. Where the partial pressure is high, as it is in the ground atmosphere, the quantity of gas dissolved in the water is larger. Well and other ground waters are commonly higher in carbon dioxide content than surface waters.

TABLE 6.—SOLUBILITY OF OXYGEN IN FRESH WATER AND IN SEA WATER OF STATED DEGREES OF SALINITY AT VARIOUS TEMPERATURES WHEN EXPOSED TO AN ATMOSPHERE CONTAINING 20.9 PER CENT OF OXYGEN UNDER A PRESSURE OF 760 MM., INCLUDING PRESSURE OF WATER VAPOR[1]

Temperature, °C.	Chloride as Cl in sea water, p.p.m.					Difference per 100 p.p.m. chloride, p.p.m.
	0	5,000	10,000	15,000	20,000	
	Dissolved oxygen, p.p.m.					
0	14.62	13.79	12.97	12.14	11.32	0.0165
1	14.23	13.41	12.61	11.82	11.03	0.0160
2	13.84	13.05	12.28	11.52	10.76	0.0154
3	13.48	12.72	11.98	11.24	10.50	0.0149
4	13.13	12.41	11.69	10.97	10.25	0.0144
5	12.80	12.09	11.39	10.70	10.01	0.0140
6	12.48	11.79	11.12	10.45	9.78	0.0135
7	12.17	11.51	10.85	10.21	9.57	0.0130
8	11.87	11.24	10.61	9.98	9.36	0.0125
9	11.59	10.97	10.36	9.76	9.17	0.0121
10	11.33	10.73	10.13	9.55	8.98	0.0118
11	11.08	10.49	9.92	9.35	8.80	0.0114
12	10.83	10.28	9.72	9.17	8.62	0.0110
13	10.60	10.05	9.52	8.98	8.46	0.0107
14	10.37	9.85	9.32	8.80	8.30	0.0104
15	10.15	9.65	9.14	8.63	8.14	0.0100
16	9.95	9.46	8.96	8.47	7.99	0.0098
17	9.74	9.26	8.78	8.30	7.84	0.0095
18	9.54	9.07	8.62	8.15	7.70	0.0092
19	9.35	8.89	8.45	8.00	7.56	0.0089
20	9.17	8.73	8.30	7.86	7.42	0.0088
21	8.99	8.57	8.14	7.71	7.28	0.0086
22	8.83	8.42	7.99	7.57	7.14	0.0084
23	8.68	8.27	7.85	7.43	7.00	0.0083
24	8.53	8.12	7.71	7.30	6.87	0.0083
25	8.38	7.96	7.56	7.15	6.74	0.0082
26	8.22	7.81	7.42	7.02	6.61	0.0080
27	8.07	7.67	7.28	6.88	6.49	0.0079
28	7.92	7.53	7.14	6.75	6.37	0.0078
29	7.77	7.39	7.00	6.62	6.25	0.0076
30	7.63	7.25	6.86	6.49	6.13	0.0075

[1] Under any other barometric pressure B the solubility can be obtained from the corresponding value in the table by the formula:

$$S' = S\frac{B}{760} = S\frac{B'}{29.92}, \text{ in which } S' = \text{solubility at } B \text{ or } B',$$

S = solubility at 760 mm. or 29.92 in.
B = barometric pressure in millimeters
B' = barometric pressure in inches.

TABLE 7.—SOLUBILITY OF CARBON DIOXIDE IN WATER

Temperature, °C.	Cc. per liter of water for partial pressure of 1 part per 10,000	Parts per million for stated partial pressures of CO_2 in the atmosphere			
		1 part per 10,000	4 parts per 10,000	6 parts per 10,000	8 parts per 10,000
0	0.1713	0.34	1.4	2.0	2.8
4	0.1473	0.29	1.2	1.7	2.4
8	0.1283	0.26	1.0	1.5	2.1
12	0.1117	0.22	0.9	1.3	1.8
16	0.0987	0.19	0.8	1.2	1.6
20	0.0877	0.17	0.7	1.0	1.4
24	0.0780	0.15	0.6	0.9	1.2
28	0.0780	0.15	0.6	0.9	1.2

When sewage in contact with the atmosphere is only partially saturated with oxygen or with some other gas found in the atmosphere above, it will absorb the gas from the air slowly, the rate of solution depending upon the surface area of the sewage exposed and upon the degree of undersaturation. When sewage contains more of a gas in solution than is consistent with the partial pressure of the gas in the atmosphere, the gas will go out of solution until balance is attained. This explains why carbon dioxide, which is commonly present in the atmosphere to the extent of only 0.04 per cent, corresponding to about 1 p.p.m. of CO_2 in the sewage, cannot be used as a measure of oxidized carbonaceous matter in sewage. Too much of it is lost to the atmosphere as soon as it is produced.

It is evident from the foregoing that most of the oxygen dissolved in sewage comes from the atmosphere. In water courses containing green plants, however, some of the oxygen may be contributed by photosynthesis. Under such conditions it is not unusual to find supersaturated values for oxygen, particularly if highly nitrified effluents have found their way into the watercourses.

Sewage and sewage effluents generally hold in solution not only gases absorbed from the atmosphere but also small quantities of other gases, together with a considerable quantity of carbon dioxide, that are produced as a result of the decomposition processes going on. Such decomposition may result in the formation of methane, commonly found in marshes as a result of the decay of bottom deposits and hence called marsh gas. This is a colorless, odorless, hydrocarbon compound, CH_4, of high fuel value. Nitrogen may be liberated in gaseous form by the decomposition of nitrogenous organic matter. Carbon dioxide may be produced by the breaking down of carbonaceous matter, resulting in a combination of the carbon with oxygen. Hydrogen sulfide, H_2S, is often formed in considerable quantities from the disintegration of sulfur-bearing organic matter or the reduction of mineral sulfites and sulfates. This gas is a colorless, inflammable compound, having the characteristic odor of rotten eggs. It is not formed in the presence of an abundant supply of oxygen. The blackening of sewage and sludge is usually due to the formation of hydrogen sulfide that has combined with iron present to form ferrous sulfide, FeS.

Fresh sewage possesses a somewhat disagreeable odor, probably due mainly to the characteristics of certain compounds contained in the sewage matters. It is not so objectionable as the odor of septic sewage.

The anaerobic decomposition of sewage, as explained above, may result in the production of hydrogen sulfide. This is the most important gas to be formed from the standpoint of the causation of odors. Other volatile compounds, such as indol, skatol, cadaverin and the mercaptans, may be formed in decomposing sewage in small quantities and by virtue of their strong physical properties may cause odors more offensive than that of hydrogen sulfide. There is little information at present as to their presence in sewage or the manner in which they may be produced by its decomposition. If sewage can be supplied with dissolved oxygen, its decomposition will not result in the production of the offensive odors so evident when undergoing anaerobic changes. Therefore, an effort is generally made to bring about the necessary changes in sewage under aerobic conditions.

The Hydrogen Ion Concentration of Sewage and Sewage Matters.—According to the theory of electrolytic dissociation, all aqueous solutions contain free *hydrogen ions*, H^+, and free *hydroxyl ions*, OH^-. These are derived by dissociation of molecules. When the number of each is the same, the solution is neutral. When the number of H ions is greater than that of the OH ions, the solution is acid. When the reverse is true, with OH ions in excess, the solution is alkaline. Since acidity is due to the presence of H ions, the acidity increases as the number of H ions increases. Conversely, since alkalinity is due to the presence of OH ions, the alkalinity increases as the number of OH ions increases. A strong acid is one that dissociates in water to give a large number of H ions that are positively charged, together with negatively charged ions of the acid radical.

Thus hydrochloric acid dissociates as follows:

$$HCl \leftrightharpoons H^+ + Cl^-$$

A strong base, similarly, is one that dissociates to give a large number of negatively charged OH ions, together with positively charged metal ions. Sodium hydroxide dissociates thus:

$$NaOH \leftrightarrows Na^+ + OH^-$$

Pure distilled water contains, besides molecules of H_2O, free hydrogen ions and free hydroxyl ions by reason of the dissociation of the molecules. $H_2O \leftrightarrows H^+ + OH^-$. This dissociation is very slight, affecting only one molecule in 550,000,000, but by virtue of it the water exhibits conductivity in slight degree to the passage of an electric current. According to the mass law,

$$\frac{\textit{Concentration of H ions} \times \textit{concentration of OH ions}}{\textit{Concentration of undissociated } H_2O} = \textit{a constant.}$$

In view of the fact that the proportion of undissociated water is extremely large, approaching unity, relative to the quantities of H or OH ions, the denominator can be taken as a constant, and then

$$\textit{Concentration of H ions} \times \textit{concentration of OH ions} = \textit{a constant.}$$

The constant has been measured electrically and found to be 10^{-14}, or 1/100,000,000,000,000, at 22°C. The number of H ions and OH ions in perfectly pure water is the same; therefore, the concentration of H ions and of OH ions will be 10^{-7}, or 1/10,000,000, *e.g.*, 1 gm. in 10,000,000 liters of water, in order to satisfy the equation above. This is the value for a neutral solution. An acid solution has more H ions, say a concentration of 10^{-5}, or 1/100,000, and a correspondingly lower number of OH ions, 10^{-9}. An alkaline solution has a greater number of OH ions, say 10^{-6}, and a correspondingly smaller number of H ions, which would be 10^{-8}. Thus, whatever the value of OH ions, there will still be some H ions in solution. Hence reference is commonly made to the H ion concentration alone, whether the solution be acid or alkaline, instead of both the H ion and OH ion concentration or the OH ion concentration alone.

For convenience of statement the H ion concentration is expressed in terms of the exponent that measures the magnitude of the decimal fraction. This is, therefore, the logarithm of the reciprocal of the H ion concentration. The term, as suggested by Sörenson, is called the "pH value"; hence $pH = \log \frac{1}{H^+}$. The unit of pH expression is grams of H per liter, as it is in stating the value of normal solutions of acids or bases. The pH value is, then, the logarithm of the reciprocal of the H ion concentration expressed in grams per liter. It should be remembered that pH 7.0 denotes neutrality, while pH values above 7.0 signify alkalinity and pH values below 7.0 acidity, and that increasing H ion concentration, therefore, is associated with decreasing pH values and vice versa. In considering pH values it is well to keep in mind, also, that large differences are denoted by small numbers. Thus a difference of 1 in the pH value signifies 10 times the H ion concentration and a difference of 2, 100 times the H ion concentration. Measurements of pH are sufficiently accurate to give values to more than one significant figure; two are commonly employed and three or four in very fine measurements.

Formerly the acid or basic properties of water and sewage were determined by finding the quantities of standard acid or alkali necessary to produce color changes in certain indicators, such as methyl orange or phenolphthalein, that are less sensitive than the ones now used. Accordingly the departure of the reaction of the solution from true neutrality was not so accurately registered. The method is known as *titration* and a solution was spoken of as alkaline to methyl orange or acid to phenolphthalein.

In sewage there occur certain substances known as *buffers*. When acid or alkaline compounds are added to sewage, these oppose or offset dissociation, so that in their presence considerable amounts of acid or alkali can be added without changing the pH. This is called *buffer action* and, as will be seen, is an important characteristic of sewage and sewage matters. The determination of hydrogen ion concentration is particularly significant in connection with three problems: the life processes of bacteria that decompose sewage matters; the coagulation and precipitation of suspended and colloidal matter; and the dewatering of sewage sludge. The activities of the microorganisms of decay are stimulated or retarded by changes in pH. There is an optimum value, and artificial adjustment to this value is sometimes worth while. Coagulation and precipitation of suspended and colloidal solids take place most rapidly at certain pH values and are therefore aided by bringing the liquid to these values, by the addition of acids or alkalies, or of acid or alkaline salts. A similar statement may be made for the dewatering of sewage sludge.

Living Things in Sewage.—Among the living things that play a part in the disposal of sewage are rats, the scavengers of sewers, in which many rats live; gulls and other birds that feed upon floating organic matter discharged from sewers; fish that often congregate about sewer outlets for food; certain aquatic plants that assimilate some of the end products of sewage decomposition; and minute living things, plant and animal microorganisms, too small as individuals to be seen with the naked eye. The last, the lowest organized forms of life, play the most important role. By common acceptance these microorganisms array themselves in two groups, the *bacteria*[1] and the *plankton*[2]

[1] Singular *bacterium*.

[2] The term *plankton*, denoting an assemblage of organisms, commonly is used in the singular. The individual organisms composing the plankton are known as *planktons* or, perhaps better, *plankter*.

or *microscopic organisms*.[1] The terms bacterium and plankton are derived from Greek words signifying "a staff" and "wandering," respectively, the first bacteria noted having been staff- or rod-shaped organisms and the first microorganisms, other than bacteria, having been observed as a free-floating, or wandering, assemblage of minute living things. Of the two, the bacteria are the more active.

These living organisms are of interest particularly from the standpoint of their food habits, which are largely responsible for the changes that are brought about in the so-called biologically activated treatment processes and in the self-purification of streams.

The position in the scale of life of these minute forms, together with some others that have a bearing on the disposal of sewage and some that do not, is given in the outline that follows, taken from Whipple (4). This outline includes both animals and plants, arranged in order from the more complex to the simplest forms.

Outline of Plant Groups

Spermatophyta or seed plants
Pteridophyta or ferns
Lycopsida or club mosses
Bryophyta or mosses and liver worts
Thallophyta or thallus plants
- **Fungi**
- **Characeae**
- **Algae**
  - *Chlorophyceae* or green algae
  - *Xanthophyceae* or yellow-green algae
  - *Diatomaceae* or diatoms
  - *Phaeophyceae* or brown algae
  - *Rhodophyceae* or red algae

Schizophyta or fission plants
- **Schizophyceae** or fission algae, **Cyanophyceae** or blue-green algae
- **Schizomycetes** or fission fungi, including the bacteria

The **Schizophyceae** or **Cyanophyceae** are more commonly classed with the algae and the **Schizomycetes** with the fungi.

[1] Sedgwick suggested that the microorganisms be divided according to a scheme that is given by Whipple (4):

Microorganisms Organisms, either plants or animals, invisible or barely visible to the naked eye.	*Microscopic Organisms or Plankton* Not requiring special culture. Easily studied with the microscope. Microscopic in size, or slightly larger. Plants or animals. *Bacterial Organisms* Requiring special cultures. Difficultly studied with the microscope. Microscopic or submicroscopic in size. Plants.

OUTLINE OF ANIMAL GROUPS

VERTEBRATA including the *Pisces* or fish and the *Amphibia*
MOLLUSCA or mollusks
BRYOZOA (*Polyzoa*) or moss animalcules
ARTHROPODA including the *Insecta*, *Arachnida* or water spiders, mites and bears, and the *Crustacea*
VERMES including the *Coelhelminthes* or segmented worms, *Trochhelminthes*[1] or trochal worms, *Nemathelminthes* or roundworms, and the *Platyhelminthes* or flatworms
HYDROZOA or polyps and medusae
PORIFERA or sponges
PROTOZOA or single-celled animals
Sarcodina or amoeboid protozoa
Mastigophora or flagellate protozoa
Infusoria or ciliate protozoa
Sporozoa, exclusively parasitic

In the outline of plant groups the descent in the scale of life is from the seed plants, *Spermatophyta*, to the thallus plants, a group that is not differentiated into root, stem or leaf. There are three subdivisions of this last group: the *Fungi*, of which many genera are found in sewage and sewage-polluted waters, and which are devoid of chlorophyll; the *Characeae*, water plants that occupy an intermediate position between the algae and the higher plants; and the *Algae*, chlorophyll-bearing organisms that mostly live entirely submerged in water. They are primitive plants capable of producing their own carbohydrate supply by photosynthesis. The bulk of the plankton population is usually made up of the algae. They thrive in sewage-polluted waters that have largely recovered their stability by oxidation processes.

Finally, there are the *Schizophyta*, a group of plants that multiplies by the process of cell division, or fission. This group includes the *Cyanophyceae*, chlorophyllaceous plants often placed in schemes of classification with the *Algae*, and the *Schizomycetes* or bacteria. The science of bacteriology concerns itself with this last class, so many representatives of which are found in sewage.

The outline of animal groups starts with the *Vertebrata* under which are placed the fish. Lower in the scale of life are the *Arthropoda* and the *Vermes*, members of which groups have important bearing on the self-purification of streams. Finally come the *Protozoa*, the simplest of animal forms. It is difficult to differentiate between these unicellular animal forms and the unicellular plants. Definitions may be found applicable to the higher types of life, but they become obscure when applied to the lowest forms. Two of the subdivisions under the *Protozoa*, namely, the flagellates and the ciliates, are common inhabitants of the

[1] The *Rotifera* or wheel animalcules are the largest group of the *Trochhelminthes*.

plankton assemblage. They are apt to be numerous in waters that have partially recovered from heavy sewage pollution.

The Bacteria.—Bacteria vary greatly in size. They are seldom more than 2 or 3μ[1] in their greatest dimension and usually about 0.5μ in diameter. Bacteria have a considerable variety of shapes but all are modifications of the three fundamental shapes, spheres, rods and spirals. These shapes give rise to three divisions, or families, the *cocci*, *bacilli* and *spirilla*. The cocci are nonmotile; the bacilli and spirilla are either motile or nonmotile.

The living substance of the bacterial cell is surrounded by a gelatinous envelope called a *capsule*. The *cell wall* is a slightly altered layer of the *cytoplasm* or living matter, that preserves the form and functions of the cell itself. The cytoplasm, which is made up of highly nitrogenous substances, can assimilate nourishment from the food reaching it in a way either to obtain vital energy or to form new cell wall and more cytoplasm, both of which result in growth. The cell wall permits transfusion of soluble substances but keeps out solid particles. Since much bacterial food is in solid form, the organisms must dissolve this food outside the cell. This they do by substances called *ferments* or *enzymes*, carried and secreted by the cell.

During disintegration of food substances by bacteria, acids and other products injurious to them are commonly formed. These may accumulate until further multiplication is stopped. Checks to bacterial growth are also offered by an insufficient food supply, unsuitable temperature, competition of different species of bacteria, ingestion by other organisms, such as the bacteriverous protozoa, and the presence of certain poisonous chemicals, such as chlorine compounds, acid or caustic substances, copper salts and arsenic. In an unfavorable environment some bacteria, assuming that the cells are not actually destroyed, have the ability to form spores, a resting, more resistant stage, analogous to the hibernation or estivation[2] encountered among higher forms of life.

The growth and activities of bacteria are dependent upon the consumption of food, as is the case with every other form of life. According to their food habits, bacteria are classified as *saprophytes* and *parasites*. The former are of greatest importance in sewage treatment and disposal; the latter are significant from the hygienic standpoint. The organic and mineral matter in sewage usually affords suitable food for the development of saprophytic bacteria. Some parasitic organisms can live also on dead organic matter. Most parasitic organisms, however,

[1] For convenience, the dimensions of small particles are given in microns for which the abbreviation is the Greek letter mu—μ. One micron equals one thousandth of a millimeter. The millimicron, mu mu, which is one thousandth of a micron or one millionth of a millimeter, is also useful. Since 1 mm. is about $\frac{1}{25}$ in., $1\mu = 1/25{,}000$ in. and $1\mu\mu = 1/25{,}000{,}000$ in. (approx.)

[2] Summer torpor.

do not develop to any extent outside the body of their host. The disease-producing bacteria die off quite rapidly in sewage.

Bacteria, like all other living organisms, need oxygen to carry on their life processes. Life, indeed, is a process of combustion. Some bacterial organisms require free oxygen and obtain it from the oxygen dissolved in the water. These are called *aerobic*[1] bacteria. Others can obtain their oxygen supply from the oxygen radicals of organic compounds or such mineral substances as nitrites, nitrates and sulfates. These are called *anaerobic* bacteria. Many bacteria are neither strict aerobes nor strict anaerobes and are said to be *facultatively* aerobic or anaerobic. This relation to oxygen is shared by other primitive forms of life, such as the protozoa and rotifera. The higher the scale of life, the more strictly it becomes aerobic. Classification of bacteria with respect to oxygen conditions is particularly important in sewage problems. The tremendous development of bacteria and other organisms in sewage often depletes the dissolved oxygen and establishes so-called anaerobic or septic conditions.

An important characteristic of bacterial growth is that the cells are sensitive to the reaction of their environment, *i.e.*, its acidity or alkalinity, as measured by the H ion concentration. A favorable reaction will promote growth, an unfavorable one will arrest it or even destroy the organisms completely. Bacteria are likewise sensitive to the temperature of their environment. They are destroyed at high temperatures and their activities are inhibited at low ones. Each species has its optimum temperature. For some it is relatively high and for others relatively low.

Bacteria find their way into sewage through many channels. They are present in the water supply of the community as well as in the ground water leaching into the sewers. Some doubtless fall into the sewage from the air. The wastes of certain industries contain large numbers of bacteria. Many more are washed into sewers by storm water. By far the greatest number, however, come from human and animal excreta, which teem with these minute living cells.

The mixture of all kinds of discarded substances in sewage gives it a high bacterial content and a diversified bacterial flora. There are present saprophytic and parasitic forms, aerobic, anaerobic and facultative species, spore formers and nonspore formers, beneficent and pathogenic types. A cubic centimeter of sewage will contain from a few hundred thousand to several million bacteria, the number depending upon the temperature, the age of the sewage, the character of the community producing it and the hour of the day.

Plankton and Higher Organisms.—The plankton by virtue of their food habits bring about changes in sewage matters which are second in

[1] From the Latin and Greek words for "air" and "life."

importance only to those induced by the activities of the bacteria. The changes are most apparent in natural bodies of water into which sewage is discharged. When the sewage is in concentrated form, as it is in sewage filters, the action is principally due to the bacteria, although not wholly so. In polluted natural waters the changes wrought are due to the life processes, not only of the bacteria but also of the higher organisms. The nature of the substances in the water determines very largely the predominant species of organisms and the changes wrought by them.

The role of the microorganisms in promoting the striking changes that are manifest subsequent to the pollution of water with sewage is discussed in greater detail in Chap. VII.

THE DECOMPOSITION OF SEWAGE

Changes Taking Place in Sewage Matters.—Transformations of a striking nature take place in the physical, chemical and biological properties of sewage as time elapses. Large coarse particles tend to be comminuted and some finely divided, colloidal particles agglomerate into flocculent masses. There are manifest both dispersion and coagulation of solid matters. The ultimate tendency is toward clarification that results from precipitation of such substances.

Biologically, the picture is ever changing. Bacterial species that were once predominant find existence too difficult and give way to others that find the environment better suited to their needs. As purification goes forward, microscopic organisms and the larger forms of aquatic life constantly shift in variety as conditions change. There is a regular sequence of forms, as physical and chemical factors are altered.

Chemically, great differences appear in the nature and structure of the complex organic substances that make up part of the solid content of sewage. Organic matter is more or less readily broken up into other and simpler compounds, usually by biochemical action. For example, the extremely complex nitrogenous compounds known as proteids, when attacked by the bacteria through the agency of their enzymes, undergo hydrolysis and cleavage, meaning that water is absorbed and simpler compounds are split off. Amino acids are formed by this process, compounds containing the group NH_2. The amino acids yield further ammonia and carbon dioxide, if oxygen is present; when it is absent the products may be several, as ammonia, organic acids, carbon dioxide, methane, hydrogen or even elemental nitrogen. These substances differ greatly from each other in their properties and are remote from the original proteid substance in both properties and chemical structure. This, too, is the case with the carbohydrates and the fats. Biochemical forces act upon them to produce new and quite different substances.

Some of the chemical changes to which sewage matters are subject result in the *liquefaction*, or change from an insoluble to a soluble substance, and *gasification* of solid particles, thus reducing the content of suspended solids. Certain species of bacteria, through their enzymes, possess marked ability to liquefy solid material. Nitrification is another change in chemical nature that nitrogen undergoes. With the aid of nitrite- and nitrate-forming bacteria, ammonium compounds are ultimately converted to nitrates. The reverse process, denitrification, is also encountered at times, whereby nitrates are reduced to nitrites, ammonia and even elemental nitrogen.

Biological action, under conditions in which oxygen is absent, involves chemical changes in organic matter that lead to the formation of compounds possessing offensive odors. The process is known as *putrefaction*. In contrast to this is the process of *oxidation*, which goes forward in the presence of ample oxygen supply. Under such conditions there are formed stable organic matter and mineral matter, both of which are free from offensive odors and, under ordinary conditions of nature, will not undergo further changes that lead to the production of bad odors. The problem of sewage treatment is primarily one of devising means and methods of utilizing the above mentioned forces most effectively and economically, in order to convert speedily the putrescible organic matter into stable organic and mineral matter without creating offensive conditions. It is to the metabolism of minute organisms, bacteria and plankton, that the transformation of obnoxious sewage to an unobjectionable liquid is largely due.

Indestructibility of Matter.—However striking the transformations in mutable substances may be, the student of sewage-disposal problems must always keep in mind the fundamental fact that matter is indestructible, that no form of treatment, natural or artificial, will destroy it, regardless of how great may be the changes in its form and characteristics. There is at times a loss of volatile constituents from sewage to the atmosphere, but the sum total of the end products of decomposition is always equal to the substances from which they are derived, pound for pound. In the transformations that substances undergo, they may be changed in physical form, as sugar and salt are changed by solution in water, or they may be changed in chemical structure, as when several new substances are formed with entirely different properties. It follows, therefore, that the objectionable characteristics of sewage must be overcome, either by removing the matter causing them and disposing of it apart from the sewage, by transforming the objectionable into unobjectionable matter, or, in the case of living organisms, by killing them directly or indirectly.

Aerobic and Anaerobic Decomposition.—The complexity of the reactions involved in the decomposition of sewage makes impossible of statement, in the light of existing information, the exact nature of the changes that go on. With respect to oxygen two types of decomposition are recognized, anaerobic decomposition, or putrefaction, and aerobic decomposition, or oxidation. Both are due to the activities of organisms, among which the bacteria predominate. Biologically, the two types differ greatly in the genera that are at work, because the oxygen concentration is not the same in one as in the other. Anaerobic decomposition is brought about by organisms that live and work in the relative absence of oxygen; aerobic decomposition by those that require free oxygen to promote their activities. It should be remembered that all species of bacteria require the element oxygen for energy purposes. If free oxygen is not available in sewage, species will develop that can utilize the oxygen in nitrites, nitrates and sulfates and that in organic combination.

Metabolism varies with species and so it is found that putrefaction produces end products different from those of oxidation. The relative absence of oxygen gives rise to the formation of such compounds as hydrogen sulfide, H_2S, methane, CH_4, ammonia, NH_3, hydrogen, H, nitrogen, N, carbon dioxide, CO_2, and possibly to intermediate compounds like indol, C_8H_7N, skatol, C_9H_9N, mercaptan, C_6H_5SH, and cadaverin, $C_5H_{14}N_2$. Many of these substances are volatile and ill-smelling. They give to sewage that is subjected to putrefaction its offensive properties. If sewage can be supplied with dissolved oxygen, its decomposition will not result in the production of the offensive odors so evident when it undergoes anaerobic changes. Decomposition under an ample supply of oxygen yields end products like carbon dioxide, nitrates, NO_3, and sulfates, SO_4. These compounds are stable in composition and with inoffensive properties.

The oxidation of sewage does not imply the complete mineralization of all organic matters. A large proportion, particularly of the suspended solids, is converted into humus-like organic compounds that are relatively stable in character. Oxidation goes on to some extent during anaerobic decomposition, but it is usual to consider that aerobic conditions are essential to oxidation that accomplishes mineralization. That one form of the latter, nitrification, is due to bacterial action is apparent from the fact that it is checked or stopped entirely by low temperatures, by heating to the point of sterilization and by certain chemicals inimical to the life of bacteria. Furthermore, temperatures favorable to bacterial development cause nitrification to proceed rapidly.

In sewage farming and intermittent filtration through beds of sand and other fine materials, the production of high nitrates is an evidence of oxidation practically as complete as can be secured. Where sewage is applied to beds of very coarse materials and conditions do not closely simulate those of the natural soil, oxidation is not so complete and nitrates are not found in such large quantities.

Throughout the changes incident to putrefaction and oxidation there are manifest both *symbiosis*, or coöperation, and *antibiosis*, or antagonism. Sometimes the products of cell activity from one species serve as nutritive substances for the growth and reproduction of another species. Thus there is an interlocking of activity. Again, the products of one kind of cell may inhibit the growth of another, making life for the latter difficult or impossible.

The decomposition of sewage is generally the result of both putrefactive and oxidizing processes. Putrefaction may be the first stage, oxidation the second. Putrefaction does not necessarily imply total absence of free oxygen. It may proceed in the presence of free oxygen inside the masses of sewage matter without, however, giving rise to

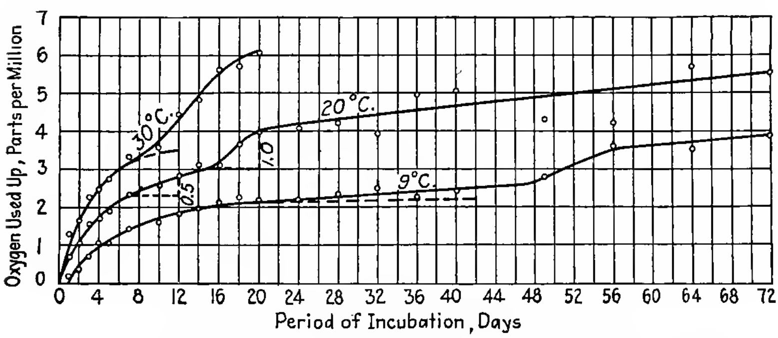


FIG. 3.—Oxygen requirements of fresh sewage.

objectionable conditions. Strictly anaerobic conditions are only found when long storage of sewage or sewage matters as in so-called septic tanks or sludge-digestion tanks depletes the dissolved-oxygen content of the sewage, or when streams or other bodies of water are overloaded with sewage. Septic conditions are often produced, however, in long outfall sewers and parts of the sewerage system in which steady or self-cleaning flow is not maintained. It is important to remember that as long as any oxygen remains in solution, even though putrefaction is in progress, decomposition will not give rise ordinarily to offensive conditions.

The Oxygen Requirements of Decomposing Sewage.—The oxygen requirements of fresh decomposing sewage, as measured by the biochemical oxygen demand, or B.O.D., test, are illustrated in Fig. 3. As shown there are two stages. At 20°C., for example, the first stage extends to about the sixteenth day and is characterized by a gradual falling off in the quantity of oxygen used up in equal time intervals.

A transition is then made to the second stage, during which transition the rate of oxygen consumption increases until about the twentieth day, after which the rate, although lower than during the transition stage, continues to be fairly high and constant for a protracted period of time. The second stage is known as the *nitrification* stage, because, during it, oxidation of the nitrogenous organic matter to nitrates takes place. At higher temperatures, the first stage is shortened in time; at lower ones, it becomes longer.

Mathematical studies of the deoxygenation curve show that during the first stage the consumption of oxygen is proportional to the oxygen requirement of the sample at any time, *i.e.*, to the amount of the undecomposed organic matter remaining.

This can be formulated as follows:

If L = B.O.D. in p.p.m. exerted by organic matter during first stage, and

L_t = B.O.D. in p.p.m. remaining after time t

then $L - L_t = X_t$ = oxygen in p.p.m. used up in t days, as determined by the B.O.D. test

$$\frac{-\dfrac{dL_t}{L_t}}{dt} = K', \text{ where } K' \text{ is a constant}$$

$$-\int_L^{L_t} \frac{dL_t}{L_t} = K' \int_0^t dt$$

$$\log \frac{L_t}{L} = \log \frac{L - X_t}{L} = -K_1 t,$$

where $K_1 = 0.4343K'$ = the deoxygenation constant

$$\frac{L_t}{L} = \frac{L - X_t}{L} = 10^{-K_1 t}$$

$$X_t = L - L_t = L(1 - 10^{-K_1 t})$$

Chemists call this a *unimolecular reaction*, because the velocity of the reaction is controlled only by the concentration of the oxygen-requiring substances, the amount of oxygen present being sufficient to oxidize these substances as soon as they are freed. The formula developed by calculus is merely a variant of the expression for compound interest,

$$L_t = L(1 + r)^t$$

where L_t = capital after time t

L = original capital

r = rate of interest

The B.O.D. decreases at a uniform rate, the decrease being determined in a unit of time by the B.O.D. remaining to be satisfied after each unit of time, just as money increases at a uniform rate, the increase in a unit of time being determined by the capital accumulated to draw interest after each unit of time. Since a decrease and not an increase takes place,

$$L_t = \frac{L}{(1 + r)^t} = L(1 + r)^{-t}$$

$$\log \frac{L_t}{L} = -t \log (1 + r) = -K_1 t$$

The deoxygenation constant K_1, it has been determined, is 0.100 at 20°C. The rate of deoxygenation naturally changes with temperature, as do all biologically activated processes. From considerations of physical chemistry and actual test, the deoxygenation constant K_1 at any temperature T, $K_{1(T)} = K_{1(20)}[1.047T^{-20}]$. Temperature, furthermore, affects the magnitude of the first-stage B.O.D. and the first-stage demand L at any temperature T has been found to bear the following relation to the first-stage demand at 20°C.: $L_T = L_{20}[1 + 0.02(T - 20)]$.

Example.—An example will illustrate the use of these equations. Let it be required to find the 1-day, 37°C. B.O.D. of a sewage whose 5-day, 20°C. B.O.D. is 100 p.p.m. The first stage, 20°C. B.O.D. is found as follows:

Since $\log \frac{L - X_t}{L} = -K_1 t$,

$$\log \frac{L - 100}{L} = -0.100 \times 5 = 0.500 - 1$$

$$\frac{L - 100}{L} = 0.316$$

$$L = \frac{100}{0.684} = 148 \text{ p.p.m.}$$

This result could have been found also from Table 8,

$$L = \frac{100}{0.68} = 148 \text{ p.p.m.}$$

The first stage, 37°C. B.O.D. is therefore:

$$L_{37} = L_{20}[1 + 0.02(T - 20)] = 148[1 + 0.02(37 - 20)] = 198 \text{ p.p.m.}$$

and since

$$K_{1(37)} = K_{1(20)}(1.047^{T-20}) = 0.100(1.047^{17}) = 0.218$$

$$\log \frac{198 - X_t}{198} = -0.218 \times 1 = 0.782 - 1$$

$\frac{198 - X_t}{198} = 0.605$ and the 1-day, 37°C., B.O.D., $X_t = 78$ p.p.m.

If the oxygen present in a sample of sewage is used up in t days by biochemical activity, the B.O.D. in parts per million of the sample X_t must be equal to the oxygen originally available. The percentage ratio of the oxygen available, X_t, to the oxygen required during the first stage, L, is then called the relative stability, S, of the sewage.

$$\text{At } 20°\text{C.}, S = \frac{X_t}{L}100 = 100(1 - 10^{-0.1t}) = 100(1 - 0.794^t).$$

This relationship is the basis of what is called *relative stability* and is employed chiefly in connection with the methylene blue test. If $t = 10$, for example, $S = 100(1 - 10^{-1}) = 90$, and the relative stability of a sample of sewage that does not decolorize methylene blue at 20°C. until 10 days have elapsed is said to be 90 per cent. The relative-stability numbers obtained in this way for a temperature of 20°C. are shown in Table 8, from "Standard Methods" (10).

TABLE 8.—RELATIVE-STABILITY NUMBERS

Time required for decolorization at 20°C., days	Relative stability, per cent	Time required for decolorization at 20°C., days	Relative stability, per cent
0.5	11	8.0	84
1.0	21	9.0	87
1.5	30	10.0	90
2.0	37	11.0	92
2.5	44	12.0	94
3.0	50	13.0	95
4.0	60	14.0	96
5.0	68	16.0	97
6.0	75	18.0	98
7.0	80	20.0	99

It is evident from the shape of the deoxygenation curve that the equations just discussed and the relative-stability table can be used in computing the B.O.D. for a given period from the 5-day or 10-day B.O.D. tests only when decomposition of the sewages examined lies within the period of the first stage. Use of the relative-stability numbers in Table 8, it must be remembered also, is restricted to 20°C. tests.

Constituents and Properties of Sewage Sludge.—Fresh sludge is made up of the settling solids from raw sewage or from sewage that has received some form of treatment as, for example, the effluent of trickling filters or the aerated sewage from the activated-sludge process. The nature of the materials found in the settling solids already has been discussed. The character of sludge, as regards appearance, putrescibility, digestibility and air drying qualities, varies greatly with the type and time of treatment that produces it. The differences are discussed in a subsequent chapter.

The volume of sludge produced from sewage is relatively large, owing to the great content of water, which normally amounts to 85 to 98 per cent by weight of the material. The proportion of water in sludge depends upon the method of separation and digestion to which it has been subjected. The same may be said of the completeness with which the excess water may be removed by drying processes. In practice it is desirable to carry as low a content of water as possible, both before and after drying, with the qualification, of course, that the sludge before drying should contain enough water to permit it to flow.

The organic content of sludge is high. There is less in digested sludges than in those from septic and plain sedimentation tanks. Organic matter in sewage after being deposited by sedimentation, has a considerable tendency to go slowly into solution, by virtue of the presence in its structure of certain substances that are soluble and which, in going into solution, exercise a dispersive effect upon the solid organic matter. Buswell (5) comments on the behavior and changes in organic matter in sludge deposited by sedimentation as follows:

> The animal and vegetable matter of which it is composed consists largely of organic compounds containing either hydroxyl, carboxyl or amine groups or all three. These groups are soluble in water and act as solution links tending to favor the dispersion of the compounds in water. Many of these compounds are characteristically sticky or gluey, tending to take up water, first swelling to gels and then dispersing to sols. If solution, or perhaps more properly dispersion, of these solids is allowed to take place in such a manner that the liquefied products are returned to the main body of the sewage liquor, the reduction in oxygen demand accomplished by sedimentation will be proportionately offset. The solids are of such a nature that they cannot be removed and exposed in the open air without creating a nuisance. They are allowed to undergo a rotting process while submerged and in a compartment or tank separated from the sewage liquors. The object of this rotting . . . is to produce a relatively insoluble, inert, and stable organic residue of a compact granular texture which may be separated readily from the associated liquid by draining. This process is most often referred to as digestion but is sometimes called septicization.

Not all the organic matter in sludge is capable of digestion, even after a period of years. After sludge has undergone digestion, however, the resultant material is relatively inert and humus-like in its properties. In practice, the total solids in digested sludge may amount to 55 to 65 per cent by weight of the original solids in fresh sludge from plain sedimentation. The volume relations, however, are somewhat different. Buswell (5) states:

> The volume of the sludge is reduced in a much greater ratio than is the weight, due to the destruction of the water-binding colloidal character of

the organic matter. The volume reduction may frequently amount to 75 per cent.

Included in the organic content of sludge is a small percentage of "grease," a general term that includes saponifiable and hydrocarbon oils, fats from kitchen refuse and industrial wastes, and soaps. These materials are carried down from the sewage. They are particularly resistant to natural decomposition, tend to clog the pores of filters and are a detriment to the use of the sludge for fertilizing purposes, inasmuch as they interfere with the full utilization of fertilizing constituents and clog the interstices of the soil.

The fixed-solids content, or mineral matter, of fresh sludge constitutes about two fifths of the total material. It is made up largely of silica in the form of sand or combined as clay or rock material. In addition, there is a small quantity of aluminum, calcium, magnesium and sulfur compounds. Iron is another mineral constituent of sludge, occurring in the form of sulfide and hydroxide.

Among the more important constituents of sewage sludge are those that are responsible for whatever fertilizing properties it may possess. Chief among these are organic nitrogen and phosphorus compounds. Potash salts are soluble and so are not retained in material quantity. The organic nitrogen is not directly available for plant use but a portion of it becomes so, by virtue of the activity of bacteria in the soil which convert it to ammonium salts and nitrates. Phosphorus is in combination as phosphates of calcium, iron or aluminum, being present to the extent of 1 or 2 per cent, computed as P_2O_5. Most of the phosphates in sludge are only slightly soluble and so may be assimilated only slowly by plants. Further discussion of the fertilizing value of sludges is given in subsequent chapters.

Bibliography

1. Rept. Metrop. Sewerage Comm. of New York, 1910.
2. Phelps, E. B.: *Tech. Quart.*, 1905; **18**, 40.
3. *Pub. Health Bull.* 132, U. S. Pub. Health Service, 1923.
4. Whipple, G. C.: "Microscopy of Drinking Water," 4th ed., 1927.
5. Buswell, A. M.: "Chemistry of Water and Sewage Treatment," 1928.
6. Thom, Charles, and A. C. Hunter: "Hygienic Fundamentals of Food Handling," 1924.
7. Kinnicutt, L. P., C.-E. A. Winslow and R. W. Pratt: "Sewage Disposal," 2d ed., 1919.
8. Heuser, Emil: "Textbook of Cellulose Chemistry," trans. by C. J. West and G. J. Esselen, Jr., 1924.
9. Boruff, C. S., and A. M. Buswell: *Ind. Eng. Chem.*, 1929; **21**, 1181.
10. "Standard Methods for the Examination of Water and Sewage," Am. Pub. Health Assoc. and Am. Water Works Assoc., 7th ed., 1933.

CHAPTER IV

ELEMENTS OF SEWAGE ANALYSIS

Broadly speaking, it is the aim of sewage treatment to remove the suspended or readily settling portion of the sewage matters as an inoffensive or even marketable sludge and to oxidize the soluble and colloidal portion. It should be the aim of sewage analyses, therefore, to provide information upon which treatment methods can be based and from which the efficiency of performance can be gaged.

Significance of Analyses in Sewage Treatment and Disposal.—Removal of suspended matter is accomplished by *screening* or by *sedimentation*, the former collecting only the coarser particles, the latter those matters which will settle readily when the velocity of flow is checked. The *fresh solids* obtained, being in part organic in nature, remain offensive. Before they can be disposed of, therefore, it is commonly necessary that they be rendered stable. This is accomplished best by permitting them to decompose at the plant in *digestion tanks.* Design of screens and of sedimentation and digestion tanks, therefore, requires a knowledge of the quantities and nature of the suspended matters in sewage. It is necessary to know how much material probably will be removed by screening or sedimentation, what the screenings or sludge will be like and how much material must be handled, dried and finally disposed of.

Oxidation of soluble and colloidal matter is accomplished by *filtration* or *aeration*, which may remove and at the same time dispose of suspended and colloidal material not removed by sedimentation. Analyses of the sewage therefore must furnish information upon the load to be handled by these devices. This is gaged by the analyses for organic matter in the sewage and its oxygen demand. In the past, determinations of nitrogen and of carbonaceous matter have been used extensively for this purpose. In recent times the quantity and condition of the organic matter are being measured more directly by the *biochemical-oxygen-demand* test.

Destruction of organic matter in sewage treatment is brought about by the activities of hosts of living organisms, particularly bacteria. Although they are, therefore, of great importance, the identification and enumeration of these organisms seldom are attempted except in a rough way because of the difficulties involved. Sewage-treatment processes, however, are selected and controlled with a view to providing the desirable forms of life with environmental conditions in which their activities will be promoted. Among the analytical determinations that are of assistance are those of temperature and hydrogen ion concentration.

Performance of the various treatment processes is gaged by operating records that give the analyses of the untreated and treated sewage. The efficiency of each process is determined by suitable tests whose judicious selection is, therefore, an important matter.

Sewage, whether treated or not, ultimately must be disposed of either on land or into water. Most commonly, sewage is discharged into streams, lakes or tidal estuaries. Here the sewage undergoes the natural purification processes active in all waters, standing or flowing. The processes of self-purification are much like those that have been developed for the artificial cleaning of sewage in treatment works. The load of sewage which a given body of water can receive without the creation of offensive conditions is strictly limited.

In sewage disposal, the quantity and character of the suspended solids and the oxygen requirements are again the most significant information to be obtained. Knowing the character of the watercourse into which the sewage is discharged and the uses to which its waters are put, these tests may be employed to determine whether the sewage can be discharged in a raw state or whether it must be treated first and how far the treatment processes must be carried. It should be remembered that the real object of sewage treatment is to prepare the sewage for disposal. The use of many watercourses for sources of drinking-water supplies or for recreational purposes necessitates the use of most of the common tests germane to a water analysis. These, however, are performed on the water rather than on the sewage it receives.

In the operation of treatment and disposal works the information presented in sewage analyses will serve a fourfold purpose as follows:

1. To measure the efficiency of plant operation and of the various treatment processes employed.
2. To indicate modifications of operating methods for greater efficiency.
3. To provide information for the design of extensions to the plant or for the design of other similar plants.
4. To establish the character of the effluent from the plant, particularly in regard to stream pollution.

PHYSICAL AND CHEMICAL EXAMINATION

Collection of Samples.—Owing to the variable nature of sewage and industrial wastes, from the standpoint of locality as well as time of production, it is necessary to obtain samples, planned and taken intelligently, in order to derive from them a correct knowledge of the character of the sewage or wastes. The results of analysis of a single sample usually mean little. It is advisable to secure a series of samples and to be guided by the average results of analyses, giving due consideration to the variations. It is possible, however, to be misled by both average results and analyses of isolated samples, unless the conditions of sampling are known and appreciated.

Sampling of sewage, industrial wastes and effluents from treatment works commonly involves the collection of composite daily or sometimes weekly samples. Individual portions are taken at frequent intervals, half-hourly or hourly, and are mixed at the end of the sampling period or combined in a single container as collected. Whenever possible the individual portions should be combined in volumes proportionate to the rates of sewage flow. Proportioning is done conveniently by taking a simple multiple in cubic centimeters of the flow in million gallons daily or some other unit of flow. A suitable factor must be chosen to give the desired volume for the composite sample, which should not exceed 4 liters, according to "Standard Methods" (1).

Preservation of Samples.—Since sewage decomposes rapidly on standing for a comparatively short period of time at room temperature, it is important that the analysis be made as soon as possible after the sample is collected. Certain determinations, such as that of dissolved oxygen, should be made on the spot. The rate of decomposition may be lessened materially by keeping the samples at a low temperature, as in a refrigerator. Even at this temperature, however, changes will take place. When the analysis cannot be performed within 4 to 6 hr. after collection, a preservative must be added to the sample. Chloroform, formaldehyde or sulfuric acid is used for this purpose. Some sterilizing agents interfere with certain analytical determinations, so that sometimes it is found desirable to sterilize two different portions of the sample, each with a different germicide.

The quantity of sterilizing agent required depends upon the character of the sample. For strong sewage, it may be necessary to use the equivalent of 1 to 2 cc. of the concentrated chemical solution to each 500 cc. of the sample. For the determination of relative stability or biochemical oxygen demand, samples must be free from preservatives.

Mechanical Equipment for Sampling.—For the purposes of eliminating errors due to the human element, reducing the personnel required in taking samples and obtaining a more continuous and uniform sample than can be obtained by hand sampling, an automatic sampler has been designed and developed by the Sanitary District of Chicago. One of these devices, designed for use at the West Side works, is illustrated in Fig. 4.

The sampler consists of a bucket attached to a piston head on the end of a piston rod, operating vertically and driven through a crankpin and connecting rod by a horizontal-shaft motor and reducing gear, as well as a sampling compartment or weir box, all attached to a rigid structural-steel frame. The liquid enters the weir box on one side near the bottom and flows over a weir. The bucket picks up the sample below the surface at the bottom of the stroke. At the top of the stroke the bucket trips and spills into a gallon collecting-bottle through a funnel. The bucket is operated by a ⅓-hp., 1140-r.p.m. motor and 4000: 1 reducing gear, thereby giving one sample every 4 min., and is of such capacity as to fill a gallon bottle in 24 hr.

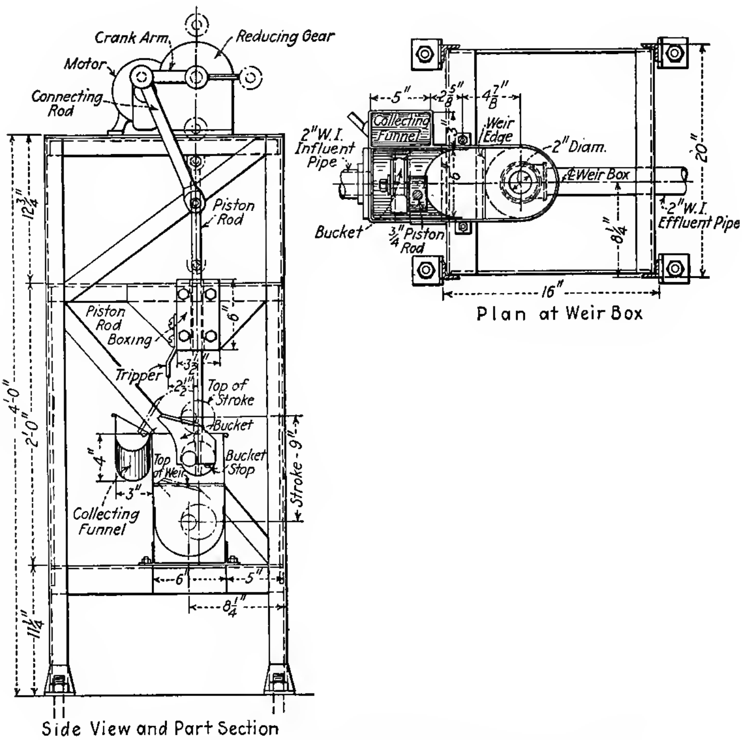


FIG. 4.—Automatic sampler, West Side treatment works, Chicago.

Expression of Analytical Results.—In reporting the results of chemical analyses of sewage, it is customary to employ for most determinations the units, *parts per million*. The exceptions to this general rule, as well as the method of expressing the results of physical tests, will be discussed in connection with the methods of making the individual tests. Formerly chemical results were expressed in *grains per gallon* or in *parts per* 100,000 and these methods of notation are still in use at some laboratories. Table 9 will be found convenient for converting results expressed in one way into those expressed in another.

Table 9.—Relations between Grains per Gallon, Parts per 100,000 and Parts per Million

Unit	Grains per U. S. gal.	Grains per Imperial gal.	Parts per 100,000	Parts per million
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 million.........	0.058	0.07	0.10	1.0

"Parts per million" really means the weight of the specified substance present in 1,000,000 parts by weight of the sewage examined. As a matter of practice, however, the weights of the substances found are so small, relatively, that the specific gravity of the liquid is neglected or, more accurately, is assumed to be equal to that of pure water, unity, and a measured volume of the liquid is used for analysis.

As indicating the efficiency of a treatment plant, it is often the practice to record the "percentage removed" of various constituents. This term may be applied accurately in the case of certain ingredients, as suspended matter like silt; if 1 part is removed from 1,000,000 parts of sewage and 3 parts remain, it is correct to state that 25 per cent of this suspended matter has been removed. If, however, instead of actually being removed, a substance merely has been altered in character, it may not be strictly accurate to refer to the changed condition in terms of "percentage removed." In order to avoid a misunderstanding it is preferable to use the term "percentage reduced."

Physical Tests.—As a part of the analysis of sewage and effluents, observations sometimes are made of the temperature, turbidity, suspended matter and color of samples. Usually such observations are not considered essential, although they may be helpful in special cases.

The temperature of raw sewage is sometimes recorded as it comes to a treatment plant, for this temperature may have an important bearing on the efficiency of various treatment processes. In collecting a sample for the determination of dissolved oxygen, the temperature of the sample must be taken, in order to find the percentage saturation, and commonly is recorded to the nearest degree of the centigrade scale. Likewise, the temperature of individual components of a composite sample may be noted at the time of collection.

In making turbidity observations of polluted waters it is not ordinarily necessary to go to the refinement required with potable water. It is generally sufficient to designate the turbidity as "slight," "distinct," "decided," "extreme," etc. Only in rare cases is it necessary to record the visible suspended matter and in such cases it should be reported as "slight," "distinct," or "decided."

The color of effluents or waters into which sewage and effluents are discharged is not ordinarily significant. In unusual cases, however, owing to the discharge of industrial wastes, it may be important to note the color in general terms.

Observations of the odor of water containing sewage or effluent, or of an effluent itself, may furnish valuable information, but this is not usually the case with sewage. Water in a stream may appear quite free from odor, when observed in the usual way, but if it is examined in a bottle, a very different impression may be gained.

For such an examination the sample should be shaken vigorously in a collecting bottle when it is one half to two thirds full, and the odor observed by quickly placing the nose to the mouth of the bottle. Some samples, which yield only a slight odor when cold, will give off a decided odor upon heating, owing to either of two causes: the odor may be distributed through the water in such a way that it is not expelled readily upon shaking but is driven off upon heating, or the water may contain organic matter, from which odoriferous compounds are distilled upon heating. Where odors are caused by gases dissolved in the sample, they are sometimes liberated so completely by shaking the sample that a second shaking will not produce further odor. Therefore, in cases of doubt, another sample should be tested. Where the observed odor is caused by organic substances, it usually will persist and may change in quantity and character as the sample is kept, owing either to multiplication of organisms, as in the case of surface waters, or to decay of such organisms or of other organic matter, as in the case of sewage.

The quality of an odor generally is expressed by a descriptive epithet, such as aromatic, disagreeable, earthy, fishy, grassy, musty, sweetish and vegetable. The intensity of an odor may be recorded by a numeral prefixed to the term expressing quality. "Standard Methods" (1) suggests the following numerical scale for intensity of odor: 0, none; 1, very faint; 2, faint; 3, distinct; 4, decided, and 5, very strong.

Determination of Solid Matters.—The quantity of solid matter in sewage is determined by evaporating a known volume of sewage and drying and weighing the residue. This residue is a measure of the *total solids* in the sewage. If this residue is ignited, *i.e.*, heated sufficiently to burn off the volatile matter, the weight of the remaining substances gives the so-called *fixed solids*, or approximately the mineral matter. The difference in the weights of the total solids and fixed solids is called the *loss on ignition* and is a measure of the *volatile solids*, or approximately of the organic matter. Ignition gives an idea of the quantity but not of the character of the organic matter present.

By filtering the sewage through filter paper or some other filtering medium before evaporation and ignition, it is possible to determine the quantities of fixed and volatile *dissolved solids* and *suspended solids* either from the filtrate or from the matter deposited on the filtering medium, or from both. This approach does not isolate the colloidal matter from the suspended and dissolved solids. Some of the colloidal solids will be reported as suspended solids and some as dissolved solids. The method of filtration always should be stated. There is at present no simple or standard test for colloidal matter and the term is used somewhat loosely as applying to finely divided matter that does not settle readily.

By *settling solids* is understood commonly the quantity of sludge or sediment, expressed in cubic centimeters per liter, in per cent by volume or in parts per million by weight, which will be deposited from a given quantity of sewage in a certain interval of time. The readiest method of approximately measuring the volume of settling solids is with the aid of an "Imhoff cone," a glass 4 in. in diameter at the top, 16 in. tall and tapering to the size of a thimble at the bottom. It is graduated in cubic centimeters at the bottom and holds 1 liter, as shown in Fig. 5. The cubic centimeters of sludge commonly are read after allowing the sample to stand for 2 hr. The method of settling and the time should be noted. If the weight of the settling solids is to be found, which is the only way to make an accurate determination, the supernatant liquid is decanted and its content of total solids determined. The settling solids are then obtained by computing the difference between the total solids content of the unsettled sample and that of the settled sample.

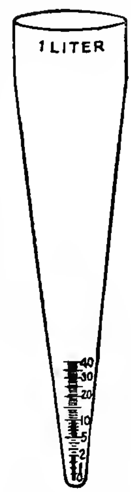


Fig. 5.—Imhoff cone.

At present no test is made to determine the quantity of floating matters in sewage, apart from the determination of oils and fats. This result will include some matter which will settle and some which will be dispersed throughout the liquid.

The physical condition of the principal constituents of sewage of medium strength and the average quantities of each are shown in Fig. 1.

Determination of Organic Matters.—The tests for organic matters in sewage shown in Table 3 are designed to give information on:

1. The quantity and physical character of organic matter. (Tests for volatile solids, total, suspended and dissolved.)

2. The quantity and character of nitrogenous matter. (Tests for albuminoid and total nitrogen, which commonly are considered together with the tests for nitrogen in mineral form, ammonia,[1] nitrite and nitrate nitrogen.)
3. The quantity and nature of carbonaceous matter. (Test for oxygen consumed.)
4. The oxygen avidity of the sewage. (Tests for biochemical oxygen demand and relative stability.)
5. The presence of fats and oils. (Test for ether-soluble matter.)

The tests for volatile solids or loss on ignition already have been discussed. They do not measure the nature of the organic matter present but only the state in which organic matters occur.

The nitrogenous matter in sewage is derived from urea and the proteins and their products of decomposition. *Organic nitrogen* is determined by the Kjeldahl method. The sewage is boiled with strong acid which liberates the nitrogen in the complex organic substances. This is converted into ammonia and determined as such. *Albuminoid nitrogen* is supposedly that part of the organic nitrogen which is broken down most readily. It is determined by boiling sewage with a chemical oxidizing agent in an alkaline solution and catching the ammonia distilled off. *Ammonia nitrogen,* formerly called "free ammonia," is driven off when sewage is boiled or distilled without the addition of chemicals. It can be determined also by adding suitable reagents to the cold sample, a method known as direct nesslerization. *Nitrite* and *nitrate* nitrogen commonly are found by adding to the sewage chemical reagents that will impart certain colors to it. These colors are compared with those produced by known quantities of nitrites and nitrates. Other methods are also available, such as the chemical reduction of these oxidized products to ammonia and the determination of the latter.

There is no reliable test for determination of the carbonaceous matter in sewage. If total carbon could be found readily it would be a valuable measure of total organic matter present. The test for *oxygen consumed,* although sometimes thought of as measuring carbonaceous matter, actually gives only the quantity of oxygen that sewage will absorb from potassium permanganate when boiled or heated with acid for a given length of time. It does not give the total oxygen needed for chemical oxidation of all organic matter. The time and temperature employed in this test should be stated.

Decomposition and mineralization of organic matter by bacterial activity are accompanied by depletion of the dissolved oxygen. The avidity of sewage for oxygen, therefore, reflects both the nature and quantity of the organic matter it contains. There are two tests that measure this characteristic. The first is known as the *biochemical*

[1] Includes both organic and inorganic substances.

oxygen demand, B.O.D., of the sewage and the second as its *relative stability*.

The *biochemical oxygen demand* commonly is ascertained by diluting the sewage with distilled water to which 300 p.p.m. of sodium bicarbonate has been added, holding the sample at a constant temperature for a certain length of time and noting the difference in the dissolved oxygen content at the beginning and end of the test. It is common practice to determine the B.O.D. at 20°C. after incubation for either 5 or 10 days. The sample must fill a glass-stoppered bottle, so that no oxygen can be absorbed from the air. This is a delicate test and requires a thorough appreciation of the conditions requisite for its satisfactory conduct.

In the *relative-stability* test, use is made of the fact that methylene blue, an organic dye substance, is decolorized when the oxygen in the sample is exhausted. The test is performed by adding a small volume of methylene blue solution to the sewage in a filled glass-stoppered bottle, holding it at a constant temperature, commonly 20°C., and noting the time elapsed before the blue color disappears. The value is a measure of the *relative stability* of the sample as compared with one which would not become decolorized during the initial stage of oxidation. The results of this test usually are reported in percentage, as in Table 8.

The reduction in oxygen content in both these tests is due to the activity of living organisms in the sewage. For this reason a disinfected or highly acid or alkaline sewage or effluent must be adjusted to normal conditions and seeded with living organisms before the tests are performed. The laws of oxygen depletion have been formulated and can be used to extend the information gained from either test.

Fats and oils in sewage are derived from animal and vegetable matter or from wastes such as those of garages and gas works. Their quantity is determined as *ether-soluble matter*, by liberating with acid the fatty acids, after evaporating the water, and dissolving them in ether which is then driven off, leaving behind the ether-soluble matter which can be weighed.

Aside from the test for fats and oils, the tests for organic matter attempt to measure chiefly the concentration and condition of that portion of the sewage which draws upon the oxygen resources of the bodies of water into which the sewage is discharged. For this reason the tests that measure directly the oxygen avidity of the sewage have come to be the most important. The tests for nitrogen and oxygen consumed are time-honored tests which have been applied to sewage after being developed for water analysis. Of the nitrogen determinations, the tests for organic nitrogen and for nitrite and nitrate nitrogen are reliable. The tests for albuminoid and ammonia nitrogen are quite unreliable, especially the former. Measuring only a portion of the decomposable substances, the nitrogen determinations yield but scanty information as far as the broader aspects of sewage treatment and disposal are concerned. While they, as well as the oxygen consumed test, have lost a great deal of their former importance since the introduction of the biochemical oxygen demand test, they are still in general use, largely because of the store of information available through them from the past. Certain of the tests, too, remain of specific value in a number of treatment processes. The test for ammonia nitrogen, for example, appears to be of value under some conditions. Comparison of effluents from treatment works on the basis of nitrification, *i.e.*, nitrate production, is not justifiable, since it depends upon the quantity of nitrogen originally present in the sewage and, like the other nitrogen determinations, measures only part of the work accomplished. The degree of nitrification is of value, however, in judging the efficiency from some points of view of a particular plant.

Dissolved Oxygen.—It is important not to confuse the determination of dissolved oxygen with that reported as "oxygen consumed," which is used as a measure of the carbonaceous matter present in sewage. The dissolved oxygen test is made to ascertain the quantity of atmospheric oxygen which is dissolved in the given sample of sewage, effluent or polluted water. In collecting a sample for this test, extreme care must be exercised in order to avoid the entrainment or absorption of any oxygen from the air. The test should be carried out immediately after collection of the sample.

The quantity of dissolved oxygen in the sample is determined by using it to liberate iodine from an excess of alkaline potassium iodide, added to the sample in the presence of manganous sulfate, and determining the quantity of iodine thus liberated. The results of the dissolved oxygen test may be reported either in parts per million, cubic centimeters per liter or percentage of saturation.

Hydrogen Ion Concentration.—The hydrogen ion concentration is an expression of the intensity factor of alkaline or acid properties as opposed to the quantity factors, "alkalinity" and "acidity." As explained in Chap. III, the H ion concentration generally is represented by a series of numbers, called "pH values." In determining pH values in sewage analysis, use commonly is made of the fact that certain organic dye substances, called indicators, manifest sensitive color changes at different H ion concentrations. Comparison of these colors with colors produced in solutions of known H ion concentration is made readily and will give the desired information. For details in regard to this determination, reference may be made to Clark's book on the subject (2).

Determination of Mineral Matters.—The determination of a number of the mineral constituents of sewage, such as the fixed solids and mineral forms of nitrogen, already has been discussed. Most of the remaining forms of mineral matter are only of passing interest.

When testing to determine the degree of *alkalinity*, it is usual not to ascertain by analyses what alkaline salts are present but simply to determine the alkalinity from all sources. This is done by titrating against a standard acid, reporting the results in terms of parts per million of calcium carbonate.

Acidity is determined by titrating against a standard alkali, expressing the results in terms of calcium carbonate required to neutralize. The results of such tests do not show to what acids or salts the acidity is due but simply the total acidity of the sewage.

The *hardness* or "soap-consuming power" of water is a measure of its content of calcium and magnesium. Since no lather will form until sufficient soap has been added to combine with the salts of these elements, this fact is used for determining hardness. The results of this determination also are reported in terms of calcium carbonate.

The *chloride* radicals are determined readily by titrating against a standard solution of silver nitrate.

Iron in sewage may be determined easily by adding to the sewage chemical reagents which impart a pink color to it. This color is compared with that produced by known quantities of iron.

Mineral analyses, as far as their wider application to sewage-disposal problems are concerned, are of secondary value. The test for chlorides sometimes is used to measure the concentration of sewage. Furthermore, owing to hourly variations in concentration of sewage, the determination of chlorides in influent and effluent of a treatment plant may afford a check upon the reliability of sampling methods. The determination, however, is not of sufficient value to be made part of the routine examination of sewage. Of similar local or specific value are most of the other tests for mineral matter, such as alkalinity, acidity, hardness, sulfates, iron and heavy metals. The latter substances are sometimes of importance because of their germicidal action. The tests for alkalinity and acidity in their more general significance have been superseded by the determination of the hydrogen ion concentration.

BACTERIOLOGICAL EXAMINATION

Collection of Samples.—In collecting samples of effluents and polluted waters for bacterial examination, extreme care should be exercised against the contamination of the samples. They should be collected in bottles with glass stoppers, preferably of the flat mushroom type, holding for ordinary purposes about 4 oz. Prescott and Winslow (3) recommend wide-mouthed bottles. The bottles should be thoroughly cleansed in the laboratory with sulfuric acid and potassium bichromate, or alkaline permanganate of potash followed by sulfuric acid. Then they should be thoroughly rinsed with distilled water, dried by draining and sterilized with dry heat at 170°C. for at least an hour and a half, or in an autoclave at 120°C. for 15 min. Before sterilization, the bottles should be wrapped in cloth or paper, or the stopper and neck should be wrapped with tinfoil.

When the sample is taken, the paper, cloth or tinfoil should be removed and the stopper withdrawn, care being exercised not to let the fingers touch anything which will later come into contact with the water collected. When collecting a sample of running water, the mouth of the bottle should be pointed upstream, in order that contamination from the fingers may be carried away from the bottle rather than into it. When samples are collected from still water, the same result may be attained by rapidly pushing the bottle through the water while it is being filled. When the sample is being taken from a tap or pump, the water should be allowed to run for a period of several minutes before the sample is collected. After the bottle has been filled, the stopper should be replaced immediately and the stopper or bottle rewrapped with the cloth, paper or foil originally protecting it.

Preservation of Samples.—After a sample has been obtained, the number of bacteria in it may increase or decrease rapidly and it is therefore desirable to adhere to the recommendations of "Standard Methods," (1) that the interval between sampling and examination should not exceed 12 hr. in the case of relatively pure waters and 6 hr. in the case of impure waters. During the period of storage, the temperature should be kept between 6 and 10°C. If the sample must be transported a considerable distance before it is plated, the bottle should be packed in ice. This will prevent a rapid increase in the bacterial content, but it must be remembered that long-continued exposure to low temperature is inimical to bacterial life.

Bacterial Counts.—Determination of bacteria in sewage, as in water, makes use of the fact that bacteria can be cultivated on suitable nutrient media and that different groups of bacteria will react differently to artificial environmental conditions. Three tests are in common use: the count of bacteria that develop on agar or gelatin at 20°C. in 48 hr., the count of bacteria that develop on agar at 37°C. in 24 hr. and the quantitative test for organisms of the *coli aerogenes* group.

Agar and *gelatin* are so-called solid culture media. They contain besides certain nutrient substances enough agar-agar, a Japanese seaweed with gelatinous properties, or gelatin, itself a nutrient substance, to render the mixture solid at the temperatures of incubation. If a known volume of sewage is placed in a sterile glass dish and melted agar or gelatin is added, a solid medium, in which the sewage bacteria are entrapped, results upon cooling. The organisms are isolated thus at the same time that they are provided with an abundant food supply.

If the glass dish is covered to keep out bacteria that might fall into it from the air and is held at a favorable temperature, single bacteria that were invisible to the naked eye will multiply and form colonies so large that they can be seen easily without the aid of a microscope. A count of the colonies with the aid of a magnifying glass will show the number of bacteria originally present in the sewage that will develop under the conditions of test. No method is known for readily ascertaining the exact number of bacteria in a liquid. The 20 and 37°C. counts merely give the number of organisms that develop on the particular types of culture medium at the specified temperatures in the particular time. Many of the most important sewage bacteria cannot grow under these conditions and, therefore, are not enumerated. The 20°, or room-temperature, count is supposed to measure the normal saprophytic *flora* of sewage. The 37° count is employed to enumerate those organisms that grow at the body temperature of man and warm-blooded animals and represents, therefore, in a general way the organisms of parasitic origin. As a matter of fact, no rigid demarcation of bacterial groups can be obtained by these two tests.

The quantitative test for organisms of the *coli aerogenes* group, commonly called *B. coli.*,[1] if carried to completion, is probably the best index of sewage pollution now available. *Bacillus coli* has been chosen as the standard indicator organism, because it occurs in large numbers in the excreta of man and the higher animals and is identified readily. While parasitic, it is not pathogenic, although it bears close resemblance to the causative organism of typhoid fever, from the standpoint of the environment in which both are found, as well as the length of life of the two germs under a variety of conditions. The standard test for *B. coli* is based on the fact that this organism ferments milk sugar, or lactose, to produce hydrogen and carbon dioxide gas, at the same time rendering the medium acid. Gas production in a nutrient medium containing lactose is, therefore, presumptive evidence of the presence of *B. coli*. By adding definite volumes of water or sewage to lactose broth quantitative information can be obtained, such as that coli is presumably present in 10-cc. but not in 1-cc. samples. For the complete test "Standard Methods" (1) should be consulted.

The numbers of *B. coli* may be determined also by direct count of the colonies developing on eosin-methylene-blue agar from incubation for 24 hr. at 37°C. On this medium colonies of *B. coli* appear as very dark, almost black, spots and experienced observers have no difficulty in differentiating between these colonies and colonies of other organisms. Furthermore, this medium has a selective action and inhibits development of organisms other than *B. coli*.

[1] An abbreviation of *Bacillus coli*, recently renamed *Escherichia coli* after its discoverer. Escherich.

In the case of sewage, bacterial counts are reported in numbers per cubic centimeter. Although bacterial examinations are used chiefly in connection with water-supply problems, they often are employed in sewage-disposal studies where bathing beaches or shellfish grounds are involved. Such examinations are also matters of routine at sewage-treatment plants using disinfection processes.

Differentiation of Bacteria Other than B. Coli.—Much labor has been expended in isolating different species of bacteria and describing them, in order to make their future identification practicable. This is important, for while much can be learned by a study of a mixture of organisms, most progress will be made when the action of the several species is known and the environments required for the optimum development of each are defined. Up to the present time, however, tests for differentiating species other than *B. coli* have not been made practicable as a matter of routine.

One of the most significant bacteria in sewage treatment would be *B. typhosus*, the causative organism of typhoid fever, provided it could be identified easily. The isolation of this bacillus from infected water is, however, a matter of great difficulty and uncertainty. Furthermore, most questions relating to the contamination of water supplies require data not only with reference to the actual presence of *B. typhosus* but also as to its probable or possible presence at some future time under similar conditions. Hence the consensus at the present time appears to be that the test for *B. coli* is the most satisfactory method of determining the extent of bacterial contamination and that an effort to isolate *B. typhosus* usually is not warranted.

MICROSCOPICAL EXAMINATION

Sewage and effluents rarely are subjected to microscopical examination in routine analytical work. Under certain circumstances, however, such a test may be of value in studies of the disposal of sewage into natural waters. Microscopical analysis may be used to indicate the progress of the self-purification of streams, the presence in water of an excess of toxic industrial wastes or the presence of sewage contamination. The examination consists in the enumeration of the kinds of microscopic organisms in the water and an estimate of their quantity, together with a brief examination of the amorphous matter. For the method of making a microscopical examination reference should be made to "Standard Methods" (1) and for further details Whipple's book (4) should be consulted.

EXAMINATION OF SEWAGE SLUDGE AND MUDS

The routine examination of sewage sludge is important in the operation of most sewage-treatment plants, particularly those in which the sludge is subjected to a process of digestion in preparation for further treatment and disposal. A similar problem sometimes is encountered in a study of the pollution of streams, where the character of deposits of sludge and muds may have to be studied. The more common tests to which these substances are subjected are outlined below.

Collection and Preservation of Samples.—For the analysis of sludge and mud care must be exercised to obtain representative samples. In the case of wet sludge and of mud which is at or above the water surface, a number of individual samples may be collected and combined into a composite sample. The samples should be taken and composited in wide-mouthed bottles and kept in a refrigerator until analyzed. Ordinarily no preservative is required for sludge samples, unless they are to be kept for a week or more before analysis. If the hydrogen ion concentration is to be determined, however, a preservative should be added, unless the analysis is to be made within a few hours of collection. If determinations of fats are to be made by ether extraction, chloroform must not be used as a preservative.

A representative sample of dried sludge may be obtained by shoveling it into a heap, dividing the heap into four approximately equal parts and discarding two opposite quarters. The two remaining quarters are then shoveled into a heap and the process of division is repeated until the quantity remaining is suitable for analysis. Sludge from drying beds should be freed from adhering particles of sand or gravel as much as possible.

Samples of mud may be collected from the bottoms of streams and lakes by means of valved or slotted pipes, conical dredges, clam-shell buckets or other sampling devices. According to "Standard Methods," (1), samples may be preserved by adding 5 gm. of sodium benzoate or 2 cc. of concentrated sulfuric acid to 80 gm. of mud.

Moisture and Specific Gravity.—The moisture content of sludge or mud is determined by subjecting a weighed quantity to evaporation on a steam bath and then in an oven at 103°C. The loss in weight is reported as moisture, which is expressed in percentage of the original sample.

In determining the specific gravity of sludge or mud, a wide-mouthed flask or glass-stoppered bottle of known weight is used. It is filled first with distilled water, then with the substance which is being tested, and the weight is determined each time. If a bottle is used, care must be taken to avoid undue pressure in inserting the stopper. The specific gravity is equal to the ratio of the weight of sludge or mud to that of an equal volume of water. It should be recorded to the second decimal place.

Fixed and Volatile Solids.—Fixed and volatile solids are determined in the same way for sludge as for sewage. The results usually are reported in percentage of the dry solids in the original sludge.

Reaction.—In determining the reaction or the hydrogen ion concentration of sewage sludge or muds, the standard practice is to make an aqueous suspension of the substance by diluting it with an appropriate quantity of distilled water. According to Holmquist (5), dilution should not be greater than 15 parts of water to 1 part of sludge or mud, in testing the hydrogen ion concentration.

Whether the sludge or mud is acid or alkaline may be determined in the aqueous suspension by the use of either methyl orange or methyl red and phenolphthalein as indicators. The hydrogen ion concentration may be measured in the manner outlined above for sewage and effluents.

Fertilizer Analysis.—In examining a sludge from the standpoint of fertilizing value, the principal constituents of interest are organic nitrogen, phosphoric acid and potash. The nitrogen can be determined most accurately by distillation into acid according to the Kjeldahl method, as outlined above for sewage and effluents. The phosphoric acid is determined after precipitation in the form of ammonium phosphomolybdate in nitric acid solution, using ammonium molybdate as the precipitant. Similarly, the potash is found by precipitation in the form of potassium platinic chloride, using platinic chloride as the precipitant and removing the sodium compounds by washing with alcohol. The results of fertilizer analyses are expressed in percentage by weight of the dried sludge.

For details in regard to such analyses, reference should be made to the methods used by agricultural chemists (6).

Grease.—The determination of the quantity of grease or fats in sewage sludge is made by ether extraction, the grease being found in the form of ether-soluble matter, as outlined for the determination of fats and oils in sewage. Continuous extraction for 6 hr. or longer with ethyl ether extracts gums, resins, waxes, etc., in addition to true grease. It is standard practice to use petroleum ether, which is inert with respect to gums, etc., but is as effective as ethyl ether in dissolving grease or fat. Continuous extraction is necessary for the complete removal of grease. The results of this test also are reported in percentage by weight of the dry solids in the sludge.

Special Tests.—In addition to the tests commonly made upon sewage sludge and muds, which have been outlined above, several other constituents are sometimes investigated when there are special reasons for doing so. For instance, research has been made upon the bacteria and other microorganisms which are connected with the various stages of sludge digestion. The gases evolved in this process also have been the subject of much study, with a view to determining the practicability of utilizing them. Another determination which has been made in special investigations is the biochemical oxygen demand of sludge deposits and muds from the bottoms and banks of polluted streams. Such tests are valuable in special cases, but with the possible exception of the analysis of sludge gases, they have not been adopted as routine procedure in the operation of treatment plants.

EXAMINATION OF INDUSTRIAL WASTES

The methods employed in analyzing liquid wastes from industrial establishments are practically the same as those utilized in the case of domestic sewage, treatment-plant effluents and polluted waters. On account of the different kinds of wastes produced by various industries, however, certain tests are more important in some cases while other tests are more significant in other cases. For instance, in examining the wastes from iron works, wire-drawing plants or coal mines, one of the most important tests is the determination of acidity. In the analysis of iron-works wastes, another significant constituent is iron. Similarly, suspended solids may be important in wastes from rubber-reclaiming plants and tanneries; biochemical oxygen demand and bacterial counts in wastes from stockyards, tanneries and wool-scouring establishments; and fats in wastes from laundries and wool-scouring works.

In some instances, special tests may have to be employed in order to reveal significant facts about certain wastes. This is exemplified by a test for tendency to form foam, which was used by the authors in examining the wastes from a glue factory. The wastes were discharged into a river just above a dam and at times the foam which formed at this dam floated downstream for several miles, causing complaints from riparian owners. Hence it was necessary to test the foam-forming tendency of the wastes as a matter of daily routine.

In brief, the choice of determinations to be made in the routine analysis of industrial wastes should be based upon local conditions, as well as upon the characteristics of the wastes themselves.

SEWAGE-WORKS LABORATORIES

In view of the fact that sewage analyses play an essential part in maintaining the operating efficiency of sewage-treatment works, it is important that laboratories where such analyses are made, be suitably designed and equipped to fulfill their purpose. For this reason some of the main principles which govern the design and equipment of sewage-works laboratories are given below.

Location and Layout of Space.—In order to conserve time and energy, the laboratory sometimes is located as near as possible to the points where samples of sewage and effluents are to be collected regularly. This may involve the erection of a building solely for laboratory purposes, but in many cases the laboratory can be located in the administration building of the treatment plant. The latter plan has the advantage of combining the control and operating departments under one roof. If the laboratory is located in a building of more than one story, it is often on the top floor, where facilities for good lighting and ventilation are afforded.

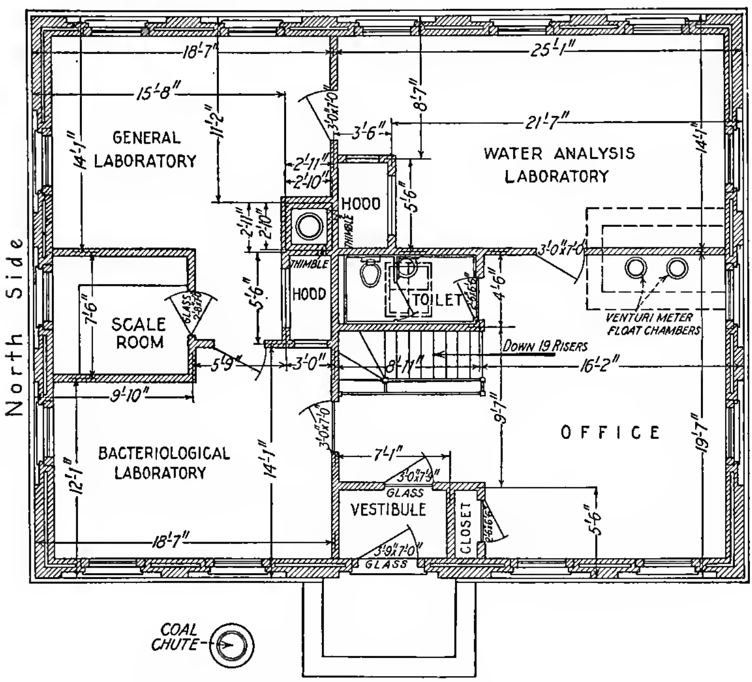


Fig. 6.—Plan of sewage-works laboratory at Fitchburg, Mass.

The most completely equipped laboratories contain facilities for biological research as well as for chemical analysis. The difference in character of the two kinds of work makes it advisable to divide the laboratory into two distinct parts, chemical and bacteriological. In the case of large plants it may be advisable also to provide one or more constant-temperature rooms, well insulated against heat and cold, where a given temperature can be maintained by thermostatic control. For small plants the expense of such an installation probably is not justified. A stock room, for the storage of a reserve supply of apparatus and materials, is often included in the design, as well as an office containing desks, bookcases and filing cabinets. A plan of the sewage-works laboratory at Fitchburg, Mass., which was designed for bacteriological as well as chemical work, is shown in Fig. 6.

Equipment and Special Features.—Among the important items to be considered in the design of a laboratory are ventilation and lighting. Since artificial ventilation is usually necessary, various methods have been devised, ranging from a mere opening in the ceiling to an intricate system involving air shafts, fans and blowers. In addition to such a ventilating system provision generally is made for a fume hood, through which dangerous and obnoxious gases can be conducted to the outside atmosphere. Many hoods are equipped with blowers, which tend to hasten the removal of fumes.

Several kinds of analytical work, but especially those involving the observation of colors, require good light, the most satisfactory results being obtainable with daylight. Since light from the north is most uniform throughout the day, many laboratories have windows on the north side and the apparatus for which natural light is to be used is arranged on that side of the room. In some laboratories where colorimetric work must be carried out by artificial light, so-called "daylight" lamps are employed.

The problem of equipping a laboratory with fixtures, apparatus and chemicals is one which requires study in relation to the kind of analytical work to be performed, the frequency of performing it, the personnel of the laboratory, the funds available, and other factors. Weiner (7) lists the following major pieces of apparatus, constituting permanent fixtures, which are necessary for fully equipping a laboratory for sewage-disposal investigations:

Laboratory tables	Kjeldahl supports with heaters
Fume hood	Kjeldahl distilling outfit
Cabinets	Incubator
Analytical balance	Dry sterilizer
Water bath	Autoclave
Drying oven	Water still
Centrifuge	Gas-analysis apparatus
Extraction support and accessories	Refrigerator
Muffle furnace	Microscope and accessories

Additional suggestions in regard to the construction and equipment of sewage-works laboratories may be obtained from a committee report of the National Research Council (8).

Bibliography

1. "Standard Methods for the Examination of Water and Sewage," Am. Pub. Health Assoc. and Am. Water Works Assoc., 7th ed., 1933.
2. Clark, W. M.: "The Determination of Hydrogen Ions," 3d ed., 1928.

3. Prescott, S. C., and C.-E. A. Winslow: "Elements of Water Bacteriology," 4th ed., 1924.
4. Whipple, G. C.: "The Microscopy of Drinking Water," 4th ed., 1927.
5. Operation and Control of Sewage Treatment Plants, N. Y. State Dept. of Health, 1930.
6. "Methods of Analysis of the Association of Official Agricultural Chemists," 1920.
7. Weiner, Henry: *Sewage Works Jour.*, 1929; **1**, 358.
8. Rept. Nat. Res. Council Comm. on Construction and Equipment of Chemical Laboratories, 1930.

CHAPTER V

COMPOSITION OF MUNICIPAL SEWAGE

Municipal sewage usually contains liquid and water-carried wastes of several different kinds. Liquid wastes from kitchen sinks are called *sink drainage* and the conduits through which they flow *sink drains*. When to these are added the wastes from laundry tubs, bath tubs and water-closets, the combined wastes appropriately are termed *house sewage* or *domestic sewage*, for they have been collected by a part of the building's plumbing system and conveyed from the house by the house sewer or building connection. The wastes from manufacturing processes generally are termed *industrial wastes*, although they sometimes are called *manufacturing* or *trade wastes*. Often sinks and water-closets used by employees discharge into the pipes carrying industrial wastes and there is then some question whether the combined liquids may be termed "sewage." This is largely a question of degree and the answer depends principally on the relative proportions of industrial wastes and house sewage.

Rain water running directly from roofs is called *storm water* or *roof water*. That running off the ground also is called storm water and that running from streets is sometimes termed *street wash*, because it is the water with which the street surfaces have been washed. All storm-water run off is wash water and has washed the surfaces over which it has flowed, frequently taking up large quantities of impurities in doing so. The term "storm water" should be applied only to that portion of the rainfall which runs off during or shortly after a storm. Ordinary stream flow is distinguished from storm water by designating it as *surface water*. This term includes storm water but not those waters which are retained in or flow through the ground, the latter being called *ground water*. Waters escaping from the ground into streams and lakes become surface waters.

Origin of Sewage.—In the simplest sewered community, a small group of houses drained by a single sewer, from each of the houses comes "house sewage." Flowing from the "house sewer" to the "sewer" at this point, it may be said to have changed from "house sewage" to "sewage." No individual house sewage changes its composition by its change of location, but it mingles with the other house sewages coming from buildings where the activities are different and consequently where the house sewage is different. In some houses the waste water from the early morning baths is being discharged; in others, sink water from the breakfast dishes; from still others, perhaps, the laundry wastes. The sewage from this simplest sewered community thus consists of mingled house sewages of different qualities but still remains "domestic sewage."

Passing on down the sewer, the sewage of the simple sewerage district is mingled with other domestic sewages from other similar communities and with the wastes from business houses, hotels, and garages. If there are industries like dye works, tanneries or woolen mills, the spent dyes, tanning liquors and wash waters may be discharged into the sewer, materially modifying the quality of the sewage. In many places ground water finds its way into the sewers, sometimes in considerable quantities, having a marked effect on the quality of the sewage.

Storm water is not admitted intentionally to separate sewerage systems, but in many cases there may be leakage through manhole covers; some street inlets may be connected with the sewers and roofs may be connected either with or without permission from the authorities. Such storm water also may have a marked influence upon the quality of the sewage. The authors have known a dry-weather flow of 300,000 gal. daily in a separate sewerage system to be increased to 3,000,000 gal. by storm water and ground water which found their way into the sewers.

Combined sewers are designed to take storm water, for which inlets are provided on the streets, and in such sewers the composition of the sewage at times is materially influenced by storm water. In the early part of a storm large quantities of mineral and organic matter are contributed by street washings. The roof water and run-off from other areas, reaching the sewers somewhat later, afford considerable dilution. In sewers there are sometimes considerable deposits of sewage matter, which are washed out by the scouring action of storm water. Such deposits are often extremely foul and high in organic matter. The effect of this material and the wash from the streets is often to make the early part of the storm flow much stronger than the ordinary sewage. Such flows are frequently alluded to as *first flushings*.

Variations in Sewage Quantity.—Variations in the quantity of municipal sewage produced by different cities, as well as variations at different times in a given city, have been described at length in Vol. I, Chap. V.

Variations in Sewage Quality.—The quality of the sewage from a given community changes rapidly from time to time, because of the varying kinds and quantities of constituents contributed and also on account of the varying quantities of ground, surface and storm water entering the sewers. The night sewage is much weaker than that of the day and hourly analyses of sewage show that its composition depends directly upon the activities of the contributing population.

The quality of sewage varies from day to day, the greatest variation from the average being on Sunday when industrial wastes and household activities are checked. Where the week's washing is done on Monday, the quantity of sewage is increased on that day by the discharge of wash waters and the soaps and other ingredients affect the quality of the sewage.

During the spring or "wet" months sewage may be considerably weaker than during the summer and early autumn, owing to the storm and ground waters which find their way into the sewers. The composition is also affected to some extent by temperature and bacterial action as well as by dilution.

In many places the chlorine content of the sewage is practically proportional to the dilution and if the quantity of chlorine in the diluted sewage is known, the quality of the undiluted sewage and the effect of dilution may be calculated.

In view of the fact that sewage is not a homogeneous substance, it is difficult to collect representative samples. Furthermore, skill and experience are required on the part of the analyst, if consistent and reasonable results are to be obtained.

Table 10.—Variations by 2-hr. Periods in Quantity and Quality of Columbus Sewage, Expressed in Percentages of Daily Average

Period	Rate of flow	Oxygen consumed			Nitrogen as		Chlorine	Suspended matter			Fats	Bacteria
		Total	Dissolved	Suspended	Free ammonia	Organic		Total	Volatile	Fixed		
12 P.M.– 2 A.M.	88	35	42	27	73	47	72	59	29	81	33	75
2 A.M.– 4 A.M.	83	69	23	123	56	31	74	43	28	53	26	47
4 A.M.– 6 A.M.	80	28	29	27	42	23	73	37	18	52	30	36
6 A.M.– 8 A.M.	89	28	29	27	49	28	69	37	29	43	22	56
8 A.M.–10 A.M.	113	176	181	169	108	167	94	213	156	257	81	92
10 A.M.–12 M.	117	116	136	92	109	117	143	145	160	134	111	167
12 M. – 2 P.M.	115	123	149	92	110	115	123	110	129	96	111	139
2 P.M.– 4 P.M.	117	117	149	81	167	126	111	110	109	112	81	167
4 P.M.– 6 P.M.	115	197	149	254	147	225	111	165	229	116	307	103
6 P.M.– 8 P.M.	103	88	81	96	99	84	101	94	109	83	163	125
8 P.M.–10 P.M.	92	90	87	92	99	80	114	54	64	46	81	106
10 P.M.–12 P.M.	89	58	68	46	84	63	80	52	50	54	74	81

In Table 10 are given by 2-hr. periods the proportional variations in flow and quantity of various constituents of the sewage of Columbus, O., which are typical of average hourly changes (1).

Causes similar to those effecting the hourly changes make the quality of sewage different on different days. Analyses typical of the daily variations in composition of Columbus sewage are given in Table 11 (1).

TABLE 11.—DAILY VARIATIONS IN WEIGHT IN CONSTITUENTS OF COLUMBUS SEWAGE, EXPRESSED IN PERCENTAGES OF WEEKLY AVERAGE

Day of week, 1904–05	Oxygen consumed	Nitrogen as				Chlorine	Residue on evaporation								
							Total			Volatile			Fixed		
		Total organic	Free ammonia	Nitrites	Nitrates		Total	Dissolved	Suspended	Total	Dissolved	Suspended	Total	Dissolved	Suspended
Sun.	61	75	102	112	93	84	86	92	64	71	76	64	90	95	63
Mon.	114	101	104	89	93	100	105	101	121	118	107	131	102	100	115
Tues.	110	105	103	112	93	105	103	103	102	108	107	109	101	102	99
Wed.	106	107	100	89	93	104	101	103	93	99	102	95	101	103	93
Thur.	98	102	94	100	139	100	99	100	98	98	98	99	100	100	97
Fri.	104	102	101	100	93	102	102	100	110	101	103	99	102	100	117
Sat.	100	104	97	100	93	102	102	101	107	101	103	100	102	100	111

TABLE 12.—MONTHLY VARIATIONS IN WEIGHT IN CONSTITUENTS OF WORCESTER SEWAGE, EXPRESSED IN PERCENTAGES OF ANNUAL AVERAGE

Month, 1907	Ammonia		Oxygen consumed	Chlorine	Suspended solids			Precipitation, in.
	Free	Total albuminoid			Total	Volatile	Fixed	
January	93	110	106	104	92	92	93	2.76
February	112	109	117	94	98	115	58	1.84
March	95	95	93	80	102	91	129	1.69
April	88	88	90	81	84	85	81	2.72
May	115	103	103	103	85	91	73	2.92
June	115	104	99	117	123	121	129	3.82
July	127	122	122	148	148	162	115	2.55
August	134	128	143	149	131	147	92	1.08
September	119	114	110	125	139	118	186	9.38
October	81	85	89	88	97	94	104	4.63
November	68	85	79	72	74	53	122	6.06
December	86	76	74	75	61	75	27	4.53

TABLE 13.—COMPARATIVE ANALYSES OF WATER AND SEWAGE
Parts per Million

Municipality	Date of analyses	Residue on evaporation	Ammonia nitrogen	Albuminoid nitrogen	Nitrite nitrogen	Nitrate nitrogen	Chlorine	Oxygen consumed	Hardness
Attleboro, Mass.									
Water supply	1930	52.0	0.004	0.011	0.00	0.08	4.4	......	22
Sewage	1930	338.0	24.6	3.3	....	....	29.1	35.2	
Dayton, Ohio									
Water supply	1921	540.0	0.010	0.036	0.00	3.60	16.0	0.20	366
Sewage	1930	878.0	16.0	.....	....	....	93.0	120.0	
Easthampton, Mass.									
Water supply	1930	73.0	0.002	0.006	0.00	0.17	1.9	......	37
Sewage	1930	638.0	34.6	5.3	....	....	69.2	66.2	
Fitchburg, Mass.									
Water supply	1924–28	34.1	0.039	0.123	....	....	1.8	......	9
Sewage	1924–28	473.0	12.2	4.3	0.08	0.89	55.4	135.6	
Mansfield, Ohio									
Water supply	....	391.0	0.081	0.012	0.00	1.50	8.5	0.05	285
Sewage	....	876.0	13.3	.....	0.00	0.20	108.7	51.6	
Worcester, Mass.									
Water supply	1926–29	41.8	0.036	0.127	....	....	2.3	......	13
Sewage	1925–30	811.0	14.8	6.5	0.08	0.75	97.1	130.7	

NOTE: Analyses of Attleboro water and sewage, Easthampton water and sewage, Fitchburg water and Worcester water from annual reports of Massachusetts Department of Public Health; Dayton water analysis by Ohio State Department of Health; Dayton sewage analysis from Morehouse; Fitchburg sewage analysis from annual reports of Commissioner of Public Works; analyses of Mansfield water and sewage from Dittoe; Worcester sewage analysis from Lanphear.

In Table 12 are given the proportional variations in character of sewage from month to month at Worcester, Mass.

Comparative Analyses of Water and Sewage.—It is a popular opinion that sewage is the municipal water supply defiled by use and discharged into the sewers. Were this wholly true, the quantity of sewage would correspond closely to that of the water supply. In a general way it does, but the whole water supply does not reach the sewers, as indicated in Vol. I, page 173, and much water from other sources enters them, such as ground water and private water supplies. It is, therefore, important to consider as the basis of sewage the composition of water supplies and ground water and to estimate the quantity of each entering the sewers.

The average analyses of a few water supplies and the corresponding sewages are given in Table 13. The water supplies and sewages at Fitchburg and Dayton illustrate the point just made. If the water supply and ground water at Fitchburg had contained the same quantity of total solids as those at Dayton, the total solids in the Fitchburg sewage would have been 506 p.p.m. greater, an increase of 107 per cent, effected only by the change in the water supply. In other words, waters which are high in residue, which is the case with all hard waters, contribute a considerable proportion of the solids in the corresponding sewage, while waters low in residue furnish to the sewage only an insignificant proportion of its solids.

The quantity of nitrogenous matter in municipal water supplies is usually so low that its influence upon the composition of the sewage may be disregarded. At Fitchburg the organic solids in the water supply, estimated at 10 p.p.m., are equivalent to about 2 per cent of the total solids in the sewage and to about 4 per cent of the organic solids in the sewage.

Substances Present in Sewage.—To the substances contained in the water supply are added urine, feces and paper from water-closets. Researches of Wolff and Lehmann (2) gave the quantities of urine and feces and their constituents stated in Tables 14 and 15.

The Metropolitan Sewerage Commission of New York in 1910 estimated the quantity of paper in sewage as 20 gm. per capita daily.

Kitchen sinks contribute soap, grease, extracts of meats and vegetables, sugar, salt, milk and waste food washed from dishes. Sometimes substantial quantities of garbage are thrown in water-closets, instead of being disposed of in a suitable manner. Laundries and baths contribute soapy water containing starch, dissolved and suspended impurities from clothing and excretions and tissues from the body.

Garages and gasoline filling stations may have untrapped drains that discharge oil and sand into the sewers. Automobile washstands

Table 14.—Weight of Solid and Liquid Excrements of a Mixed Population

Number, sex and age of persons contributing	Feces		Urine	
	Pounds per 100,000 persons yearly	Grams per cap. daily	Pounds per 100,000 persons yearly	Grams per cap. daily
37,610 men	4,521,664	150	45,217,782	1,500
34,630 women	1,237,040	45	37,458,512	1,345
14,060 boys	1,239,504	110	6,423,670	570
13,700 girls	274,736	25	5,041,344	460
Total	7,272,944	...	94,141,308	
Average	....	90	....	1,170

Table 15.—Average Composition of Human Excrements

Kind	Water	Total solids	Organic matter	Inorganic matter	Nitrogen	Phosphoric acid	Potash	Lime	Magnesia
Per cent									
Fresh human feces	77.2	22.8	19.8	3.0	1.00	1.10	0.25	0.62	0.36
Fresh human urine	96.3	3.7	2.4	1.3	0.60	0.17	0.20	0.02	0.02
Mixture of the two[1]	95.0	5.0	3.6	1.4	0.63	0.24	0.20	0.06	0.04
Grams per capita daily									
Fresh human feces	69.5	20.5	17.8	2.7	0.90	0.99	0.22	0.56	0.32
Fresh human urine	1127.0	43.0	28.1	14.9	7.04	1.99	2.34	0.23	0.23
Mixture of the two	1196.5	63.5	45.9	17.6	7.94	2.98	2.56	0.79	0.55

[1] Calculated from a table by Rafter and Baker, to make these quantities comparable with those in Table 14.

contribute large quantities of water, more or less heavily laden with mineral matter, some organic matter, oil and grease.

Waste water used for floor washing in dwellings and office buildings is usually foul and the washings from cuspidors may contribute pathogenic organisms. Hospital sewage often contains pathogenic organisms from the excreta, as well as from other discharges of patients.

Because of the great variations in the quantities of solids discharged from industrial establishments, as indicated in Table 28, it is necessary that these quantities be obtained for each separate city by an industrial survey, as outlined in Chap. XXXII.

Storm water washes much mineral and some organic matter into the sewers. Generally, the first flush of storm water and, more particularly, of street wash contains large quantities of suspended matter, which increase the strength of the sewage. After long-continued rain has washed the streets and flushed the sewers, the quantity of solids carried into the sewers by the storm water may be considerably reduced, so that the effect of the storm water will be only to dilute the sewage.

TABLE 16.—COMPOSITION OF TYPICAL AMERICAN SEWAGE
Parts per Million

Substances	Total	In solution	In suspension
Residue on evaporation	800	500	300
Mineral matter or ash	400	300	100
Organic or volatile matter	400	200	200
Nitrogenous matter	150		
Nitrogen	15		
Carbon	75		
Hydrogen, oxygen, sulfur, phosphorus, etc	60		
Non-nitrogenous matter	250		
Fats, etc	50		
Carbon	35		
Hydrogen, oxygen	15		
Carbohydrates	200		
Carbon	90		
Hydrogen, oxygen, etc	110		
Total carbon	200		
Total nitrogen	15		
Total hydrogen, oxygen, sulfur, phosphorus, etc	185		

The organic matter in sewage may be said, in a general way, to consist of urea and proteids, which include most nitrogenous organic matter, and of carbohydrates, including cellulose, woody fiber, fats and soap. Of these, the most important are the proteids and urea, because they are most likely to cause offensive conditions. In filtration problems, the fats become of importance if present in large quantities, as they tend to coat the surfaces of filter media and to fill the voids thereof. The general relations of these ingredients are shown in Table 16, giving a hypothetical analysis typical of American sewage (3).

Of these constituents, most attention has been given to the nitrogenous compounds, usually considered the active agencies in producing offensive conditions. Dunbar has stated that too much importance has been placed upon them, because nitrogen does not enter into the composition of offensive gases as sulfur does, and that more attention should be given to the latter. In Table 16 Winslow and Phelps estimated that about one third of the total organic matter is nitrogenous and of this portion only 10 per cent is nitrogen, whereas 50 per cent is carbon.

The mineral substances in sewage consist principally of calcium and magnesium carbonates and sulfates, mineral constituents of the organic matter, many inorganic industrial wastes and, especially where combined sewers are used, the sand, clay and other earth washed into the sewers in times of storm. The mineral content of domestic sewage is rarely important.

Sewage Analyses.—The data given in Tables 29 to 33 are from actual analyses of sewage and are grouped according to the presence or absence of storm water and industrial wastes. They are chiefly valuable as illustrations of actual analytical results and as guides in making estimates of the probable quality of sewage from a community of known size and character. All data upon which the tables are based are not strictly comparable and there was much diversity in the methods of taking the samples analyzed. Where they were taken once or twice an hour throughout the entire 24 hours of the day, 7 days a week, for long periods of time, the averages represent closely the general character of the sewage; but where isolated samples were taken at relatively long intervals, it is uncertain whether they fairly represent the sewages. Differences in methods of analysis also have an effect upon the results and there is an ever-present difficulty in obtaining representative proportions of coarse suspended matter. It is important when interpreting analyses of sewage, to procure as complete information as possible regarding conditions affecting the composition of the sewage. This often leads to discarding analyses which otherwise would have been accepted and sometimes such information may make the results of certain analyses of unexpected value. It is desirable, therefore, when reporting analyses, to give in great detail all available data relating to the production, collection, sampling and analysis of sewage and effluents.

Tables 29 to 33 show that, even within groups of cities of presumably similar characteristics, there is an apparent variation in the quantity per capita of different constituents. Reasons for this are that it is difficult to obtain reliable estimates of the population tributary to the sewers; errors in gaging the flow may be considerable; and errors in sampling are difficult to avoid. There are also differences due to varying characteristics of the cities and of their inhabitants, which are real and not apparent. Furthermore, changes in economic conditions and in industrial processes often result in wide variations in the composition of sewage affected by industrial wastes.

The character of sewage varies according to the type of sewerage system, whether separate or combined, and according to the quantity and quality of industrial wastes discharged into it. Average analyses of sewages grouped according to these major factors are brought together in Table 34 for ready comparison. While it is somewhat hazardous to draw general conclusions from such comparative analyses, this table indicates that sewage from which industrial wastes are excluded is, on the whole, weaker, or more dilute, than that which contains industrial wastes, no matter whether the sewage is carried in separate or combined sewers. The sewage from industrial cities having separate sewerage systems and allowing industrial wastes to be discharged into the sewers appears to be the strongest. In the case of industrial cities having combined sewerage systems with industrial wastes present, the strength of the sewage is reduced somewhat, owing to the diluting effect of storm water.

Table 17.—Approximate Relative Quantities of Organic and Inorganic Matter in Solution and in Suspension[1]

	Large cities[2] with combined sewers	Combined sewers		Separate sewers	
		With industrial wastes	Without industrial wastes	With industrial wastes	Without industrial wastes
Total solids, p.p.m.	660	1,640	840	800	780
Dissolved, per cent	75	80	60	70	80
Suspended, per cent	25	20	40	30	20
Organic, per cent	45	35	50	55	50
Inorganic, per cent	55	65	50	45	50
Organic solids, p.p.m.	300	540	420	460	400
Dissolved, per cent	60	65	...	60	60
Suspended, per cent	40	35	...	40	40
Inorganic solids, p.p.m.	360	1,100	420	340	380
Dissolved, per cent	85	90	...	80	90
Suspended, per cent	15	10	...	20	10

[1] Adjusted from Table 34.

[2] Chicago, Cleveland, Milwaukee, Philadelphia and Rochester.

Suspended Matter.—In chemical analysis all solids floating or suspended in sewage are reported as suspended solids. As stated in detail in Chap. IV, the quantity of this material may be determined either by weight or by volume; in the latter case, only such solids as will settle in a reasonably short time, generally 2 hr., are reported and are referred to as settling solids. The proportion of matter floating and in suspension is generally less than one half of the total solids. Table 17 shows the relative proportions of organic and inorganic solids in solution and in suspension, in sewages of various kinds.

This table indicates the fairly uniform proportions, on the average, of the different classifications of solids in sewage, irrespective of type of sewer system and presence or absence of industrial wastes. In certain sewages, however, the proportions may vary widely from the average.

When considering analyses in which the quantities of suspended solids are given, it is well to bear in mind that it is impossible to take samples which represent fairly the coarser particles. Where suspended matter is removed from the sewage by grit chambers or sedimentation tanks, the quantity and character of the material so removed may be determined also by careful measurements and analyses of the sludge produced.

Unless great care is exercised in the selection of the point of sewage sampling, the segregation of suspended solids in horizontal strata, both in closed conduits and in open channels, may introduce serious error in the results of suspended-solids determinations.

Dissolved Matter.—The relative quantity of dissolved matter in sewages is on the average perhaps about three quarters of the total solids. It includes all substances readily soluble in water, such as those contained in urine, meat and vegetable extracts and a large variety of soluble industrial wastes, such as spent dyes, acid liquors and tanning solutions. As previously explained, the quantity of dissolved solids in the water supply also affects the composition of the corresponding sewage. In seaboard cities salt water, which contains about 36,000 p.p.m. of total solids, sometimes gains admission to the sewers and has a material effect upon the quality of the sewage, increasing its mineral content and diluting the organic matter. Of the organic matter, about one half generally is in solution.

The proportion of dissolved to suspended matter is not always constant, even in the same sewage, as chemical, physical, and bacterial action is constantly going on, converting dissolved into suspended matter and the reverse. An approximate idea of the average proportions of organic and mineral matter in solution and suspension in sewage may be obtained from Table 17. Caution is necessary in drawing general conclusions from these figures, which may be useful, however, as showing the approximate proportions of different classes of solids likely to occur in sewage.

TABLE 18.—SETTLING SOLIDS IN COLUMBUS SEWAGE, 1905

Tank	Detention period, hr.	Total suspended solids, p.p.m.		Solids settling in specified time, p.p.m.	Suspended solids removed, per cent	Volatile suspended solids, p.p.m.		Volatile solids settling in specified time, p.p.m.	Volatile suspended solids removed, per cent
		Influent	Effluent			Influent	Effluent		
Grit chamber *B*	0.3	242	188	54	22	74	62	12	16
Grit chamber *A*	1.3	196	129	67	34	87	60	27	31
Sedimentation tank *B*	6.0	211[1]	80	131	62	85[1]	30	55	65
Sedimentation tank *A*	8.0	210[1]	73	137	65	78[1]	35	43	55

[1] In influent to grit chambers, through which sewage passed before entering sedimentation tanks.

Settling Solids.—The suspended matter in sewage may be divided into two parts, that which will not settle readily when allowed to stand quiescent and that which will. The latter portion is called the "settling solids."

In 1905, Johnson (1) ascertained the quantities of suspended solids in the sewage of Columbus, Ohio, which were capable of settling in different periods of time. His results are shown in Table 18. During these experiments, the sewage flowed continuously through the tanks, so that the results are not strictly comparable with results obtained in similar periods of time with quiescent sewage. The sewage first passed through grit chambers, but the period of sedimentation in them was so much shorter than in the tanks that probably no serious error was introduced by attributing the precipitation of suspended matter to sedimentation in the period required for flow through the sedimentation tanks.

The settling solids in the sewage of Providence, R. I., as determined by Bugbee (4), are given in Table 19. It is significant that the solids capable of settling in 4 hr. amount to but 15 per cent of the total solids of the sewage, although they are equivalent to 55 per cent of the suspended matter.

TABLE 19.—SOLIDS AND ALBUMINOID AMMONIA IN PROVIDENCE SEWAGE, 1913

Determination	Parts per million	Percentage of total solids	Percentage of suspended solids
Total solids	1155.0	100	
Suspended solids	314.0	27	100
Settling solids, 4 hr.	171.0	15	55
Total albuminoid ammonia	8.46	100	
Suspended albuminoid ammonia	5.18	61	100
Settling albuminoid ammonia	2.27	27	44

At Columbus, as indicated in Table 20, it was found that only about 60 per cent of the suspended matter could be removed from the sewage by any economical period of sedimentation (1). As the period of sedimentation and length of travel of the sewage are increased, they rapidly approach points where additional time has but little effect as regards the removal of the residual suspended matter.

Table 21 summarizes information furnished to the authors by Hommon regarding the settling and suspended solids in the sewage of Atlanta, Ga. While the Gooch-crucible determinations indicate a possible removal of 81 and 51 per cent of the total suspended solids by sedimenta-

TABLE 20.—PROPORTION OF SUSPENDED SOLIDS CAPABLE OF SETTLING FROM COLUMBUS SEWAGE IN STATED PERIODS OF CONTINUOUS FLOW

Length of travel, ft.	Period of flow through tank, hr.	Suspended solids removed, per cent	Suspended solids remaining, per cent
40	0.8	35	65
80	1.7	50	50
120	2.5	55	45
160	3.3	57	43
200	4.2	60	40

tion in Imhoff tanks at the two plants, Hommon states that the percentage removal of settleable solids, as determined by the Imhoff settling glasses, is practically 100. He also states that the 60 and 56 p.p.m. of total suspended solids in the Imhoff-tank effluents are nearly all colloidal material, incapable of settling.

TABLE 21.—SETTLING AND SUSPENDED SOLIDS IN ATLANTA SEWAGE
Parts per Million

Plant	Number of samples	Suspended solids in tank influent[1]	Settling solids	Suspended solids in tank effluent[1]
Proctor Creek plant..........	250	320	260	60
Peachtree Creek plant.......	200	114	58	56

[1] By Gooch-crucible method.

The volume of solids settling out of average sewage at Worcester, Mass., in 2 hr., as determined by the conical-glass method, is given in Table 22. As there was a noticeable error in the observed volume, due to voids in the settled material, a correction, often quite large, was made by subtracting the estimated volume of voids from the actual reading. There was a marked variation in the volume of settling solids during each period, but the averages followed in a general way the concentration of the sewage.

At the suggestion of the authors, Lanphear investigated the relation of the volume of settling solids determined by the conical-glass method to the true settling solids by weight. The weight of solids settling in 2 hr. in many cases constituted a large percentage of the total suspended matter in the sewage, as determined by the filter-paper method, which is explained in part by the precipitation of colloidal matters. The tests were made with strong day sewage. The settling solids bore no definite relation to the total suspended matter and there was a much wider variation in the settling solids by volume than by weight. In the case of several samples, compacting of the sludge after 2 hr. more than offset the increase in volume owing to additional settling solids.

TABLE 22.—SETTLING SOLIDS[1] IN AVERAGE SEWAGE AT WORCESTER, BY THE CONICAL-GLASS METHOD

Period, 1913–1914	Number of samples	Corrected volume of settling solids, cc. per liter			Excess of actual reading over true volume, per cent		
		Maximum	Minimum	Average	Maximum	Minimum	Average
Apr. 21–30	37	13.50	1.55	5.44	212	1.0	32.4
June 13–25	37	29.01	3.40	10.03	132	0.5	23.2
Nov. 4–10	24	19.20	2.65	9.07	60	1.0	14.1
Feb. 17–26	45	22.52	2.90	8.72	193	2.1	44.6
Average	..	21.06	2.62	8.31	149	1.1	28.6

[1] For 2-hr. period of sedimentation.

The erratic results obtained by the conical-glass method were shown by 3 simultaneous measurements with 4 glasses for each sample. The determinations showed in many cases a wide variation in the actual readings of settling solids, but the volumes corrected for voids were much more uniform. The ratio between weight and volume of settling solids from day to day was quite variable, being several times as much in some samples as in others. The volume of settling solids determined in this way was not a measure of the volume of sludge that would be deposited in tanks, owing to the low, variable density of the settling solids in the conical glass.

Colloidal Matter.—In Chap. III colloids are stated to be substances in a sort of pseudo-solution which will not diffuse through a parchment diaphragm; for practical purposes the colloids may be held to include those fine suspended particles usually reported as suspended solids, but incapable of settling in sedimentation tanks as ordinarily operated.

		P.p.m.
Dissolved solids	Total	2710.0
Colloidal matter	Total	2260.0
	Volatile	2125.0
Oxygen absorbed in 4 hours at 80°F.	Total	547.3
	Colloids	411.6
	Non-colloids	135.7

O'Shaughnessy (5) states that 80 per cent of the dissolved matter in water vigorously shaken with fecal matter is colloidal. The analysis at the bottom of page 103 shows how great a proportion of the organic and oxygen-consuming matter is in this colloidal condition.

It is generally agreed that agitation, such as that caused by flowing rapidly through a long sewer, over steps or through screens and by pumping, changes some suspended matter in sewage into colloids. This is illustrated by an experiment at Leeds, England, cited by O'Shaughnessy (5). The average of the somewhat erratic results from three experiments showed that 30 per cent of the albuminoid ammonia was in colloidal form before pumping and 42.5 per cent after pumping; and that 14.9 per cent of the organic matter, as indicated by the oxygen consumed, was colloidal before and 17.2 per cent after pumping. The colloids in the screened sewage before and after pumping are given in Table 23.

Table 23.—Colloidal Matter in Leeds Sewage before and after Pumping

Parts per million

	Albuminoid ammonia		Oxygen consumed, 4 hr.	
	Before pumping	After pumping	Before pumping	After pumping
Test 1	2.40	4.27	21.8	19.3
Test 2	4.65	4.95	23.2	32.8
Test 3	1.47	0.87	20.0	22.4
Average	2.84	3.36	21.7	24.8

At the Philadelphia sewage experiment station, many tests were made in 1909–1910 to ascertain the quantity of organic matter in the sewage, as indicated by oxygen consumed, in suspension and capable of settling when allowed to stand quiescent in a bottle for 2 hr., in colloidal condition and in solution (6). The averages of the results obtained from August to April, inclusive, are as follows:[1]

[1] For determination of oxygen consumed, samples were acidified at room temperature and placed in water bath at 100°C. for 30 min. Colloidal matter was precipitated by Fowler's clarification method. This method is to add to 200 cc. of the sewage 2 cc. of a 5 per cent solution of sodium acetate and 2 cc. of a 10 per cent solution of ferric ammonium alum. The sample is then brought to boiling and allowed to remain over a low flame for 2 min. Then it is cooled and filtered. In this way the colloids are precipitated, leaving a clear filtrate, which may be taken for practical purposes to contain only substances in true solution. Fowler (7) states that "this method has been found to yield as instructive results as the method of dialysis, while it occupies much less time, and probably, in consequence, is less liable to error."

Oxygen Consumed	P.p.m.
Total	79.9
Settling	14.9
Colloidal	28.8
Dissolved	36.2

These determinations indicate that about 18.5 per cent of the organic matter was capable of settling in 2 hr., about 36 per cent being in colloidal condition and 45.5 per cent in solution. This sewage contained a large quantity of industrial wastes, the effect of which is indicated in part by the analytical results for fixed solids, which averaged 35 per cent higher on working days than on holidays.

Biochemical Oxygen Demand.—During the bacterial decomposition of sewage, oxygen is utilized in the conversion of complex, unstable organic compounds into simpler, stable organic and inorganic substances. The quantity of oxygen required is a measure of the strength of sewage or of sewage effluents and is indicative of the effect of such sewages and effluents upon the oxygen content of the bodies of water into which they may be discharged. The quantity of oxygen required to satisfy this demand varies with the prevailing temperature and with the period of time over which the process of decomposition proceeds. The commonly reported oxygen demand is that for a period of 5 days at 68°F. (20°C.). For ordinary purposes, it is assumed that at the end of 20 days approximately 99 per cent of the demand is satisfied at 68°F. In practice, furthermore, it is customary to compute the "total oxygen demand" on the assumption that the 5-day demand at 68°F. is equivalent to 68 per cent of the total demand. The oxygen demand of several sewages, as reported in 1931 by the Committee on Sludge Digestion of the Sanitary Engineering Division, American Society of Civil Engineers, is given in Table 24. Additional data are given in Tables 29 to 34.

The difficulties inherent in sampling and in the technique of the oxygen demand test make it necessary to apply and interpret such data with caution and judgment.

Bacteria.—A consideration of the general composition of sewage would be incomplete without reference to its bacterial content, although this has been discussed at length in Chap. III. The number increases greatly as the sewage increases in age, especially during the first day or two. If a sample is kept in a bottle or basin without dilution, agitation or other agency for changing conditions, the bacteria reach their maximum number in the course of a few days and then gradually decrease, although they by no means entirely disappear even after long periods of time.

Of the many kinds of bacteria present in sewage, the organisms of the *coli aerogenes* group, or *B. coli*, have occasioned the most study. This is due to the fact that these organisms are found in the intestines of warm-blooded animals and are present in large numbers in sewage. While these organisms are not specifically pathogenic, they resemble the bacillus causative of typhoid fever as regards length of life under varying conditions and, therefore, wherever found, organisms of the *coli aerogenes* type indicate potential presence of pathogenic organisms. The bacteriological methods for isolation and enumeration of organisms of the *coli aerogenes* group have been described in Chap. IV.

Table 24.—Data on Oxygen Demand of Sewage

Municipality	Testing period reported	Average daily sewage flow, gallons per capita	Five-day oxygen demand	
			Parts per million	Pounds per capita daily
Indianapolis, Ind.	1930	140	338	0.394
Decatur, Ill.	1930	306	154	0.392
Rochester, N. Y.[1]	1930	169	233	0.328
Philadelphia, Pa.	1923–1930	188	204	0.320
High Point, N. C.	1930	97	358	0.289
Milwaukee, Wis.	1930	121	269	0.272
Rochester, N. Y.[2]	1930	117	244	0.237
Madison, Wis.	1930	130	...	0.235
Dallas, Tex.[3]	1930	73	445	0.230
Akron, Ohio	1930	156	...	0.190
Columbus, Ohio	1930	97	226	0.182
Flint, Mich.	1930	75	...	0.175
Dayton, Ohio	1930	92	221	0.169
Canton, Ohio	1929	97	202	0.163
Worcester, Mass.	1930	92	212	0.162
Pasadena, Cal.	1930	74	257	0.159
Marion, Ohio	1930	70	...	0.157
Alliance, Ohio	1930	97	193	0.155
Springfield, Ill.	1930	112	166	0.155
Delaware, Ohio	1930	48	344	0.137
Elyria, Ohio	1930	71	171	0.101

[1] Irondequoit works.
[2] Brighton works.
[3] Five months.

The number of these organisms in sewage varies widely and seems to have no fixed relation to the total number of bacteria as determined by the agar count. As Lederer and Bachmann (8) have pointed out, the sampling error is a serious one in bacterial examination of sewage.

TABLE 25.—BACTERIAL CONTENT OF SEWAGES FROM VARIOUS CITIES
Numbers per Cubic Centimeter

City	Date of analyses	Total agar count, 24 hr. at 37°C.	*Coli aerogenes* group		Remarks
			Presumptive test	E.M.B. count, 24 hr. at 37°C.	
Allentown, Pa.	1930				
Average	....	749,000	96,000	49,000	Average for year
Maximum	....	4,350,000	1,000,000	100,000	
Minimum	....	43,000	1,000	1,000	
Baltimore, Md.	1925	4,100,000	....	1,400,000[1]	Average for year; Back River plant.
Cleveland, Ohio	1929	1,327,000	166,000	....	Average for year; Easterly plant.
Indianapolis, Ind.	1926–1930	2,400,000	190,000	....	Average for 5 years.
Rochester, N. Y.	1929	....	....	....	Sept. 4 to 6, incl.; Irondequoit plant.
Average	....	942,000	266,000	55,000	
Maximum	....	3,528,000	1,000,000+	226,000	
Minimum	....	9,000	10,000–	10,000–	
Toronto, Ont.	1925	1,627,000	....	843,000[1]	Aug. 6, 13 and 20; Leaside plant.
Wilmington, Del.	1930–1931	....	....	....	Oct. 28 and 29, 1930, and Jan. 8 to 11, incl., 1931; experimental plant.
Average	....	12,505,000	31,700	26,800	
Maximum	....	63,000,000	100,000	44,000	
Minimum	....	100,000	10,000	10,000	

[1] Acid-forming.

The averages of duplicate tests made at 1-min. intervals for a period of 10 min. in their experiments gave extreme values of 190,000 and 550,000 bacteria per cubic centimeter. The numbers of both *coli aerogenes* and bacteria growing on agar are affected by seasonal changes in temperature, being greater during the warm months than during the cold months of the year.

Bacterial counts on agar for sewages from seven different cities, together with the numbers of *coli aerogenes* determined both by the presumptive test and by the count on eosin-methylene-blue agar, are given in Table 25. The agar counts vary from a minimum of 9,000 per cubic centimeter to a maximum of 63,000,000 per cubic centimeter, averaging about 3,000,000 per cubic centimeter. The numbers of *coli aerogenes*, determined by the presumptive, lactose-broth test, vary from a minimum of 1,000 to more than 1,000,000 per cubic centimeter, averaging about 150,000 per cubic centimeter.

Effect of Temperature.—The composition of sewage is affected by its temperature, which varies considerably from season to season and depends upon geographical location. The temperature of sewage is commonly higher than that of the water supply, because of the addition of warm water from households and industries. As the specific heat of water is about five times that of air, the sewage temperatures observed are higher than the local air temperatures during most of the year and are lower only during the hottest summer months. Since most of our modern sewage-treatment works depend upon biological activity, temperature measurements are important. When the temperature is low, the activity of bacterial life is at a minimum and the sewage may contain dissolved oxygen originally present in the diluting water, as well as some nitrates and nitrites, and it may remain in relatively inoffensive condition for some time. In warm weather, however, bacteria are quite active and consequently the dissolved oxygen, as well as the oxygen of the nitrates and nitrites, is exhausted rapidly and putrefaction begins.

Data relative to mean annual and mean monthly temperatures of sewage are given in Tables 26 and 27.

The seasonal variation of free and combined oxygen, however, is not the only change due indirectly to the temperature and perhaps in many cases is not the most important. Generally, the quantity of oxygen in sewage is so small in proportion to its oxygen requirements that the presence of the oxygen may be neglected. The activity of bacteria at high summer temperatures causes important changes in the organic matter. This is shown by a decrease of the organic nitrogen and albuminoid ammonia, with an increase of free ammonia more or less closely corresponding. Suspended matter, which in winter passes to the outfall in relatively coarse condition, may be disintegrated in summer through bacterial activity so that it is in a much finer state of subdivision. Such a change may be marked where deposits of organic matter are formed in

TABLE 26.—MEAN ANNUAL TEMPERATURES OF MUNICIPAL SEWAGES

City	Temperature, °F.	Year	City	Temperature, °F.	Year
Chicago, Ill., Calumet plant	53		Plainfield, N. J.	60	1925
Gloversville, N. Y.	53	1908–1909	Columbus, Ohio	61	1909–1910
Toronto, Ont.	53		Philadelphia, Pa., Northeast plant	62	1924–1927
Schenectady, N. Y.	55	1927	Pasadena, Cal.	62	
Rochester, N. Y.	57	1927	Cleveland, Ohio, Westerly plant	63	1927
Brockton, Mass.	57	1927	Indianapolis, Ind.	67	1927
Marion, Ohio	58	1925–1927	Durham, N. C.	69	1927–1928
Albany, N. Y.	58	1920–1921	Decatur, Ill.	80[1]	
Baltimore, Md.	59	1927			

[1] Sewers receive unusual proportion of warm industrial wastes.

TABLE 27.—MEAN MONTHLY TEMPERATURES OF SEWAGE AND AIR, 1927

Month	Schenectady, N. Y.		Cleveland, Ohio	
	Air, °F.	Sewage, °F.	Air, °F.	Sewage, °F.
January	22	52	27	53
February	24	49	34	53
March	32	48	40	53
April	39	45	47	57
May	48	51	58	62
June	58	54	64	62
July	65	59	71	65
August	73	64	65	72
September	68	64	65	73
October	64	63	56	73
November	55	60	47	64
December	43	54	35	59
Average	49	55	51	63

sewers. During cold weather these deposits may accumulate in relatively large quantities, whereas, during the higher temperatures of summer, decomposition may take place so rapidly and such large quantities of gas may be formed that great masses of the previously accumulated sludge rise to the surface and float along with the sewage until the gas is liberated, after which the material is carried in suspension because of its fine subdivision.

Effect of Industrial Wastes.—The influence of industrial wastes upon the character of sewage depends upon two principal factors, namely, the relative volumes of wastes and sewage and the composition of the wastes.

The great increase in weight of certain sewage constituents, which may be brought about by the discharge of industrial wastes into sewers is illustrated by the rough estimates in Table 28.

TABLE 28.—ESTIMATED INCREASE IN WEIGHT OF SUSPENDED SOLIDS AND OXYGEN CONSUMED IN SEWAGES, DUE TO INDUSTRIAL WASTES

City	Kind of wastes	Increase in suspended solids, per cent	Increase in oxygen consumed, per cent
Akron, Ohio	Rubber reclaiming	120	140
Chicago, Ill.[1]	Packing house	460	225
Dayton, Ohio	Paper mill	60	25
Fort Worth, Tex	Packing house	65	
Fostoria, Ohio	Wire mill and carbon grinding	75	100[3]
Gloversville, N. Y	Tannery	155	115
Milwaukee, Wis.[2]	Packing house and tannery	70	10

[1] Packing-house district only.
[2] Portion of city only.
[3] Due largely to ferrous iron.

From this table it is seen that the discharge of wastes into the sewers is equivalent to increasing the population in these cities 60 to 460 per cent, or 10 to 225 per cent, as measured by the suspended solids and oxygen consumed, respectively. By increasing the load upon the plants, such increments of wastes add greatly to the cost of treating sewage. Moreover, some wastes, such as acid pickling liquors, are detrimental to biological action and may necessitate modification in plant and processes, otherwise suitable for sewage. They may also cause troublesome sludge deposits. Occasionally, however, wastes may be utilized to advantage, as at Worcester, Mass., where the quantity of iron sulfate from pickling liquors has at times been sufficient to cause excellent chemical precipitation with the addition of lime.

Large quantities of heavy oils from industries and garages have formed troublesome scum in channels and tanks. In one town, wool-scouring liquors made treatment of the sewage upon intermittent filters impracticable, owing to clogging of the sand. At Atlanta, Ga., carbide-plant wastes caused a thick scum to form on the surface of the sedimentation compartments of Imhoff tanks.

Some cities have required partial treatment of wastes before their discharge into sewers, as at tanneries, where tanks and screens have been installed for removal of suspended solids. Many cities have ordinances prohibiting the discharge into sewers of substances which may injure sewers and other structures or interfere with the treatment of sewage, but they have not been enforced generally.

A particularly striking example of the effect of industrial wastes upon the composition of sewage and its treatment is the experience at Fostoria, Ohio, which has been described by Cameron (9). Here the discharge of large volumes of wastes from a wire mill and of suspended carbon from a carbon-products mill so modified the character of the sewage as to make it impossible to treat the sewage by biological processes, until the industrial wastes were in part excluded from the sewers and were in part given adequate treatment before discharge therein.

The nature of industrial wastes varies widely, for they come from plant, animal and mineral sources. Many of them exhibit characteristics and behavior similar to those of domestic sewage. These may be mixed with sewage and treated with it satisfactorily. There are other wastes that may interfere seriously with the operation of sewage-treatment plants and outfall works. Among the varied industries producing large quantities of wastes are food-manufacturing establishments, such as stockyards, packing houses, canneries, corn-products plants, sugar refineries and breweries, and plants for tanning, wool scouring, cloth washing, dyeing and bleaching, wire drawing and galvanizing, pulp and paper making and gas manufacture.

Laboratory examination of industrial wastes reveals the fact that, as compared with municipal sewage, they possess as a class a higher content of total, settling, colloidal and organic solids, organic nitrogen and chlorides and that they exhibit much higher oxygen-consumed and biochemical oxygen-demand values. At Chicago it was estimated that in 1920 the biochemical oxygen demand of industrial wastes was equivalent to that of the sewage from a population of 1,500,000 persons, the human population of the city being 3,000,000.

At New Haven, Conn., it was found that the copper wastes from wire works and other copper-working establishments rendered the use of biologically activated processes of sewage treatment unsatisfactory. At

Chicago the wastes from paint industries, containing copper, arsenic and lead compounds, interfered seriously with the activated-sludge treatment of the sewage at the Calumet plant, until controlled. Sulfur wastes from iron works and paper mills may lead to the production of odors of hydrogen sulfide; acid wastes may affect sludge digestion adversely; oily wastes may interfere with the activated-sludge process, and gas-plant wastes may give rise to a variety of troubles.

Notes on Tables 29 to 33, Inclusive

Chicago, Ill. The analysis given in Table 29 is an average analysis of sewage received at the Calumet works during 1929 and 1930, together with an average analysis of sewage received at the North Side works from July to September, 1930, as reported by Pearse. The tributary sewerage system is of the combined type.

Cleveland, Ohio. The analysis reported in Table 29 is the average of analytical results obtained at the Southerly plant for the year 1930, as reported by Lawrence. The sewerage system is on the combined plan with many storm-water overflows. Difficulties have been experienced with pickling liquors, but otherwise industrial wastes have little effect upon the sewage.

Milwaukee, Wis. The analysis given in Table 29 is an average analysis of sewage received at the Jones Island sewage-treatment plant during the year 1927, as reported by Ferebee. The sewerage system is largely of the combined type and industrial wastes from packing houses, tanneries, steel mills and breweries are present in considerable quantities.

Philadelphia, Pa. The analysis given in Table 29 is an average analysis of the influent at the Northeast sewage treatment works for the years 1926 to 1930, inclusive, as reported by Beaumont. The tributary sewerage system is of the combined type and after the first flush of street wash, storm-water flows are diverted to water courses. The sewage contains dye liquors and oil in considerable quantities, neither of which affect the treatment.

Rochester, N. Y. The analysis reported in Table 29 is the average of analyses made at the Irondequoit plant during the years 1926 to 1930, inclusive, as reported by Ryan. The sewerage system is of the combined type, with several storm-water overflows, and industrial wastes have little effect except during the canning season.

Akron, Ohio. The analysis given in Table 30 is an average analysis of sewage received at the Botzum plant for the seven months, June to December, 1929, inclusive, as reported by Backherms. The sewerage system is of the combined type and industrial wastes from rubber-reclaiming plants are present in considerable quantities.

Decatur, Ill. The analysis given in Table 30 is an average analysis of sewage received at the treatment plant for the years 1928 to 1930, inclusive, as reported by Hatfield. The sewerage system is of the combined type and industrial wastes from starch works are present in considerable quantities.

Fostoria, Ohio. The analysis given in Table 30 is an average analysis of sewage received at the treatment plant during the months of October,

TABLE 29.—ANALYSES OF SEWAGE FROM LARGE CITIES WITH COMBINED SEWERS

	Chicago, Ill.		Cleveland, Ohio		Milwaukee, Wis.		Philadelphia, Pa.		Rochester, N. Y.		Average[4]	
Date of analyses	1929–1930		1930		1927		1926–1930		1926–1930			
Total population	3,376,000		900,000		542,000		1,919,000					
Population served	1,145,000		70,000		378,000		175,000		328,000			
Gal. daily per capita served	253		133		145		177		144		170	
	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily
Ammonia nitrogen	6.1	5.8	14.0	7.0	30.4[2]	16.7	10.0	6.7	10.7	5.8	10.2	6.3
Organic nitrogen	7.2	6.9	8.0	4.0			26.0	17.4	.....	.....	13.7	9.4
Nitrite nitrogen	.....	.....	0.8	0.4			.....	.....	.....	.....	0.8	0.4
Nitrate nitrogen	.....	.....	7.4	3.7			.....	.....	0.25	0.14	3.8	1.9
Oxygen consumed, 30 min	.....	.....	66.0	33.2	.....	.....	87.0	58.3	.....	.....	76.5	45.8
Chlorine as chlorides	34.0[1]	32.5	67.0	33.7	.....	.....	78.0	52.3	77.0	42.0	64.0	40.1
Alkalinity	157.0	150.3	188.0	94.5	.....	.....	130.0	87.2	.....	.....	158.3	110.7
B.O.D., 5 days at 20°C	91.0	87.2	163.0	82.0	239.0	131.5	155.0[3]	103.8	181.0	98.7	165.8	100.6
Iron	.....	.....	60.0	30.2	.....	.....	.....	.....	.....	.....	60.0	30.2
Fats	.....	.....	.....	.....	.....	.....	52.0	34.8	.....	.....	52.0	34.8
Total solids:												
Total	.....	.....	678.0	341.0	.....	.....	.....	.....	633.0	346.0	655.5	343.5
Volatile	.....	.....	.....	.....	.....	.....	.....	.....	290.0	158.0	.....	.....
Fixed	.....	.....	.....	.....	.....	.....	.....	.....	343.0	188.0	.....	.....
Dissolved solids:												
Total	.....	.....	494.0	248.0	.....	.....	.....	.....	497.0	271.8	495.5	259.9
Volatile	.....	.....	.....	.....	.....	.....	.....	.....	193.0	105.2	.....	.....
Fixed	.....	.....	.....	.....	.....	.....	.....	.....	304.0	166.6	.....	.....
Suspended solids:												
Total	133.0	127.3	184.0	93.0	296.0	162.8	188.0	125.9	136.0	74.2	187.4	116.6
Volatile	86.0	82.4	.....	.....	256.0	140.8	124.0	83.1	97.0	52.8	.....	.....
Fixed	47.0	44.9	.....	.....	40.0	22.0	64.0	42.8	39.0	21.4	.....	.....

[1] North Side works only.
[2] Reported as total nitrogen.
[3] Computed from the 10-day demand.
[4] Owing to incompleteness of the analytical data, the individual figures for average results are not based upon the same number of analyses for each constituent and the averages are not weighted according to flows or populations. Note especially that the sum of the average dissolved solids and average suspended solids is not equal to the average total solids in this and the following tables.

TABLE 30.—ANALYSES OF SEWAGE FROM COMBINED SEWERS WITH INDUSTRIAL WASTES

	Akron, Ohio		Decatur, Ill.		Fostoria, Ohio		Gloversville, N. Y.		Worcester, Mass.		Average[4]	
Date of analyses	1929		1928–1930		1930		1921–1922		1925–1930			
Total population	250,000		57,000		12,790		22,100		191,800			
Population served	220,000		35,000		12,000		21,000		188,800			
Gal. daily per capita served	111		322		154		145		117		170	
	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily
Ammonia nitrogen	7.9	3.3	10.0	12.2	18.8	11.0	17.5	9.6	14.8	6.5	13.8	8.5
Albuminoid nitrogen	......	......	.....	.....	.....	.....	13.2	7.2	6.5[1]	2.9	9.8	5.0
Organic nitrogen	18.0	7.6	14.0	17.1	13.5	7.8	36.2	19.9	13.6[2]	6.0	19.1	11.7
Nitrite nitrogen	......	......	.....	.....	.....	.....	......	.....	0.08	0.04	0.08	0.04
Nitrate nitrogen	......	......	.....	.....	.....	.....	......	.....	0.75	0.33	0.75	0.33
Oxygen consumed, 30 min	154.0	64.7	112.0	136.5	78.2	45.6	259.7	142.0	130.7[1]	57.9	146.9	89.3
Chlorine as chlorides	996.0	418.0	.....	.....	125.0	73.0	358.0	196.5	97.1	43.0	394.0	182.6
Alkalinity	163.0	68.5	.....	.....	316.6	184.5	195.6	107.4	.....	.....	225.1	120.1
B.O.D., 5 days at 20°C	136.0	57.1	225.0	274.3	279.0	162.6	169.0[3]	92.5	177.2[2]	78.5	197.2	133.0
Iron	......	......	.....	.....	22.6	13.2	......	.....	49.2	21.7	35.9	17.4
Fats	......	......	.....	.....	.....	.....	489.0	268.5	.....	.....	489.0	268.5
Total solids:												
Total	2465.0	1032.0	.....	.....	.....	.....	1612.0	883.0	811.0	359.0	1642.7	758.0
Volatile	803.0	336.0	.....	.....	.....	.....	492.0	269.0	325.0	144.0	540.0	249.7
Fixed	1662.0	696.0	.....	.....	.....	.....	1120.0	614.0	486.0	215.0	1102.7	508.3
Dissolved solids:												
Total	2164.0	906.0	.....	.....	.....	.....	1224.0	670.0	551.0	244.0	1313.0	606.7
Volatile	632.0	264.0	.....	.....	.....	.....	231.0	126.0	162.0	71.8	341.7	153.9
Fixed	1532.0	642.0	.....	.....	.....	.....	993.0	544.0	389.0	172.2	971.3	452.8
Suspended solids:												
Total	301.0	126.0	195.0	237.8	295.0	172.0	388.0	213.0	260.0	115.0	287.8	172.8
Volatile	171.0	72.0	.....	.....	.....	.....	261.0	143.0	163.0	72.2	......	......
Fixed	130.0	54.0	.....	.....	.....	.....	127.0	70.0	97.0	42.8	......	......

[1] Average for 1925–1927, inclusive.
[2] Average for 1928–1930, inclusive.
[3] Average of four determinations.
[4] See Table 29, footnote 4.

TABLE 31.—ANALYSES OF SEWAGE FROM COMBINED SEWERS WITHOUT INDUSTRIAL WASTES

	Bloomington, Ill.		Dayton, Ohio		Fitchburg, Mass.		Providence, R. I.		North Toronto, Ont.		Average[9]	
Date of analyses	1929–1930		1930		1924–1928		1926–1930		1931			
Total population	.....		200,982		32,338		.....		.....			
Population served	40,000		200,000		30,500		248,000		56,000			
Gal. daily per capita served	132		92		96		141		85		109	
	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily
Ammonia nitrogen	7.3	3.7	16.0[6]	5.6	12.2	4.4	11.4[3]	6.1	9.0	2.9	12.2	4.5
Albuminoid nitrogen	.....	.....	.....	.....	4.3	1.6	4.7[4]	2.5	5.5	1.8	4.8	2.0
Organic nitrogen	8.2	4.1	11.3[6]	3.9	.....	.....	.....	.....	.....	.....	9.8	4.0
Nitrite nitrogen	.....	.....	.....	.....	0.08	0.03	.....	.....	.....	.....	0.08	0.03
Nitrate nitrogen	.....	.....	.....	.....	0.89	0.32	.....	.....	0.17	0.05	0.53	0.18
Oxygen consumed, 30 min.	48.0	24.0	120.0[6]	41.8	135.6	49.2	77.0	41.2	73.6	23.7	90.8	35.9
Chlorine as chlorides	.....	.....	93.0[7]	32.4	55.4	20.2	581.0[5]	311.0	43.3	13.9	63.9[8]	22.2[8]
Alkalinity	.....	.....	407.0[7]	141.8	.....	.....	.....	.....	.....	.....	407.0	141.8
B.O.D., 5 days at 20°C.	93.0	47.0	193.0	67.2	154.0[1]	56.0	.....	.....	253.0[2]	81.5	175.7	62.9
Total solids:												
Total	.....	.....	878.0	306.0	473.0	172.0	1716.0[5]	917.0	563.0	181.0	838.0[8]	219.7[8]
Volatile	.....	.....	.....	.....	236.0	86.0	.....	.....	.....	.....	.....	.....
Fixed	.....	.....	.....	.....	237.0	86.0	.....	.....	.....	.....	.....	.....
Dissolved solids:												
Total	.....	.....	661.0	230.5	183.0	67.0	1440.0[5]	770.0	334.0	107.0	392.6[8]	134.8[8]
Volatile	.....	.....	.....	.....	110.0	40.0	.....	.....	.....	.....	.....	.....
Fixed	.....	.....	.....	.....	73.0	27.0	.....	.....	.....	.....	.....	.....
Suspended solids:												
Total	213.0	107.0	217.0	75.5	290.0	105.0	276.0	147.0	229.0	74.0	245.0	101.6
Volatile	.....	.....	.....	.....	126.0	46.0	.....	.....	181.0	58.0	.....	.....
Fixed	.....	.....	.....	.....	164.0	59.0	.....	.....	48.0	16.0	.....	.....

[1] From *U. S. Pub. Health Bull.* 132.
[2] Average of nine determinations in Jan., 1931.
[3] Computed from "free ammonia."
[4] Computed from "albuminoid ammonia."
[5] High due to sea-water infiltration.
[6] Average for 9 months, April to Dec., incl.
[7] Average for 8 months, May to Dec., incl.
[8] Excluding Providence.
[9] See Table 29, footnote 4.

TABLE 32.—ANALYSES OF SEWAGE FROM SEPARATE SEWERS WITH INDUSTRIAL WASTES

	Clinton, Mass.		Fort Worth, Tex.		Pawtucket, R. I.		Pittsfield, Mass.		Torrington, Conn.		Average[1]	
Date of analyses	1926–1930		1926–27		1928		1930		1928			
Total population	12,900		160,000		......		50,000		25,500			
Population served	12,000		160,000		80,000		45,000		25,500			
Gal. daily per capita served	125		157		116		108		270		155	
	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily
Ammonia nitrogen	30.4	14.4	31.6	18.8	13.1	5.8	18.7	7.6	7.2	7.3	20.2	10.8
Albuminoid nitrogen	14.6	6.9	.....	.....	5.7	2.5	5.9	2.4	2.0	2.0	7.0	3.4
Organic nitrogen	37.0	17.5	35.7	21.2	13.7	6.0	......	......	......	......	28.8	14.9
Nitrite nitrogen	......	.....	0.0	0.0	.....	.....	0.07	0.03	0.15	0.15	0.07	0.06
Nitrate nitrogen	......	.....	0.0	0.0	.....	.....	0.78	0.32	1.25	1.28	0.68	0.53
Oxygen consumed, 30 min.	173.7	82.1	144.0	85.7	213.0	94.0	400.0	163.0	81.6	83.3	202.5	101.6
Chlorine as chlorides	61.5	29.1	516.5	307.5	79.9	35.0	48.5	19.3	22.7	23.2	145.8	82.7
Alkalinity	......	.....	433.0	258.0	251.0	110.5	135.0	55.3	68.6	70.3	221.9	123.5
B.O.D., 5 days at 20°C.	......	.....	594.5	354.0	141.0	62.0	......	......	82.7	84.8	272.7	166.9
Iron	2.2	1.0	.....	.....	.....	.....	......	......	......	......	2.2	1.0
Fats	358.8	169.5	.....	.....	.....	.....	......	......	......	......	358.8	169.5
Total solids:												
Total	1497.0	708.0	.....	.....	819.0	360.0	615.0	251.0	269.0	274.6	800.0	398.4
Volatile	888.0	420.0	.....	.....	365.0	161.0	423.0	173.0	145.0	148.0	455.2	225.5
Fixed	609.0	288.0	.....	.....	454.0	199.0	192.0	78.0	124.0	126.6	344.8	172.9
Dissolved solids:												
Total	817.0	386.0	.....	.....	672.0	296.0	475.0	194.0	240.0	245.0	551.0	280.2
Volatile	381.0	180.0	.....	.....	286.0	126.0	299.0	122.0	117.0	119.4	270.7	136.8
Fixed	436.0	206.0	.....	.....	386.0	170.0	176.0	72.0	123.0	125.6	280.3	143.4
Suspended solids:												
Total	680.0	322.0	311.0	185.0	147.0	64.0	140.0	57.0	29.0	29.6	261.4	131.5
Volatile	507.0	240.0	.....	.....	79.0	35.0	124.0	51.0	28.0	28.6	......	......
Fixed	173.0	82.0	.....	.....	68.0	29.0	16.0	6.0	1.0	1.0	......	......

[1] See Table 29, footnote 4.

TABLE 33.—ANALYSES OF SEWAGE FROM SEPARATE SEWERS WITHOUT INDUSTRIAL WASTES

	Allentown, Pa.		Brockton, Mass.		Dayton, Ohio		Marion, Ohio		Rome, N. Y.		Average[1]	
Date of analyses	1925		1929–1930		1923		1925–1928		1930			
Total population	90,400		63,800		190,000		35,000		32,338			
Population served	10,000		63,300		12,900		23,000		30,500			
Gal. daily per capita served	100		46		55		63		164		85	
	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily	P.p.m.	Gm. per cap. daily
Ammonia nitrogen	15.4	5.8	36.2	6.3	20.8	4.3	19.0	4.5	5.3	3.2	19.3	4.8
Albuminoid nitrogen	3.2	1.2	10.8	1.9	5.6	1.2	......	.....	2.4	1.5	5.5	1.4
Organic nitrogen	7.5	2.8	......	......	12.0	2.5	12.4	2.9	9.4	5.8	10.3	3.5
Nitrite nitrogen	0.29	0.11	0.18	0.03	.....	.....	......	.....	.....	.....	0.23	0.07
Nitrate nitrogen	0.25	0.09	0.14	0.02	.....	.....	......	.....	.....	.....	0.20	0.05
Oxygen consumed, 30 min	155.0	58.7	130.1	22.7	149.6	31.0	143.3	34.2	119.2	74.0	139.4	44.1
Chlorine as chlorides	62.0	23.5	96.5	16.8	68.0	14.1	85.9	20.4	26.6	16.5	67.8	18.3
Alkalinity	225.0	85.2	......	......	358.0	74.0	390.0	92.8	110.4	68.5	270.8	80.1
B.O.D., 5 days at 20°C	188.0	71.2	......	......	177.8	37.0	193.8	46.1	87.0	54.0	161.6	52.1
Fats	......	......	......	......	46.8	9.7	......	.....	.....	.....	46.8	9.7
Total solids:												
Total	657.0	248.9	651.0	113.7	840.0	174.0	1338.0	318.0	413.0	256.0	780.0	222.1
Volatile	382.0	144.7	372.0	65.0	332.0	69.0	......	.....	223.0	138.0	......	.....
Fixed	275.0	104.2	279.0	48.7	508.0	105.0	......	.....	190.0	118.0	......	......
Dissolved solids:												
Total	518.0	196.3	425.0	74.2	668.0	139.0	1116.0	265.0	297.0	184.0	604.8	171.7
Volatile	261.0	98.9	198.0	34.6	190.0	40.0	......	.....	135.0	84.0	......	....
Fixed	257.0	97.4	227.0	39.6	478.0	99.0	......	.....	162.0	100.0	......	......
Suspended solids:												
Total	139.0	52.6	226.0	39.5	172.0	35.0	222.0	53.0	116.0	72.0	175.2	50.4
Volatile	121.0	45.8	174.0	30.4	142.0	29.0	......	.....	88.0	54.0	......	......
Fixed	18.0	6.8	52.0	9.1	30.0	6.0	......	.....	28.0	18.0	......	.....

[1] See Table 29, footnote 4.

TABLE 34.—COMPARATIVE AVERAGE ANALYSES OF SEWAGE[1]

	Parts per million					Grams per capita daily				
	Large cities with combined sewers	Combined sewers with industrial wastes	Combined sewers without industrial wastes	Separate sewers with industrial wastes	Separate sewers without industrial wastes	Large cities with combined sewers	Combined sewers with industrial wastes	Combined sewers without industrial wastes	Separate sewers with industrial wastes	Separate sewers without industrial wastes
Gal. per capita daily	170	170	109	155	85					
Ammonia nitrogen	10.2	13.8	12.2	20.2	19.3	6.3	8.5	4.5	10.8	4.8
Albuminoid nitrogen	.....	9.8	4.8	7.0	5.5	.....	5.0	2.0	3.4	1.4
Organic nitrogen	13.7	19.1	9.8	28.8	10.3	9.4	11.7	4.0	14.9	3.5
Nitrite nitrogen	0.8	0.08	0.08	0.07	0.23	0.4	0.04	0.03	0.06	0.07
Nitrate nitrogen	3.8	0.75	0.53	0.68	0.20	1.9	0.33	0.18	0.53	0.05
Oxygen consumed, 30 min.	76.5	146.9	90.8	202.5	139.4	45.8	89.3	35.9	101.6	44.1
Chlorines as chlorides	64.0	394.0	63.9	145.8	67.8	40.1	182.6	22.2	82.7	18.3
Alkalinity	158.3	225.1	407.0	221.9	270.8	110.7	120.1	141.8	123.5	80.1
B.O.D., 5 days at 20°C.	165.8	197.2	175.7	272.7	161.6	100.6	133.0	62.9	166.9	52.1
Iron	60.0	35.9	......	2.2	......	30.2	17.4	......	1.0	......
Fats	52.0	489.0	......	358.8	46.8	34.8	268.5	......	169.5	9.7
Total solids:										......
Total	655.5	1642.7	838.0	800.0	780.0	343.5	758.0	219.7	398.4	222.1
Volatile	.....	540.0	......	455.2	......	.....	249.7	......	225.5	....
Fixed	.....	1102.7	......	344.8	......	.....	508.3	......	172.9	....
Dissolved solids:										
Total	495.5	1313.0	392.6	551.0	604.8	259.9	606.7	134.8	280.2	171.7
Volatile	.....	341.7	......	270.7	......	.....	153.9	......	136.8	....
Fixed	.....	971.3	......	280.3	......	.....	452.8	......	143.4	....
Suspended solids:										
Total	187.4	287.8	245.0	261.4	175.2	116.6	172.8	101.6	131.5	50.4
Volatile	.....	....	......	......	......	......	......	......	......	....
Fixed	.....	....	......	......	......	......	......	......	......	....

[1] See Table 29, footnote 4. Remember that the various averages are not comparable, and their relative significance varies.

November and December, 1930, as reported by Cameron. The sewerage system is of the combined type and industrial wastes from a steel mill, a carbon works, an electrical manufacturing company and a slaughterhouse are present in considerable quantities.

Gloversville, N. Y. The analysis given in Table 30 is an average analysis of sewage received at the treatment plant from February, 1921, to October, 1922, inclusive, as reported by Robinson. The sewerage system is largely of the combined type with storm-water overflows and industrial wastes from tanneries are present in large quantities.

Worcester, Mass. The analysis given in Table 30 is an average of the results for the years 1925 to 1930, inclusive, as reported by Lanphear. The sewerage system is largely of the combined type with many storm-water overflows. Industrial wastes from wire mills, carpet mills, dye works, tanneries and gas works are present in considerable quantities.

Bloomington, Ill. The analysis given in Table 31 is an average analysis of sewage received at the treatment plant during the year ending May 31, 1930, as reported by Taylor and Woltmann. The sewerage system is of the combined type with storm-water overflows and the sewage contains only small quantities of industrial wastes.

Dayton, Ohio. The analysis given in Table 31 is an average analysis of sewage received at the treatment plant for the year 1930, as reported by Morehouse. The sewerage system is primarily of the separate type and small quantities, only, of industrial wastes are present.

Fitchburg, Mass. The analysis given in Table 31 is an average analysis of sewage received at the treatment plant during the years 1924 to 1928, inclusive, as published in the annual reports of the Commissioner of Public Works. The sewerage system is a combination of the combined and separate systems. Practically no industrial wastes are present.

Providence, R. I. The analysis reported in Table 31 is the average of analytical results obtained for the years 1926 to 1930, inclusive, as reported by Bugbee. The sewerage system is of the combined type with storm-water overflows. Relatively small quantities of industrial wastes are present, but there is considerable infiltration of sea water.

Toronto, Ont. The analysis given in Table 31 is an average analysis of sewage received at the North Toronto treatment plant during January and February, 1931, as reported by Harris. The sewerage system of this district is of the combined type and no industrial wastes are present.

Clinton, Mass. The analysis given in Table 32 is an average analysis of catch samples collected during the years 1926 to 1930, as published in the annual reports of the Massachusetts Department of Public Health. The sewerage system is of the separate type, though a small quantity of storm water does enter the sewers. Industrial wastes from wool scouring are present in considerable quantities.

Fort Worth, Tex. The analysis given in Table 32 is the average of analytical results on nine 24-hr. composite samples taken during the summer of 1926 and five 24-hr. composite samples taken during the winter of 1927, as reported by Mahlie. The sewerage system is of the separate type and industrial wastes from packing houses are present in considerable quantity.

Pawtucket, R. I. The analysis given in Table 32 is an average analysis of 24-hr. composite samples from five different sewer outlets, collected June 29–30, 1928, and analyzed by the authors. The sewerage system is of the separate type and industrial wastes from cotton, woolen and rayon mills are present in considerable quantities.

Pittsfield, Mass. The analysis given in Table 32 is an analysis of a 24-hr. composite sample of sewage collected April 28–29, 1930, and analyzed by the authors. The sewerage system is of the separate type and industrial wastes from the manufacture of electrical equipment, paper and woolen goods are present in considerable quantities.

Torrington, Conn. The analysis given in Table 32 is an analysis by the authors of a 24-hr. composite sample of sewage collected Sept. 25–26, 1928. The sewerage system is of the separate type and industrial wastes from the manufacture of brass and woolen goods are present in considerable quantities.

Allentown, Pa. The analysis given in Table 33 is an analysis by the authors of sewage collected from Oct. 26 to Nov. 2, 1925, from one district having separate sewers and no industrial wastes.

Brockton, Mass. The analysis given in Table 33 is an average analysis of sewage received at the treatment plant during the years 1929 and 1930, as reported by Crocker. The sewerage system is of the separate type and industrial wastes are relatively immaterial.

Dayton, Ohio. The analysis given in Table 33 is an analysis by the authors of a 24-hr. composite sample collected Oct. 9–10, 1923, from one district having separate sewers and no industrial wastes.

Marion, Ohio. The analysis given in Table 33 is an average analysis of sewage received at the treatment plant during the years 1925 to 1928, inclusive, as reported by Browne. The sewerage system is mainly of the separate type and practically no industrial wastes are present.

Rome, N. Y. The analysis given in Table 33 is that of a 24-hr. composite sample collected Sept. 9–10, 1930, as determined by the authors. The sewerage system is of the separate type and industrial wastes are absent, though ground-water infiltration is high.

Bibliography

1. Johnson, G. A.: Rept. on Sewage Purification at Columbus, Ohio, 1905.
2. Rafter, G. W., and M. N. Baker: "Sewage Disposal in the United States," 1894.
3. Water-Supply and Irrigation Paper No. 185, U. S. Geological Survey, 1906.
4. Ann. Rept. of City Engineer, Providence, R. I., 1913.
5. O'Shaughnessy, F. R.: "Significance of Colloidal Matter in Problems of Sewage Disposal," 1914.
6. *Rept. Bur. Surveys*, Comprising Work at Sewage Experiment Station, Philadelphia, 1910.
7. Fowler, G. J., Sam Evans and A. C. Oddie: *Jour. Soc. Chem. Ind.*, 1908; **27**, 205.
8. Lederer, Arthur, and Frank Bachmann: *Proc.*, Third Meeting of Ill. Water Supply Assoc., 1911, p. 145.
9. Cameron, A. B.: *Ann. Rept.*, Ohio Conference on Sewage Treatment, 1930, p. 30.

CHAPTER VI

OUTLINE OF METHODS OF SEWAGE TREATMENT AND DISPOSAL

The following outline, adapted from the *American Journal of Public Health* (1), will serve to show the relation of the common sewage-treatment and disposal methods to the character and behavior of the various constituents of sewage which have been discussed in Chap. III.

Sewage-disposal Methods

I. Disposal without treatment
- *A*. Dilution or disposal into water, including fish ponds
- *B*. Irrigation or disposal on land, including subsurface irrigation

II. Disposal after treatment by one of the following methods or a combination of these methods
- *A*. Separation of solids and liquids
  1. Floating solids and coarse suspended solids by
     - *a*. Racks
     - *b*. Screens
     - *c*. Skimming tanks, including aeration
     - *d*. Detritus tanks
     - *e*. Storm-water tanks
  2. Grit by
     - *a*. Grit chambers
     - *b*. Detritus tanks
     - *c*. Storm-water tanks
  3. Coarse and fine suspended solids by
     - *a*. Fine screens
     - *b*. Sedimentation tanks
       - (a) Plain-sedimentation tanks
       - (b) Chemical-precipitation tanks
     - *c*. Filters, contact aerators, activated-sludge treatment (see "Treatment of Liquids")
- *B*. Treatment of liquid by
  1. Oxidation through
     - *a*. Dilution
     - *b*. Irrigation
     - *c*. Sand filters
     - *d*. Contact beds
     - *e*. Trickling filters

f. Activated-sludge treatment
g. Contact aerators
h. Chlorination
2. Sedimentation following (*d*), (*g*).
3. Disinfection by chlorination

C. Disposal of effluent by
1. Dilution
2. Irrigation
3. Reclamation in the form of water

D. Treatment of solids by
1. Digestion, usually in Imhoff or separate sludge-digestion tanks
2. Dewatering on sand beds
3. Dewatering by presses, vacuum filters, or centrifuges
4. Drying of dewatered sludge in heat dryers

E. Disposal of solids
1. As fill
2. As fertilizer
3. By dumping in water
4. By incineration.

Sewage Disposal by Dilution.—The disposal of sewage by dilution is the discharge of sewage into natural waters where, if the process be successful, the sewage 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 America, it is common practice to dispose of sewage by dilution in rivers, lakes and tidal waters. The active forces of purification are physical, chemical, bacteriological and biological. The principal physical forces are sedimentation and dilution; the chemical, biological and bacteriological processes are complex and authorities are not agreed as to the exact nature of the actions or the relative importance of the various agencies. The transformation of putrescible organic substances into more stable organic matter and finally into inorganic and inert compounds, nitrates, carbon dioxide and water, is a well-known and fundamental change. The problem of sewage disposal by dilution is to bring about the change without causing offensive conditions.

In considering disposal by dilution, it is particularly important to pay attention to the suspended solids in the sewage, for while similar in composition to the dissolved substances, they are subject to somewhat different laws and actions. It is also of fundamental importance to remember that organic matter constitutes food for living organisms, as pointed out in Chap. III. Much of the suspended matter is not suitable food, being inorganic. There are probably rarely sufficient birds, fish and other food consumers present to remove all suspended food matter immediately, and a portion is either deposited about the sewer outlet or carried along by the current.

The velocity of the currents into which sewage is discharged may have an important bearing upon the results of such discharge. If sewage be discharged into a swiftly moving, relatively small stream, the suspended matter will be carried away from the community so rapidly that it will not have an opportunity to form deposits or to putrefy and cause offensive conditions within the community where it originates. When the stream reaches a lower riparian town, its slope may be flattened, with consequent reduction of velocity, thus allowing it to deposit a part of the suspended matter it has been carrying and affording time for the organic matter to decompose so as to produce offensive conditions. Such deposits may tend to annoy nearby inhabitants as well as to cause objectionable shoaling of the stream.

Sedimentation is also important because by it a substantial proportion of the organic matter of sewage may be removed from the diluting waters and retained at the bottom of deep pools in rivers, lakes and tidal waters. The decomposition of such sludge deposits is relatively slow. Thus, in some rivers sedimentation will remove the settling solids from the waters, throwing upon the rivers principally the burden of the dissolved and colloidal organic matter. In time of freshet, when there is ample volume of water and swift currents, such deposits may be scoured out and the rivers relieved of the burden of changing this organic matter to mineral or more stable organic substances. Under such conditions sedimentation in the river performs a function similar to that of sedimentation tanks, through which the sewage may be passed before its discharge into the stream.

If the proportion of sewage to diluting water is large, the bacteria may thrive to such an extent that their demand for oxygen will exhaust the available supply and anaerobic processes will set in, followed by the offensive conditions of putrefaction. In any event, such processes are likely to go on in the sludge banks formed by the precipitation of organic sewage matters. If, on the other hand, there is enough oxygen to meet the demands of the bacteria, the aerobic organisms will predominate and the organic matter will be oxidized. By bacterial oxidation the organic matter is converted into simpler compounds, some of which are suitable for plant food, and the oxidation of sewage under conditions favorable to the growth of plants is certain to be followed by such growth. Just what the functions of the plankton are under all conditions is not clear, but it is certain that the oxygen exhaled by them is an important factor in maintaining the supply of dissolved oxygen in water under certain conditions.

The effect of temperature upon sewage disposal by dilution is more important than is sometimes recognized. A river may receive in winter, without creating objectionable conditions, a quantity of sewage which, in summer, would cause it to be extremely offensive. Bacterial action at low temperatures is relatively slow and sewage may be carried by the stream receiving it to tide water, or to a point where ample dilution is afforded, before there is enough bacterial development to cause objectionable conditions. This has an important economic bearing, 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 winter conditions by providing a less complete treatment, care being taken to avoid sludge deposits which may prove objectionable during the succeeding warm season.

An ample supply of dissolved oxygen must be present constantly in natural waters receiving sewage, if putrefactive conditions are to be prevented. The two sources from which the supply can be renewed are the atmosphere and the plankton. The latter is most effective in the northeastern part of this country during August and September.

Absorption of oxygen from the atmosphere is usually the chief source of dissolved oxygen in water. It is this ability of water to absorb oxygen from the air rapidly which has maintained the purity of most ponds, lakes and oceans notwithstanding the large quantity of organic matter and other contaminating material washed into them from the surface of the earth by the natural runoff. Moreover, there is no evidence of deterioration of the quality of these waters except in isolated cases, where the digesting capacity of the water has been exceeded. The rapidity with which a water, whose dissolved oxygen has been reduced by sewage oxidation, will absorb oxygen from the atmosphere varies greatly under different conditions, as explained in Chap. VII.

Limitations of Sewage Disposal by Dilution.—In Chap. II the limitations upon disposing of sewage by discharging it into natural waters have been outlined. To prevent putrefactive conditions it is often necessary to resort to the treatment of sewage prior to its discharge. If floating matter alone is objectionable, it is necessary to remove that only. If deposits are the source of complaint, the removal of the settleable solids may be enough. Where dilution fails because of lack of oxygen, the treatment of the sewage may be carried far enough to reduce its oxygen demands to such a degree that the oxygen supplied by the natural water may meet them. It is essential, before determining the exact type of treatment to be adopted, to ascertain the requirements of the situation and the extent to which it is necessary to remove the objectionable constituents from the sewage.

Aeration of Sewage Prior to Disposal.—One method of treating sewage, which has been tried in a few cases, provides simply for the aeration of the sewage before its discharge into water, in order to increase its oxygen content and lengthen the period of time before putrefaction begins. However, the consumption of oxygen by the organic matters in the sewage goes on long after the effect of such aeration is past, for the quantity of oxygen introduced by such means is small compared with that necessary to render sewage nonputrescible indefinitely. This process seldom has been applied for the treatment of raw sewage, but occasionally has proved advantageous in conjunction with other treatment processes, to increase the oxygen in plant effluents prior to discharge into the diluting waters.

Such a process might be used in a flowing stream as an oxygen booster to carry the organic load past one or more stretches where offensive conditions might be produced in the absence of some treatment to reduce the load or increase the dissolved oxygen. A similar effect may be brought about by treating with sodium nitrate a stream which has a low oxygen content because of the discharge into it of relatively large quantities of sewage or industrial wastes. In this case the nitrate furnishes oxygen to assist the dissolved atmospheric oxygen of the stream in preventing objectionable conditions.

Impounding Reservoirs as a Substitute for Sewage Treatment.—Another method of treatment, which is applied to a stream rather than to the sewage discharged into it, is the construction of a series of impounding reservoirs in order to retard the flow of the stream and and promote self-purification. This scheme, as described by Imhoff, has been adopted to lessen the pollution of the Ruhr River in Germany (2). One reservoir, Lake Hengstey, had been completed by 1929, when it was decided to construct seven more. Upon completion of the undertaking, the time of flow of the Ruhr River between the first reservoir at Hengstey and the last one at Kahlenberg will be increased to 84 days during extreme low water, as compared with 18 days under similar conditions during 1929. The oxygen saturation, it is estimated, will be approximately 100 per cent, without making any substantial increase in the quantity of sewage subjected to biological treatment before discharge into the river. In 1929 the oxygen saturation was about 40 to 60 per cent.

Imhoff states that there are two prerequisites to the construction of impounding reservoirs as a substitute for sewage-treatment plants, as follows:

First, the municipal sewage must be freed from sludge-forming solids as far as possible in sedimentation tanks. Otherwise sludge would accumulate on the reservoir bottom and putrefy in summer. The reservoirs, therefore, may well replace biological treatment, but not sedimentation.

Secondly, in winter, the river must carry an adequate amount of water; for during cold weather natural self-purification is greatly reduced, also in reservoirs. Adequate dilution of the sewage by river water is then required. Both requirements are met in the Ruhr District.

Fish Ponds.—A method of sewage disposal involving dilution is the discharge of settled sewage into shallow artificial ponds which are stocked with fish. Reclamation of the nutritive elements contained in sewage, through the intermediate activity of bacteria, plankton and other small aquatic organisms, in the form of fish flesh was first emphasized by Hofer, Professor of Zoology in the University of Munich. The feasibility of this process must be apparent from what has been said about the succession of life in sewage-polluted streams. The difficulties encountered relate to the maintenance of a sufficiently high oxygen concentration by keeping the sewage fresh, avoiding sludge deposits and destroying surface growths which prevent re-aeration or absorb oxygen from the water; the elimination of toxic substances, such as hydrogen sulphide, and of flocculent masses of iron hydrate and the like which deposit on the gills of the fish and cause asphyxiation; and the maintenance of a biological balance that will yield adequate quantities of fish food.

This process has not been applied in the United States. As practiced in Central Europe, dilutions of 2 to 5 volumes of clean water are employed for settled sewage. The ponds are 1 to 2½ ft. deep, one acre of pond surface being provided for 800 to 1200 persons. The ponds are drained and cleaned during the winter, when ice conditions and retardation of life processes interfere with the raising of fish. The sewage is disposed of by other means during this time. Special hibernating basins are provided at Strasbourg for the plant and animal pond life. In the spring, the ponds are filled and stocked with fish. Ducks keep the ponds clear of undesirable weeds.

The fish raised are usually carp and tench, both highly prized in Europe, but not in America. When the dilution is great, rainbow trout also are stocked. A pond one acre in area is said to produce from 400 to 500 lb. of fish flesh and from 200 to 250 lb. of duck meat a year. The most notable plant in Europe is the one at Strasbourg, where the fish ponds remove 88 per cent of the organic material and 80 per cent of the nitrogen from the sewage. The effluent is clear and contains about 10,000 bacteria per cubic centimeter. A large plant is in process of development at Munich.

Sewage Disposal by Irrigation.—The alternative to sewage disposal in water, called *dilution,* is sewage disposal upon land, called *irrigation.* Both are essentially disposal processes as opposed to treatment processes, because uncontrolled, natural purifying agencies are relied upon to render the sewage matters inoffensive. Dilution makes use of the forces of purification operative in water; irrigation calls upon those active in soil. If irrigation also is classified as a treatment process, this is due chiefly to the fact that irrigation ditches and sometimes underdrains must be provided and that many sewage-irrigation plants have effluents which are discharged into streams or other bodies of water for final disposal.

There are two distinct types of irrigation projects, *sewage farming* and *subsurface irrigation.* Sewage farming, also known as *broad irrigation* and *land treatment,* dates back to the earliest days of sewerage systems. In this process sewage is caused to flow over cultivated fields or to percolate through the ground until it joins the natural ground water or passes into underdrains, incidentally watering and to some extent fertilizing the growing crops. In subsurface irrigation sewage is distributed beneath the surface of the ground and penetrates into the soil from open-jointed pipes. While sewage farming has been employed for very large communities, as well as for smaller ones, subsurface irrigation is confined to small water-carriage systems, more particularly to those of isolated dwellings, hotels, country clubs and institutions.

Irrigation with sewage in the form of sewage farming is carried out with two objects in view: proper disposal of the sewage and cultivation of crops from which revenue may be obtained. The disposal of sewage should be the primary object, in most cases, and the raising of crops secondary, controlled so as not to interfere with the purification of the sewage. In subsurface irrigation, the cultivation of crops is considered only incidentally.

Limitations of Agricultural Utilization of Sewage.—Sewage farming can compete with the so-called artificial treatment methods only when large tracts of suitable land are available at low cost, efficient management, from both sanitary and agricultural viewpoints, is assured, and water is scarce and hence valuable.

In America sewage farming seems practically restricted to the semiarid regions of the Southwest. Sewage is produced constantly, whereas the growing of crops, the prevalence of rain and other factors governing the farming requirements are intermittent. Frequently the requirements for the disposal of sewage conflict with those for the growing of crops, resulting in deficiencies in disposal. Objectionable odors may at times be created and annoyance may be caused by the breeding of flies. Sewage should not be used in the irrigation of vegetables which are to be eaten raw. The possibility of pollution of underground water must also be considered. Broad irrigation or sewage farming has been largely superseded by modern methods of sewage treatment. In some cases, this will result because of expanding areas of population which will encroach upon the sewage irrigation area. In others, it will be caused by the difficulties of disposing of increased volumes of sewage in a sanitary manner by irrigation.

Need for and Extent of Sewage Treatment.—As previously stated, there are limitations to the disposal of sewage by dilution or irrigation and, in order that sewage may be disposed of without creating objectionable conditions, it frequently becomes necessary to provide treatment by artificial means. This may involve only the removal of floating matter or settleable solids from the sewage, or some form of oxidation may be required.

Sedimentation of sewage for the purpose of separating the solid matter from the liquid seldom reduces the putrescibility of the sewage by more than one third as measured by its biochemical oxygen demand. The oxygen demand of settled sewage is exerted by organic materials that are not subject to gravitation deposition, because they are present in the sewage largely in the colloidal state and to a less extent in solution or in very fine suspension. While chemical precipitation will carry down part of this putrescible matter, experience has shown that these substances are removed or stabilized most effectively by a group of treatment processes which are variously termed "oxidation" or "biological" methods, among which trickling filter treatment and the activated-sludge process are at present most prominent. The combination of the sedimentation of sewage with some form of oxidation process is sometimes called "complete" treatment.

Hygienic considerations, such as the contamination of water supplies, natural ice, shellfish and bathing-beach waters, may require the complete treatment of the sewage and the disinfection of the treatment-plant effluent. The exact type of treatment to be adopted depends upon the requirements of the situation. In selecting the type of plant, consideration must be given to the characteristics of the sewage and of the plant site, as well as to the extent to which the objectionable constituents must be removed from the sewage.

Reclamation of Sewage.—In some sections of the country water is scarce and must be transported long distances to points where it is needed. This difficulty has been partly overcome in certain instances by subjecting sewage to complete treatment by the activated-sludge process, purifying the effluent of the sewage-treatment plant and utilizing the resulting water. Such a scheme has been in effect at Grand Canyon, Ariz., since 1926, as a substitute for hauling water a considerable distance in tank cars. Here the reclaimed sewage is used for all purposes except drinking, cooking and washing.

Since 1930 the city of Los Angeles has been demonstrating the practicability of reclaiming sewage and utilizing it for the purpose of increasing the ground-water supply. At the demonstration plant sewage is purified by sedimentation, activated-sludge treatment and filtration and is then applied to sand beds where it joins the ground water. Goudey (3) reports that frequent analyses of the purified sewage have shown it to be better than the water brought to Los Angeles by aqueduct from the high Sierras and about the same in quality as the city ground-water supply.

REMOVAL AND DISPOSAL OF COARSE SUSPENDED MATTER

Coarse suspended matter is removed from sewage by one or more of the following devices, depending upon the character of the material and the purpose of its removal: racks, screens, grit chambers, skimming tanks, detritus tanks and storm-water tanks.

Racks.—Racks are commonly used for the protection of appliances for conveying and pumping sewage and sewage sludge, the general protection of sewage-treatment plants and in conjunction with sewage disposal by dilution. They are constructed of parallel bars or rods and may be termed "fixed" or "movable," according to their characteristics, and "hand-cleaned" or "machine-cleaned," according to the method of cleaning. Racks may remove from 0.5 to 6 cu. ft. of material, called "rakings," from each million gallons of sewage treated. In conjunction with sewage disposal by dilution, the chief value of racks is the removal of large, coarse, unsightly matters, which will float on the surface of the diluting waters or become stranded on their shores.

Screens.—Screens are similar to racks, but the openings in the former, through which the sewage passes, are considerably smaller. They are commonly used for the removal from sewage of the coarser floating and suspended matters, such as cloth, paper, kitchen refuse, pieces of wood, cork, hair fiber and uncomminuted fecal solids. They are constructed of perforated plates or wire cloth, the former being most commonly used. The principal objects of screening are: the removal of matters that tend to form scum on settling and aeration tanks or on sludge-digestion tanks; the removal of solids likely to clog trickling filter nozzles or the surface of filters or irrigation areas; the removal of solids that may settle to the bottom of aeration units; the removal of coarse solids and uncomminuted fecal matters that are not readily penetrated by chlorine when sewage is disinfected; the removal of unsightly matters which will float on the surface of diluting waters or become stranded on their shores, or the reduction in the quantity of sludge settling to the bottom of slow-moving bodies of water and likely to form sludge banks or cause offense by decomposing. Screens may remove from 5 to 30 cu. ft. of material, called "screenings," from each million gallons of sewage treated and from 2 to 10 per cent of the suspended solids.

Disposal of Rakings and Screenings.—The material removed from sewage by racks and screens is unsightly and offensive. Hence it is desirable to clean these devices frequently and make prompt disposition of the rakings and screenings. The most common method of disposal is by burial, although, if buried too deeply, the screenings may remain as deposited without material improvement in their condition and, if not sufficiently covered, they may cause odors and attract rodents and flies. At Baltimore rakings are ground and returned to the sewage for treatment and disposal with the other solids.

Other methods of disposal include dumping at sea and incineration. Before incineration, it is common practice to dewater the material partially by pressing or centrifuging. Incineration of screenings, like that of garbage, must be carried on at high temperatures, to prevent the escape of odors from the stack.

Grit Chambers.—Another device for the removal of coarse suspended matter from sewage is the grit chamber. This is an enlarged channel or long tank placed at the influent end of a siphon, pumping station or treatment plant which it is designed to protect. The cross section of the grit chamber is designed so as to reduce the velocity of the flowing sewage, but only enough to cause the deposition of the heavier solids. The object is to remove the heavy mineral matter, such as sand, gravel and bits of coal and cinders, as well as the heavy, but nearly inert, organic matter, such as coffee grounds, fruit seeds, mash and similar substances, in order to protect or to secure the economical and satisfactory operation of sewerage and sewage-treatment systems.

Grit chambers may be desirable for the following purposes: to prevent clogging of inverted siphons and submerged outfall pipes by material that can be removed therefrom only with difficulty; to prevent silt deposits in bodies of water into which sewage is discharged through outfalls or storm-water overflows; to prevent injury to pumping machinery; to avoid excessive wear on mechanical devices; to facilitate the handling and treatment of sludge in sedimentation and digestion compartments; or to prevent interference with operation of diffuser plates in aeration tanks. Grit chambers provide, at relatively small cost, a satisfactory means for removing heavy mineral matter from sewage.

The material removed by grit chambers at some plants may be fairly free from organic matter and is said to have been used for walks or drives, for renewal of surfaces of sand beds or for covering more objectionable material. If the grit is offensive, it is commonly limed and covered over at the sludge dump. A plant is provided at Milwaukee for the incineration of grit with screenings, although at present it is used for the disposal of screenings alone.

Skimming Tanks.—Floating matter may be removed from sewage in skimming tanks, which are chambers so arranged that floating matter rises and remains on the surface of the sewage until skimmed off, while the liquid flows out continuously under partitions, curtain walls or deep scum boards or through submerged outlets.

The material collected on the surface of skimming tanks includes oil, grease, soap and vegetable and fruit débris, originating in household and industry. The removal of these substances, notably oil and grease, is desirable where the formation of unsightly scum or sleek on waters receiving otherwise untreated sewage is to be avoided; where similar conditions are to be obviated in the tanks of sewage-treatment works, and where heavy discharges of oil and grease would interfere with aeration in the activated-sludge process of sewage treatment or with the re-aeration of sewage-polluted waters.

Recently a short period of aeration has been used to cause cohesion and flotation of grease particles and oily substances, which then may be collected in the form of scum. The efficiency of skimming tanks is said to be materially increased thereby. The scum may be removed from the sewage in compartments adjacent to the aeration chamber. Detritus tanks, sedimentation tanks and sedimentation compartments of Imhoff tanks may be designed and equipped for the retention, collection and removal of floating matter, serving thereby as skimming tanks.

Skimmings are commonly disposed of by burial, incineration or digestion.

Detritus Tanks.—Detritus tanks or chambers usually provide a longer detention period than grit chambers, commonly with provision for removing the sediment without interrupting the flow of sewage. They are essentially settling tanks providing short detention periods, designed primarily to remove heavy settleable solids. Provision has been made at certain treatment plants in this country for drawing off 10 to 15 per cent of the average sewage flow with the settled sludge and passing this concentrated liquor through grit chambers, where the grit is separated out, then through fine screens, where the coarser suspended solids are removed, and thence returning the screened sewage to the effluent of the detritus tanks.

All of the solid matter settling in the detritus tanks may be removed and disposed of, to lighten the burden on subsequent treatment units. This is done at Indianapolis, where so-called "concentrate thickener" tanks are provided and the heavy sludge is continuously removed by mechanical means.

The material removed from sewage by detritus tanks is generally potentially offensive and should be dewatered and incinerated or otherwise disposed of promptly and in a manner to avoid causing offense. In some cases it is limed and covered with less offensive material or with fresh earth at the sludge dump.

Storm-water Tanks.—At many sewage-treatment plants provision is made for by-passing all sewage and storm water in excess of a certain quantity and discharging it without treatment into a stream. At a few plants, however, storm-water tanks are provided, in order to give the lower flows in excess of those provided for in the treatment plant a brief period of sedimentation prior to disposal by dilution. These tanks are in effect detritus tanks for storm flows, from which the settled solids are removed following the return to normal flow. The most important function of storm-water tanks is the retention of putrefactive solids, which, if discharged into a watercourse, might accumulate in sludge banks and continue to give trouble as they decompose. Such tanks serve to catch the first flush of storm water, which usually contains very large quantities of suspended solids. The solid matter retained in storm-water tanks may be disposed of in the same manner as the solids removed from detritus tanks.

REMOVAL OF FINE SUSPENDED MATTER

Classification of Sedimentation Tanks.—If sewage, partly treated sewage or other liquid containing settleable solids is retained in tanks long enough and at velocity low enough, a part of the suspended matter may be settled out. A distinction is usually made between the removal of heavy mineral solids, called grit, in grit chambers, the removal of heavy and coarse solids, called detritus, in detritus or storm-water tanks during a short sedimentation period, and the settling out as sludge of lighter, more organic, sewage solids in sedimentation tanks during a more prolonged detention period.

Sedimentation tanks may be classified as follows: *plain-sedimentation tanks*, in which sedimentation occurs without the addition of chemicals to precipitate the suspended solids and without a sufficient detention period to produce anaerobic decomposition; *chemical-precipitation tanks*, in which the sewage receives treatment by chemicals to precipitate the solids and thereby increase the quantity of suspended solids settled out; *septic tanks*, in which the sludge is in immediate contact with the sewage flowing through the tank, the sludge being retained in the tank for a sufficient period to secure satisfactory decomposition of organic solids by anaerobic bacterial action; and *Imhoff tanks*, consisting of an upper or continuous-flow sedimentation chamber and a lower or sludge-digestion chamber, the floor of the upper chamber sloping steeply to trapped slots through which solids may settle into the lower chamber, where they are retained for anaerobic bacterial digestion.

In addition to these four common types of tanks, which are used for the removal of fine suspended matter from sewage, certain other processes occasionally have been utilized to accomplish the same purpose. Among the latter may be mentioned the Miles acid process, the electrolytic treatment of sewage and the magnetite filter which is utilized at Dearborn, Mich., as an adjunct to chemical precipitation.

Plain-Sedimentation Tanks.—Sedimentation may be effected in fill-and-draw or continuous-flow tanks, the latter type being most commonly used. They are provided for the separation of settling solids from sewage, and the sludge settled out is removed from the tanks either continuously or intermittently but before anaerobic bacterial digestion is produced. Prior to disposal, the sludge may be subjected to treatment such as digestion in separate tanks and dewatering on sand beds.

The detention periods employed vary according to the purpose for which the tanks are provided, the efficiency of sedimentation required and the character of the sewage treated. Periods of 1 to 6 hr. are common, although experiments indicate that ordinarily little may be accomplished in the way of sedimentation by prolonging detention periods beyond 2 hr.

By plain sedimentation it is possible to remove from 40 to 75 per cent of the suspended matter and 30 to 35 per cent of the organic matter, as measured by the biochemical-oxygen-demand test, from sewage of average strength in 1 to 4 hr. In well-designed tanks from 80 to 95 per cent of the settleable solids may be deposited. Bacterial removal often approximates that of suspended matter.

The development of mechanical equipment for facilitating continuous sludge removal has added to the usefulness of plain-sedimentation tanks. Tanks thus equipped may be used for sedimentation alone, as at Syracuse; as preliminary-sedimentation tanks prior to the activated-sludge process, as at the North Side plant at Chicago, or prior to trickling filters, as at Springfield, Mo.; as final-sedimentation tanks in the activated-sludge process, as at the Chicago North Side plant; and as humus tanks following trickling filters, as at the Southerly plant at Cleveland.

Chemical-Precipitation Tanks.—In order to increase the efficiency of sedimentation of suspended matter and induce deposition of colloidal matter, the process of chemical precipitation sometimes is utilized. This process involves the addition to sewage of chemicals that form floc in the liquid. Flat-bottom horizontal-flow tanks have been employed commonly in connection with this type of treatment. Their design does not differ materially from that of plain-sedimentation tanks, except that provision for larger volumes of sludge may be required. The recent installations of chemical-precipitation tanks have included mechanical equipment to facilitate continuous sludge removal.

The most common substances used as precipitants are lime, alum, lime and copperas, ferric salts, sulfuric acid and sulfur dioxide. The degree of clarification obtained depends upon the quantity of chemicals used and the care with which the process is controlled. It is practicable by chemical precipitation to remove 80 to 90 per cent of the total suspended matter, 70 to 80 per cent of the biochemical oxygen demand and 80 to 90 per cent of the bacteria.

The handling and disposal of sludge resulting from chemical precipitation is one of the greatest difficulties with this method of treatment. Although the effluent may have a fairly satisfactory appearance, it is ordinarily putrescible and not comparable with the effluents produced by oxidation processes. These drawbacks, together with the expense of chemicals, have curtailed the use of chemical precipitation. Nevertheless, during the years 1932 to 1935, experiments with this process of sewage treatment have been carried on in a number of places. Since 1932 a few sewage-treatment plants using a chemical-precipitation process have been put into operation. A field of usefulness for chemical precipitation is in the treatment of industrial wastes difficult of treatment by oxidation processes.

Miles Acid Process.—The Miles process of treatment attempts, by the addition of an acid to the sewage, to precipitate the bulk of the solids in the form of a sludge which can be dried and degreased, thereby producing for sale both fertilizer and grease. An important sanitary feature of the treatment is that the acidified sewage contains few bacteria and but little suspended matter. The process has not been adopted in this country for treatment of domestic sewage, although the sewage of Bradford, England, has been treated with acid for many years. Here large quantities of grease are present in the sewage, because of the wool-scouring liquors discharged into it. The acid process is used in America for the treatment of certain kinds of industrial wastes.

Septic Tanks.—Septic tanks are essentially single-story sedimentation tanks, in which the settled sludge is allowed to remain for decomposition. The tanks are commonly of the horizontal-flow, flat-bottom or hopper-bottom type. The detention period provided allows the sewage to undergo anaerobic decomposition in direct contact with the decomposing sludge. The detention period is in general from 8 to 24 hr. The sludge should be retained in the tank for a sufficient period to secure a satisfactory digestion of organic solids, although this is difficult to obtain in such a tank because of the continuous addition of fresh solids to the accumulated sludge.

Except for small, usually private or institutional installations, in which septic tanks find wide application, this type of tank has been displaced by Imhoff tanks or by plain-sedimentation tanks combined with separate sludge-digestion tanks. Among the factors unfavorable to the septic tank are: the large quantities of suspended solids at times in the effluent; the often offensive, septic character of the effluent; the operation difficulties due to scum, odors and sludge of poor drying quality; and the greater economy of other sedimentation and sludge-digestion methods.

Imhoff Tanks.—A variation of the septic tank is a deep, two-story tank, developed by Imhoff, in which the sewage passes through the upper or continuous-flow sedimentation compartment and the settling solids fall through trapped slots into the lower or sludge-digestion chamber. The digestion compartment receives no fresh sewage directly. Theoretically, the tank carries on simultaneously and independently, by means of its two-story construction, the functions of both plain-sedimentation and sludge-digestion tanks. The Imhoff patent on this type of tank has expired.

Most of the Imhoff tanks in this country are rectangular in plan and of the horizontal-flow type. The bottom of the tank is usually constructed in the form of hoppers or troughs and the sludge is withdrawn through pipes which extend to within a short distance of the hopper or trough bottom.

Imhoff tanks have ordinarily proved to be more satisfactory than septic tanks and have been included in many recent large sewage-treatment plants. They are in competition with plain-sedimentation and separate sludge-digestion tanks. The relative advantages and disadvantages are dependent upon local conditions and requirements.

The efficiency of sedimentation in Imhoff tanks is substantially the same as that in plain-sedimentation tanks. In general, some 40 to 75 per cent of the suspended solids are removed from the sewage in the sedimentation compartment and from 30 to 35 per cent of the organic matter as determined by the B.O.D. test.

Electrolytic Treatment.—A number of proprietary processes have been promoted in which an electric current is passed through sewage with one or more of the following aims: production of a chemical precipitant for the removal of suspended solids; neutralization at the electrodes of the electrical charges of colloidal matter with resultant precipitation; reduction and subsequent oxidation of organic matter by the nascent hydrogen and oxygen produced by electrolysis of the water; deodorization of the sewage; or disinfection of the sewage.

In some early installations, precipitating chemicals were formed by decomposition of the electrodes, iron hydrate being produced, for example, by the use of iron electrodes. In one of the later processes, known as the *direct oxidation process*, lime is added to the sewage before it is electrolyzed. Disinfection and deodorizing are commonly attributed to the production of hypochlorites from the salt contained in the sewage, or, when lime is added, to the excess hydrate alkalinity.

A sewage-treatment plant employing the lime-electrolytic process has been in operation for several years at Winston-Salem, N. C.

FILTRATION AND OXIDATION OF LIQUID

Classification of Oxidation Processes.—*Oxidation* may be defined as the process whereby, through the agency of living organisms in the presence of free oxygen, the organic matter is converted into a more stable form or into mineral matter. Generally speaking, oxidation of sewage is accomplished by filtration or by the activated-sludge process.

Filtration units may take the form of intermittent filters, contact beds or trickling filters. An *intermittent filter* is a natural or artificial bed of sand or other fine-grained material, to which sewage is applied in intermittent doses and through which it percolates to underdrains. A *contact bed* is an artificial bed of coarse material, such as broken stone or clinkers, in a watertight basin, which is operated in cycles of filling with sewage, standing full, being emptied and resting empty. A *trickling filter* is an artificial bed of coarse material, over which sewage is distributed intermittently and through which it trickles to underdrains.

In the *activated-sludge process*, sewage flowing through a tank is brought into intimate contact with air and biologically active sludge previously produced by the same process. Occupying a position intermediate between filtration and activated-sludge treatment is the submerged contact aerator, or Emscher filter, in which a filtering medium that is artificially aerated is suspended in flowing sewage.

Contact Aerators.—Contact aerators may be built by submerging a crate holding veneer mats, laths, brushwood, coke or other medium in a sedimentation compartment and providing for the admission of air from below, to cause the sewage and air to flow upward through the medium and return downward on the outside of the crate. Growths which form on the medium must be removed at frequent intervals, in order to make the contact aerators effective.

It has been stated that the contact aerator may effect a 30 per cent increase in the efficiency of treatment obtained by the sedimentation unit without such provision.

Intermittent Filters.—The action of the intermittent filter may be separated into two functions, mechanical straining and transformation of dissolved organic substances into stable material. When sewage is applied to sand filters, the coarser suspended particles are retained on the surface, forming a thin compact mat, which does not allow water to pass readily. Where unsettled sewage is applied, such a mat will be formed in a relatively short time, and must be removed frequently in order to maintain the efficiency of the filter. It has come to be the custom, therefore, to pass the sewage through sedimentation tanks before applying it to filters, thus reducing the frequency of cleaning the beds. This, however, is accomplished somewhat at the expense of the porosity of the sand below the surface, for the fine particles in the settled sewage are capable of penetrating to a depth of 1 in. or more. Where unsettled sewage is applied, a portion of such matter is retained by the surface mat.

The oxidizing action of the filter depends upon the life processes of bacteria. Every particle of sewage comes into contact with the sand in such a way that a portion of the colloidal and dissolved matters may be thrown out of solution by attraction or adsorption. These substances, together with the bacterial growths or zoogloea, attach themselves to the grains of sand in a gelatinous film covering each grain. This film, while appearing to adsorb from the passing sewage dissolved and colloidal matter, is also the home of bacteria which feed upon and break down the complex adsorbed organic matter and transform it into stable substances.

The results obtained by the intermittent filtration of ordinary municipal sewage leave little to be desired. The effluent from a well-designed and carefully operated plant is usually practically clear, free from suspended matter, nearly colorless and without odor, and contains little organic matter and but an extremely small part of the bacteria applied to the filter. It is not to be inferred, however, that the effluent from an intermittent sand filter is suitable for domestic consumption. It is possible that at times some of the bacteria of the sewage pass through the sand bed and such effluent must be treated further by disinfection if it is necessary to reduce the danger of the transmission of such organisms.

The process has been found generally applicable only in localities having natural deposits of sandy soil. Because of the extensive filtration areas required, it is not suitable for large cities but will probably continue to be used successfully by many small communities and by institutions. Intermittent filtration is generally considered to be more reliably efficient than other methods of treatment, especially for small plants where skilled supervision is not available.

In some instances intermittent filters have been adopted for the treatment of effluents from contact beds, trickling filters and activated-sludge units. In such cases the rate of dosing the beds may be greatly increased and the area of beds required may be comparatively small.

Contact Beds.—Contact-bed treatment consists in applying sewage to a watertight tank filled with broken stone, cinders, coke or other inert substances, commonly ½ to 1½ in. in size and about 4 ft. deep. In England the 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 in many cases the contact material has had to be removed, cleaned and replaced after a service of about 5 years. Such beds are often built in series of two or three; the effluent from the first or primary bed passes to the second and from the second to the third, being improved in quality by each successive treatment.

Contact beds are filled with sewage, allowed to stand full, emptied and allowed to rest. The cycle is then repeated. Schedules vary, according to the design and the rate of sewage flow, for the time of resting depends upon the number of times a bed is filled each day. The following schedule may be assumed to illustrate such a cycle with three fillings per day:

Time of filling	1.0 hr.
Time of contact	0.75 hr.
Time of emptying	0.25 hr.
Time of resting	6.0 hr.
Time of cycle	8.0 hr.

While sewage is commonly settled before being applied to contact beds, it still retains a portion of its settleable solids and much fine suspended and colloidal matter. In the contact bed, the colloids are withdrawn from the sewage by the physical attraction of the contact material and the settling solids are deposited and largely retained in the interstices of the stone. Certain sewage bacteria find a favorable habitat in the organic matter attached to the stones and doubtless increase its quantity by their rapid growth, thus forming a gelatinous film adhering to the contact material. Here the organisms thrive and convert the organic matter into soluble stable organic substances, into mineral matter, into gases which either pass out of the filter dissolved in the effluent, or are liberated into the atmosphere, and into humus-like solids, some of which are washed out of the bed; the remainder accumulates until the contact material must be removed, cleaned and replaced.

The efficiency of contact beds varies greatly with the many factors of design and operation which influence the degree of purification provided. In general, it may be said that from 60 to 80 per cent of the organic matter and 50 to 75 per cent of the bacteria are removed. The effluent of coarse-grained beds must undergo final sedimentation for the removal of suspended matter, if a high degree of efficiency is to be secured. The effluent from fine-grained contact beds is fairly clear and stable.

Contact beds have seldom been used when the population to be served is much in excess of 5000. They are rarely retained when plants are enlarged, being superseded for the most part by other processes in which it is possible to secure a higher rate of treatment and degree of purification. However, they continue to serve a limited field of usefulness for small plants, as they require less head than trickling filters and provide less opportunity for the dissemination of odors.

Trickling Filters.—A trickling filter is an artificial bed of coarse material, commonly broken stone or clinkers, over which sewage is distributed in drops, films or sprays, from troughs, drippers, moving distributors or fixed spray nozzles, and through which it trickles to the underdrains. As in the case of contact beds, the filtering material of trickling filters becomes coated with a gelatinous film, inhabited by certain bacteria. Suspended and colloidal matters collect upon this film and by bacterial action, with the aid of atmospheric oxygen, reduction of the organic matter takes place rapidly. The resulting more stable solids created in the bed leave the filter periodically and final-sedimentation tanks are generally required, in order to remove such solids from the filter effluent. The sludge from these tanks is not so offensive as that from tanks in which suspended matter from raw sewage has been deposited and under the most favorable conditions may have an odor resembling that of garden mold. This has led to the application of the term "humus tanks" to sedimentation tanks following trickling filters.

To reduce the clogging of distributing nozzles and to prevent the voids in the filter medium from being filled with solid matter, preliminary treatment by sedimentation is commonly employed.

Filter depths generally provided are from 5 to 10 ft. The design is sometimes based on providing one acre of filters to serve each 20,000 persons, the range being roughly from 10,000 to 40,000 persons an acre. The rate of dosing trickling filters can be greatly increased if preliminary partial oxidation is provided in conjunction with sedimentation.

The degree of purification effected by trickling filters, as in the case of most other sewage-treatment processes, is influenced by many factors of design and operation. In general, the reduction of organic matter is 60 to 85 per cent, and of bacteria 70 to 85 per cent. The trickling filter is capable of converting putrescible sewage into highly stable effluent with low B.O.D. The settled effluent of trickling filters may be treated on sand filters at relatively high rates, to reduce further the organic matter and bacteria and to produce a well-clarified final effluent.

Trickling filters have largely superseded contact beds as a method of treatment but are now in competition with the activated-sludge process, which ordinarily produces a clearer effluent and is usually less expensive to construct. Trickling filters, however, cost less to operate, exclusive of pumping, and require less skill in operation. Among the difficulties experienced with the operation of trickling filters are clogging of the filter media, clogging of nozzles, dissemination of odors during the spraying of sewage on the beds and frequently the prevalence of a small, gray moth fly, *Psychoda alternata*, about the filters. As a rule, however, trickling filters have established a reputation among sewage-works operators for successful operation.

Activated-sludge Process.—The activated-sludge process consists essentially of agitating or stirring the sewage, in the presence of abundant oxygen and of sludge which has settled out of sewage previously passed through the activation or aeration tanks.

When an activated-sludge plant is first placed in operation, the sewage is passed through the aeration tanks and then through sedimentation tanks, where the sludge is allowed to settle. This sludge is discharged into, and mixed with, the sewage entering the aeration tanks.[1] After a

[1] The sludge thus introduced into the sewage is known as "return sludge."

period of four to six weeks, the sludge settling out after aeration will be changed in character from ordinary sewage sludge to a flocculent form, favorable to bacterial development, and when this sludge is introduced into the incoming sewage and the mixture is agitated in the presence of free oxygen, the sludge and impurities in the sewage gather together in small flocs, which readily settle out in the final-sedimentation tanks. Thus the solids and impurities of the incoming sewage are "seeded," or rendered biologically active, to secure rapid purification when aerated with the incoming sewage. Once this biologically active sludge—from which the process gets the name of activated-sludge—is well developed, a high degree of purification of the sewage is secured and the final effluent should be clear and nonputrescible. Thereafter a portion of the activated sludge, amounting to perhaps 25 per cent of the sewage flow, is added to the sewage as it enters the aeration tanks. The resultant "mixed liquor" is aerated for a certain interval of time, commonly 4 to 6 hr., and then discharged into final-sedimentation tanks, where a detention period of 2 to 3 hr. is often provided.

Aeration is usually accomplished in this country by discharging compressed air into the bottom of the tank, whence it passes up through the liquid, although mechanical means for agitating the sewage have been installed at some of the smaller plants. In such cases the supply of oxygen is absorbed from the atmosphere. In England, such aeration, as well as the agitation, is frequently accomplished by revolving paddles or other mechanical means. A combination of aeration and agitation by means of compressed air and mechanical equipment is sometimes employed.

Preliminary treatment of the sewage is generally provided prior to aeration. Usually grit and other heavy material must be removed from the sewage to be treated, because these substances are not readily maintained in suspension and tend to accumulate on tank bottoms. Furthermore, it is best to keep solids which are large in bulk, as well as floating matters, out of the aeration units, as they do not respond readily to treatment. It may be desirable to remove the settleable solids from the sewage prior to aeration. Therefore, depending upon local conditions, one or more of the following preliminary devices may be installed: racks, grit chambers, fine screens, skimming tanks, sedimentation tanks and contact aerators.

Sedimentation tanks following the aeration tanks are a necessary part of the activated-sludge plant. Where the sewage receives only partial treatment by the process, the settled effluent may be oxidized further by means of sewage filters.

A high degree of purification may be obtained by the activated-sludge process. In fact, it is possible to remove from 90 to 95 per cent of the suspended solids and to reduce the B.O.D. in about the same proportion. It is possible with this process to reduce the bacteria in sewage by more than 99 per cent.

The degree of purification obtainable under favorable conditions usually is slightly greater than by trickling filters, with which this process is in competition, and the effluent is usually of better appearance. More skill is required in the operation of the activated-sludge plant. There is, however, less danger of producing objectionable odors and no danger of annoyance from flies such as sometimes attends the operation of trickling filters. Economically, the activated-sludge process utilizing the diffused-air method of agitation is most practical where unit costs of power are low.

DISINFECTION AND CHLORINATION OF LIQUID

Disinfection, which is the partial destruction of the microorganisms likely to cause infection and disease, may be accomplished by means of heat, chemicals or actinic rays, also called ultraviolet light. As far as municipal sewage treatment is concerned, chlorine and its compounds are at the present time the only practical means of securing economical and adequate results. The application of these chemicals to sewage or water is called *chlorination*.

Chlorination of sewage or effluents may be adopted to prevent contamination of water supplies and bathing-beach and shellfish waters. In addition to disinfection for such objects, sewage may be treated with chlorine or bleaching powder for the purpose of the retardation of decomposition, the reduction of oxygen demand or the control of odors. Although chlorine may be applied primarily for one or more of the purposes mentioned, all are accomplished to some extent.

Liquid chlorine now is generally employed in large or permanent installations. Bleaching powder, on the other hand, may be employed where sewage is to be chlorinated on a small scale or as a temporary expedient.

Disinfection by chlorination is accomplished by cell destruction, death of the organisms probably being due to a combination of chlorine with the cell contents. In the chlorination of sewage, the unstable organic matter exerts a marked and rapid chlorine demand, which must be satisfied before disinfection may be expected. The time required for disinfection is relatively short, but a suitable period must be provided in order to secure effective chlorination of sewage. Disinfection of raw sewage is somewhat unreliable. The grosser solids are not penetrated by chlorine and it is only after these matters are removed from the sewage that chlorination can be given a definite efficiency rating. The efficiency of chlorination in removing bacteria from settled sewage may be 90 to 95 per cent and from oxidized sewage, 98 to 99 per cent.

The deodorizing effect of chlorine in sewage treatment is due to its affinity for hydrogen sulfide, H_2S, and other substances which are oxidized easily and quickly. A chlorine dose is required which is somewhat in excess of the quantity theoretically necessary to combine with any hydrogen sulfide which may be in the sewage.

Reduction in biochemical oxygen demand by the use of chlorine, resulting from the oxidation of some unstable sewage matters, may be of importance in preventing offensive conditions downstream from treatment works, discharging their effluents into small rivers, which provide adequate dilution only after accession of other waters or after emptying into larger bodies of water.

TREATMENT AND DISPOSAL OF SLUDGE

Volume of Sludge Produced.—The problem of the treatment and disposal of the sludge produced by various sewage-treatment processes is of considerable magnitude in treatment works of other than small communities. Sludge as removed from raw sewage is made up in large part of substances which are responsible for the offensive character of untreated sewage and these substances, if not effectively handled, treated and disposed of, may be the source of much difficulty and may produce objectionable conditions.

The volume of sludge varies with the strength of sewage and the efficiency and type of treatment. Roughly, the quantity produced from each million gallons of sewage treated may be taken as 3000 gal. by plain-sedimentation tanks, 700 gal. by humus tanks following trickling filters, 5100 gal. by chemical-precipitation tanks, and 19,400 gal. by the activated-sludge process. These sludges contain large proportions of water, ranging from 92.5 per cent for humus sludge to 98.5 per cent for activated sludge. The approximate weight of solids in the sludge for each million gallons of sewage treated may be taken as 500 lb. for trickling-filter humus, 1200 lb. for sludge produced by plain sedimentation, 3300 lb. by chemical precipitation and 2200 lb. by the activated-sludge process.

Methods of Sludge Treatment and Disposal.—Sludge is sometimes disposed of in the watery, putrescible state in which it is produced. Common methods of disposal of untreated sludge are: dumping into large bodies of diluting water, as at sea, and disposal on land in trenches or lagoons.

Frequently, however, sludge is subjected to some sort of treatment, in order that it may be disposed of more easily and with less danger of creating offense. Common methods of sludge treatment to facilitate its disposal include: reduction of the organic matter by digestion either in Imhoff or separate sludge-digestion tanks; dewatering of digested or undigested sludge, with or without conditioning of the sludge by chemicals, either by draining and drying on beds of sand or other porous material, filtering with or without pressure or vacuum, dewatering in a centrifuge, by flotation or spray drying; and further drying of the dewatered sludge by heat.

After dewatering, sludge generally is used for filling waste land or for fertilizer, although quite recently the feasibility of sludge incineration has been under investigation. Sludge which has been artificially dried by heat is utilized as fertilizer or as a fertilizer base.

Sludge-digestion Process.—By means of biochemical agencies the organic matter in sewage sludge may be gasified, liquefied, mineralized or converted into more stable organic matter. If the processes of decomposition are carried on to relative completion, the resultant sludge is rendered inoffensive, is considerably reduced in bulk, can be dewatered and dried more readily and is more suitable for use as fertilizer, as compared with the untreated sludge. In treatment works, sewage solids may be submitted to digestion either while remaining in contact with the flowing sewage, as in septic tanks, or after separation from the flowing sewage, as in Imhoff and separate sludge-digestion tanks. Normal reduction in the weight of sludge solids by disgestion is from 30 to 40 per cent. This is effected, however, entirely by the reduction of organic solids which may be reduced by 60 to 80 per cent.

The rate of digestion is greatly influenced by the temperature of the sludge. It requires about 50 per cent more time to digest sludge at 70°F. than at 85°F. and about twice as long a time at 50°F. as at 70°F.

The gas which results from the processes of sludge digestion amounts to about 8 cu. ft. per pound of organic solids added to the digestion tanks and 0.3 to 1.0 cu. ft. a day per capita served. It has a nominal heating value of 500 to 800 B.t.u. per cu. ft.

Digested sludge as drawn from tanks commonly has a moisture content of 90 to 95 per cent.

Sludge Digestion in Imhoff Tanks.—In Imhoff-tank installations sludge is added to the digestion compartments by gravitational deposition from the sewage flowing through the upper or sedimentation compartments. In some plants humus sludge from sedimentation tanks following trickling filters or excess activated sludge is introduced directly to the digestion compartments of Imhoff tanks for digestion with the primary sludge.

The temperature of sludge in the digestion compartment closely follows the temperature of the sewage in the settling compartment. In large areas of the United States, low sewage temperatures prevail during the late fall, winter and early spring months and the rate of sludge digestion is greatly retarded at such times. Provision for sludge storage for 5 to 7 months is generally made under these conditions.

It is not practicable to heat the digestion compartments of Imhoff tanks. The gas, however, is collected at some plants and utilized as fuel for the incineration of screenings, for the operation of gas engines or for heating various parts of the plant.

Chief among the difficulties encountered in the operation of Imhoff tanks are those rather directly connected with the sludge problem, such as foaming, production of odors, choking of slots and sludge pipes, failure of sludge to move toward withdrawal pipes, production of sour or acid sludge and excessive accumulation of scum. Of these, foaming has perhaps caused the greatest difficulty and is the hardest to control.

Separate Sludge-digestion Tanks.—In plants where plain-sedimentation tanks are installed, sludge from such tanks may be discharged into separate tanks for digestion. The digestion process in these tanks is similar to that which takes place in septic tanks and in the lower chambers of Imhoff tanks.

There are several advantages in the use of separate sludge-digestion tanks. By heating such tanks the sludge may be kept at temperatures favorable to rapid digestion, thus effecting a material saving in sludge-storage capacity over that required in Imhoff tanks where the rate of digestion is retarded by cold weather. Heated separate sludge-digestion tanks are especially advantageous in conjunction with mechanical dewatering, permitting the operation of the plant at a fairly uniform rate throughout the year, with the result that the necessity for storage capacity is practically eliminated. Separate digestion tanks may be placed at the elevation desired and the sludge pumped to them, thus avoiding the expense and construction difficulties sometimes incident to the great depth of Imhoff tanks, all of which must be below the flow line in the sedimentation compartment. Since by the use of separate tanks digestion capacity may be provided irrespective of sedimentation capacity, such tanks are advantageous where digestion is required in conjunction with existing sedimentation tanks or where existing facilities for sludge digestion have been outgrown while sedimentation is still adequate.

In case the tanks are heated, they are commonly roofed, the gases of decomposition being collected and utilized for heating the tanks. Both fixed and floating roofs are in common use.

Mechanical equipment for stirring the sludge, for breaking up scum layers and for facilitating the withdrawal of sludge is sometimes provided in these tanks.

Sludge-drying Beds.—In this country, the most common method of dewatering wet sludge, either untreated or as drawn from digestion compartments, is by drainage and evaporation on specially prepared sand beds. The sludge becomes dry enough for handling and is commonly removed when its moisture content is reduced to 50 to 70 per cent.

The sludge-drying area is usually divided into several units depending upon the size of the plant and the method of cleaning. The width of the unit when sludge is removed into dump cars running on industrial tracks is often 15 to 20 ft. The drying beds are usually constructed of graded sand and gravel with a depth of 18 to 24 in. Beds are commonly dosed with sludge to a depth of 8 to 12 in.

The dewatering of sludge on open drying beds is greatly influenced by climatic conditions. In the case of Imhoff sludge, which is ordinarily digested to a satisfactory degree for drying only during the warmer months of the year, the necessity of protecting the beds from the influence of climatic conditions may not be urgent. In the case of heated separate sludge-digestion tanks, however, in which digested sludge is being produced uniformly throughout the year, the provision of housed drying beds may be desirable.

Covers for sludge-drying beds are usually constructed of glass, like a greenhouse. Some degree of protection against the spread of odors is claimed for covered sludge beds.

Dewatering Sludge by Pressure or Vacuum Filters.—Prior to dewatering by filters, sludge is usually conditioned, so as to cause the smaller particles to unite and form aggregates, or large particles, which will be retained upon the surface of the filtering medium and allow the water to pass through. The following methods, either individually or in combination, are generally used for conditioning sludge: changing the H ion concentration of the sludge; adding chemical coagulants to the sludge; heating the chemically conditioned sludge; and adding absorbents, such as diatomaceous earth, to the sludge.

There are three types of filters in which conditioned sludge may be dewatered: chamber or leaf-filter presses, in which the sludge is put under pressure, the water being forced through cloth, such as heavy army duck, on which the sludge solids are retained, forming a cake; bag presses, in which bags filled with the liquid sludge are squeezed between drainage sheets by large platens; and vacuum filters, in which the water is drawn through cloth by suction and the solids retained form a thin cake.

Of these, the vacuum filter is most commonly used. The filter cloth is mounted on a drum which revolves partially submerged in the liquid sludge. A layer of sludge is picked up on the cloth by suction as the drum revolves. The suction is increased to extract the water, the filtrate passing through vacuum pipes to a receiver. The dewatered cake is loosened from the cloth by substituting air pressure for suction within the drum and the cake is then deflected by scrapers on to conveyors which carry it away. In this manner the water in sludge may be reduced from an initial content of 95 to 99 per cent to 75 to 85 per cent.

Other Mechanical Methods of Dewatering Sludge.—Other methods of dewatering sludge include dewatering in a centrifuge, flotation and spray drying.

In this country the centrifugal separation of sludge liquor and solids has been tried only on an experimental basis, although several plants in Germany have had centrifuges in operation for a number of years. Experiments with this method of dewatering have been carried on at Baltimore, Md., Chicago, Ill., Collingswood, N. J., and Milwaukee, Wis., among other places.

Another method of dewatering which has been studied is flotation, in which the sludge solids are coagulated and brought to the surface for ready separation from the sludge liquor. Flotation is accomplished by the use of either heat or chemicals or both. Recently a flotation process has been used at Plainfield, N. J., to treat and concentrate digested sludge preliminary to spray drying.

In the latter process, sludge is fed into a centrifugal spray machine in a heated chamber and is atomized in a horizontal plane near the top of the chamber. The dried product falls to the floor and is removed by a revolving rake. The exit gases, laden with moisture and a certain amount of dust, are carried out of the drying chamber into a cyclone dust collector. The Plainfield installation is arranged so that the dried sludge may be delivered either to the dust collector or to a furnace, where it burns as pulverized fuel.

Heat Drying of Sludge.—When sludge is to be prepared for sale as fertilizer or fertilizer base, it must generally be dried to less than 10 per cent moisture content. Rotary heat dryers, such as are used in many industrial operations, are generally employed to drive off the excess moisture from sludge that has been subjected to primary dewatering. The hot, moisture-laden gases from the dryers may carry offensive odors over long distances. Treatment of these gases may be necessary in order to eliminate odors, methods which have been used for this purpose being washing, condensing and chlorination.

Incineration of Sludge.—In an attempt to utilize the calorific power of sludge in its disposal, the burning of sludge was tried at a few places in the United States in the early days of sewage disposal, notably at Worcester, Mass., and at Coney Island, N. Y. On account of the expense involved in the construction and operation of incinerators, however, as well as the danger of disseminating offensive odors unless incineration is carried on at uniformly high temperatures, this method of sludge disposal has not been employed to any great extent.

Because of the expense of sludge-storage tanks or glass-covered beds for drying sludge, studies have been made during the past few years to determine whether partially or wholly digested sludge can be dewatered mechanically by vacuum filters and destroyed in incinerators, thus eliminating both storage tanks and sand beds. A large test plant, with a capacity of 20 to 30 tons a day, was put in operation at Chicago to carry on such investigations. At the end of 1933 this incinerator plant was replaced with one of a different type, which has been in more or less continuous operation for a number of months in 1934 and 1935. Incineration has been adopted in the plan for sludge disposal at the Calumet sewage-treatment works now under construction in the Sanitary District of Chicago.

Disposal of Sludge in Water.—The disposal of sludge by carrying it out to sea and dumping it in deep water is practiced at Providence and by the sewerage district of the Passaic Valley, N. J. Treatment of the sludge prior to disposal is not essential, except for the purpose of reducing the volume to be handled. The sludge is discharged into scows, which are towed to the disposal area and dumped.

At Cleveland, Imhoff sludge is pumped into Lake Erie and several American communities discharge sludge into inland streams when they are in flood.

Disposal of Sludge on Land.—Dewatered sludge, digested or undigested, is most commonly disposed of in sludge dumps on low lands or waste areas. It is used to some extent as a low-grade fertilizer.

Wet sludge may be disposed of by discharging it into lagoons, running it into trenches which are covered after being filled or flowing it on land in thin layers. A lagoon is a basin formed by a natural depression, often a sand pit, clay pit, or quarry, or by surrounding a tract of land with a dike of earth. The sludge is left to dry by evaporation and such losses of water through the soil as may naturally take place. Partial digestion and drying of the sludge occur over a long period of time, frequently accompanied by offensive odors when undigested sludge is disposed of in this way.

COMMON COMBINATIONS OF SEWAGE-TREATMENT PROCESSES

Governing Factors.—A number of different conditions govern the choice of sewage-treatment methods for any given municipality, sewerage district or sanitary district. Among them may be listed the following: the character and volume variations of the sewage to be treated, taking into account the concentration, composition and condition of the sewage when it reaches the treatment and outfall works; the required degree of purification with respect to the ultimate disposal of the sewage by dilution or irrigation and the satisfactory disposal of the sewage sludge; the practicability of obtaining sites for the treatment works and the conditions imposed by different sites, both as regards treatment methods and ultimate disposal of sewage and sludge; and the relative cost and efficiency of different treatment and disposal methods.

The important bearing of these conditions upon sewage treatment has been touched upon in preceding chapters. It is well to point out at this time some of the intimate relations which exist between the treatment works and the remaining parts of the sewerage system.

In a measure, for example, the arrangement of the collecting system dictates the choice of the treatment methods and vice versa, because the treatment processes must be adapted to the requirements of the disposal site and to the facilities for ultimate disposal of sewage and sludge. The nature of the collecting system, furthermore, affects the composition, concentration and condition of the sewage delivered to the treatment works. Treatment methods may vary somewhat for combined sewage and separate sewage, and for fresh and stale sewage. Different disposal sites may call for different plans of sewerage and sewage treatment.

In a large city two or more treatment plants may be cheaper and better than a single one. Where land is expensive, certain processes may become economical which would not be financially practicable where land is cheap. One of the reasons for this is illustrated in the following schedule prepared by the authors in a study of three different types of treatment works for a city of 600,000 persons.

	Area and volume of treatment plant	
	Acres	Million cu. ft.
1. Sedimentation tanks, sludge beds and intermittent sand filters	800	140
2. Imhoff tanks, sludge beds and trickling filters	60	17
3. Activated-sludge process, mechanical dewatering and drying of sludge	10	5

In preparing estimates of cost, the provision of land for extensions to meet future requirements must receive attention. This is particularly important where the treatment plant includes intermittent filters or structures requiring a large area in proportion to their capacity or where it is probable that the degree of purification effected by the plant must be increased with the lapse of time. Overworking treatment plants which it is impracticable to enlarge usually results in offensive conditions and generally plans for sewage treatment should be adopted which allow for a suitable increase of capacity during the period for which the general plan provides. This period may be from 25 to 40 years, depending on local conditions. This may not always be feasible, however. For example, it may be reasonably certain that clarification of sewage by sedimentation, with disinfection of the effluent, will be a satisfactory treatment for 25 years and enough land may be available near the city for a plant adequate for the requirements during that period, but no longer. Such a plant may prove more economical than the construction of a trunk sewer to a site much farther away but large enough for the ultimate plant of several times the largest possible capacity of that nearer the city. It is true that the plant close at hand may be serviceable for only a comparatively short period, but its total cost during that period, including amortization of such portions as have to be abandoned, may be less than the total cost during the same period of the works required for the more distant site. It is hardly possible to estimate closely the future requirements of a city for more than 25 to 40 years and in such a period of time sewage-treatment methods may progress in many ways. Hence it is generally unwise to plan works "for all time."

Common Combinations of Processes.—Of the various processes of sewage treatment, the following may be considered as common major methods, which may or may not involve other processes of sewage and sludge treatment and disposal:

1. Fine screening
2. Sedimentation
3. Intermittent sand filtration
4. Trickling-filter treatment
5. Activated-sludge process

With the many types of treatment processes available and the variety of conditions and requirements to be met, it is only natural that the works of different communities should vary appreciably in the combination of treatment processes employed. The following pages contain a few examples of typical American sewage-treatment plants.

Fine-screening Plants.—Of the treatment plants in which fine screening is the major process provided, those at Long Beach, Cal., and at Bridgeport, Conn., may be taken as examples. Both plants provide fine screening of the sewage prior to discharge through an ocean outlet. At Long Beach the screenings are incinerated and the resulting ash is buried. At Bridgeport the screenings are buried without other treatment.

Sedimentation Plants—The treatment works at Syracuse, N. Y., Newark, N. J., Dayton, Ohio, and Erie, Pa., may be taken as examples of the sedimentation process with different types of plants.

At Syracuse the sewage passes through grit chambers and through both coarse and fine racks to plain-sedimentation tanks. The settled sewage is discharged into Onondaga Lake for disposal by dilution. The sedimentation tanks are mechanically equipped to facilitate sludge removal and the sludge is disposed of by pumping into the waste lagoons of the Solvay Process Company. A plan and longitudinal profile of the Syracuse plant are shown in Fig. 7.

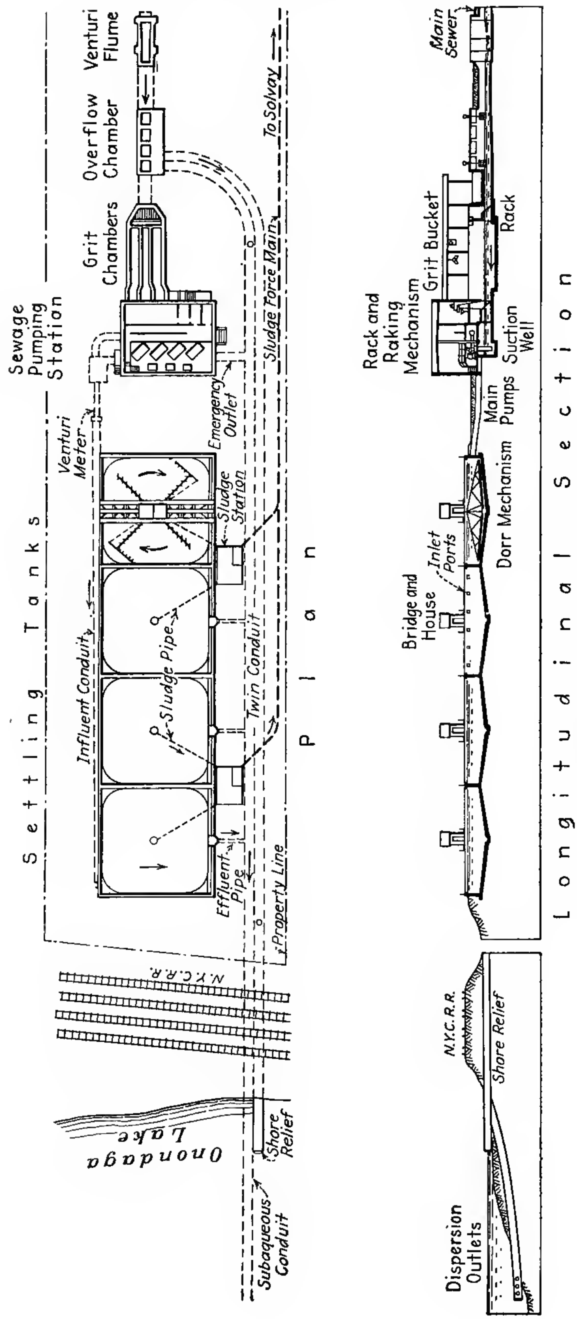
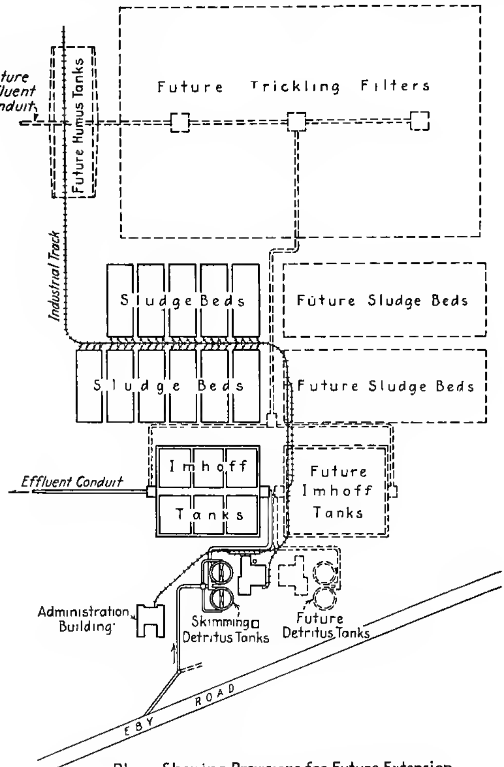
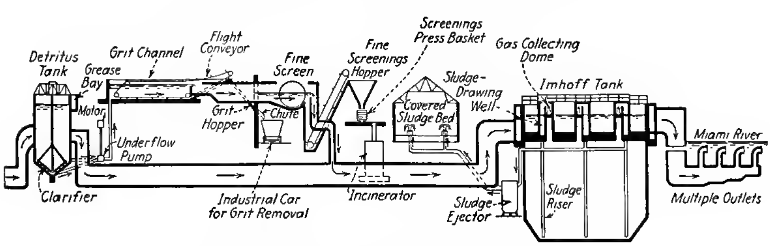


FIG. 7.—Sedimentation plant at Syracuse, N. Y.

The sedimentation plant of the Passaic Valley Sewerage Commission, located at Newark, N. J., comprises simply plain-sedimentation tanks with hopper bottoms from which the sludge is discharged into sludge



(a)-Plan, Showing Provisions for Future Extension



(b)-Diagrammatic Profile of Existing Plant

FIG. 8.—Sedimentation plant at Dayton, Ohio.

barges. The latter are towed out to the disposal area at sea, where the sludge is dumped. The settled sewage flows to a diffusion area in New York Bay.

The sedimentation plant at Dayton comprises racks, skimming-detritus tanks, Imhoff tanks and glass-housed sludge-drying beds. The concentrated liquor drawn from the bottom of the detritus tanks is passed through grit chambers and fine screens, the screened liquor

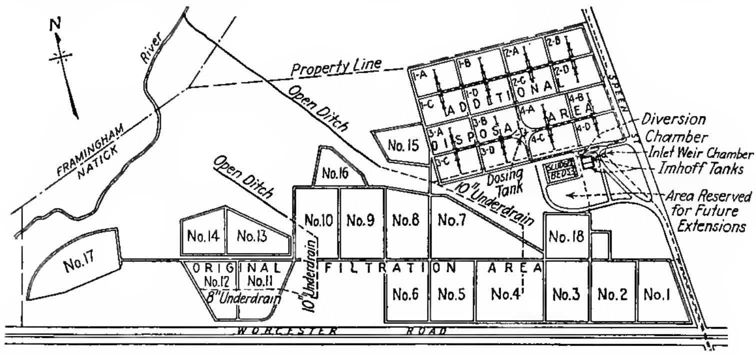


FIG. 9.—Intermittent sand-filter plant at Framingham, Mass.

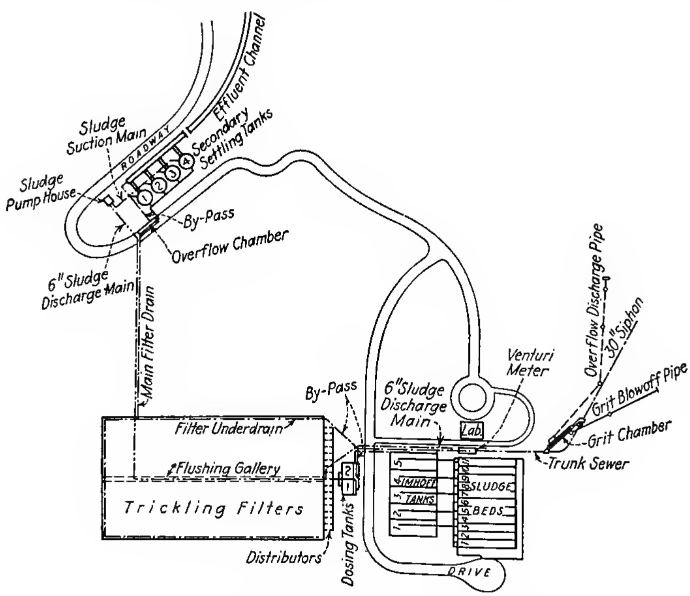


FIG. 10.—Trickling-filter plant at Fitchburg, Mass.

returning to the sewage and passing to the Imhoff tanks. The effluent of the Imhoff tanks is discharged into the Miami River for disposal by dilution. The digested sludge drawn from the Imhoff tanks is dewatered on glass-housed drying beds. Part of it is disposed of as fertilizer on

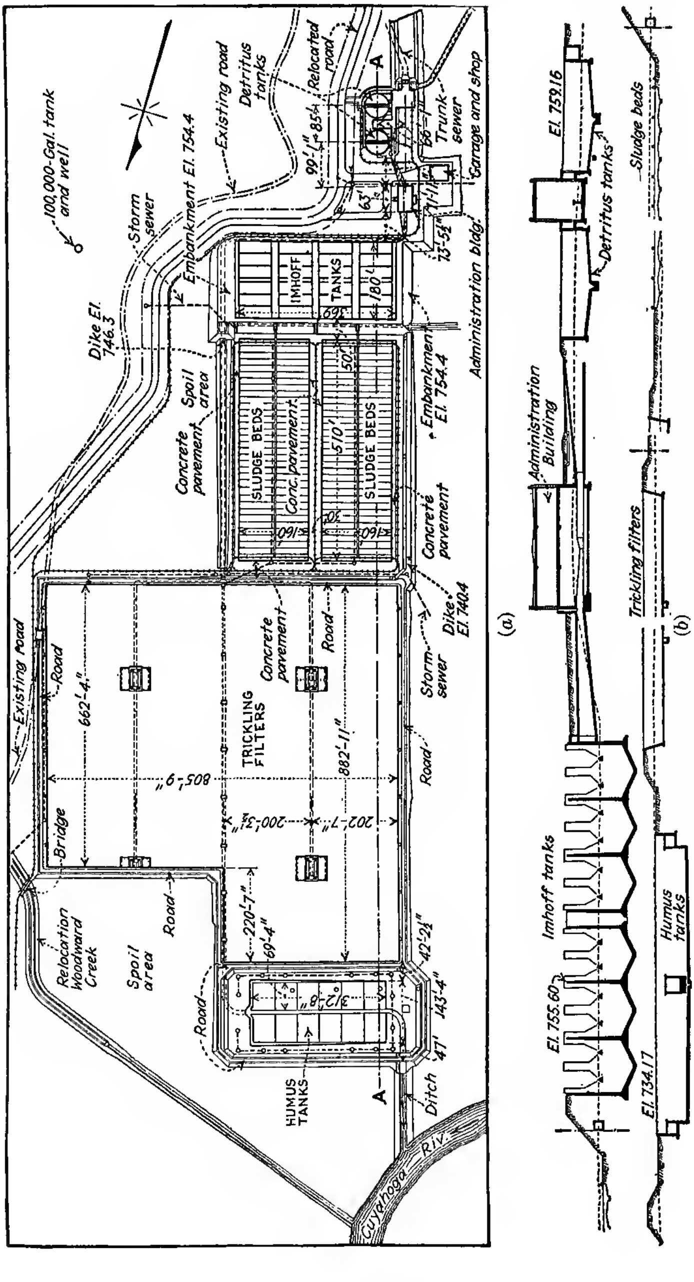


FIG. 11.—Trickling-filter plant at Akron, Ohio. (*a*) Plan; (*b*) diagrammatic longitudinal section.

farms and golf courses and the remainder is used for filling low areas. A plan and diagrammatic profile of the plant are shown in Fig. 8.

At Erie, Pa., the sewage passes through bar racks, grit chambers and plain-sedimentation tanks. The settled sewage is discharged into Lake Erie for disposal by dilution. The racks and grit chambers are provided with mechanical equipment for cleaning. The rakings are burned in an incinerator and the grit is disposed of by burial. Mechanical equipment also is installed in the sedimentation tanks to facilitate sludge removal. The sludge is digested in separate tanks, which are roofed and equipped with sludge-stirring and scum-breaking devices. The sludge gas is collected and used for incinerating the screenings and heating the diges-

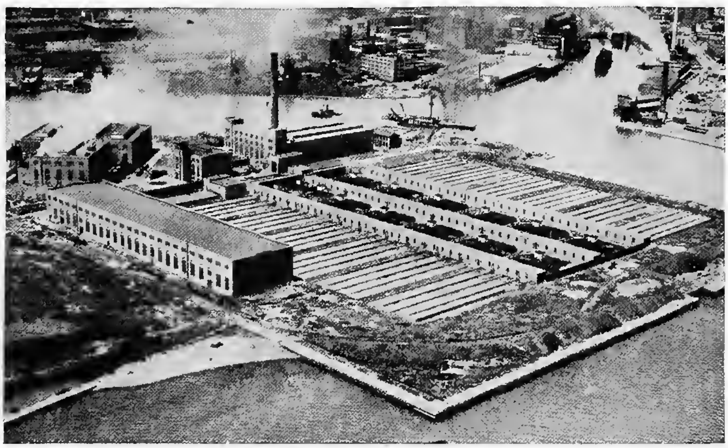


Fig. 12.—Activated-sludge plant at Milwaukee, Wis. (*Chicago Aerial Survey Co.*)

tion tanks. The digested sludge is dewatered on open drying beds. The dewatered sludge during 1932 was disposed of to farmers for fertilizer.

Intermittent Sand-filter Plants.—In 1929 there were 19 municipal intermittent sand-filter plants in Massachusetts, with an aggregate area of about 244 acres of sand-filter beds. At Marlborough, the sewage is passed through plain-sedimentation tanks prior to filtration. At Framingham, part of the sewage is passed through Imhoff tanks prior to filtration on 29 acres of sand filters. A plan of the sand-filter plant at Framingham is shown in Fig. 9. The digested sludge from the Imhoff tanks at this plant is dewatered on underdrained sand beds.

Trickling-filter Plants.—A plan of the trickling-filter plant at Fitchburg, Mass., is shown in Fig. 10. The plant comprises coarse racks, grit chambers, Imhoff tanks, dosing tanks, trickling filters, final-sedimentation or humus tanks of the Dortmund type and sludge-drying beds.

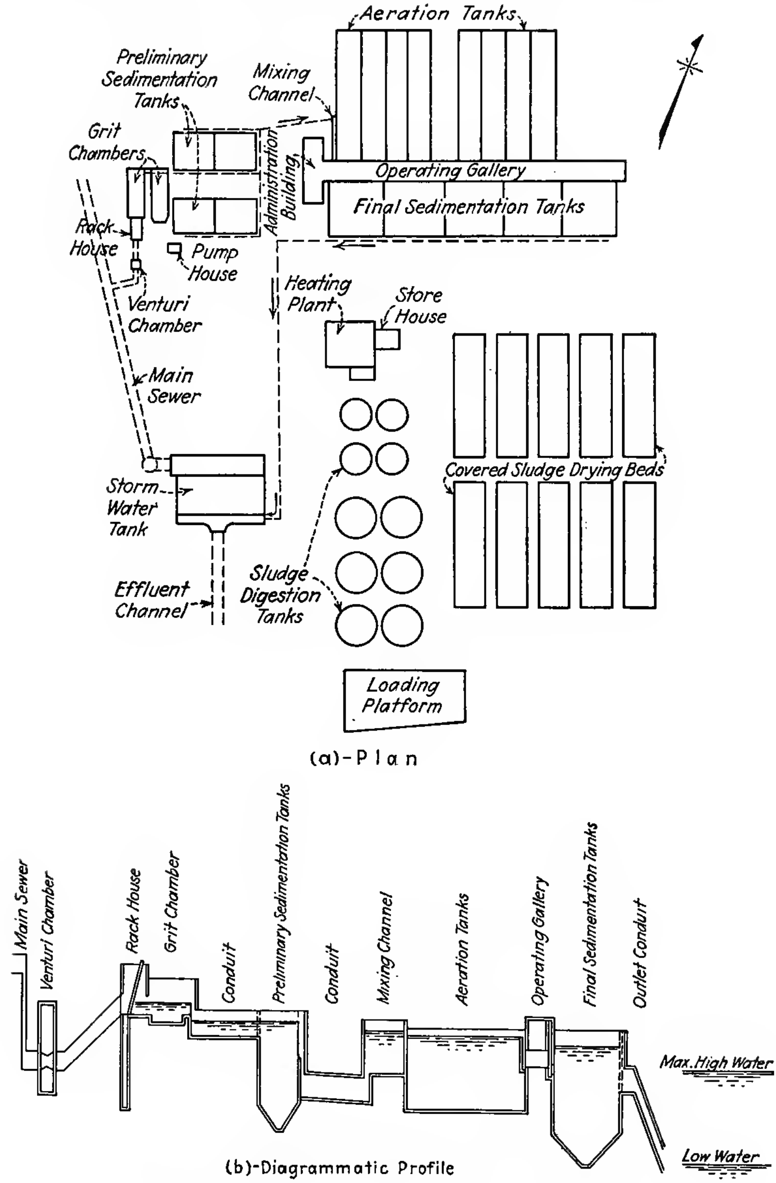


FIG. 13.—Activated-sludge plant at North Toronto, Ont.

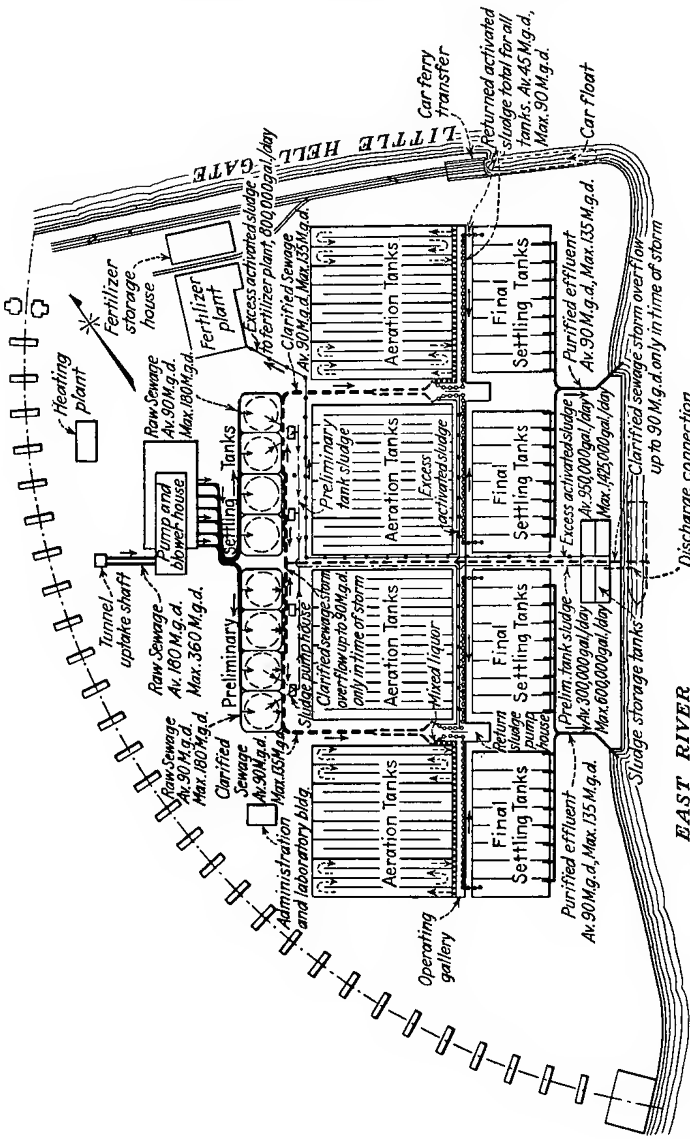


FIG. 14.—Activated-sludge plant at Wards Island, New York, N. Y.

A plan and diagrammatic longitudinal section of the trickling-filter plant at Akron, Ohio, are shown in Fig. 11. In general, the works consist of coarse bar racks, skimming-detritus tanks from which the underflow passes through grit chambers and fine screens, Imhoff tanks, trickling filters, humus tanks of the plain-sedimentation type, and sludge-drying beds.

Activated-sludge Plants.—The plants at Milwaukee, Wis., and North Toronto, Ont., and the proposed Wards Island plant in New York City may be taken as representative of present activated-sludge plants.

At Milwaukee the sewage is passed through coarse racks, grit chambers and fine screens and then treated by the activated-sludge process in aeration tanks of the ridge-and furrow type, with sedimentation in final tanks some of which are equipped in part with Dorr sludge plows and in part with Tow-Bro sludge-removal mechanisms. The effluent is discharged into Milwaukee harbor for final disposal by dilution. The excess activated sludge is conditioned, dewatered in Oliver vacuum filters, dried in direct-indirect heat dryers of the Atlas type and sold as a fertilizer or fertilizer base. A view of the plant is shown in Fig. 12.

At the North Toronto plant the sewage passes first through racks, grit chambers and preliminary-sedimentation tanks. The settled sewage is treated by the activated-sludge process in aeration tanks of the spiral-flow type, with sedimentation in final tanks equipped with Fidler-type spiral blades to assist in sludge removal. The effluent flows into the Don River for final disposal by dilution. The primary and excess activated sludges are heated and digested in separate sludge-digestion tanks, some of which are equipped with mechanical stirrers. The digested sludge is dewatered on glass-housed drying beds and used to fill low areas within the plant site. Storm flows in excess of the quantity provided for in the treatment plant are passed through storm-water tanks for sedimentation prior to discharge into the river. A plan and diagrammatic profile of the plant are shown in Fig. 13.

The plant under construction at Wards Island, New York City, will provide for primary sedimentation, activated-sludge treatment in aeration tanks of the spiral-flow type and final sedimentation in tanks equipped for continuous sludge removal facilitated by mechanical means. The effluent will be discharged into the East River for final disposal by dilution. The primary and excess activated sludges will be pumped to sludge-storage tanks, from which they will be discharged by gravity to ocean-going sludge boats and disposed of at sea. A plan of the proposed plant is shown in Fig. 14.

Bibliography

1. *Am. Jour. Pub. Health,* 1925; **15**, 334.
2. Imhoff, Karl: *Sewage Works Jour.,* 1931; **3**, 120.
3. Goudey, R. F.: *Jour.,* Am. Water Works Assoc., 1931; **23**, 230.

CHAPTER VII

POLLUTION AND SELF-PURIFICATION OF NATURAL WATERS

Appreciation of the many problems involved in the disposal of sewage by dilution requires thorough familiarity with the theory of pollution and self-purification of natural waters. For sewage disposal by dilution is not, as its name seems to imply, merely a means for getting rid of sewage matters by diluting them with or dispersing them in large volumes of water so that their origin can no longer be recognized. That is only one aspect of the process, whose chief merits are associated with the fact that in sewage-polluted waters there are operative the same or similar forces of purification which are employed artificially and more intensely in sewage-treatment plants. It might be better to speak of sewage disposal by dilution as "treatment by natural purification in water." To this end self-purification may be defined as the "natural process or combination of natural agencies that tend to render stable and innocuous foreign substances that find their way into water and so to restore the water to its natural condition of purity" (1).

Progressive Changes in Sewage-polluted Streams.—When sewage is discharged into water, a progression of changes is set in motion which is manifested by changing physical, chemical and biological characteristics of the water. Such changes take place in standing as well as in running water. They have been studied more particularly in connection with sewage-polluted streams, because it is in such waters that the effects of pollution and subsequent self-purification are most conspicuous and, therefore, defined most clearly. For this reason discussion of this subject will be hinged upon the events that take place in a clean stream into which a large volume of sewage is discharged at a single point.

Degradation.—The sewage, to begin with, renders the water turbid. Sunlight is shut out and green plants, thus deprived of their energy for growth, soon die. The fresh organic matter is relished by some varieties of fish which, therefore, gather in the vicinity of the outfall. Decomposition of the organic matter, as the result of bacterial activity, soon begins and quickly reduces the available oxygen in the stream, making conditions unsuitable for all forms of life except the most primitive organisms, which subsist on decaying matter. These saprophytes—bacteria, fungi and protozoa—develop in tremendous numbers, until one drop of the polluted water literally teems with microscopic life. The reduction in available oxygen is accompanied by an increase in the carbon dioxide content of the stream. Reaeration takes place but is not able to keep pace with deoxygenation. In all but rapidly flowing streams some of the suspended sewage matter settles to the bottom and forms sludge deposits or a *pollutional carpet* in which small reddish worms, *Limnodrilus* and *Tubifex*, together with other organisms, make their appearance. These "work over" the sewage sludge and are potent factors in its stabilization. The so-called sewage fungi, among them *Sphaerotilus natans*, also make their appearance near the outfall and are found in white bulbous masses on the bottom and attached to stones and sticks. During periods of high flow, the sludge deposits are often washed away and the stream is thus relieved of much of its sludge load.

It is evident that the stretch of river immediately below the sewer outfall is one in which life is reduced from a more highly organized to a more primitive plane and the physical and chemical quality of the water is abased. Hence this first zone of pollution is referred to as the *zone of degradation*. Using the dissolved-oxygen content of the stream as a criterion of its pollution, the zone of degradation occupies that river stretch immediately below the outfall in which the oxygen saturation is reduced to about 40 per cent.

Active Decomposition.—The zone of degradation is followed by a stretch of river in which the processes of decomposition which gradually establish themselves in the first zone become extremely active. For this reason it is called the *zone of active decomposition*. In this zone the water remains grayish in color. Decomposition of the complex organic materials is characterized by reducing and splitting processes. Soluble, volatile and gaseous compounds are formed. As long as the water contains dissolved oxygen, seriously offensive odors are not likely to be prevalent. In grossly polluted streams, however, the dissolved-oxygen content may be completely exhausted, in spite of the fact that the rate of re-aeration increases as the oxygen content is reduced. Anaerobic or septic conditions will then prevail. During the summer months, foul odors are apt to arise, gas bubbles are evolved and the sticky, blackened sludge deposits may be lifted by the gases of decomposition into the water, turning the stream into a black putrefying watercourse. As the available organic material is consumed, the bacteria and protozoa which have held sway die off and decomposition slackens. A point is reached when re-aeration first balances and then outweighs the immediate biochemical demand for oxygen and a new store of dissolved oxygen is acquired gradually. More highly organized life reestablishes itself. The trend of self-purification becomes positive rather than negative as heretofore. The dissolved-oxygen saturation first sags below 40 per cent, sometimes to zero, and then gradually climbs back to above 40 per cent.

Recovery.—There follows the zone of active decomposition a third zone in which the stream gradually recovers its former appearance and normal condition. The water becomes clearer; algae and larger aquatic vegetation reappear. The dissolved-oxygen content is slowly increased to full saturation by re-aeration and usually to a less extent by the photosynthetic activities of green plants. Small animal forms of more highly organized life reestablish themselves and some of these serve as food for fish. Judged by chemical standards this *zone of recovery* is one of mineralization. Nitrogen end products are carried to nitrites and finally to nitrates, sulfur to sulfates and carbon to carbon dioxide or carbonates.

Cleaner Water.—Below these three zones of vigorous self-purification is found the *zone of cleaner water,* in which natural and desirable stream conditions are established once more. The water becomes attractive in appearance but may still harbor pathogenic organisms. Long periods of flow are often required to destroy these.

The zones of pollution and natural purification neither occupy a fixed position in streams nor are they sharply bounded. They shift with changes in temperature and variations in river flow and sewage discharge. During the summer, when life and with it decomposition are most active, the zones are more pronounced. Depending upon the load of sewage that the stream receives, the zones occupy longer or shorter river stretches. In slightly polluted streams, the zone of active decomposition may be suppressed; with heavy pollution septic conditions may prevail for many miles. The progress of self-purification may be interrupted or accelerated by the discharge of new wastes on the one hand or by dilution due to the entrance of cleaner water on the other. In lakes and tidal estuaries pollution and self-purification are influenced by winds and by tides or other water movements. Bottom deposits, being less subject to the changes which may occur in the water itself, are more representative of average conditions and are therefore an especially useful guide in pollution studies.

Processes of Self-purification Classified.—If the individual factors associated with the self-purification of polluted waters are separated, they are found to be physical, chemical and biological in nature. Like all processes involved in the purification of sewage, they are interrelated closely and mutually dependent, the resultant of their operations being a gradual return of polluted water to cleanly condition. The most important processes of self-purification are outlined below, together with a brief explanation of their operation.

Physical Processes:

1. Dispersion.—Mixing of sewage with water results in the dispersion of the sewage matters, animate and inanimate. A dilution, for example, of

raw sewage in 5 c.f.s. of water per 1000 population signifies that the daily waste matters of one person are dispersed in about 3300 gal. of water instead of in 100 gal. as in sewage. Improvement in appearance and odor and in concentration of possible pathogenic organisms is the natural result.

2. Sedimentation.—Heavy suspended particles settle out in standing water, slowly running water or in the backwaters of rapidly flowing streams. Light suspended particles and colloidal matters settle as aggregated or coagulated masses. While this removes these matters from the water, it transfers them to the bottom where they must be cared for. The formation of sludge deposits is often objectionable and intensifies the problems of pollution in certain localities, such as ferry slips, docks or other reaches of quiescent water. This results in a type of decomposition which differs from that taking place in the water itself.

3. Light.—Sunlight is a disinfectant, destroying many objectionable bacteria. It is also a stimulant to the growth of green plants, large and small. These, by photosynthesis, give off oxygen and remove carbon dioxide. Sunlight also bleaches color.

4. Aeration.—As water comes into contact with the atmosphere, an interchange of gases takes place whenever the concentration of the gases in the liquid and gaseous phases are not in equilibrium. This is particularly important in connection with the oxygen requirements of polluted water. As the oxygen dissolved in the water is used up by the decomposition of the organic-sewage load, new volumes of oxygen are added by absorption from the atmosphere. This is called *re-oxygenation* or, more generally, *re-aeration.* Carbon dioxide and other gases of decomposition escape to the atmosphere.

Chemical Processes:

1. Oxidation.—Aerobic organisms convert complex chemical substances into mineral matter, gases or relatively stable organic matter. Certain dissolved mineral constituents, such as iron and manganese, are oxidized and precipitated as insoluble compounds.

2. Reduction and Hydrolysis.—Anaerobic organisms liquefy and split the complex organic constituents of sewage and thus pave the way for stabilization by oxidation. In the complete absence of oxygen, offensive odors and gases are produced, which escape to the air. Certain mineral substances are taken into solution. True anaerobic decomposition may take place in the bottom sludge or ooze even when the supernatant water contains dissolved oxygen, because diffusion of oxygen is not sufficiently rapid to supply the deficiencies created in the bottom deposits.

3. Coagulation.—Mixing of certain wastes, especially industrial wastes, with sewage or other wastes sometimes results in chemical coagulation with consequent precipitation of dissolved, colloidal and suspended matters.

4. Disinfection.—Certain poisonous substances, such as copper and arsenic, derived from industrial wastes, destroy bacteria and other living organisms. As a result they interfere with the biologically activated purification processes, until they are reduced in concentration to the limits of tolerance of the living cells.

Biological Processes:

1. Bacteria.—The food habits of these and other living reagents result in the decomposition and stabilization of the sewage matters by biochemical means. As the food substances are used up or the environment is unfitted for continued existence by the accumulation of the waste products of their activity, each group of organisms dies off and makes way for other groups which in turn flourish and succumb.
2. Plankton.—Plant forms—algae and fungi—utilize simple chemical substances resulting from bacterial activity. Green plants, by photosynthesis, give off oxygen while taking in carbon dioxide. Animal forms—protozoa, rotifera and crustacea—are scavengers and, as such, destroy organic substances much as the bacteria do. Some forms are predatory; protozoa ingest bacteria and are themselves consumed by crustacea. Rotifera feed on algae.
3. Larger Aquatic Life.—Rooted plants utilize food substances contained in bottom deposits. Worms "work over" sewage sludge. Insect larvae and other forms live on food substances in the water or bottom ooze. Fish may be scavengers or may live on plankton and insect larvae.

OXYGEN RELATIONS

The Oxygen Balance.—The importance of the oxygen conditions existing in polluted waters, reflecting as well as deciding the nature of the progress of decomposition of sewage which is disposed of into water, must be evident from the foregoing discussion. The relation between the biochemical-oxygen demand of the sewage and the oxygen available in the diluting water is called the *oxygen balance.* Since offensive odors ordinarily will not be given off as long as there is sufficient oxygen present to meet the demand, the oxygen balance is probably the best measure of the quantity of sewage which a given watercourse can handle without the creation of offensive conditions.

The following factors determine the oxygen balance:

1. The biochemical-oxygen demand of the sewage as discharged, which at times must be considered to be composed of (*a*) the demand of the liquid; (*b*) the demand of the suspended solids; and (*c*) the demand of the deposited sludge.
2. The biochemical-oxygen demand of the diluting waters. This is sometimes quite appreciable, owing to prior pollution by sewage or by industrial wastes.
3. The dissolved-oxygen content of the sewage at the time of discharge.
4. The dissolved-oxygen content of the diluting water.
5. The oxygen absorbed by re-aeration from the atmosphere.
6. The oxygen given off by green plants.
7. The oxygen readily available from mineral matter such as nitrites and nitrates.

Determinations of the biochemical-oxygen demand and the oxygen available in sewage and water, together with the formulation of the *deoxygenation curve*, have been described in previous chapters. Re-aeration, however, has been touched upon only lightly. Two types of re-aeration may be distinguished, atmospheric and photosynthetic, or biological. Of these the former has been formulated mathematically and is of more general applicability; the latter depends so much on local conditions that formulation and prediction of its effects are not made readily.

Atmospheric Re-aeration.—The quantity of oxygen absorbed by water from the air in a unit of time is dependent upon the following factors: the degree of undersaturation of the water; the area of water surface exposed per unit volume; the agitation of the water surface by wind, currents and other disturbances such as those due to rapids, dams and ships' propellers; and the downward diffusion of the oxygen.

Re-aeration, therefore, varies greatly in different bodies of water. In a turbulent stream the rate of re-aeration is high, because new surfaces of water are being brought constantly into contact with the atmosphere. In a calm, deep lake the rate is relatively low, because in the absence of agitation the lower layers of water are dependent for re-aeration upon the diffusion of oxygen from the upper ones. This is a slow process.

The laws governing atmospheric re-aeration have been studied most thoroughly by the workers of the United States Public Health Service in investigations of the Ohio and Illinois Rivers (2, 3). The findings are summarized below. In order to appreciate the limitations of experience, however, reference should be made to the original publications.

Since chemists have shown that the rate of solution of a moderately soluble gas by an undersaturated liquid varies directly with the degree of undersaturation of the liquid and is constant for a given liquid and gas (4), the law of oxygen absorption was formulated as follows, in the Ohio River studies:

Let D = initial oxygen-saturation deficit, or difference between full saturation value and observed value, p.p.m.

D_t = oxygen deficit at any time t, p.p.m.

K_2 = atmospheric-re-aeration constant.

Then

$$\frac{dD_t}{dt} = -K_2t$$

Whence

$$\log \frac{Dt}{D} = -K_2t$$

Experiments by Dibdin, Adeney and Becker afford experimental confirmation of this law. It should be noted that this expression is analogous to that defining the rate of deoxygenation or B.O.D. of sewage in water. K_2 cannot be determined in the laboratory as was

K_1, however, but varies with the body of water under consideration. Thus it includes the factors of relative water surface and degree of agitation which were previously listed. In the Ohio River studies K_2 was found to be modified by stream depth and by various physical conditions which influence the turbulence of flow, among them the velocity of the current and the slope and irregularity of the channel. In the Ohio River these relations could be expressed by the equation:

$$K_2 = CV^nH^{-z}$$

where V = velocity of flow, feet per second
H = mean river depth, feet above extreme low water
and C and n are constants for a particular river stretch.

The river constant C is defined chiefly by physical characteristics producing different degrees of turbulence in various river stretches under similar flow conditions, the slopes and irregularity of the channel apparently being the most prominent of such characteristics. In the Ohio River studies the slope, S, in feet per mile at extreme low river stage, was considered the most reliable index of the slope of the channel bottom, and irregularity was expressed in terms of an "irregularity factor" established by noting the number of changes in slope per mile, each one of which produced a change of elevation of 1 ft. in the longitudinal profile of the channel bottom. River stretches with irregularity factors of 2 to 3 were classified as relatively smooth, whereas those having factors between 3 and 5 were classified as relatively rough. The following empirical relations between the river constant C and the slope S were established for the two classes of channel roughness, from studies of different river stretches.

Channel relatively smooth, irregularity factor 2 to 3:

$$C = 11S^{2.3}$$

Channel relatively rough, irregularity factor 3 to 5:

$$C = 0.39 \times 10^{1.16S} + 17$$

Broadly the river constant n defines the range of variation in rates of re-aeration in a given river stretch under different flow conditions. In the Ohio River studies it was found that the effect of a given variation in river stage upon the velocity of flow, modifying turbulence, in a given river stretch is closely related to the value of n. This relationship was expressed empirically by

$$n = \frac{y - 0.17}{y - 1.17}$$

in which y is the mean relative increase in velocity of flow per 5-ft. increase in gage height.

The rate of re-aeration as expressed by K_2 is further modified by the water temperature, the controlling element apparently being the fact that the rate of absorption of oxygen at the surface is limited by the process of diffusion which, as shown by Black and Phelps (5), is governed by a similar temperature relation. Experiments by Becker, Haslam, Hershey and Keen (4) have permitted the derivation of the following temperature-correction equation:

$$K_{2(T)} = K_{2(20^\circ\text{C.})} \times (1.0159^{T-20})$$

in which T is the observed temperature, degrees centigrade.

The magnitude of the re-aeration constant has been computed by the Public Health Service for certain stretches of the Ohio and Illinois Rivers from measurements of the "oxygen sag," the values being listed in Table 35.

TABLE 35.—MEASURED VALUES OF THE RE-AERATION COEFFICIENT, K_2, IN THREE STRETCHES OF THE OHIO RIVER AND TWO STRETCHES OF THE ILLINOIS RIVER

Month	Values of re-aeration coefficient, K_2				
	Ohio River			Illinois River	
	Miles below Pittsburgh			Miles above mouth	
	11–19	23–65 (Beaver River to Steubenville, Ohio)	103–349 (Moundsville, W. Va., to Scioto River)	263–240 (Morris to Ottawa, Ill.)	148–122 (Pekin to Havana, Ill.)
May	0.25	0.20	0.18	0.31	0.47
June	0.19	0.33	0.27	0.31	0.28
July	0.29	0.23	0.21	0.21	0.20
Aug.	0.22	0.26	0.21	0.19	0.27
Sept.	0.14	0.19	0.17	0.31	0.14
Mean	0.22	0.24	0.21	0.27	0.27

The average value of K_2 for these river stretches is 0.24 and has been tentatively set at 0.20 for temperatures of 20°C. and similar river characteristics. In the shallow rapids of the Des Plaines River below Joliet, Ill., where the channel is steep and rough, however, the average value of K_2 was computed to be 2.00. Further observations such as these are much needed for streams as well as for lakes and tidal estuaries, before it will be possible to predict with reasonable certainty the rate of re-aeration of different bodies of water.

In standing water the value of K_2 at any temperature varies inversely with some power of the depth and directly with the amount of mixing that takes place owing chiefly to winds, currents and convection currents, and mechanical disturbance of the surface by water craft. A study of experiments by Adeney (6) yields a value of 0.115 for K_2 at 20°C. for each foot depth of water and $K_{2(T)} = K_{2(20°C.)} \times (1.020^{T-20})$. This value of K_2 is only slightly more than one half the average magnitude of the re-aeration constant for the normal stretches of the Ohio and Illinois rivers.

The Oxygen Sag.—Deoxygenation and re-aeration combine to produce in streams dissolved-oxygen values which, when plotted as ordinates with time of flow below the point of sewage discharge as abscissas, yield a curve characteristic of self-purification and known as the *dissolved-oxygen sag*. The equation of this curve is given by combining the

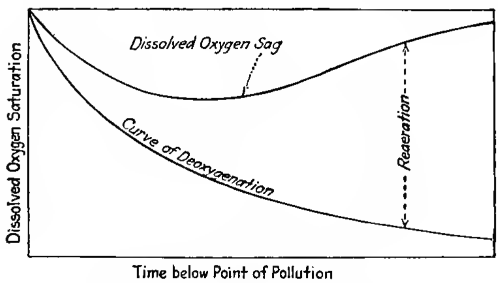


FIG. 15.—The dissolved-oxygen sag.

equations of deoxygenation and atmospheric re-aeration, obtaining the following formula:[1]

$$D_t = \frac{K_1 L}{K_2 - K_1}(10^{-K_1 t} - 10^{-K_2 t}) + D \times 10^{-K_2 t}$$

where D_t = oxygen-saturation deficit of the water in p.p.m. at any time t

D = initial oxygen-saturation deficit at point of pollution, p.p.m.

L = initial biochemical-oxygen demand of the diluted sewage, p.p.m.

K_1 = the deoxygenation constant

K_2 = the re-aeration constant.

The general outline of the oxygen sag is shown in Fig. 15, together with the oxygen conditions which would obtain in the absence of re-aeration. The latter may occur in highly polluted streams when they are covered by a continuous sheet of ice.

[1] For derivation of this formula see Appendix A, *Pub. Health Bull.* 146, U. S. Pub. Health Service.

In computing the probable dissolved-oxygen sag of a stream into which sewage is discharged, the following factors must be known or estimated:

A. For the sewage, assuming that it is fresh:
1. Quantity (Q_s)
2. Temperature (T_s)
3. Dissolved-oxygen content (O_s)
4. B.O.D. during first stage, commonly computed from 5-day value (L_s)
5. Deoxygenation constant, commonly computed from 20°C. value (K_1)

B. For the water, assuming that its own B.O.D. falls within the first stage:
1. Quantity (Q_w)
2. Temperature (T_w)
3. Dissolved-oxygen content (O_w)
4. B.O.D. during first stage, commonly computed from 5-day value (L_w)
5. Re-aeration constant, commonly computed from 20°C. value (K_2)

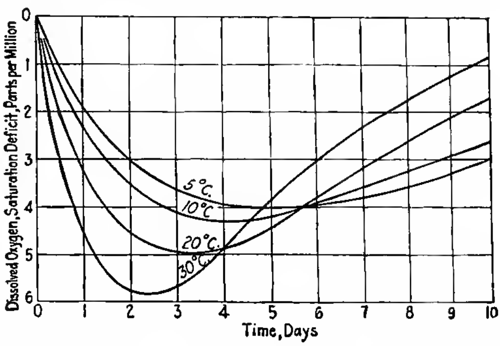


FIG. 16.—Effect of temperature variations on the progressive changes in the dissolved-oxygen deficit. $L = 20$ p.p.m. $D = 0$. (*After Streeter*.)

The various values in the oxygen-sag equation will then be known or will be given by the following relations:

The temperature of the mixture, $T = \dfrac{T_sQ_s + T_wQ_w}{Q_s + Q_w}$

The oxygen-saturation deficit of the mixture,

$$D = (\text{saturation value at } T°\text{C.}) - \frac{O_sQ_s + O_wQ_w}{Q_s + Q_w}$$

The initial B.O.D. of the mixture,

$$L = \frac{L_sQ_s + L_wQ_w}{Q_s + Q_w}$$

The various forms that the oxygen sag may take at different temperatures, as calculated by the equation developed by the Public Health Service, are illustrated in Fig. 16. The curves are based upon an assumed initial B.O.D. of $L = 20$ p.p.m. and an initial oxygen-saturation deficit of $D = 0$ p.p.m. The constants K_1 and K_2 are taken as equal respectively to 0.1 and 0.2 at 20°C.

A comparison of computed and observed values of dissolved-oxygen content of a polluted river, made by the Public Health Service for five stations on the Illinois River, is given in Table 36.

TABLE 36.—COMPARISON OF CALCULATED AND OBSERVED DISSOLVED-OXYGEN CONTENTS OF UPPER ILLINOIS RIVER AT SUCCESSIVE SAMPLING STATIONS

Station, miles above mouth of river	Oct, 1921		May, 1922		June, 1922		July, 1922	
	Calculated	Observed	Calculated	Observed	Calculated	Observed	Calculated	Observed
	Dissolved-oxygen saturation deficit, p.p.m.							
263	8.4	9.1	8.6	8.0	8.6	8.8	6.2	8.6
240	6.4	7.1	7.0	6.3	6.4	7.6	4.8	8.0
227	6.5	7.4	7.0	6.4	6.5	8.1	4.9	8.0
196	4.3	5.3	4.4	4.3	3.6	5.9	3.0	7.5
179	4.9	6.6	3.6	4.6	2.6	6.3	2.7	8.1
	Dissolved oxygen, percentage of saturation							
263	20	14	11	19	3	1	28	2
240	39	32	28	35	28	15	44	6
227	38	29	28	34	27	9	43	8
196	59	50	55	61	60	31	65	12
179	63	37	63	52	71	26	69	9

It will be noted that the computed and observed values check each other closely for May and reasonably well for October, but that they diverge widely for June and July. This divergence is probably due to oxygen absorption by the bottom sediments, a factor that must not be neglected.

In making estimates of the dissolved-oxygen sag by the method developed by the Public Health Service the limitations of the method should be clearly realized. Actual conditions may be profoundly modified by the hydrography of the stream, the deoxygenation characteristics of the polluting substances, the presence of wastes other than sewage, especially toxic industrial wastes, the entrance of diluting waters or additional waste materials, the growth of green plants, the existence of dams and other obstructions, and many other factors. To illustrate some of these modifications, Fig. 17 is given. It shows the dissolved-oxygen sag of the Skaneateles outlet in central New York, as affected by a number of wastes which enter the stream along its course (7).

If the velocity of flow of a stream is known, the oxygen sag can be determined with respect to distance below the point of pollution. The average velocity naturally varies considerably. In the Ohio River studies previously referred to, monthly mean velocities ranging from 0.16 to 3.60 miles an hour were encountered in 1914–1915 between Pittsburgh, Pa., and Louisville, Ky. An average value for velocity of flow of large rivers is ⅔ mile an hour.

Biological Re-aeration.—Waters that support appreciable growths of green plants and related organisms may draw a considerable quantity

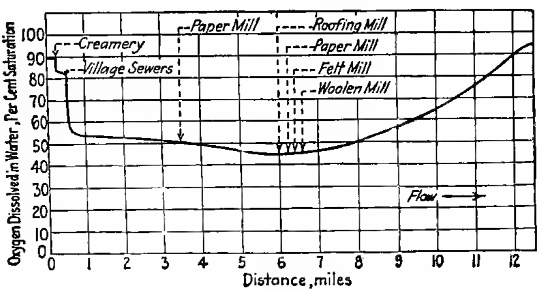


FIG. 17.—Dissolved-oxygen sag of Skaneateles outlet. (*After Suter.*)

of oxygen from the photosynthetic activities of these organisms. An example of the reoxygenation of polluted water by green plants, particularly the plankton, is afforded by the authors' observations of a small stream at Cincinnati, Ohio. A sewerage system serving a population of about 1700 persons discharged sewage into the upper reaches of this stream which had a sluggish flow for about a mile below the sewer outlet and then spread out into a shallow creek with occasional pools. The odors from the water just below the sewer outlet were 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 downstream green algae appeared and their growth increased in luxuriance as the distance from the outlet increased. Where it was heaviest, there were evolved by the green masses enormous numbers of gas bubbles 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 with oxygen and free from objectionable appearance. Aquatic growths of this type are active both in the re-aeration of polluted water and in the sedimentation and straining out of suspended material.

Observations similar to these have been made at other places. The most notable ones are those of the United States Public Health Service on the Potomac River flats below Washington, D. C. (8). Of special interest in this investigation are the studies of the effect of sunlight on plant activities. Purdy found that the average saturation with dissolved oxygen of the water flowing from the flats was 99 per cent on sunny days and only 69 per cent when the sky was cloudy or overcast. The photosynthetic activity of the aquatic flora supported by the river flats is further illustrated by the fact that the average concentration of dissolved oxygen in the river channel was 71 per cent, whereas that of the flats was 87 per cent.

It is evident, however, from what has been said about the progressive changes which take place in a sewage-polluted stream, that heavy pollution will greatly restrict, and often eliminate, the development of green plants and related organisms in the zones of degradation and active decomposition.

BIOLOGICAL RELATIONS

Biological Processes of Self-purification.—An understanding of the theory of self-purification of polluted waters is closely linked with a knowledge of the food habits and life processes of the living organisms commonly encountered in these waters. This knowledge has been made available by the researches of numerous workers and has been summarized admirably by Marsson, from whose lecture, as translated by Kuichling, the following conclusions are taken (9):

In water most diversified interrelations between plants and animals, or between food producers and food consumers, are found. Among the latter there is a constant struggle for existence, the final victors being the predatory fishes which are chiefly used for human consumption, and whose food has been evolved in a complex manner from the dead organic matter originally present in the water. On their own death, their bodies become a source of nutrition for the other animals whose progenitors served to feed them.

Although these considerations show how readily streams and their denizens can cope with natural waste waters, *i.e.*, the sewage of cities and the wastes of certain industries including their sludge-forming matters, and how often relatively clean water can be encountered only a short stretch below the point of pollution, we must remember that the quantity of putrescible or decomposing matters must always remain in proper proportion to the volume of water and current velocity. The maximum load of municipal and industrial wastes that a body of water can digest depends not only upon its physical properties but also upon its biological and chemical characteristics.

The problem has a somewhat different aspect in the case of certain industrial wastes that contain acids, alkalies or other substances that are destructive to animal life and that kill not only the larger flora and fauna but also the microscopic organisms which drift in the stream and are constantly at work in maintaining its healthful purity.

The self-purification of a stream thus involves a large number of factors, all of which stand in mutual relationship and interaction like the wheels of a clock, and the main object of the foregoing considerations was to point out the great importance of the action of animal and plant life alone in conserving the purity of waters.

Increase in Aquatic Life Due to Sewage Pollution.—It stands to reason that the addition to natural waters of large quantities of food substances, such as are contained in sewage, commonly will permit the waters to support life in increasing varieties and numbers. The quantity and seasonal distribution of the plankton in the Illinois River before and after opening the Chicago Drainage Canal, as given by Forbes and Richardson (10), are illustrated in Table 37.

TABLE 37.—ILLINOIS RIVER PLANKTON AT HAVANA, ILL., BEFORE AND AFTER OPENING OF CHICAGO DRAINAGE CANAL

Cubic Centimeters of Plankton per Cubic Meter of Water

Month	Before opening			After opening
	1895–1896	1896–1897	1897–1898	1909–1910
Sept.	1.52	0.38	6.33	0.10
Oct.	0.57	1.10	5.94	2.58
Nov.	3.02	0.02	1.00	1.38
Dec.	1.19	0.76	0.56	0.38
Jan.	0.01	....	0.45	0.01
Feb.	0.01	0.04	0.27	0.21
Mar.	0.07	0.38	0.33	2.18
Apr.	5.69	5.11	4.40	29.60
May	1.30	5.62	11.30	12.27
June	0.71	0.27	3.96	11.89
July	1.44	4.78	0.58	0.23
Aug.	1.17	3.65	0.91	0.06
Average	1.39	2.10	3.00	5.07

One of the authors has described the stimulation of aquatic growths along the banks and in the bed of the Hockomock River, which receives the effluent of the Brockton, Mass., sewage-treatment works through its tributaries (11). In this case excessive vegetation was held to be responsible for the clogging of the channels by the growths and by banks of silt and sand, the formation of which was fostered by these growths.

Excessive growths of green algae during the summer months have also been reported in the Back River in the neighborhood of the Baltimore sewage-treatment works (12).

LePrince and Headlee (13, 14) have stated that the breeding of the house mosquito, *Culex pipiens*, is stimulated by the discharge of sewage into a sluggish stream. The larvae of this variety of mosquito are found in almost all sorts of stagnant fresh-water pools and in salt marshes which have become polluted with sewage, but they are rarely found in clean water. Although the *Culex* does not transmit disease, it is annoying on account of its habit of entering dwellings and "biting" during the night. When present in large numbers, these mosquitoes are apt to range out for two miles from the breeding place, in search of a supply of human blood.

Indicator Organisms.—There are numerous organisms whose ecology —or mutual relations between living organisms and their environment— is sufficiently definite and well established to permit their use as indicators of pollution and self-purification. Engineers are generally familiar with the use of *B. coli* as the bacterial index of pollution. Other organisms may be used in a similar way. An extensive ecological classification of aquatic organisms has been prepared by Fair and Whipple (1) and will prove useful in studies involving the biology of the self-purification of natural waters. Use of these living indicators, however, is predicated upon an appreciation of the fact that final judgment cannot be passed on the basis of single organisms. In this respect biological analysis does not differ from chemical analysis. In the words of Fair and Whipple,

> . . . a multiplicity of indications is required for reasonable interpretation of the pollutional status of the water. Instead of considering the different indicator organisms separately, therefore, they should be examined as biological associations that by mutual admission and exclusion of evidence indicate the correct conclusion to be drawn.

Furthermore, other tests, physical and chemical as well as bacterial, should be utilized in order to obtain adequate evidence of existing conditions.

Among the living indicators of pollution and self-purification, the bottom-dwelling or sessile organisms are particularly useful in gaging the pollutional character of bodies of water. As expressed by Fair and Whipple, "the character of the bottom sediments found in any stretch of a stream is a reflection of the cumulative variations that have taken place in the supernatant water while the sediments were being deposited.

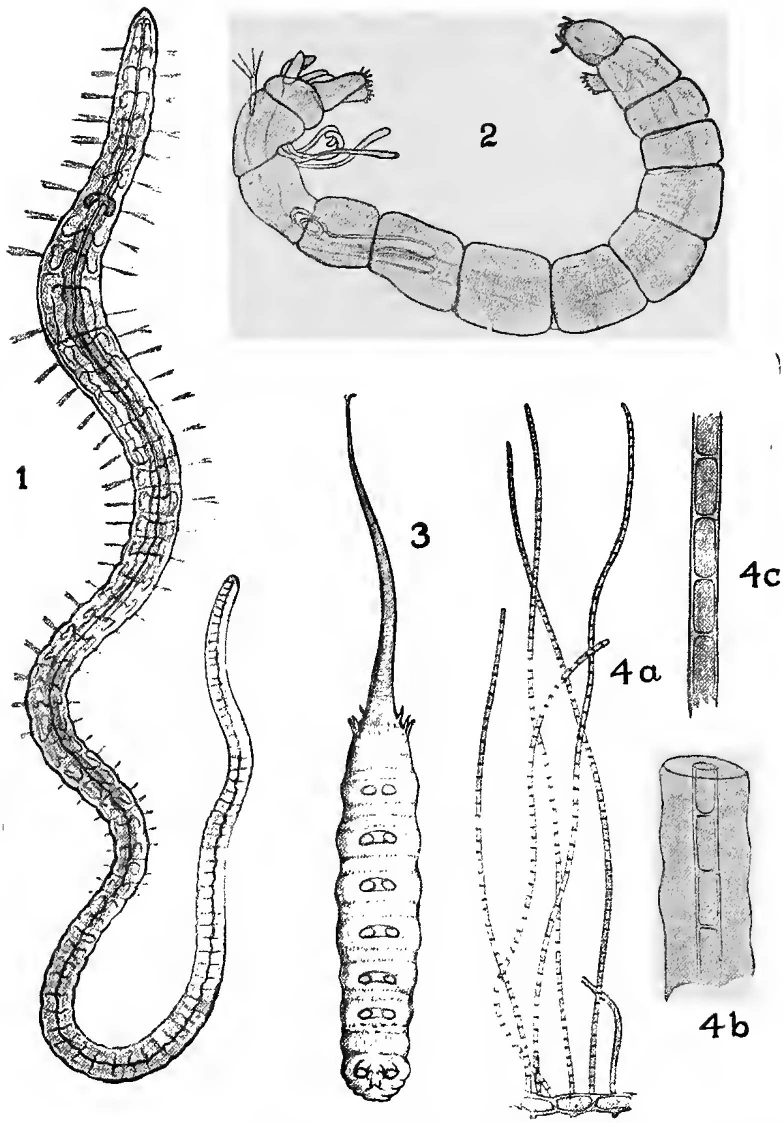


FIG. 18.—"Type organisms" found on the bottom of polluted streams. (*From "The Microscopy of Drinking Water," G. C. Whipple, 4th ed., John Wiley & Sons, Inc.*, 1927.)

1. *Tubifex tubifex* × 5;
2. *Chironomus plumosus* × 10;
3. *Eristalis tenax* × 5.
4. *Sphærotilus natans:*
   a. Tuft of filaments × 400;
   b. Cells in both sheath and slimy envelope × 200;
   c. Cells in sheath only × 200.

Bottom sludges, therefore, are in some respects more significant of the average condition of a stream than its flowing waters." Some of the "type organisms" that are often found on the bottom of polluted streams and that are readily recognized with the naked eye are shown in Fig. 18.

Bacterial Death Rates in Polluted Waters.—If all the bacteria that develop in sewage-polluted water could be readily identified on a quantitative basis, an interesting picture of zonal differentiation, comparable to that presented by the oxygen sag and the sequence of plankton and bottom organisms, probably would be obtained. The present common methods of quantitative sanitary bacteriology, however, merely indicate a maximum development or dispersion of bacteria a short distance below the point of sewage pollution and a gradual, more or less constant, reduction in numbers thereafter.

From the hygienic standpoint, the fate of the pathogenic organisms which may find their way into sewage and thence into streams and other bodies of water is a most important question. The isolation of pathogens from polluted water is such a difficult matter, however, that there is no evidence of actual conditions. As far as *Bacillus typhosus*, the causative organism of typhoid fever is concerned, the Metropolitan Sewerage Commission of New York, after a study of the literature, reported in 1910 as follows:

> There is unanimous opinion that typhoid bacilli will live in sewage whether or not the sewage be sterilized before these germs are added; that typhoid bacilli do not usually multiply in crude sewage, but retain their vitality for some days, and that typhoid bacilli may live a considerable time in sea water.

In general, experimental evidence seems to show that typhoid organisms die more rapidly in polluted and warm water than in clean and cold water. With reference to temperature Houston (15) obtained the results shown in Table 38.

It has been found that the viability of *B. typhosus* is similar to that of *B. coli*, but that, of the two, the latter is the more resistant. For this reason the fate of *B. coli* is commonly assumed to yield information on what happens to pathogenic bacteria.

The most elaborate study of bacterial death rates in polluted waters has been made by the Public Health Service in its investigations of the Ohio and Illinois rivers. Hoskins (3) has summarized the findings in part as follows:

> Quite extensive observations of the decrease of bacteria in polluted waters indicate that such changes follow a fairly regular course, modified by variations in environment, such as temperature and other factors, but yet having an orderly arrangement of reduction. Just what agency is primarily

TABLE 38.—INFLUENCE OF TEMPERATURE ON VITALITY OF THE TYPHOID BACILLUS IN RAW RIVER WATER

Average of 10 Experiments at Each Temperature. Average Initial Number, 103,328

Temperature, degrees		Percentage of typhoid bacilli remaining after various periods of time in weeks								
C.	F.	1	2	3	4	5	6	7	8	9
0	32	46.23	0.948	0.063	0.033	0.003	0.003	0.002	0.001	0.000
5	41	14.41	0.025	0.006	0.003	0.000				
10	50	0.067	0.014	0.003	0.000					
18	64.4	0.038	0.003	0.000						
27	80.6	0.018	0.000							
37	98.6	0.005	0.000							

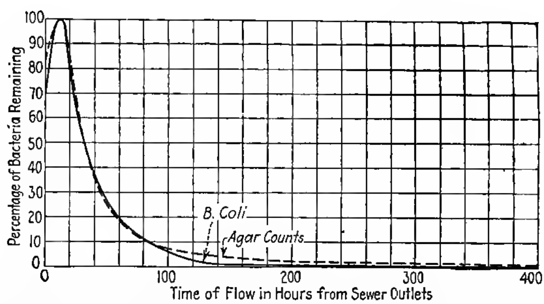


FIG. 19.—Bacterial purification in Ohio River between Cincinnati and Louisville in relation to time of flow from sewer outfalls of Cincinnati metropolitan district,—summer months, April to November, 1914, 1915 and 1916.

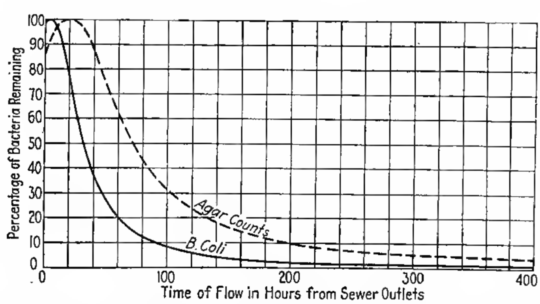


FIG. 20.—Bacterial purification in Ohio River between Cincinnati and Louisville in relation to time of flow from sewer outfalls of Cincinnati metropolitan district,—winter months, December to March, 1914, 1915, and 1916.

responsible for the death of such bacteria has not been definitely determined. However, there is considerable evidence suggesting that plankton activity rather than lack of food supply is the dominant influence in bacterial diminution.

Observations of the pollution of the Illinois and Ohio Rivers have indicated that the numbers of bacteria contributed per capita by the sewered populations of various cities are reasonably constant; these numbers change, however, with seasonal temperature, being much greater in summer than in winter. Such bacteria tend to increase in numbers in the receiving stream for a short period and then decrease at orderly rates as the time from the point of maximum density is increased. These rates of decrease were found to be affected by water temperature and apparently by concentration, being most intensive during the warmer months and under conditions where the density of bacteria was greatest.

Illustrations of bacterial death rates in polluted waters are given in Figs. 19 and 20, showing the bacterial purification in the Ohio River between Cincinnati and Louisville, under both summer and winter conditions. The curves were drawn from data on *B. coli* and agar counts of bacteria found in the studies of the Public Health Service (16). Curves based on counts of bacteria developing on gelatin would be quite similar to those based on agar counts.

HYDROGRAPHIC RELATIONS

Quiet and Running Water.—The forces of self-purification are essentially the same in quiet and running water but differ markedly in their intensity of action, some being more efficacious in running water, others in quiet water. The old adage that running water purifies itself seems to exclude the possibility of quiet water doing the same. Opposing this erroneous impression Sedgwick (17) has stated:

> Sedimentation and the destruction of microorganisms 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."

While Sedgwick thus stressed the action of those forces of self-purification, *i.e.*, sedimentation and sunlight, that are more effective in quiet than in running water, he did not give adequate consideration to the oxygen requirements of grossly polluted waters, in which atmospheric re-aeration becomes of considerable moment. It is easily conceivable that a body of water may be so seriously polluted that it would become putrid, if practically quiescent, while it would maintain an adequate supply of oxygen if its flow were sufficiently turbulent. The relative rates of re-aeration of quiet and running water have been previously noted. The flow of deoxygenated water over dams presents even more striking evidence of the beneficial effect of re-aeration. Thus Forbes and Richardson (10) record an increase in the dissolved-oxygen content of the Illinois River at the Marseilles Dam during July and August, 1911, from 0.64 to 2.94 p.p.m.

A factor favoring quiet water is the greater development of photosynthetic life, owing to the absence of turbulence and the improved penetration of sunlight resulting from the rapid deposition of suspended matter. In the Illinois River system, for example, Kofoid (18) noted a ratio of channel plankton to backwater plankton of 1 to 3.4 during the years 1894 to 1899. He remarks, furthermore, that the river plankton in a large degree has its source in the impounded backwaters.

Just how great is the relative cumulative effect of the various forces of self-purification in quiet as compared with running waters cannot be estimated without reference to the pollution load and the characteristics of the different bodies of water.

Goodnough (19) has stated from experience with ponds at Easthampton, Attleboro and elsewhere, that sewage discharged into a pond or slow-moving stream, such as the Charles River Basin, has a less noticeable effect than an equal volume of sewage has upon a rapidly moving stream of equal volume. He further said:

> In connection with public water supplies, the advantages of the storage of polluted water in large reservoirs in the removal of the effect of pollution have been recognized for many years, and the available evidence furnished by the observations of the effects of the discharge of sewage into ponds in the State indicates that, whatever effect the sewage discharged into the proposed basin may have upon its waters, the effect is likely to be less than it would be in the case of the discharge of an equal quantity of sewage into a flowing stream receiving the same quantity of water.

The question of sludge deposits, which represent a characteristic difference between quiet and running waters, is discussed in a subsequent section of this chapter.

Fresh and Salt Water.—There are a number of possible differences in the behavior of sewage in salt water relative to self-purification. Among the conditions which influence this behavior the following are worthy of comment: degree of sedimentation obtained; biochemical-oxygen demand; dissolved oxygen available; rate of re-aeration; and biological considerations.

When sewage is discharged into salt water, there is a greater tendency for the suspended solids to settle and thus to form sludge banks than when it is discharged into fresh water. This increase in sedimentation, in spite of the greater density of sea water or brackish water, is due in part to the coagulation of the suspended solids by the sea salts.

The rate of deoxygenation of mixtures of sewage and salt water has not been adequately explored. Clark (19) concluded from experiments with bottles of polluted sea and distilled waters that the oxygen in the salt-water mixture was absorbed much more rapidly than in the fresh-water mixture. Adeney and Letts (20, 21), on the other hand, judged from independent studies that there was no great difference between the relative behavior of the two kinds of water.

The oxygen-saturation value of sea water is normally about 20 per cent less than that of fresh water. Hence under open-air conditions of oxygen saturation the oxygen reserve of salt water is lower than that of fresh water by amounts varying from 3 p.p.m. at 32°F. to 1.5 p.p.m. at 80°F. (see Table 6).

Adeney concluded from experiments that salt water, when depleted of oxygen, reabsorbed this gas from the air at more than double the rate of distilled water. He attributed this to a "streaming effect" by which the saturated surface water was carried down more quickly in salt than in fresh water. The streaming is associated by Adeney with increase in density of the exposed surface film, due to cooling and concentration of salts by evaporation. Phelps (5), on the other hand, has argued that salt water will probably absorb oxygen less rapidly than fresh water. Experimental results of the Metropolitan Sewerage Commission of New York, presented by Gould (22), show no marked difference in the rate of re-aeration of salt and fresh water.

The effect of brackish water upon the plankton appears to be quite marked. In this matter osmosis undoubtedly plays an important part. Quoting from Kayser (23):

> This is the tendency of solutions of different degree to mingle by passage of water through a permeable membrane. The membrane is permeable to water molecules, but not to the dissolved salt molecules. The osmotic pressure of sea water is 0.65 atmosphere for each 1 per cent of salt content, or a head of more than 6 meters. This is of importance biologically. Organisms that live in water generally have a skin that is not impermeable to water, but they are also adapted to a certain degree of salinity of the water; hence if they are suddenly put in water of lower salinity, the water will pass through the skin into the body, and conversely, they will exude water. Both of these actions are usually unfavorable to the life of the organism, and, therefore, the organisms endeavor to avoid such conditions as much as possible. Osmotic pressure thus tends to confine most marine animals within certain strata, and frequently the stratum is thin, corresponding to a narrow range of salinity.

If this theory is true, it follows that the plankton of fresh water cannot thrive in salt or brackish water and marine plankton cannot thrive in fresh or brackish water. Hence a body of salt water into which a fresh-water stream flows, being subject to variations in salinity due to the relative proportions of fresh and salt water, is not well adapted to the plankton, and it may be expected that the normal plankton of fresh water and the normal plankton of salt water will be present, diminished in numbers and in activity, or that species of plankton will be found which have the power of adapting themselves to the variable conditions. There are some reasons for believing that such bodies of water have not so great a capacity for disposing of sewage as other bodies of fresh or salt water. Little is known upon this subject, however, and a positive statement of the relative capacities of waters of varying salinity cannot be safely given at this time.

An actual example of the effects of varying salinity is cited by Field (19). He made a close study of Point Judith Pond, which was subject to a variable mixture of fresh and salt water, on account of frequent breaches made by the sea through a narrow neck of land separating the pond from the ocean. As a result of these studies a permanent opening to the sea was made, resulting in a constant degree of salinity and in improved conditions.

Relative to bacteria Clark (19) decided from the results of tests that:

1. The greater number of bacteria in the supernatant liquid were found under fresh-water conditions.
2. The greater bacterial growth in the muds occurred under fresh-water conditions.
3. The greater relative number of anaerobic growths occurred under salt-water conditions, both in the mud and in the supernatant liquids.
4. In the salt-water experiments the number of bacteria which, when the water was plated, would grow in hydrogen—*i.e.*, under anaerobic conditions—exceeded in number in some instances those which would grow in air—*i.e.*, under aerobic conditions.

Temperature Effects.—Of the various factors that must be considered in the self-purification of polluted waters, temperature is one of general significance, since it affects such physical forces of self-purification as sedimentation and re-aeration, as well as such biochemical or biological forces as deoxygenation and the life processes of aquatic growths. Both the rate of deoxygenation and the subsequent rate of re-aeration increase with rising temperatures and decrease with falling ones, the former more so that the latter. This, as shown in Fig. 16, is reflected in the shape of the oxygen sag.

At low temperatures the sag is flat, but long; at high temperatures it is deep, but short. Taking into account the fact that at high temperatures water can carry less oxygen in solution, it follows that septic conditions are more likely to prevail during warm weather. During cold weather, on the other hand, self-purification is evidently retarded and the zones of self-purification extend over longer river stretches. In the disposal of sewage by dilution, therefore, rivers, lakes or tidal estuaries may receive in winter, without causing objectionable conditions, quantities of sewage which in summer would give rise to such conditions. In streams, furthermore, the flows are commonly higher in the colder seasons of the year than during the summer. The combined effect of runoff and temperature variations upon the dissolved-oxygen content of the Merrimack River at Lawrence is shown in Fig. 21, the discharge of the stream being based on reports of the United States Geological Survey and the analyses on the reports of the Massachusetts State Board of Health for 1906 and 1907. Goodnough estimated the amount of polluting matter in the Merrimack River above Lowell as equivalent to that produced by a population of 70,000, and the equivalent population discharging sewage below Lowell but above Lawrence at 182,300. These observations were taken in years of average run-off; in dry years conditions must be worse. Conditions in the river have at times been

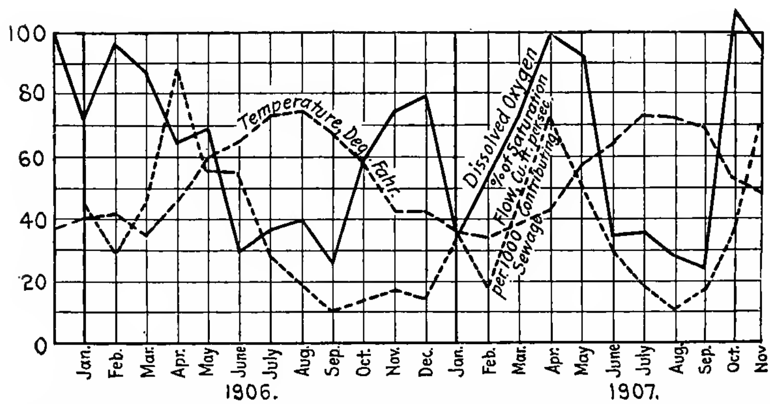


FIG. 21.—Relations of temperature, flow and dissolved oxygen in the Merrimack River at Lawrence.

such as to result in legislative inquiries and special studies by the State Department of Public Health.

Bacterial action at low temperatures is relatively slow and sewage may be carried by the water receiving it to a point where ample dilution is afforded before the bacteria have caused any offensive conditions. When freezing temperatures are reached, however, and streams become covered with ice, re-aeration is prevented and offensive conditions may obtain. This factor may be of considerable importance in heavily polluted waters in which the oxygen in the waters below the ice sheet may be depleted completely. Anaerobic decomposition will then be established with resulting noxious conditions. Cases of more or less complete deoxygenation of rivers used as public water supplies, when atmospheric re-aeration was cut off by ice cover, have been noted in the Schuylkill River at Philadelphia, Pa., 1882–1883, Maumee River at Toledo, Ohio, 1917, and Spring Creek at Richmond Hill, Ont.

Prevention of re-äeration by ice coverage and the resulting depletion of oxygen are strikingly illustrated by results obtained by the Public Health Service in studies of the Mississippi River below Minneapolis, Minn. (24). During July, 1926, with a water temperature of 22.6°C., the oxygen saturation of the river water was 24.4 per cent and its 5-day biochemical-oxygen demand 2.90 p.p.m. above Lake Pepin, the corresponding values at the outlet being 66.8 per cent and 0.99 p.p.m., showing an appreciable increase in dissolved oxygen and a significant decrease in B.O.D. during passage through Lake Pepin. During February, 1927, however, when ice coverage of the lake prevented re-aeration, the dissolved-oxygen saturation of the river water was 52.7 per cent at 0°C. above the lake and only 17.0 per cent at the outlet. This drop may have been due in part also to lack of biological re-aeration. During this month the 5-day B.O.D. values decreased from 2.62 p.p.m. above the lake to 1.55 p.p.m. at the outlet, thus indicating some stabilization of the pollution load even at this low temperature.

Of importance in connection with the oxygen resources and other factors of self-purification of polluted waters is their thermal stratification. This is particularly noticeable in large, deep lakes and is related to currents and mixing of waters and with them to the diffusion of oxygen, dispersion of pollution, distribution of aquatic life and other important considerations.

Sludge Deposits.—The settling of sewage solids in streams, lakes and tidal estuaries, while an aid to the clarifying of their waters, may intensify the pollution problems wherever sludge banks are formed. Offensive conditions may be created in the vicinity of such banks. Generally speaking, sludge deposits appear to become stabilized more slowly than do the transported solids, but whereas the latter are removed to points of greater dilution, the settled sewage matters remain to decompose until scoured away in rivers by floods, in lakes by wind-induced bottom currents and in the sea by tides and waves. It has been found by Baity, in the course of research at Harvard University, that oxidation takes place only at the surface of sludge deposits and anaerobic decomposition generally prevails within the sludge mass. In warm weather, therefore, gases and offensive odors may be given off from sludge banks even when the supernatant waters contain oxygen. During the winter, furthermore, decomposition is greatly retarded, and unless removed by spring freshets, tides or high winds, the winter's accumulation of sludge may be left to putrefy during the summer and thus add greatly to the pollution load of the waters. Anaerobic conditions are favored by thermal stratification in fresh water and by differences in density due to salt concentrations in brackish water.

In salt water Baity has found that decomposition of sludge deposits appears to be more protracted and hence less violent than in fresh water.

A striking example of the effect of sludge deposits was afforded by the condition of the Fens Basin at Boston in 1902. This basin, formerly a tidal estuary, had its waters held at a certain elevation by a dam and tidal gates. Muddy River and Stony Brook, two streams draining a thickly populated territory, discharged water which was practically weak sewage into the upper end of the basin. Heavy sludge deposits formed in the basin, amounting in 1903 to about one-fourth of its cubic contents. Clark investigated the dissolved-oxygen conditions in this basin and found marked stratification, due to a mixture of salt and fresh water, interfering with vertical circulation, so that the underlying water was rapidly deprived of oxygen as it moved over the sludge.

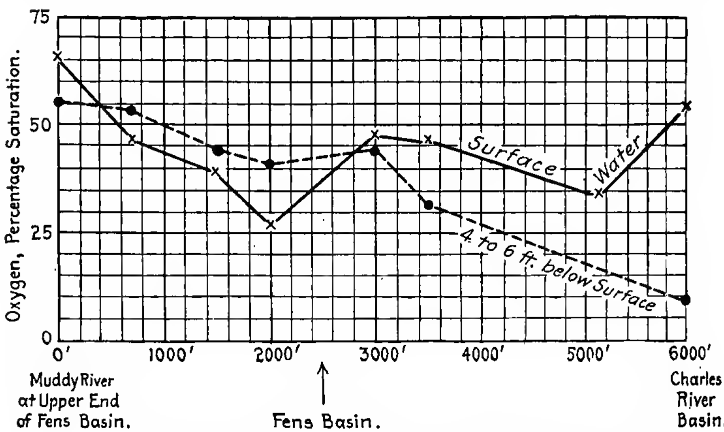


FIG. 22.—Effect of sludge deposits on oxygen content of Fens Basin.

His findings are plotted in Fig. 22. Quantitative estimates of the effects of sludge deposits upon the oxygen balance are discussed in the next chapter.

OBSERVED POLLUTION AND SELF-PURIFICATION OF NATURAL WATERS

A large number of comprehensive investigations have been made, especially during the past decade or two, of the pollution and self-purification of American streams, lakes and tidal waters. Much can be learned from a study of the reports incorporating the findings and the reader should consult the original documents for an adequate exposition of the factors involved in each case.

From the wealth of material a few examples are selected for the purpose of illustrating the different types of problems encountered and amplifying with actual observations the information presented in the preceding sections of this chapter.

Illinois River Studies.—Probably no river has been investigated as thoroughly through so many years as the system of watercourses of which the Illinois River forms the main artery. This system of waterways extends from Lake Michigan through the Chicago River, Chicago Drainage Canal, Des Plaines River and Illinois River proper for 357 miles to the mouth of the latter in the Mississippi River at Grafton (see Fig. 37).

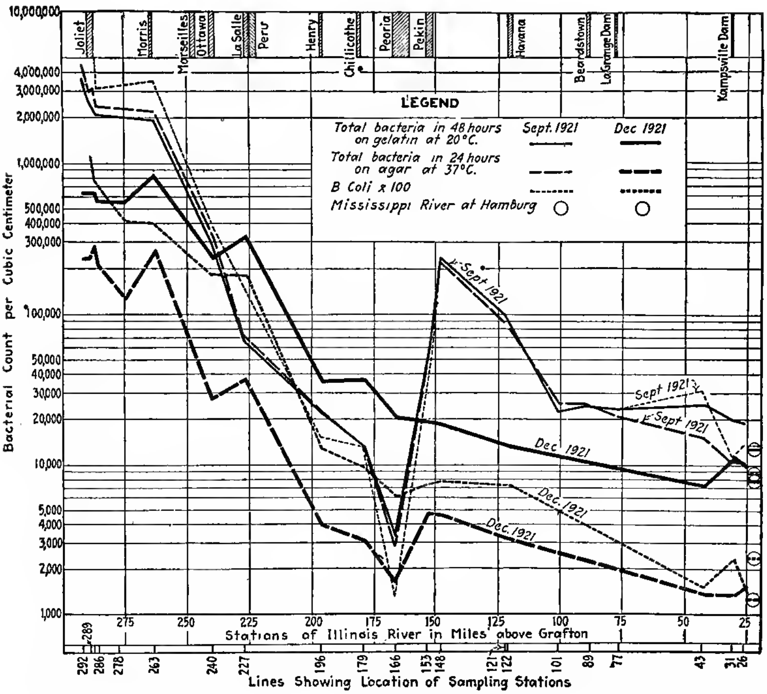


Fig. 23.—Comparison of summer and winter bacterial conditions in Illinois River.

Biological studies of the river system were begun by the Illinois State Laboratory of Natural History in 1894, even before the opening of the Chicago Drainage Canal, pollution reaching the Illinois River, however, through the Illinois and Michigan canal. These studies were at first conducted by Kofoid and have been continued to the present time by Forbes and Richardson. Chemical and general sanitary investigations have been made by Cooley in 1889 and 1891 and by Palmer in 1897 for the Illinois State Board of Health and by Bartow and Buswell of the Illinois State Water Survey. Extensive examinations of the self-purification of the Illinois River have been conducted in connection with the suit of the State of Missouri to restrain the State of Illinois and the Sanitary District of Chicago from discharging sewage diluted with water from Lake Michigan into tributaries of the Mississippi. Further studies have been made by the Chicago Sanitary

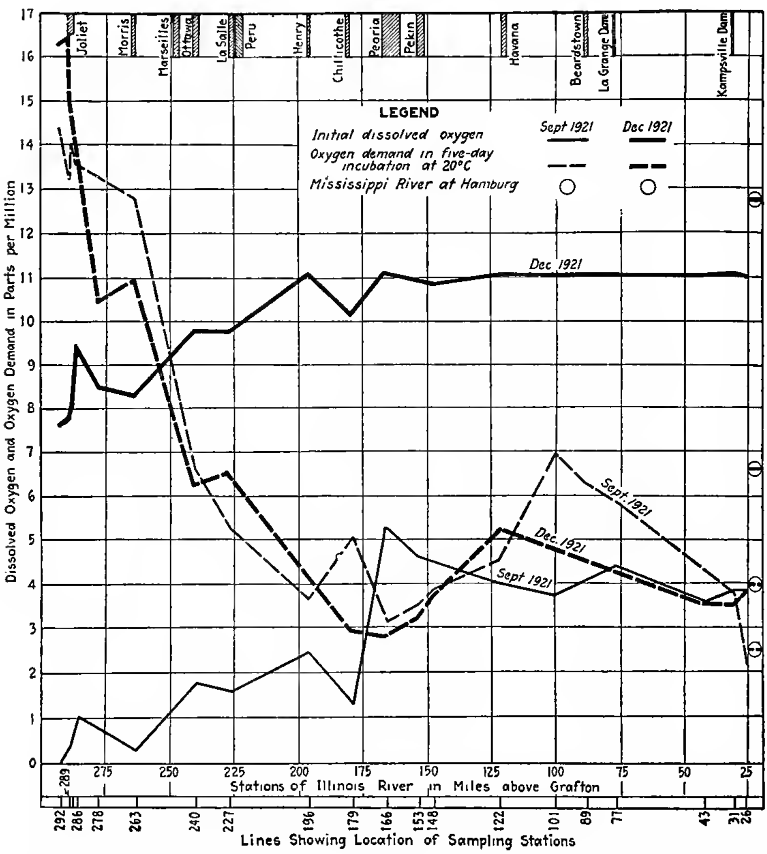


FIG. 24.—Comparison of summer and winter oxygen conditions in Illinois River.

District, by the Public Health Service and others, some of them in connection with the controversy over the lowering of the Great Lakes, a matter involving many and varied interests. No summary of the many findings is possible within the scope of this chapter. In order to exemplify the self-purifying powers of a heavily polluted stream, however, the following summary of results obtained by the Public Health Service in 1921 is taken from a paper by Hoskins (25):

During the summer months the total bacterial counts on agar at 37° (24 hr.) immediately below the drainage canal ranged from 2,000,000 to 5,000,000 per cubic centimeter, whereas in the winter months they have ranged from 100,000 to 300,000 (monthly averages). Since the volume of the Illinois River at stations 292-286 [see Fig. 23] is nearly constant, this indicates a wide seasonal variation in the total bacterial content of the sewage of the Sanitary District. Similar seasonal variations are noted in the gelatin count and *B. coli*. This is in accordance with observations at Cincinnati and Louisville on the Ohio River. . .

The decrease in bacterial content from the outlet of the Drainage Canal to station 166, immediately above Peoria, is at all times very great, and is practically parallel in the agar count, the gelatin count and the *B. coli*. The decrease is absolutely and relatively much greater in summer than in winter. Dilution by tributaries is a very small factor in this decrease, which is largely due to death of the bacteria. From a bacteriological standpoint the river immediately above Peoria is not highly polluted—not more so than many streams draining rural sections.

The pollution resulting from wastes received in the Peoria district is very considerable in summer and the decrease below this point, though steady, is less rapid than in the upper half of the river.

In the upper river the oxygen demand greatly exceeds the supply as represented by the dissolved-oxygen content, but in passing downstream from Joliet to Peoria the dissolved oxygen progressively increases and the biochemical-oxygen demand decreases until the former exceeds the latter at a point well above Peoria, establishing an "oxygen balance" [see Fig. 24]. In summer the extent of oxygen recovery in passage through Peoria Lake (a large body of wide shallow water between stations 179 and 166) is remarkable; in winter it is less marked but noticeable. Below Peoria the oxygen demand increases gradually to a point just above Beardstown, with, in summer, a corresponding decrease in dissolved oxygen and a reversal of the oxygen balance. Although a distinctly measurable increase in oxygen demand below the urban district of Peoria is to be expected, this gradual increase over a distance of 50 miles is not yet fully explained.

Other notable studies of stream pollution and natural purification by the Public Health Service are those of the Potomac, Ohio and Mississippi rivers.

Cowesett Brook.—The Illinois River studies exemplify the natural purification of a stream that is heavily polluted by crude sewage and putrescible industrial wastes. The effect of discharging the nitrified effluent of a sewage-treatment plant into a small stream is demonstrated by studies made by Weston and Turner (26) during 1914 and 1915 of the Cowesett Brook at Brockton, Mass. This stream has a flow of approximately 0 to 25 m.g.d. above the treatment works. It drains a sparsely settled farming region and less than 2 miles from its source it receives the effluent from the Brockton sewage-treatment works. About 0.7 mile below the works it is joined by a normal stream having a drainage area slightly larger than its own. During seasons of high runoff the discharge of the two streams is practically proportional to the drainage areas, but during periods of low flow the Cowesett may contain much more treated sewage than water and the ratio of flow in the two streams may be 3 to 1, that of the Cowesett being the larger. During the period of study the Brockton sewage, averaging 2.1 m.g.d. of septicized domestic sewage and industrial wastes from the shoe factories, was discharged intermittently on to 37 acres of sand beds at a rate of 60,000 gal. an acre daily, part of the sewage passing first through a trickling filter 0.5 acre in area. The principal conclusions reached by Weston and Turner are given below.

The whole study reveals the extreme importance and the energetic action of certain plants and animals in purifying a stream receiving sewage effluents.

The Cowesett Stream, which receives a more or less nitrified sand-bed effluent, first lays down a pollution carpet by precipitation of organic and mineral matter, including silica.

This false bottom contains forms of life typical of pollution and purification which furnish a better index of the true condition of the stream than do organisms found in the supernatant liquid.

The entrance of organic matter into the stream increases plant and animal life in a series, beginning with the lowest and ending with the highest forms, each class appearing along with a definite food material.

Comparison of conditions on the Cowesett with investigations of other streams shows that the smaller, shallower and more nearly stagnant the body of water, and the more highly nitrified and clarified the effluent, the more rapid is the succession of zones of higher animal life, and the more rapid and complete the purification process. . . .

The return of the stream to a normal biological condition is more rapid than its return to a normal chemical condition; and the rate of chemical change is much less rapid below the point where biological processes have ceased their unusual activities. . . .

In this stream the dissolved oxygen tends to decrease until the excessive growth of animal life, favored by pollution, ceases. This factor outweighs both the normal re-aeration of the stream and the oxygenating effect of plant growth.

The carbon dioxide introduced with the effluent tends to decrease downstream, at first rapidly and afterwards very slowly.

The rate of the oxidation of nitrogenous compounds varies with the season, and with the intensity of organic growth; it is very rapid in summer and hardly discoverable in winter.

The types of organisms which most clearly indicate the degree of pollution are bacteria, protozoa and diatoms.

Chemical changes in the stream are due to bacteria more than to any other group of organisms. The higher animals are important by removing an excess of lower organisms.

The rate of decrease of bacteria is most rapid at the point of entrance of the effluents, and except during the winter floods very slow below the first mile. . .

Different species of higher plants will tolerate pollution in different degrees. They are important in holding back suspended matter and preventing small organisms from being washed down by the current; they also assist in the removal of colloidal suspended matter and in checking the flow of the stream.

Fishes (trout, suckers) were occasionally observed even in the most polluted part of the stream.

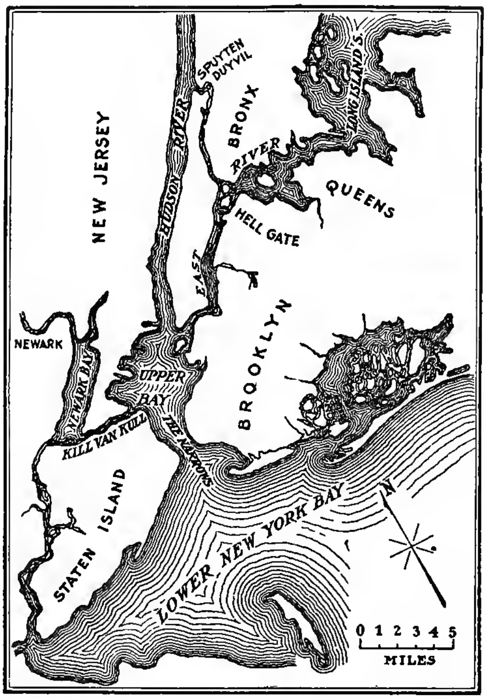


FIG. 25.—New York Harbor and adjacent waters.

The trouble from aquatic growths observed on the Hockomock River, to which the Cowesett is tributary, has been discussed on page 171.

Lake Michigan at Milwaukee.—An example of the pollution and self-purification of lake waters is furnished by conditions in Lake Michigan at Milwaukee, Wis., in 1909 to 1911, which are discussed in Chap. VIII.

New York Harbor.—Among tidal estuaries the pollution and self-purification of New York Harbor (Fig. 25) have been investigated most thoroughly by the Metropolitan Sewerage Commission and by other authorities. Extensive examinations were made also in connection with the suit of the State of New York to restrain the State of New Jersey from discharging into Upper New York Bay the sewage and trade refuse of 37 municipalities and townships in the Passaic Valley watershed in that State.

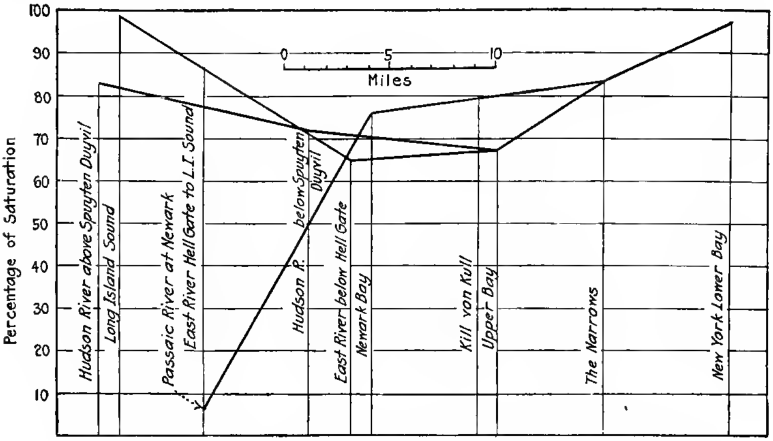


FIG. 26.—Dissolved oxygen in New York Harbor and adjacent waters.

The main results obtained by the MetropolitanSewerage Commission are given in Fig. 26, plotted from figures in the 1910 report. This shows

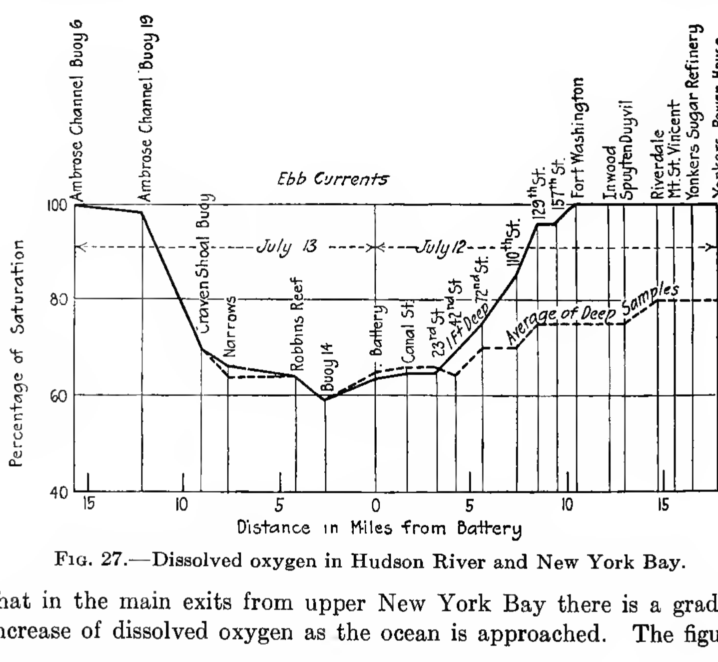


FIG. 27.—Dissolved oxygen in Hudson River and New York Bay.

that in the main exits from upper New York Bay there is a gradual increase of dissolved oxygen as the ocean is approached. The figures are averages of the oxygen found at various depths and at various conditions of tidal currents. The percentage of saturation at the same time at different points in a cross section of channel varies considerably. The samples were taken in the center of the current. In practically all cases, the waters of a flood current had more oxygen than those of the corresponding ebb current. In the Hudson River it was found that the saturation at the surface down to Fort Washington was 100 per cent on July 12 and 13, 1911, but the average of deep samples was much less, as shown in Fig. 27.

Bibliography

1. WHIPPLE, G. C.: "Microscopy of Drinking Water," 4th ed., 1927.
2. *Pub. Health Bull.* 146, U. S. Public Health Serivce, 1925.
3. *Trans.*, A. S. C. E., 1926; **89**, 1331.
4. *Jour. Ind. Eng. Chem.*, 1924; **16**, 1215.
5. BLACK, W. M., and E. B. PHELPS: Rept. to Board of Estimate and Apportionment on Discharge of Sewage into New York Harbor, 1911.
6. ADENEY, W. E.: "The Principles and Practice of the Dilution Method of Sewage Disposal," 1928.
7. Rept. on Investigation of the Pollution of Streams, N. Y. State Conservation Comm., 1923.
8. *Hyg. Lab. Bull.* 104, U. S. Public Health Serivce, 1916.
9. *Eng. News*, 1911; **66**, 246.
10. *Bull.*, Ill. State Laboratory Natural History, vol. 9, art. 10, 1913.
11. EDDY, H. P.: *Am. Jour. Pub. Health*, 1929; **19**, 641.
12. *Ann. Rept.*, Maryland State Board of Health, 1928.
13. LE PRINCE, J. A.: *Munic. Sanitation*, 1931; **2**, 10.
14. *Bull.* 348, N. J. Agric. Experiment Station, 1921.
15. *Tenth Rept. on Res. Work*, Metrop. Water Board, London, 1910.
16. *Pub. Health Bull.* 143, U. S. Pub. Health Service, 1924.
17. Rept. of Pittsburgh Filtration Comm., 1899.
18. *Bull.*, Ill. State Lab. Natural History, vol. 6, art. 2, 1903.
19. Rept. of Comm. on Charles River Dam, 1903.
20. Fifth Rept., Royal Comm. on Sewage Disposal, App. 6, 1908.
21. Seventh Rept., Royal Comm. on Sewage Disposal, 1911.
22. GOULD, R. H.: *Trans.* A. S. C. E., 1922; **85**, 699.
23. KAYSER, JOHANN: "Physik des Meeres," transl. by Emil Kuichling, 1911.
24. Second Ann. Rept., Metrop. Drainage Commission, Minneapolis and St. Paul.
25. HOSKINS, J. K.: *Eng. News-Record*, 1922; **89**, 1078.
26. Contributions from San. Research Lab., Mass. Inst. Tech., vol. 10, 1917.

CHAPTER VIII

SEWAGE DISPOSAL BY DILUTION

As reported in Chap. I in the discussion of the present status of sewage disposal in the United States, records of the sewerage works of cities with populations in excess of 100,000 in 1930 show that the sewage from about 70 per cent of the total population of 36,450,000 represented by these cities was discharged into water without treatment other than fine screening in certain instances. According to these records, furthermore, the sewage from 16 per cent of this population received only tank treatment and from 14 per cent complete treatment prior to discharge into water. Taking into account the limitations of the process described in Chap. VI and the varied uses of water, the magnitude of the problem of sewage disposal by dilution is evident.

Basic Information for Planning Dilution Works.—As shown in Chap. VII, the disposal of sewage by dilution is rendered practicable not merely by the dispersion of the waste matters in large volumes of water but, in the last analysis, by the self-purifying powers inherent in natural bodies of water. Intelligent planning of dilution projects requires a thorough appreciation of the processes of self-purification and the various factors that determine the characteristics and behavior of sewage in water. It presupposes, furthermore, a knowledge of such hydrographic elements as govern, in one way or another, the dilution secured and the interlocking operation of the many forces of self-purification.

The information required varies greatly under different circumstances. It may involve the following: studies of the wastes to be disposed of, including the value of treatment prior to discharge; hydrographic surveys or examination of hydrographic records, including, (*a*) in the case of streams, runoff records and characteristics of flow, both at and below the point of disposal, (*b*) in the case of lakes, current observations and the effects of winds, seiches and temperature stratification upon the dispersion of the sewage, (*c*) in the case of tidal estuaries, tides and the effects of winds, salinity and temperature stratification upon the movement of the sewage; studies of possible locations for, and forms of, the sewer outfall in its relation to hydrographic conditions, particularly in the case of lake and ocean outfalls; and studies of the various uses of the water receiving the sewage, including the protection of water supplies and the relative value and economy of sewage treatment and water purification, the safeguarding of bathing beaches and other recreational facilities, the conservation and protection of useful aquatic life and its commercial value in relation to sewage treatment, the avoidance of unsightly or offensive conditions created by the sewage matters on or in the waters or along the shores, and the prevention of sludge-bank formation and of the resulting encroachment on waterways.

Hydrographic Surveys.—As far as hydrographic studies are concerned, the discussion of runoff records and currents due to the normal hydraulic displacement of water requires no elaboration in such a book as the present. Special problems of horizontal as well as vertical water movement induced by winds, tides and other meteorological as well as hydrographic factors, however, are worthy of some comment.

The path of currents that are not defined by the normal flow of water along the hydraulic gradient may be studied in various ways, the selection of which must depend upon local conditions. Thus it is possible to observe pronounced currents of tidal origin by means of floats. Interpretation of the results obtained by float observations may be aided, however, by salinity observations and in some cases by temperature studies. Although float observations may have some value in the tracing of the less pronounced horizontal currents in lakes and other standing bodies of fresh water, recourse is generally taken to temperature observations and chemical and biological analyses in defining both the vertical and horizontal travel of water and sewage under the influence of wind and other factors. The methods employed, with the exception of analytical procedures, and the underlying principles are described in succeeding sections, those applying particularly to ocean outfalls being considered first.

HYDROGRAPHIC SURVEYS FOR OCEAN OUTFALLS

Float Observations.—The Metropolitan Sewerage Commission of New York, in its observations of currents, used floats of the types shown in Fig. 28 and described as follows in its report for 1910:

Can Floats.—The first consisted of tin cylinders, an upper and a lower one connected by a wire. The upper cylinder was 5⅜ in. in diameter by 5 in. in length; it was empty and sealed and carried a small red flag on a staff set in a socket on the upper end of the cylinder. From this upper can, which in action was partly submerged, a larger can 6¼ in. in diameter by 14 in. in length was suspended by a copper wire of such length as to permit the larger can to float in the current whose velocity was to be determined. This larger can was weighted with sand until the top of the upper can was nearly level with the surface of the water. This type of float had the advantage of ease of handling, ease of preparation for use, a small area exposed to wind and small cost. On the other hand, where traffic was congested—as in the

East River—they were destroyed by the paddles and propellers of steamers. When required for night work they were unable to carry a lantern.

Spar Floats.—Another type of float was made of a 3- by 3-in. by 6-ft. stick buoyed at one end by being built up to 12 by 12 in. for 24 in. from the top and weighted at the other by four vanes of No. 14 gage iron 18 by 21 in. in size, secured by bolts. A 3/8-in. rod projected about 4 ft. above the top and was provided with two arms, from which were suepended red and white lanterns at night. As this float was heavy and difficult to handle, and as the rod was easily bent, a light stiffening frame of 1/8- by 2-in. iron was

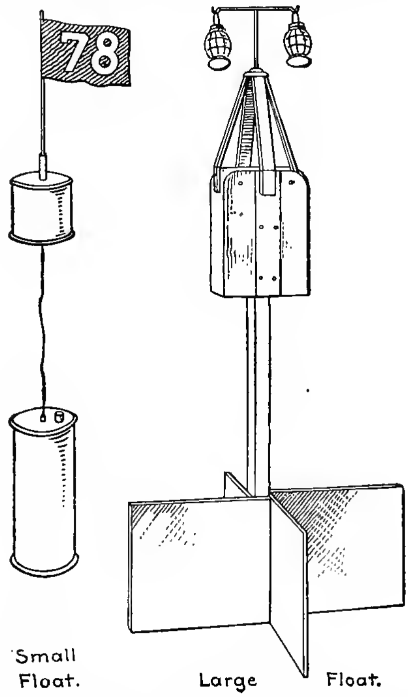


FIG. 28.—Types of floats used by Metropolitan Sewerage Commission.

attached to the head of the float and supported the rod just below the lanterns. To this frame was welded a hook to be grappled in removing the float from the water. This design proved satisfactory for the rough seas experienced in December, 1909, in the Lower Bay and among the tugs, car floats and ferries of the East River.

These floats were followed day and night and their position was determined at frequent intervals by means of a sextant or by estimating the bearing from some known point.

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 tend to rise to the surface and will then move over the surface more rapidly than will deep 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 (1).

Salinity Observations.—The measurement of the salinity of the water in a somewhat polluted tidal basin sometimes affords important information concerning the possibility of discharging more sewage into it. For instance, the tidal currents in the East River at New York have been reported by the U. S. Coast and Geodetic Survey to have maximum, average and minimum velocities of 7.8, 5.0 and 2.5 ft. per second, respectively. It was held for a long time that in the tidal movements there was a resultant southerly flow from Long Island Sound to New York Upper Bay amounting to about 11 per cent of the northerly flow. The subject was reviewed for the Metropolitan Sewerage Commission of New York by Tittman, who decided that it was difficult to detect a net flow in either direction and if there was an excess toward the south, it could not exceed 1 per cent. Float observations by the Commission confirmed this opinion that there was no resultant flow. Observations of the salinity of the water were finally made, which established the fact that there is such a flow, and consequently nonsettling sewage matter discharged into the East River will eventually be carried out to sea, if not removed in some other way earlier.

This method of studying tidal flow is based upon the fact that, while 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. Off New York, it is about 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 chlorides. In this case, a sample of harbor water having a specific gravity of 1.015 must be a mixture of 60 per cent of sea water and 40 per cent of fresh water.

A description of the Commission's methods has been given by Allen (2) in a paper on "The Use of the Salinometer in Studies of Sewage Disposal by Dilution." The instrument is 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 hydrometric reading is corrected by amounts ranging from -0.0011 at 35°F. to $+0.0028$ at 82°F. The corrections were determined by experiment. The salinity of brackish water and sea water may also be found from the chlorine content of water samples as discussed in Chap. III.

The salinity of tidal waters has a direct effect on the vertical currents of the sewage discharged into them. According to Allen, experiments with colored water or sewage discharged below the surface in waters of different densities and at different depths did not always give harmonious results. In general, where the injected liquid had a specific gravity from 0.004 to 0.016 less than that of the harbor water and the depth of discharge was between 20 and 40 ft., its rate of ascent was from 0.10 to 0.17 ft. per second. This rate is about one third that of varnished wooden balls, carefully fitted up to have a specific gravity of unity, which were released at different depths below the surface.

Tides and the Tidal Prism.—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 back and forth 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 sewer system, in order to be sure that offensive conditions will not arise at any stage of the tide and that bathing beaches and shellfish layings will not become polluted.

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 landward with increasing velocity until the maximum velocity, or *strength of tide*, is reached. The velocity then falls off until ebb tide begins, when the whole mass of water flows seaward 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 in a tidal estuary 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 generally decreases in going from the northern to the southern limits of the United States, as follows:

East Coast		West Coast	
City	Range, ft.	City	Range, ft.
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		

In order to find the quantity of diluting water, the volume of the tidal prism must be computed. An example of the necessary calculations follows:

Example.—Find the dilution afforded a sewage flow of 2.3 m.g.d., which is discharged at the upper end of a tidal estuary with a surface area of 50 million sq. ft. at mean low water and 53 million sq. ft. at mean high water, the mean range of tide being 9.5 ft. Float observations show that three successive ebb tides are required to carry the sewage seaward from the outfall to a point from which it will not return to the estuary. The average cycle of tides occupies a time of 12 hr. 20 min.

$$\text{Volume of tidal prism} = \frac{50 + 53}{2} \times 1{,}000{,}000 \times 9.5 \times 7.48$$

$$= 3660 \text{ million gal.}$$

$$\text{Volume of sewage} = \frac{2.3 \times 3 \times 12.33}{24} = 3.55 \text{ million gal.}$$

$$\text{Dilution} = \frac{3.55}{3660} = 1{:}1030$$

Actual conditions encountered in practice are seldom capable of as simple statement as is this example. Studies of currents are complicated by the necessity of considering the inertia of the mass of moving water, the configuration of the bottom and the frictional resistances. Where there are several inlets from the ocean to a tidal basin that is fed also by several fresh-water streams of appreciable volume, as in New York Harbor, the conditions are sometimes quite complicated.

If the volumes of water moving in and out of a tidal basin on the flood and ebb tides and also the volume of upland water that passes out on each tide are known, the volume of new sea water that enters during each flood tide to assist in replenishing the supply of dissolved oxygen can be computed. Hazen's formulas (3) for this purpose are as follows:

If E = the volume of ebb tide passing any place

R = the volume of river water in the ebb tide which does not return

S = the average proportion of sea water in ebb tide

then $E(1 - S)$ = the river water passing out at one ebb

and $\frac{R}{E}(1 - S)$ = the proportion of river water that passes out and does not return

Using these formulas, Allen has computed that under normal conditions about 17 per cent of the water coming into New York Upper Bay was fresh sea water.

Extent of Sewage Field in Salt Water.—Experiments have been carried on by Rawn and Palmer (4) at Los Angeles, Cal., with a view to determining the dilution of sewage obtained at the ocean surface above a submerged outlet, from which sewage is discharged, as well as the area and extent of the sewage field formed on the surface above such an outlet.

HYDROGRAPHIC SURVEYS FOR LAKE OUTFALLS

Temperature Observations and Wind-induced Movements.—The thermometry of lakes, as well as of other bodies of water, will yield information that is often of assistance in tracing and explaining the magnitude and direction of vertical and horizontal currents which are largely induced by wind action. Temperatures may be determined by means of mercury thermometers, either read directly at the surface or placed in sampling bottles for subsurface explorations. There are many useful types of thermometers, among which the thermophone, developed by Warren and Whipple, has been employed by the authors with success. This instrument is an electrical resistance thermometer, in which a current interrupter and a telephone permit the determination of the temperature at any desired depth to which the resistance coil is lowered.[1]

Another instrument depending upon similar principles is manufactured by The Leeds Northrop Company, Philadelphia, Pa., but in this case the telephone and current interrupter are replaced by a galvanometer. The latter instrument has been found by the authors to be more satisfactory in calm weather, but in rough weather, when the

[1] For a description of this instrument, manufactured by the Sanborn Co., Cambridge, Mass., see G. C. Whipple, "Microscopy of Drinking Water," 4th ed., 1927.

roll of the boat caused the needle of the galvanometer to oscillate, the thermophone proved more satisfactory.

Thermal Stratification.—The relation between temperatures and wind-induced currents is idealized in Fig. 29, adapted from Whipple (5). As shown the wind blowing over the surface of a body of water tends to carry the surface water in the direction of wind movement, owing to the frictional forces exerted by the currents of air upon the water surface. On-shore and off-shore winds, by piling up the water along the leeward shore or carrying it away from the windward shore, furthermore, induce complementary currents in opposite directions at greater depths. If the wind sweeps along the shore, the water currents may be unidirectional for long distances. If points of land jut out into the

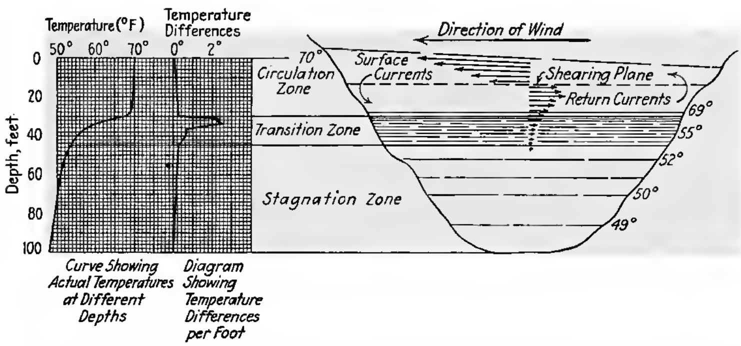


FIG. 29.—Idealized temperature gradient and relative horizontal velocity of wind-induced currents in a lake.

water, the surface water approaching them may be deflected outward and continue at the surface as eddy currents.

As shown in Fig. 29, the depth to which the surface and return currents penetrate is closely related to the thermal stratification of the water. The vertical distribution of temperatures, or the temperature gradient, indicated in Fig. 29 is characteristic of summer conditions. It will be noted that the temperature differences are small in the waters near the surface that lie within the "zone of circulation," that they are followed quite suddenly by large differences within the "transition zone," to which attaches the "zone of stagnation" with temperatures near the value of maximum density of water. Under summer conditions the greater the difference in temperature between successive layers of water, the greater is the difference in density of the water, and the greater, therefore, the "thermal resistance" to mixture, *i.e.*, the force required to displace colder, lower strata of water by warmer, upper strata. Therefore circulation is restricted to the layers of water above the zone of appreciable differences in temperature. As the surface water cools in the autumn, it becomes heavier and sinks, thus causing currents, which reach to increasing depths until all the water acquires the temperature of maximum density, 39.2°F. If it were not

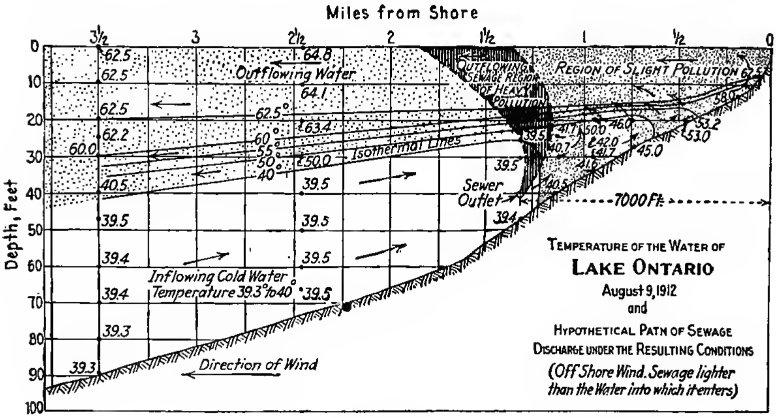


Fig. 30.

for the wind, circulation would then cease but the slight differences in density near this temperature facilitate action by the wind, so that the whole body of water probably reaches a somewhat lower temperature than 39.2°F. In the spring, when the icy surface water becomes

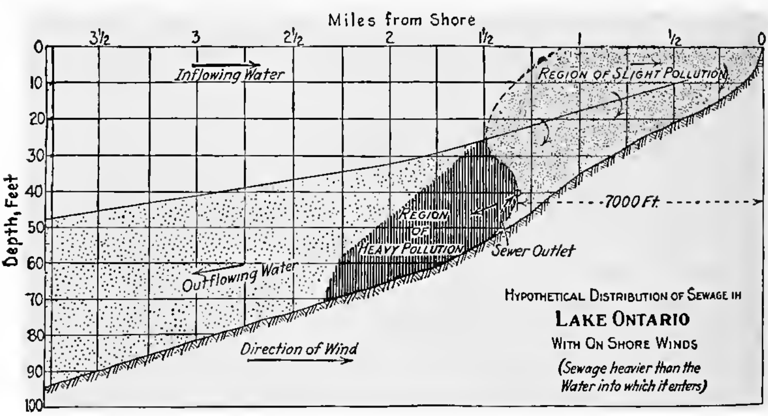


Fig. 31.

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°, owing to wind action, as previously noted. This subject has been investigated by FitzGerald (6) and others.[1]

[1] For a summary of these studies see Whipple's "Microscopy of Drinking Water."

When sewage is to be discharged into fresh water, its path can generally be predicted from wind and temperature observations made in the vicinity of the proposed outfall. Whipple's estimates, made in 1912, for the Rochester, N. Y., outfall in Lake Ontario with on- and off-shore winds during the summer, when the possibility of pollution of near-by bathing beaches is of importance, are shown in Figs. 30 and 31 (7). It was assumed in these estimates that the sewage is warmer than the bottom water, but colder than the surface water.

Whipple has given the following explanation of his estimates:

> If the warm summer sewage were discharged into the cold bottom water, as it would be with an off-shore wind, being lighter, it would rise quickly toward the surface and be caught by the outward surface current and carried away from the shore. Only a comparatively small proportion of the sewage would reach the shore under these conditions. Just what proportion would be carried toward the shore cannot be calculated from the data at hand, but it seems certain that the amount would be very small. Thus the effect of southerly winds which ordinarily prevail at this season of the year would be to carry the sewage out into the lake.
>
> With the warm surface water flowing out at the bottom, the sewage would enter bottom water of approximately its own temperature; at times the sewage might be even colder than the water. Under these conditions there would be less tendency for the sewage to rise to the surface, and a large proportion of it would be carried outward by the outgoing bottom current. Even if the temperature were slightly warmer than the bottom water of the lake at the outlet, the difference in specific gravity would be so small that sewage would rise slowly into the upper currents, so that the amount carried toward the shore under these conditions would be relatively small, although probably greater than during the prevalence of off-shore winds.

As a result of his studies Whipple concluded that the sanitary quality of the water along the beaches would not be impaired by the discharge of settled sewage into the lake at the point proposed. The authors' findings during the summer of 1929, substantiating Whipple's conclusions, are discussed later in this chapter.

Wind Velocity and Velocity of Surface Water.—On Lake Michigan the travel of the surface currents is about 5 per cent of the wind travel and they may extend to a depth of 30 to 40 ft. Judson (8) reports observed surface-current velocities of 2.3 ft. per second and probable velocities of 4.4 ft. Wheeler (9) states that continuous winds from one direction in the Gulf of St. Lawrence produced an appreciable velocity of flow at a depth of 30 ft. In the North Atlantic ocean a study of many observations by the United States Hydrographic Service led to the conclusion that the set of the surface currents amounted to about 3.2 per cent of the wind velocity (10). Somewhat similar figures were obtained by Whipple (11) in Lake Erie, where the currents due to wind ranged from 3 to 6 per cent of the wind travel. In small lakes the ratio of water to air velocities is less than in large bodies of water with a considerable "fetch" or free sweep of the winds.

A study of the effect of wind movement on the distribution of sewage through water was made in 1911 on Lake Owasco, from which the water supply of Auburn, N. Y., is obtained (12). This is a long, narrow lake having an average width of about 1 mile and a length of about 10 miles. Floats 5, 10, 15 and 20 ft. long, provided with large wings or sheets of metal which were supported at the designated depths, were used in the study. Their motion indicated a movement of the water at a depth of 5 ft. varying from 3¼ per cent of the wind velocity, when it was blowing 5 m.p.h., to 1 per cent, when blowing 30 m.p.h. They also indicated that, while at low velocities the currents at 5-ft. depths were two to three times greater than those at a depth of 20 ft., at high wind velocities the differences between the currents at these depths was not great.

While the direction of water movement is generally the same as that of wind movement, the shore and bottom topography of small lakes may change the direction of currents appreciably.

Depth of Wind and Wave Action.—As previously noted, wind-induced water movement appears to extend to depths of 30 or 40 ft. in the Great Lakes. Observations on these bodies of water have shown that deposits of sludge near the shore at these depths have been removed by scour.

If the depth at which the bottom changes from mud or clay to sand is a criterion of the depth of influence of wave action, the influence reaches a depth of 55 to 60 ft. at Duluth, 40 to 45 ft. at Chicago and Milwaukee, and 33 to 38 ft. at Cleveland. In tidal basins, the scouring action of the currents is at times sufficiently reinforced by the wind to carry away sludge banks that otherwise would not have been moved.

The vertical circulation in the Charles River Basin, Boston, Mass., due to wind and waves has been studied by Sanborn (13). In the spring of 1909 the basin contained a mixture of fresh and salt water and the effect of winds was determined by observing the vertical distribution of chlorides through the water. The greatest depth at which complete mixing of the water was caused in this way was 5 ft. with a wind velocity of 5 m.p.h., 10 ft. with a wind of 7 m.p.h., 15 ft. with a wind of 9½ m.p.h., and 20 ft. with a wind of 14 m.p.h. Sanborn doubted if the same results could have been obtained at other seasons of the year, owing to different proportions of fresh and salt water, and he did not consider them applicable to a lake of similar dimensions containing fresh water only.

The depth and force of wave movements are, naturally, also of importance in the structural design of outfalls. This phase of the subject has been studied particularly by Gaillard (14).

Seiches.—Besides the influences of wind and temperature there are certain movements in large bodies of enclosed water caused by local changes in barometric pressure and by winds blowing for a long time in one direction, known as seiches. These movements consist first of a piling-up of the water at the leeward end of the lake or the end at which there is a low barometric pressure. After the immediate cause is removed, the body of water tends to oscillate in a rhythmic period which,

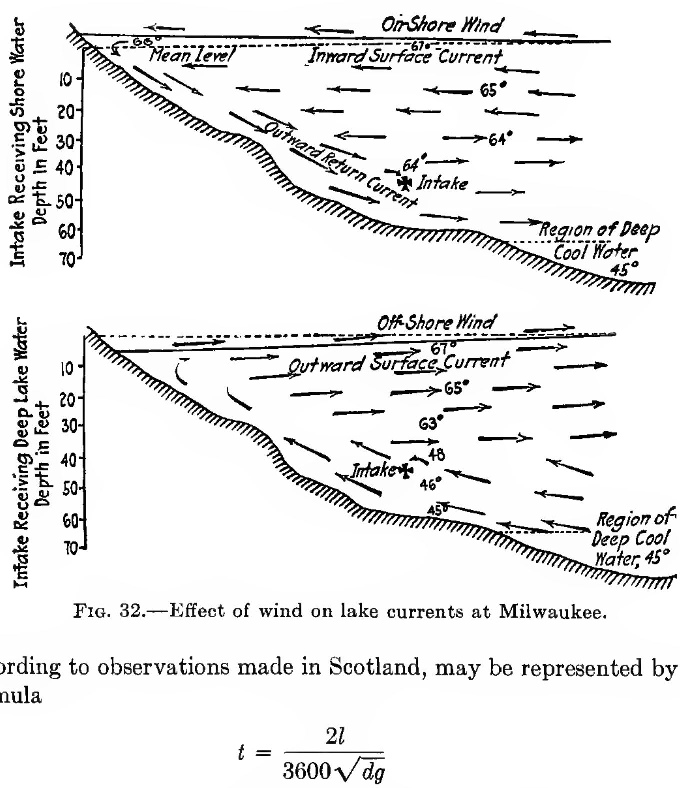


FIG. 32.—Effect of wind on lake currents at Milwaukee.

according to observations made in Scotland, may be represented by the formula

$$t = \frac{2l}{3600\sqrt{dg}}$$

where t is the time of oscillation in hours, l is the length of the lake in feet, or its width in the case of a transverse seiche, d is the mean depth in feet of the lake along the line of oscillation, and g is the acceleration of gravity.

This formula has been found by Whipple (11) to agree very well with observed conditions on Lake Erie during storms. The period of seiches is of less importance to the sanitary engineer than the horizontal water movements that accompany them.

Investigations at Milwaukee, Wis.—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. As the storage 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.

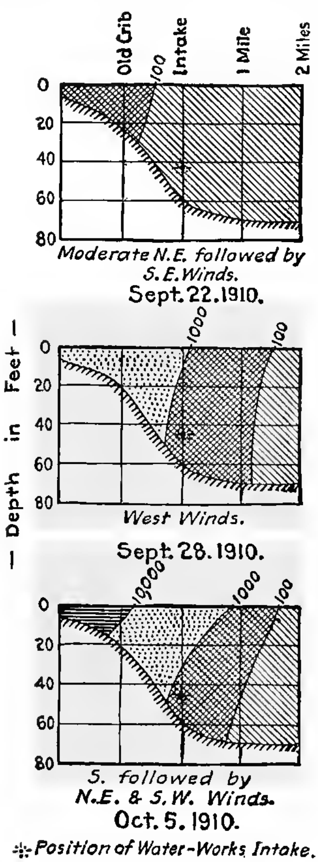


Fig. 33.—Effect of winds on vertical distribution of bacteria in a plane passing east and west through the intake of the Milwaukee waterworks.

Figure 32 illustrates the manner in which temperature records define the cause of wind-induced shore currents. An on-shore wind carries surface waters to the intake; an off-shore wind brings in the lower water strata. The quality of these waters depends upon the location of the source of pollution, as shown in Figs. 33 and 34. Thermophone readings showed that the water below a depth of about 65 ft. was 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 bacteria in samples of water collected at numerous points, both at the surface and at the bottom, and by determinations of the chlorides in the water at the same points. The results obtained on three days are given in Figs. 33, 34 and 35, illustrating how the wind affected the distribution of the sewage through the lake in the vicinity of the city.

On September 22, 1910, 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 quantity of chlorides, was driven northward and kept near the shore and the largest numbers of bacteria were found near the shore. The sewage of the city was discharged at that time into the lake by the rivers shown in Fig. 34.

On September 28, there was a light off-shore wind and there was little difference between the surface and bottom temperatures. The

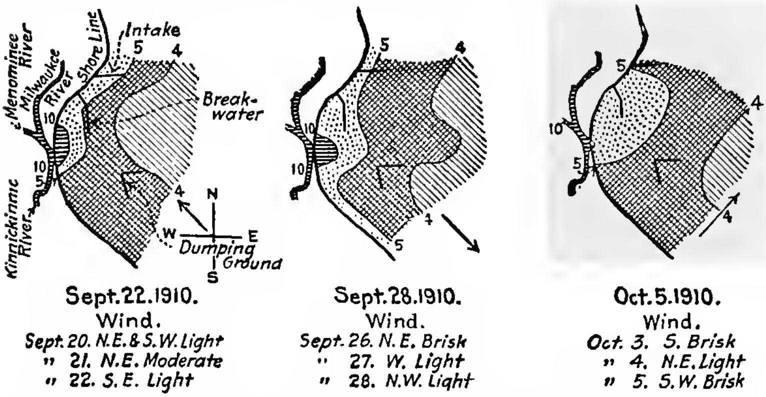


Fig. 34.—Effect of winds on distribution of chlorides in surface waters of Milwaukee Bay.

turbidity was high on this day. The chlorides extended more directly from the mouth of the river, whence the pollution came, and were distributed more widely in the bay. The surface water, containing the largest number of bacteria, was then spread out from the shore.

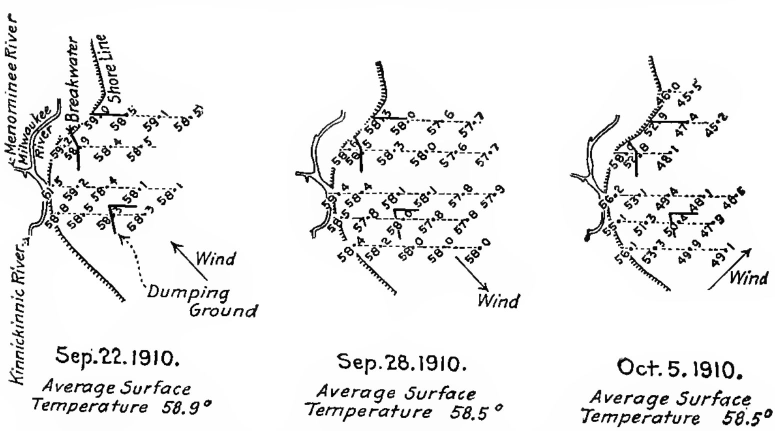


Fig. 35.—Effect of winds on temperaturés of bottom waters in Milwaukee Bay.

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 13.3°F. The surface water, containing the largest quantity of chlorides, 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. The importance of a study of prevailing winds and the currents produced by them is evident.

DILUTION REQUIREMENTS

Standards of Dilution.—As previously stated, the dilution method of sewage disposal is limited in any particular case by the uses to which the water receiving the sewage is to be put. Standards of dilution, therefore, must be set with this in mind. Where the available dilution is small, the standards must include standards of sewage treatment. In the United States no general standards of efficiency of sewage disposal have been adopted as yet. There has been amassed, however, as a result of practical experience and research, certain information which will serve as a guide in problems of disposal by dilution. This information relates in particular to the following:

1. The prevention of offensive conditions
2. The safeguarding of water supplies
3. The protection of bathing beaches
4. The conservation of useful aquatic life

In the British Isles, where rivers are small and their catchment areas are relatively densely populated, the Royal Commission fixed upon certain standards for sewages and sewage effluents which are outlined in Table 39.

TABLE 39.—STANDARDS FOR SEWAGES AND EFFLUENTS ADOPTED BY THE ROYAL COMMISSION ON SEWAGE DISPOSAL, 1912

	Required characteristics of sewage effluent		Type of treatment that will satisfy the standards
	5-day B.O.D., 18.3°C., p.p.m.	Suspended solids, p.p.m.	
General standard	≦ 20	≦ 30	Complete treatment
Special standards where sewages are diluted with the following volumes of water			
150 to 300........	...	≦ 60	Chemical precipitation
300 to 500........	....	≦ 150	Plain sedimentation
Over 500.........	....	.....	No treatment required

English sewages are two to four times as concentrated as American sewages.

Prevention of Offensive Conditions.—In the statement of the dilution required for the prevention of offensive conditions, recourse commonly is taken in American practice to the investigations and expressed opinions of recognized authorities in the field. The most important of these are briefed below.

In 1887, Hering, Williams and Artingstall (15) recommended that the Chicago Drainage Canal be designed to provide 4 c.f.s. of lake water per 1000 persons. In the adopted design this value was later reduced to 3.33, a figure that experience showed to be too low, owing in part, at least, to the fact that the population-equivalent of the industrial wastes was not considered and sludge deposits made large demands upon the diluting waters.

After an investigation for the Massachusetts State Board of Health, Stearns (16), then (1890) its Chief Engineer, concluded that if the flow is less than 2½ c.f.s. per 1000 inhabitants, an offense would be almost sure to arise.[1] With larger volumes than 7 c.f.s. per 1000 inhabitants, the pollution would be too small to cause any offensive conditions.

In 1897 the Ohio State Board of Health had an investigation of the condition of certain Ohio rivers made under the direction of Hazen (17). In discussing the results he stated that in the case of sluggish streams, or streams the waters of which are already polluted to some extent, the quantity required for adequate dilution may become 6, 8 or even 10 c.f.s. per 1000 population.

In 1902 another investigation was made for the Massachusetts State Board of Health by Goodnough (18), who succeeded Stearns as Chief Engineer of the Board. He narrowed the range of dilution fixed by Stearns and summarized the results as showing that, where the quantity of water available for the dilution of the sewage in a stream exceeds 6 c.f.s. per 1000 persons contributing sewage, objectionable conditions are unlikely to result. Where sewage is 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 c.f.s. per 1000 persons objectionable conditions had resulted.

In 1902, in a letter to the Committee on Charles River Dam, Goodnough (19) 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.

[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.

TABLE 40.—CHRONOLOGICAL SUMMARY OF CONCLUSIONS OF AUTHORITIES ON NECESSARY DILUTION OF SEWAGE

Year	Dilution c.f.s. per 1000 population	Authority	Remarks	Reference
1887	3.0	Rudolph Hering	Chicago—channel to be designed for 10,000 c.f.s.	Testimony: Committee of General Assembly, Illinois (1887)
1887	3.0	Dr. Rauch	Chicago—winter conditions	Ditto
1887	4.0	Lyman E. Cooley	Chicago—channel capacity to be 10,000 c.f.s.	Ditto
1887	2.5–3.3	Rudolph Hering	General discussion	Transactions American Public Health Association (1887)
1887	4.0	Hering, Williams and Artingstall	Chicago—this figure used for purposes of estimate	Preliminary Report, Drainage and Water Supply Commission (1887)
1889	2.33	Dr. Rauch	Minimum figure exclusive of winter conditions	Illinois State Board of Health (1889)
1889	2.5–7.0	F. P. Stearns	Range cited is debatable ground	Massachusetts State Board of Health (1889)
1897	6.8 or 10	Allen Hazen	In sluggish streams	Ohio State Board of Health (1897)
1898	3.0–7.0	Griggs and Alvord	For Scioto River, Columbus, Ohio	Report, "Proper Disposal Sewage of City," p. 20
1900	4.0	Rudolph Hering	*Re* Chicago Drainage Canal	Missouri *v.* Illinois (1900)
1901	4.0	Rudolph Hering	Scioto River, assumed settled sewage	Report, "Sewage Disposal, Columbus, Ohio," p. 9
1901	2.0	Dupre	Quoted by Rideal	"Sewage," Rideal, p. 17
1902	3.5–6.0	X. H. Goodnough	Narrowed Stearns' range	Massachusetts State Board of Health (1902)
1902	5.0	J. Herbert Shedd	Charles River Dam testimony	Metcalf and Eddy, "American Sewerage Practice," Vol. 3, p. 255, 2d ed.
1903	3.5–6.0	X. H. Goodnough	Charles River Dam (based on domestic sewage)	Report, Committee on Charles River Dam, p. 307
1903	8.0	John R. Freeman	Charles River Dam (based on domestic sewage)	Ditto, p. 49
1906	3.33	Hering and Fuller	Minimum figure *re* Chicago Drainage Canal	Report, Chicago Drainage Canal, 1906
1907	3.33	E. Kuichling	*R* Rochester—Sewage Disposal	Report, "Sewage Disposal System," p. 13
1908–1913	3.4	X. H. Goodnough	*Re* Condition Merrimack River	Massachusetts State Board of Health (1908–13)
1910	4.7	Met. Sewerage Com., N. Y.	1910 condition in New York Harbor	Report, Metropolitan Sewerage Commission (1910)
1912–1913	3.0–6.0	H. S. Morse and H. P. Eddy	*Re* Cincinnati minimum in range cited	Report, "Plan of Sewerage," p. 549
1913	4.0	G. W. Fuller	Crude sewage	J. W. Society of Engineers, Vol. 18, p. 388
1914	4.0–6.0	Conn. State Bd. Health	For sewage with less than 150 p.p.m. suspended solids	Report, "Investigation Pollution of Streams," p. 8
1915	31.0	Kershaw	Quoting Royal Sewage Commission	"Sewage Purification and Disposal," p. 60
1916	6.0*	E. B. Phelps	*Re* Canal between Lakes Erie and Ontario	International Joint Commission Report, "Remedial Measures."
1918–1921	3.86 and 6.7	Conn. State Bd. Health	*Re* Naugatuck & Hockanum rivers, respectively	Report, Industrial Wastes Board, p. 254

* Would be inadequate.

J. Herbert Shedd (19) testified in 1902 before the Committee on Charles River Dam that about 5 c.f.s. of the ordinary flow of the river, per 1000 persons contributing sewage, would render the presence of the sewage unobjectionable.

In 1908 and 1913 Goodnough made for the Massachusetts State Board of Health 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 c.f.s. or less per 1000 persons objectionable conditions followed. Where serious pollution was observed with higher rates of dilution, such conditions were usually due to the discharge of large quantities of industrial wastes into the stream.

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 c.f.s. per 1000 population, which would be reduced to 2.65 c.f.s. in 1940.

A chronological summary of conclusions of authorities on the necessary dilution of sewage, compiled in 1925 by the Engineering Board of Review of the Sanitary District of Chicago (20), is presented in Table 40.

Dilution factors of 3.5 to 6 c.f.s. per 1000 persons are used as common guides by engineers. They represent at best rough measures of the degree of dilution required and must be modified by considerations of prior pollution, industrial wastes, sludge deposits and successive increase in pollution or dilution. The American equivalent of the dilution ratio of 500:1, suggested for raw sewage by the Royal Commission, corresponds to a flow of 27 c.f.s. per 1000 population. This standard evidently takes into account a relatively intensive use of the streams by successive municipalities and industrial establishments.

As more is learned about the re-aeration characteristics of different bodies of water, it will become possible to interpret and estimate more closely the requirements of dilution for treated or untreated sewage that will prevent offensive conditions. The studies of the Public Health Service on oxygen demand and supply, described in Chap. VII, have laid the foundation for this approach to the problem. The principles expounded have been employed with marked success by the Engineering Board of Review of the Sanitary District of Chicago (20) in formulating opinions as to the probable future condition of the waters receiving the sewage of the district.

Studies of Chicago Board of Review.—The comprehensive studies by the Chicago Board of Review (20) of the disposal by dilution in the Illinois River system of the sewage and industrial wastes of the Sanitary District include both studies of the present and future oxygen demand of the waste materials, liquid, suspended and deposited, with and without treatment, and studies of the present and future quantities of available oxygen supplied by the diluting waters, notably water drawn from Lake Michigan, by re-aeration, and by effluents from treatment works. Basic information was obtained by determinations of the B.O.D. of the wastes, the oxygen supplied by the diluting waters and the B.O.D. of these waters, and from re-aeration studies of the river system, made by the Public Health Service in 1921 and 1922.

B.O.D. of Wastes.—Analyses of sewage at the 39th Street pumping station during 1914 yielded a first-stage B.O.D. value of 0.24 lb. per capita daily for raw sewage and this value was adopted as a reasonable one after comparison with results obtained at other places. The correction for dissolved oxygen contained in the sewage at this station during February, 1922, near the maximum therefor, was equivalent to less than 0.01 lb. per capita daily and was considered to lie within the accuracy of the estimated B.O.D. value. Analysis of stockyards wastes made during 1917 showed a B.O.D. population equivalent of 1,100,000 persons.

With this information and population studies as a basis, the oxygen-demand loads of Chicago sewage and industrial wastes, shown in

TABLE 41.—OXYGEN-DEMAND LOADS OF CHICAGO SEWAGE AND INDUSTRIAL WASTES IN 1920, 1930, 1945 AND 1955

	1920	1930	1945	1955
Population:				
Human	3,000,000	3,710,000	4,785,000	5,500,000
Industrial equivalent	1,500,000	1,700,000	2,000,000	2,200,000
Total	4,500,000	5,410,000	6,785,000	7,700,000
Gross oxygen demand of raw sewage, lb. daily:				
Human	720,000	890,000	1,150,000	1,320,000
Industrial equivalent	360,000	410,000	480,000	530,000
Total	1,080,000	1,300,000	1,630,000	1,850,000
Gross oxygen demand of sewage as discharged, lb. daily:				
Untreated sewage	1,080,000	651,600	None	None
Tank treatment only	None	40,000	501,000	None
Complete treatment	None	51,600	75,300	216,600
Total	1,080,000	743,200	576,300	216,600
Oxygen available in effluents, lb. daily:				
Dissolved oxygen	....	11,500	18,900	67,200
Nitrite and nitrate oxygen	....	29,000	45,700	150,400
Total	....	40.500	64,600	217,600
Net oxygen demand—lb. daily	....	702,700	511,700	None

TABLE 42.—RELATION BETWEEN TREATMENT WORKS AND OXYGEN-DEMAND LOADS OF CHICAGO SEWAGE AND INDUSTRIAL WASTES IN 1930, 1945 AND 1955

	Des Plaines, activated sludge	North Side, activated sludge	Corn Products, trickling filter	Stockyards, activated sludge	Miscellaneous, trickling filter	Calumet, trickling filter	West Side, trickling filter	Southwest Side, trickling filter	Total
Composition of sewage:									
Oxygen demand, p.p.m.									
Raw sewage	288	132	720	900		115	135	126	
Effluent	17.7	20.0	45.0	50.0		20.0	90.0	84.0	
Reduction, per cent	94	85	94	94		83	33	33	
Available oxygen in effluent, p.p.m.									
Dissolved	4.9	7.0	1.7	1.7		7.0	0.0	0.0	
Nitrite and nitrate	15.3	15.0	33.3	4.2		15.0	0.0	0.0	
Population served									
1930: No treatment					103,300		1,546,700	1,065,000	2,715,000
Tank treatment						250,000			250,000
Complete treatment	60,000*	833,300*	400,000†	1,100,000†	51,700†				2,445,000
1945: Tank treatment							1,775,500	1,356,250	3,131,750
Complete treatment	105,000*	1,171,000*	475,000†	1,250,000*	298,000†	354,250†			3,653,250
1955: Complete treatment	141,000*	1,394,700*	530,000†	1,357,000*	389,000†	419,250†	1,923,800†	1,545,250†	7,700,000
Flows to be completely treated, m.g.d.									
1930	6	175	16	35	5⅓	0	0	0	237⅓
1945	10	200	19	40	30	72	0	0	371
1955	14	222	21	44	39	84	440	344	1,208
Gross oxygen demand of effluent from complete treatment, lb. daily:									
1930	890	29,200	6,000	14,600	890	0	0	0	51,580
1945	1,480	33,000	7,130	16,700	5,000	12,000	0	0	75,310
1955	2,060	37,000	7,870	18,300	6,500	14,000	73,500	57,400	216,630
Dissolved oxygen in effluent, lb. daily:									
1930	245	10,200	225	495	330	0	0	0	11,495
1945	410	11,700	270	565	1,755	4,200	0	0	18,900
1955	570	12,920	300	620	2,270	4,900	25,600	20,000	67,180
Nitrite and nitrate oxygen in effluent, lb. daily:									
1930	765	21,900	4,440	1,220	710	0	0	0	29,035
1945	1,275	25,000	5,280	1,400	3,750	9,000	0	0	45,705
1955	1,786	27,800	5,830	1,540	4,900	10,500	55,000	43,000	150,355

* Activated-sludge treatment.
† Trickling filters.

Table 41, were obtained for the years 1920, 1930, 1945 and 1955, consideration being given to the reduction in B.O.D. by contemplated treatment plants and the oxygen available in the effluents of these works. The information utilized in arriving at the estimates of the effect of treatment is shown in Table 42. Imhoff-tank treatment was estimated to reduce the oxygen demand by 33⅓ per cent, so that the oxygen demand of Imhoff tank effluent was set at 0.16 lb. per capita daily. The oxygen demand of the effluent from complete treatment works, as shown in Table 42, was estimated to vary somewhat according

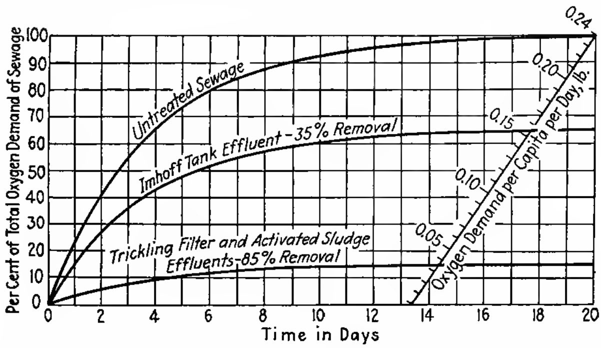


FIG. 36.—Oxygen demand of untreated sewage, Imhoff tank effluent and effluents from trickling filters and activated-sludge treatment at 23°C.

to the strength of the sewage or industrial wastes treated. The general shape of the oxygen-demand curves assumed for untreated, settled and oxidized sewage in relation to time is shown in Fig. 36 for a temperature of 23°C., the prevailing summer temperature of the canal system.

The oxygen demand of sludge deposits was computed from observations made by the Public Health Service during 1921 and 1922, along different stretches of the river system, including relatively clean bottoms and bottoms with appreciable sludge deposits. For the relatively clean stretch from Ottawa to La Salle (see Fig. 37), for example, the gross rate of re-aeration during July, 1922, was found to be 2.8 lb. daily per 1000 sq. ft., of water surface. Applying this rate to the entire stretch between Brandon's Bridge and Chillicothe, the total oxygen demand exerted by sludge deposits during July was computed as follows:

Area of water surface in thousand sq. ft.	422,170
Gross re-aeration, lb. daily (422,170 × 2.8)	1,180,000
Observed improvement in oxygen balance, lb. daily (difference between dissolved-oxygen content and B.O.D.)	275,800
Total oxygen demand exerted by sludge deposits, lb. daily	904,200
Total oxygen demand exerted by sludge deposits, lb. daily per cap. (tributary population and industrial waste equivalent estimated at 4,450,000)	0.203

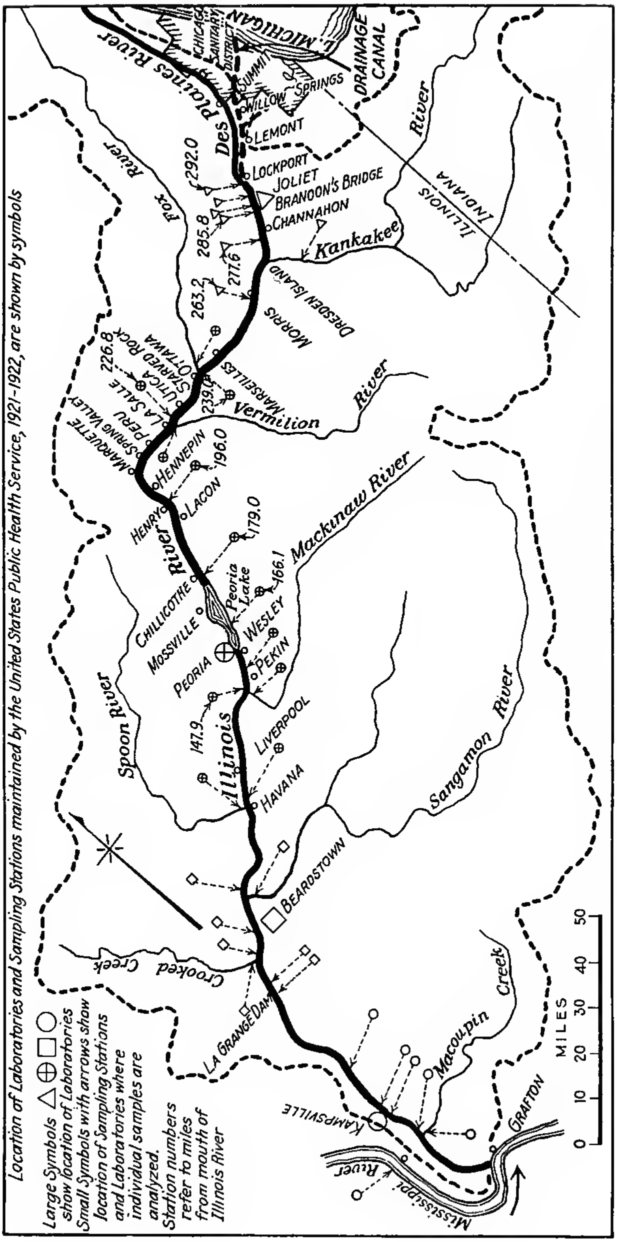


FIG. 37.—Map of Des Plaines—Illinois River.

In a similar manner the total oxygen demand exerted by the sludge deposits in pounds daily per capita was found to be 0.077 above Lockport, 0.154 between Brandon's Bridge and Ottawa and 0.051 between La Salle and Chillicothe, a total of 0.282 lb. daily per capita, or more than the estimated 20-day B.O.D. of the sewage, namely, 0.24 lb. daily per capita. Since sedimentation in the different river stretches removes the settling solids from the flowing waters, allowance was made for the resulting reduction in B.O.D. by assuming that the average time of flow of the solids and with it the B.O.D. exerted before deposition is one half the time of flow for the particular river stretch under consideration. The B.O.D. of the settling solids, being considered as equal to 35 per cent of the total B.O.D., is equivalent to $^{35}/_{65} \times 100$, or 54 per cent, of the B.O.D. exerted by the nonsettleable or liquid portion of the sewage.

With these estimates as a basis it was possible to compute the oxygen demands of the untreated sewage, made up of the nonsettleable or liquid portion, the flowing solids prior to deposition and the sludge deposits, including gas-lifted solids, together with the demand of the sewage subjected to tank treatment only and to complete treatment. The computations for arriving at the oxygen demand of the flowing solids

TABLE 43.—COMPUTATION OF OXYGEN DEMAND OF FLOWING SOLIDS IN CRITICAL RIVER STRETCHES FOR 1930

	Above Lockport	Brandon's Bridge to Ottawa	La Salle to Chillicothe
Estimated B.O.D. of sludge deposits from observations, lb. daily per capita	0.077	0.154	0.051
Per cent of total B.O.D. of sludge deposits (total = 0.282 lb. daily per capita)	27	55	18

	Above Lockport	Lockport to Starved Rock
B.O.D. reduction by settling of flowing solids, in terms of B.O.D. of liquid portion of sewage.	$\frac{27}{2} \times 0.54 = 7.3$ per cent	$\frac{55+18}{2} \times 0.54 =$ 19.7 per cent
B.O.D. of flowing solids	$\left(\frac{27}{2} + 55 + 18\right) \times 0.54 = 47$ per cent	$\frac{55+18}{2} \times 0.54 =$ 19.7 per cent

NOTE: Computations are approximate. A more accurate method is given in an addendum to the Report of the Board of Review.

are summarized in Table 43 for the year 1930. It was assumed that the proportion of settleable solids in the untreated sewage that would settle above Lockport was the same as the proportion of the total oxygen demand exerted by sludge deposits which would be exerted above Lockport and that the remainder of the settleable solids would all settle between Lockport and Starved Rock, on account of construction of proposed navigation dams. In 1945 and 1955, according to the program of construction, no sewage is to be discharged untreated and it was assumed that no sludge deposits would occur, although some solids will be discharged through storm overflows and in treatment-plant effluents.

Oxygen Supply.—The sources of oxygen supply in the canal system and Des Plaines-Illinois River are: dissolved oxygen in water drawn from Lake Michigan; dissolved oxygen in effluents from complete treatment; available oxygen in form of nitrites and nitrates in effluents from complete treatment; oxygen absorbed by atmospheric re-aeration; dissolved oxygen from tributary streams; and oxygen absorbed by biological re-aeration.

Of these sources, biological re-aeration was not considered in the estimates, because it is too uncertain a factor with present knowledge. Nitrite and nitrate oxygen, which are relatively small in quantity and which are drawn upon only after the dissolved oxygen of the water is substantially exhausted, were considered factors of safety against putrefaction. Although the tributary streams above Chillicothe furnish large supplies of dissolved oxygen during months of high stream flow in the winter and spring, they have little or no available dissolved oxygen during the critical summer period.

The dissolved-oxygen content of Lake Michigan water was placed at 8 p.p.m. in summer and 13 p.p.m. in winter, on the basis of analyses made from 1921 to 1923 at the 39th Street pumping station. The critical value of 8 p.p.m. is equivalent to 43.2 lb. of oxygen daily per cubic foot of lake water per second. Estimates of dissolved oxygen in effluents have been given in Table 42. Rates of re-aeration were estimated for different river stretches, based upon the Public Health Service investigations for July and August, 1922. These estimated rates at zero dissolved oxygen, or complete dissolved-oxygen saturation deficiency, are as follows:

	Pounds of Oxygen Daily per 1000 Sq. Ft. of Water Surface
Canal system above Lockport	2
Lockport to Brandon's Bridge	16
Brandon's Bridge to Starved Rock	4
Starved Rock to Chillicothe	2

TABLE 44.—ESTIMATE OF RESIDUAL DISSOLVED OXYGEN AT LOCKPORT IN 1930, WHEN 10,000 C.F.S. OF WATER ARE DRAWN FROM LAKE MICHIGAN, TO DILUTE A FLOW OF 1400 C.F.S. OF SEWAGE

Time of flow, days	0.97
Oxygen absorbed from air, lb. daily per 1000 sq. ft. of water surface	2.16
Uncorrected B.O.D. of sludge deposits, lb. daily per capita	0.077
Per cent of settleable solids deposited	27
B.O.D. reduction by settling of flowing solids, in terms of B.O.D. of liquid, per cent	7.3
B.O.D. of liquid, lb. daily per capita	
West Side	0.036
Southwest Side	0.028
Miscellaneous	0.019
B.O.D. of settled solids, lb. daily per capita	
West Side	0.036 × 0.073 = 0.003
Southwest Side	0.028 × 0.073 = 0.002
Miscellaneous	0.019 × 0.073 = 0.001
Accumulative total B.O.D., lb. daily per capita	
West Side	0.003 + 0.077 = 0.080
Southwest Side	0.002 + 0.077 = 0.079
Miscellaneous	0.001 + 0.077 = 0.078

	Time of flow, days	Oxygen demand exerted	
		Pounds daily per capita	Total pounds daily
North Side	1.83	0.011	9,130
West Side	1.00		
Liquid	....	0.036	55,800
Flowing solids	....	0.017	26,350
Sludge deposits	....	0.080	124,000
Southwest Side	0.8		
Liquid	....	0.028	29,820
Flowing solids	...	0.013	13,845
Sludge deposits	....	0.079	84,135
Packingtown	0.8	0.002	2,200
Corn Products	0.5	0.001	400
Calumet	0.8	0.028	7,000
Miscellaneous	0.5		
Liquid	....	0.019	2,945
Flowing solids	...	0.009	1,395
Sludge deposits	....	0.078	12,090
Total oxygen demand exerted	....	.....	369,110
Equivalent oxygen demand in p.p.m., based on total flow	....	.....	6.0

Dissolved oxygen of lake water, p.p.m	8.16
Dissolved oxygen of lake water and sewage, p.p.m	7.16
Additional dissolved oxygen from treatment-works effluent (11,500 lb.), p.p.m	0.19
Initial dissolved oxygen of total flow, p.p.m	7.35
Oxygen demand in stretch, p.p.m	6.0
Residual dissolved oxygen, neglecting re-aeration, p.p.m	1.35
Average dissolved oxygen in stretch, including reaeration, by trial, p.p.m	4.79
Average dissolved-oxygen saturation deficiency, p.p.m. (saturation = 8.7 p.p.m.)	3.91
Rate of re-aeration at zero dissolved oxygen, lb. daily per 1000 sq. ft	2.16
Corresponding re-aeration for 56,200 thousand sq. ft., lb. daily	121,500
Corresponding re-aeration, p.p.m. at zero dissolved oxygen	1.97
Re-aeration at average dissolved-oxygen saturation deficiency, p.p.m	0.88
Residual dissolved oxygen at Lockport, p.p.m	2.23

In computations for future conditions, allowances were made for reduction in rates of re-aeration by dam construction as well as increase in these rates by enlarged water surfaces. The rates of re-aeration for different stretches and flows were assumed to be directly proportional to the velocities and inversely proportional to the times of flow. Starting with these rates at zero dissolved oxygen for the different stretches, the corresponding rates of re-aeration were determined for the times of flow with different quantities of water drawn from Lake Michigan and the total re-aeration was computed for each stretch.

Oxygen Balance.—The arithmetical method employed in estimating the oxygen balance for different years and varying quantities of lake water is exemplified for the year 1930 and the canal system above

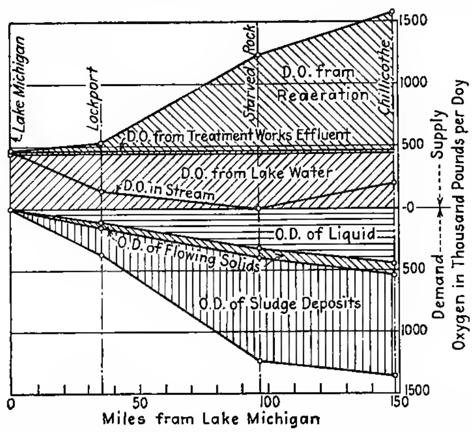


FIG. 38.—Estimated dissolved-oxygen supply and oxygen demand in canal system and Des Plaines-Illinois River in summer of 1930, with assumed flow of 11,400 c.f.s. of lake water and sewage at Lockport.

Lockport in Table 44, for a flow of 11,400 c.f.s. of lake water and sewage at Lockport. The principles of deoxygenation and re-aeration discussed in Chaps. III and VII were employed.

The estimates arrived at for the canal system in the summer of 1930, with a flow of 11,400 c.f.s. at Lockport, are shown graphically in Fig. 38, together with estimates for the remaining river stretches within the critical zone.

Safeguarding of Water Supplies.—In 1914 the International Joint Commission on the Pollution of Boundary Waters between the United States and Canada concluded among other things that, in general, protection of public water supplies is secured more economically by water purification at the intake than by sewage treatment at the sewer outlet, but that under certain conditions both water purification and sewage treatment may be necessary. The Commission was unwilling to fix more than a tentative standard of 500 *B. coli* per 100 cc. of water as the limit of safe loading of water-purification plants. Studies by Streeter of the United States Public Health Service have since yielded valuable information on the permissible loading of water purification plants drawing upon polluted sources of water (21, 22). Using bacteriological standards, it was found that the purification accomplished could be expressed by the following formula:

$$E = CR^n$$

where E = effluent count or *B. coli* index
R = raw-water count or *B. coli* index
C and n = purification constants, C measuring the magnitude of efficiency and n the relative constancy in bacterial quality.

The numerical values of C and n for *B. coli* were found to be 0.15 and 0.22, respectively, for the chlorinated effluent of 10 Ohio River plants studied in 1923–1924: East Liverpool, Steubenville, Ironton, Portsmouth and Cincinnati, Ohio, Huntington, W. Va., Ashland, Louisville and Henderson, Ky., and Evansville, Ind. If these values are substituted in the above formula, the permissible number of *B. coli* in the raw water is found to be 5000 per 100 cc. in order that purified, chlorinated water may meet the revised "Treasury Standards" of 1 per 100 cc.

The revised Treasury Department *B. coli* standard, adopted in 1925 and described in *Public Health Reports*, contains the following limiting provisions relative to the *B. coli* content of drinking waters served to the public by interstate carriers (23):

1. Of all the standard (10 cubic centimeter) portions examined in conformance with a prescribed procedure, not more than 10 per cent shall show the presence of organisms of the *B. coli* group.
2. Occasionally 3 or more of the 5 equal (10 cubic centimeter) portions constituting a single standard sample may show the presence of *B. coli*. This shall not be allowable if it occurs in more than (*a*) 5 per cent of the standard samples when 20 or more have been examined; (*b*) 1 standard sample when less than 20 samples have been examined.

Expressed in terms of the *B. coli* index, the limiting average density of *B. coli* prescribed by the revised standard is 1 per 100 cc., with occasional variations such that a *B. coli* index of 6 per 100 cc. may be exceeded with a frequency not greater than 5 per cent.

The limiting load consistent with producing unchlorinated effluents meeting the same standard was found to be roughly 60 *B. coli* per 100 cc., basing this figure on the performance of three of the plants, showing a higher order of efficiency in this respect than the remaining plants of the group from which observations were available. Results secured from seven other, widely scattered plants in the East and Middle West—Albany, Elmira and Niagara Falls, N. Y., New Milford, N. J., Chester, Pa., and Toledo and Youngstown, Ohio—confirmed in a general way those obtained from observations on the Ohio River plants. For highly elaborated plants including double-stage treatment, such as double-stage sedimentation or double filtration, the raw-water *B. coli* maximum was found to range as high as 50,000 or more per 100 cc. Subsequent studies made during the year 1926 on 13 Great Lakes plants—Ashtabula, Cleveland, Lorain, Elyria and Sandusky, Ohio, Monroe, Wyandotte, Detroit and Bay City, Mich., East Chicago and Whiting, Ind., and Evanston and Winnetka, Ill.—showed that the average Great Lakes plant, though nearly as efficient from the standpoint of bacterial reduction, when aided by chlorination, as the average Ohio River plant, is decidedly less efficient from the same standpoint, when unaided by chlorination. The raw-water *B. coli* index consistent with the primary requirement of the revised Treasury Department standard was 3300 per 100 cc. for the average plant of the group and 4500 for the average plant of the more representative majority, when the effect of chlorination is included. Without the aid of chlorination the same standard was met by the average plant of the more representative majority from raw water having a *B. coli* index of 5 to 10 per 100 cc. and by a small minority of the plants from raw water having a *B. coli* index of approximately 60 per 100 cc.

With the aid of the die-away curves of *B. coli* in polluted waters, it is possible to estimate the dilution and treatment of sewage required to safeguard water supply. This requires, however, a knowledge of the stream characteristics, such as temperature, degree of pollution, time of flow and relative dilution. Water purification commonly is dispensed with in but relatively slightly polluted waters in which long periods of flow, high dilution or long storage permit self-purification to be carried to complete recovery. Even then chlorination of the water, or of both sewage and water, is sometimes practiced as an additional safeguard.

Protection of Bathing Beaches.—Generally speaking, it would be ideal for the water of bathing beaches to approach drinking-water quality. The standard of bacterial quality of swimming-pool waters recommended by the Joint Committee on Swimming Pools and Other Public Bathing Places of the American Public Health Association and the Conference of State Sanitary Engineers (24) requires a 24-hr. 37°C. agar count of not more than 100 bacteria per cubic centimeter in less than 10 per cent of samples covering any considerable period and of not more than 200 bacteria per cubic centimeter in any single sample; furthermore, essentially a *B.coli* index of not more than 4 per 100 cc. for any single sample or 3 per 100 cc. for two consecutive samples.

Relative to bathing beaches, the committee is of the opinion that

it is very desirable that the bathing waters at public bathing places on natural streams, lakes and tidal waters should be of the same standard of bacterial quality as is required for swimming pools. It is recognized, however, that the strict application of swimming-pool water standards to all public bathing waters would probably not be practicable at present. Furthermore, since there is not now available any sufficient volume of data as to the quality of waters used for bathing, nor any sufficient record of the causation of disease by bathing in moderately polluted waters, your committee feels that any definite standard of bacterial quality which it might propose would be purely empirical.

Disinfection of the waters in the vicinity of bathing beaches by means of chlorine apparatus on boats has been practiced at a number of places.

Study at Rochester, N. Y.—The sewage from nearly the entire area within the corporate limits of the city of Rochester, together with that from a portion of the town of Irondequoit, is treated by fine screens and by sedimentation in Imhoff tanks and the settled sewage is discharged into Lake Ontario through a submerged outlet in about 40 ft. of water and 7000 ft. from shore. The population served during 1928 was about 324,000 and the average sewage flow about 48 m.g.d., of which about 45.5 m.g.d. was passed through the Imhoff tanks.

During the summer of 1929 the authors made a study of the sanitary condition of the waters of Rochester Bay in Lake Ontario, in the vicinity of the sewer outlet and of the bathing beaches. The location of the outlet with respect to the bathing beaches is shown in Fig. 39. Practically all the sewage flow discharged through the outlet receives treatment in the Imhoff tanks, but occasionally a storm flow in excess of that which can be treated in the tanks has been by-passed and discharged with the settled sewage at the outlet.

Among 19 samples of mud collected in Rochester Bay, only one gave definite evidence of being composed in substantial measure of sewage solids. This sample was collected within 50 ft. of the outlet from a deposit about 1 ft. in depth. The sample contained 18 per cent of organic matter, as indicated by loss on ignition. A comparison of the material composing the deposit at the outlet with the bottom soil as disclosed by the other samples indicated that any substantial deposit of sewage solids that may have been discharged from the outfall is confined to a small area less than ¾ mile in radius from the outlet, and that any sewage solids which may have been deposited outside this area have been dissipated by bottom currents to an extent which has rendered it impossible to detect them by any practicable method of sampling and analysis. Apparently the finely divided and light solids passing through the sedimentation tanks have been promptly dispersed and it is reasonable to assume that they are utilized for the support of the natural biological life of the lake water and thus rendered innocuous.

Tests of the lake water showed the presence of large quantities of dissolved oxygen, indicating that the settled sewage was not imposing

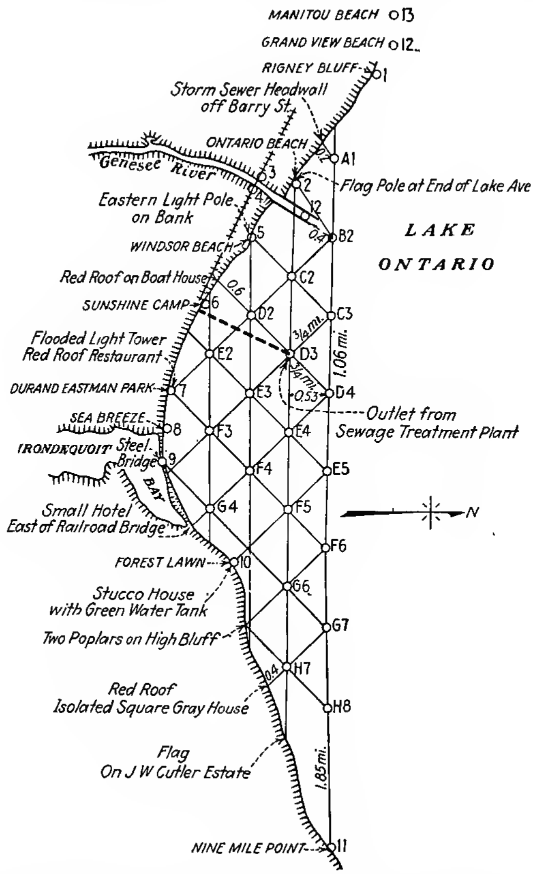


Fig. 39.—Map of waters in vicinity of Rochester, N. Y., showing sampling points used during sanitary survey, July and August, 1929.

a sufficient burden upon the lake to cause a measurable reduction in the natural supply of dissolved oxygen as a result of the decomposition of organic matter.

There was no visible evidence of sewage pollution of the waters of Lake Ontario, except in the immediate vicinity of the outlet and at times in the Genesee River. On only two occasions specks of grease were observed along the shore. The effect of bacterial pollution also was limited to the immediate vicinity of the outlet and to the area close to the mouth of the Genesee River.

During these investigations there were wide and frequent variations in the direction and intensity of the wind, which seldom persisted in the same direction and of the same intensity for more than a few hours. The effect of the wind appears to have been to set up currents in the water varying in direction and intensity according to the wind movements. This resulted in diffusion and dissipation of the sewage to such degree and to such extent that these investigations did not disclose a tendency for the sewage to travel in a well-defined path in any direction, either at the surface or on the bottom, although the general trend, indicated by bacterial examinations, was easterly, parallel to the shore.

A consideration of all the evidence accumulated in this investigation indicated that bathing in the lake waters at the several beaches was not attended by any substantial hazard to health under the existing conditions.

Conservation of Useful Aquatic Life.—Most food fish are sensitive to pollution, largely because they require the presence of a moderate quantity of dissolved oxygen in the water.

This has been emphasized in arguments over the permissible pollution of New York Bay, where the dissolved oxygen should not be reduced below 70 per cent, according to Black and Phelps (25), in order that food fishes might continue to live in the waters. Fuller stated on this topic:

> 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.

A somewhat different problem is that connected with the pollution of shellfish beds. Unlike fish, oysters and clams are often consumed raw and may therefore become vehicles of infection by harboring in their bodies pathogenic organisms ingested from the waters in which they abound. A number of epidemics of typhoid fever have been traced to contaminated shellfish.

From the idealistic standpoint, shellfish beds should be located in waters which meet the bacterial standards for drinking water. Experience indicates, however, that such a standard is unnecessarily rigorous. In Massachusetts the rating of shellfish areas is based upon the results of sanitary survey, the *B. coli* content of the waters at the areas in question and the *B. coli* content of the shellfish.

If oysters are relaid in clean water, they will quickly purge themselves of pollution. This offers a means, as yet insufficiently explored, for utilizing oysters which originate in undesirable areas. Chlorination of oysters and clams after removal from their beds also seems to render them safe for consumption. The practice of *bloating*, *freshening* or *bleaching* oysters by placing them in brackish water for a day or two appears to be regarded with disfavor. Too often the bloating is accomplished in coves, bays or the mouths of streams which, being closer to shore than the oyster beds, are apt to be more heavily polluted. The bloating process should not be confused with the fattening process of transplanting oysters to clean beds in shallow water, rich in foodstuffs, chiefly algae.

DILUTION WORKS

Types of Dilution Works.—Several types of dilution works may be distinguished according to the nature of the waters into which sewage is to be turned, namely, river systems, large fresh-water lakes and tidal estuaries and the ocean. A fourth type of dilution works is one like the Chicago Drainage Canal project or the flushing-tunnel project employed at Milwaukee, Wis., in which a watercourse otherwise not adequate for dilution is rendered so by engineering means, such as canal construction or the provision of flushing sewers carrying diluting waters from other sources. The various factors that play a part in dilution works have already been discussed.

Two other types of dilution works, briefly discussed in Chap. VI, are the discharge of settled sewage, diluted with clean water, into ponds in which fish and ducks are raised and the construction of reservoirs or artificial lakes, called "river clarifying basins," by which natural purification is aided. Both of these processes have been employed as yet only in Europe.

The relative use in the United States of river, lake and ocean outfalls and land disposal systems is shown in Table 45, which summarizes the disposal methods of the 95 cities that had a population of more than 100,000 in 1930. The values given are expressed in per cent of the total population of 36.45 million in the cities included.

TABLE 45.—USE OF VARIOUS TYPES OF DISPOSAL WORKS IN 1930 BY CITIES IN THE UNITED STATES HAVING A POPULATION IN EXCESS OF 100,000

Tributary to	Percentage of total population	
	Including as river outfalls all those discharging into rivers, even though close to lake or ocean	Including as lake or ocean outfalls, those which discharge into rivers at short distances from lake or ocean
River outfalls........	74.2	38.8
Lake outfalls........	9.2	14.0
Ocean outfalls.......	15.9	46.5
Irrigation works.....	0.7	0.7

The design and construction features of a number of outfalls together with auxiliary structures, such as regulators, overflows, silt chambers and tide gates, are discussed in Chap. XVII of Vol. I, 2d edition, of this work. There are described river outfalls at Minneapolis, Winnipeg, Louisville, New York and Washington, the lake outfall at Rochester and the ocean or tidal-estuary outfalls at Staten Island and Brooklyn. For this reason the succeeding portion of this chapter merely extends or amplifies by further examples American practice in the disposal of sewage by dilution, with special reference to the results obtained in the control of pollution.

River and Lake Outfalls.—If sewage is discharged into a stream which flows 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 quickly and at the same time thoroughly mixed with the river water. However, the outlet should be situated so that the sewage will not follow the bank for a long distance. By a study of the variations in river stage it is possible to locate the outlet so that the sewage will always be discharged directly into water and will not pass over the river bottom exposed by low water.

Cities situated on the Great Lakes and other large inland lakes are commonly dependent upon these waters both for their water supply and for the disposal of their sewage. The location of outfalls, therefore, must be investigated with particular care in relation to water-works intakes. Bathing beaches and shore fronts, too, must be protected against pollution. In order to disperse the sewage thoroughly through a large volume of water before it can rise to the surface, lake outfalls usually are submerged to a considerable depth and often extend long distances from the shore. At Rochester, N. Y., for example, a 66-in. steel outfall sewer, laid in Lake Ontario, terminates in a timber crib, 7000 ft. from shore in 50 ft. of water, the sewage discharging 10 ft. above the bottom.

Washington, D. C.—An outlet was located after careful study of currents at Washington, D. C., which had a population

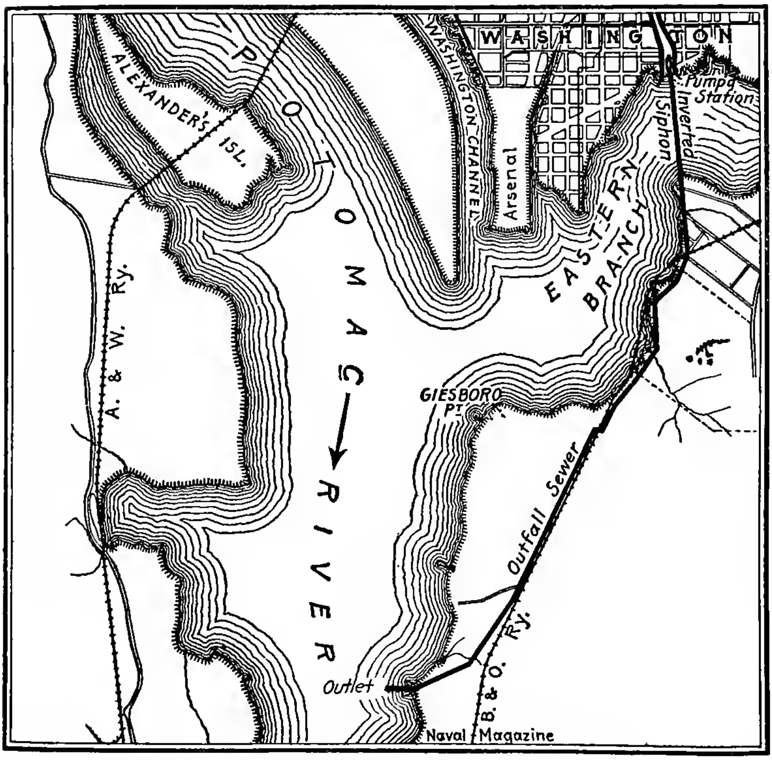


FIG. 40.—Location of the Washington sewer outlet.

of 485,716 in 1930. The sewage is discharged into the Potomac River at a point where it is subject to tidal influence (Fig. 40). In the study of the currents by Hering, Gray and Stearns in 1889, floats were considered of slight value, for observations could not be made at the lowest stages of the river and the floats frequently stranded, unless made so shallow as to be affected greatly by the wind. Reliance was placed on calculations based on the flow of the river and the tidal fluctuation, employing what is known as the "piston method," in which the total quantity of water moving up or down stream during each tide is figured. The outlet was located at a point where it was considered that sewage never would be swept back to the city and would pass but seldom into the Eastern Branch or Anacostia River.

The outlet is located in midchannel of the Potomac River, opposite Grimes, the channel at this point being close to the east bank. The outlet extends 700 ft. from shore and discharges in about 25 ft. of water. There are two 60-in. cast-iron pipes terminating in upturned elbows. The cost of the outlet pipes was about $40,000, including about $5000 for inspection. Construction was completed in 1907.

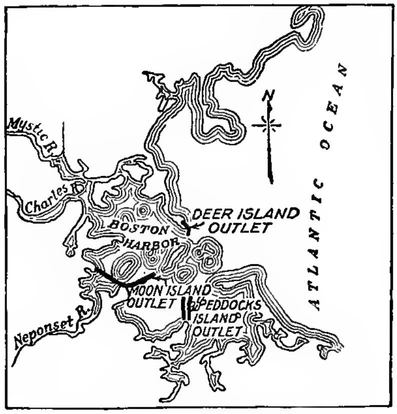
Cleveland.—The sewage of the city of Cleveland is treated at three plants, the Easterly providing for about 50 per cent of the population, the Westerly for about 30 per cent and the Southerly for about 20 per cent. The effluents from the Easterly and Westerly plants are discharged through submerged outfalls into Lake Erie (see Vol. I, Fig. 2). The sewage from the Easterly plant, completed in 1921, is passed through racks and grit chambers designed to serve a population of 575,000.[1] The Westerly plant, designed to serve 288,000 persons, was completed in 1920. It provides treatment by racks, grit chambers and Imhoff tanks.

After flowing through the Easterly treatment plant the sewage is discharged through two outfalls, a 63-in. riveted-steel pipe and an 84-in. reinforced-concrete pipe. The steel outfall, placed in operation in 1912, discharges through a single outlet, consisting of an upturned elbow, into about 25 ft. of water at a distance of approximately ½ mile from shore. The capacity of this outfall is estimated at 85 m.g.d. when operating under 6½ ft. of static head.

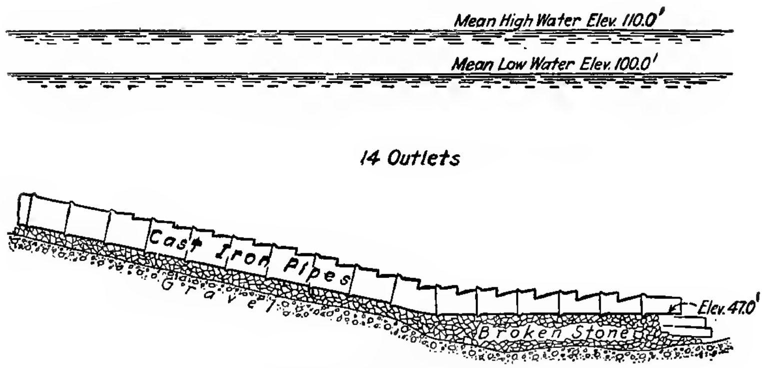
The concrete outfall, completed in 1918, was laid in a trench parallel to, and about 150 ft. distant from, the steel pipe. The total length of the concrete outfall is 3200 ft., of which the last 1000 ft. consist of a section tapering from 84 to 48 in. in diameter, fitted with multiple outlets. The sewage is discharged through 150 8-in. openings staggered on 6½-ft. centers, the axes of the openings being inclined 45 deg. from the horizontal. The depth beneath the water surface of the several points of discharge averages 30 ft. The capacity of this outfall is estimated at 145 m.g.d. when operating under 5 ft. of static head.

The settled sewage from the Westerly treatment plant is discharged through a concrete and steel pipe, 3600 ft. long, equipped with multiple outlets. This outfall, installed in 1916, consists of 400 ft. of 72-in. concrete pipe at the shore end, 2200 ft. of 60-in. riveted-steel pipe and

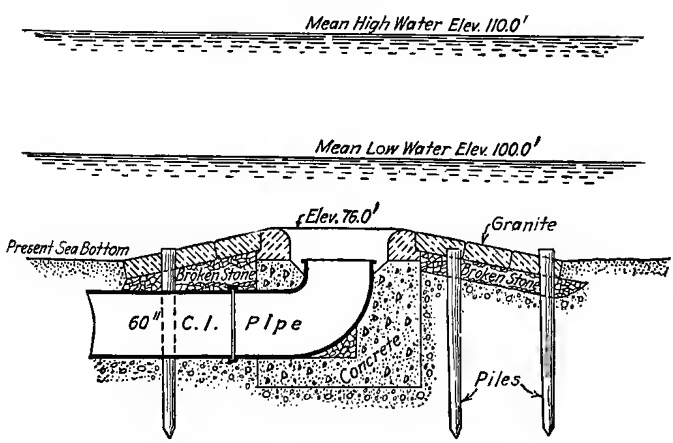
[1] A sedimentation, activated-sludge plant is under construction at the Easterly site, designed to serve a population of 770,000 with a dry-weather flow of 123 m.g.d. The work is being planned to provide treatment for 92 per cent of the sewage from the Easterly district by the end of the year 1935.



a.—Map showing locations of outfalls.



b.—Profile of Deer Island outfall.



c.—Section of Peddocks Island outlet.[1]

FIG. 41.—Outfalls of metropolitan Boston.

[1] Note that difference in elevation between outlet and low water is not shown to scale.

1000 ft. of steel pipe tapering gradually from 60 to 24 in. in diameter. Along the tapered section are openings 6¾ in. in diameter, staggered on 7½-ft. centers. The pipe terminates in a cast-iron flap gate, so weighted as to insure maximum use of the multiple outlets under all conditions of flow. The average depth of the openings beneath the water surface is approximately 30 ft. The capacity of the outfall is estimated at 85 m.g.d. when operating under 11 ft. of static head.

The Easterly outlets are about 8 miles from the Cleveland waterworks intake and the Westerly outlet is less than 4 miles from the intake. Referring to conditions which obtained previous to the construction of the sewage-treatment plants, Gascoigne and Havens (27) state that on a clear day the discoloration of the lake water about the Easterly steel outlet could be seen easily from shore. The discoloration occupied an area about one half mile in width and from 2 to 3 miles in length, gradually diminishing in intensity toward the limits of this area. While the area about the Westerly multiple outlet appeared more turbid than the surrounding lake water, this sewage field ordinarily could not be noticed from shore.

Outfalls in Tidal Estuaries or the Ocean.—As in the case of lake outfalls, 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, although not necessarily an indication of putrescible organic matter, is yet unsightly and likely to be the cause of complaint against the disposal of the sewage in this way. These undesirable conditions are reduced materially when the sewage is discharged in depths of 25 ft. or more.

Metropolitan Boston.—The development which has taken place in the design of ocean outfalls is illustrated by the three disposal systems of metropolitan Boston, shown in Fig. 41.

The first large outfall sewer which was built to discharge into Boston Harbor has its outlet near Moon Island, where float observations have shown that the average velocity of the ebb tide is 0.74 mile per hour and that the floats travel about 4 miles seaward during the whole ebb tide. The sewage is stored in tanks on the island and discharged during ebb tide. Experiments show that when 22 million gal. of sewage are discharged in 45 min., 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 action of waves. In 1929 the average daily quantity discharged at this outfall was 109 million gal.

The second large outfall sewer discharging into the harbor had its outlet originally near Deer Island Light, but a few feet below the surface and 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 ft. farther 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 others are elliptical, varying from 25 by 44 to 13 by 23 in. A typical outlet pipe is shown in Fig. 42. This outfall discharged an average of 85 m.g.d. during 1929.

The third large outfall sewer has its outlet near Peddocks Island at a depth of 24 ft. at low tide, where about 64 m.g.d. of sewage are 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 relatively small. That a considerable quantity of sewage from this outlet does come to the surface, however, is indicated by an experiment performed by the authors in 1911. Two pounds of aniline dye, eosin, were mixed with water and introduced into the sewer at the shore end of the outfall. On proceeding to the vicinity of the outlet the colored sewage could be seen rising with considerable velocity and spreading out over an area of many acres. This experiment was performed during a heavy wind and rain storm.

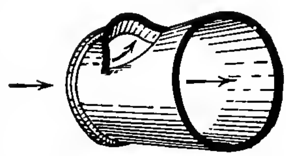


FIG. 42.—Typical outlet pipe, Deer Island outfall.

Passaic Valley Sewerage System.—The use of multiple outlets has found its greatest development in the outfall of the Passaic Valley sewerage system in New York Harbor. The arrangement of this outfall, together with the special nozzle used to insure thorough mixing of the sewage with the harbor water, is shown in Fig. 43. There are 150 nozzles or diffusers which are designed in such a way that the sewage leaves the periphery of the nozzle in a thin ribbon with a whirling outward motion. The nozzles are distributed over an area of about 3 acres and are situated at a depth of 40 ft. or more below mean low water. The outfall was placed in commission in 1924 and is designed to serve 1,600,000 people in communities along the Passaic Valley in New Jersey. Before being discharged into the harbor, the sewage is passed through racks, grit chambers and sedimentation tanks.

The Passaic Valley Sewerage commissioners have entered into an agreement with the United States Government, to the effect that the Passaic Valley sewage will not cause visible suspended particles or objectionable deposits in New York Bay; that the sewage will not cause odors in the bay or its vicinity, due to the putrefaction of organic matter; that it will cause practically no grease or color on the surface of the bay; and that the dissolved-oxygen content of the waters of the bay will not be reduced to such an extent as to interfere with major fish life, as a result of the discharge of Passaic Valley sewage.

Los Angeles, Cal.—A large part of the sewage of Los Angeles, about 109 m.g.d. in 1930, is discharged into the ocean at Hyperion, after passing through fine screens, installed principally for the protection of

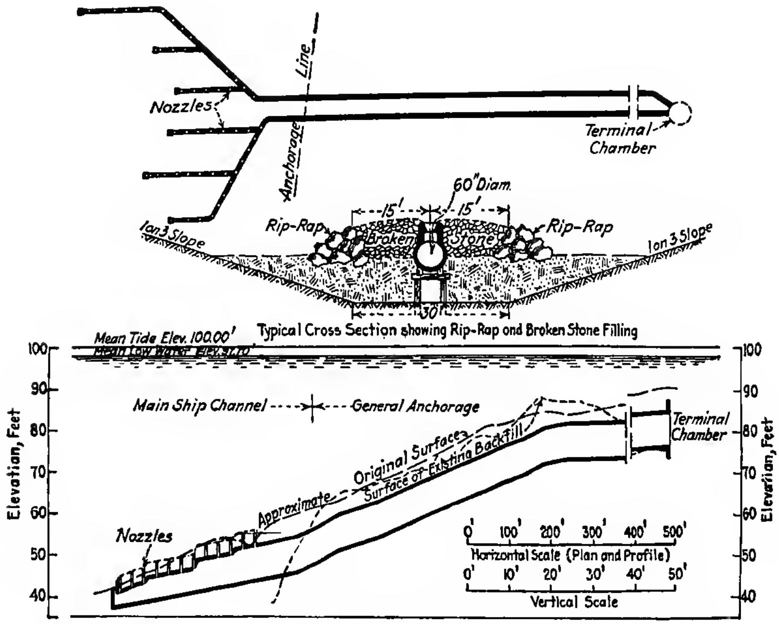


FIG. 43.—Passaic Valley outfall in New York Harbor.

bathing beaches. There are two outfall sewers, known as the North outfall and Central outfall, respectively.

According to Knowlton (28), the Central outfall, built in 1918, extends into the ocean for a distance of approximately 2000 ft. from shore. This sewer consists of a 52-in. wood-stave pipe, laid below the floor of a pier. The floor and pipe are practically level, having a fall of only 1 ft. in the entire length. At the outer end of the pier the average depth of the ocean is about 30 ft. and sewage is discharged at this end through five outlets, each extending 18 ft. below mean sea level.

The North outfall was constructed in 1924 when the screening plant was installed. It consists of a reinforced-concrete pipe sewer, 7 ft.

in diameter, laid below the bottom of the ocean for a distance of 5093 ft. from shore, to a point where the depth is 54 ft. below mean sea level. The joints of the pipe are caulked with lead wool and covered with concrete. The outer end of the 7-ft. outfall is divided into two 5-ft. branches, which have an average length of 210 ft. from their junction with the 7-ft. sewer.

During the year ending June 30, 1930, according to the Annual Report of the Los Angeles Board of Public Works, 1714 samples of surf waters were collected for bacteriological tests from 11 stations along the beach near the Hyperion screening plant. Collection of samples was made on about 13 days a month at each station. The number of *B. coli* in these samples indicates a pollution that makes the Hyperion Beach unsatisfactory for sea bathing. The report concludes that the increasing discharge of sewage into the ocean indicates that further treatment of the sewage is required to render bathing safe at all times. Two ocean surveys which were made during the year show that at the time of maximum discharge the area of the sewage field at Hyperion averages 6 acres per m.g.d. of sewage flow. Within this field there is said to be no noticeable odor.

Miscellaneous Outfalls in Tidal Waters of Massachusetts.—Table 46 contains data pertaining to certain sewer outfalls which discharge into the tidal waters of Massachusetts. Each of the examples given consists of a cast-iron pipe with a single outlet.

TABLE 46.—EXAMPLES OF SEWER OUTFALLS DISCHARGING INTO THE TIDAL WATERS OF MASSACHUSETTS

	Gloucester	Lynn	New Bedford	South Essex sewerage district	Swampscott
Date of construction	1927	1928	1913	1928	1903
Population served	24,000	102,000	113,000	102,000	10,000
Average flow of sewage, m.g.d.	1.2	14.2			
Diameter of outfall, in.	24	60	60	54	18
Material of construction	Cast iron	Cast iron	Cast iron	Cast iron	Cast iron
Length of outfall, ft.	5,950	15,240	3,300	8,300	3,050
Depth of outlet below mean low water, ft.	33	30	30	30	50
Nature of outlet	45° bend	30° bend	Upturned elbow	Upturned elbow	45° bend
Preliminary treatment of sewage	Racks	Racks	Racks and grit chambers	Racks	Racks

Flushing Sewers.—Until the intercepters along the three rivers passing through the City of Milwaukee were completed, much of the sewage of the city was discharged directly into these watercourses.

With the rapid growth of Milwaukee the streams became grossly polluted. Water was therefore pumped from the lake, as shown in Fig. 44, carried through what are called *flushing tunnels*, or *flushing sewers*, in reality water conduits, and discharged into the streams to swell their volume during times of low flow and warm weather. This is an example of artificial dilution. Flushing works of this kind were devised also for Chicago and Brooklyn. At Milwaukee intercepters

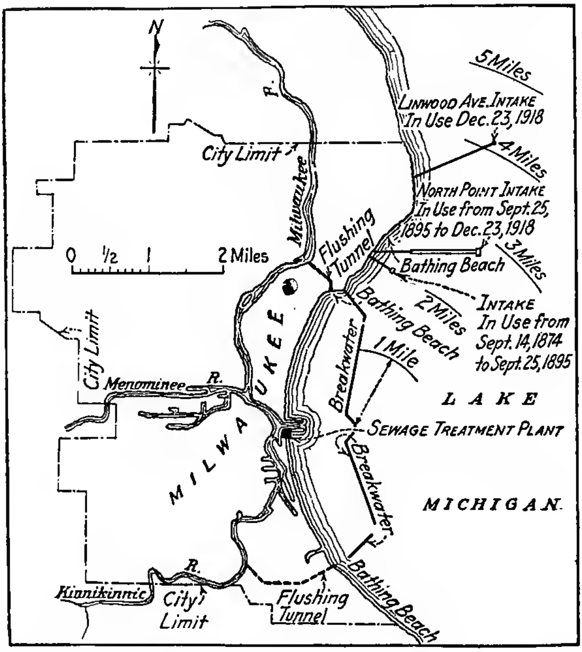


FIG. 44.—Map of Milwaukee, showing arrangement of flushing sewers.

and treatment works were constructed a few years ago. The flushing tunnels still are operated, however, to improve river conditions.

Chicago Drainage Canal.—The problem of controlled dilution in connection with the disposal of the sewage and industrial wastes of Chicago has received much attention in preceding pages and chapters of this work. The Chicago Drainage Canal was constructed for the purpose of protecting the waters of Lake Michigan, which is the source of the city's water supply. This canal connects the Chicago River with the Des Plaines River. The latter, together with the Kankakee River, forms the Illinois River, a tributary of the Mississippi River (see Fig. 45). The flow of the Chicago River was thus reversed and water abstracted from Lake Michigan serves to dilute the sewage of the

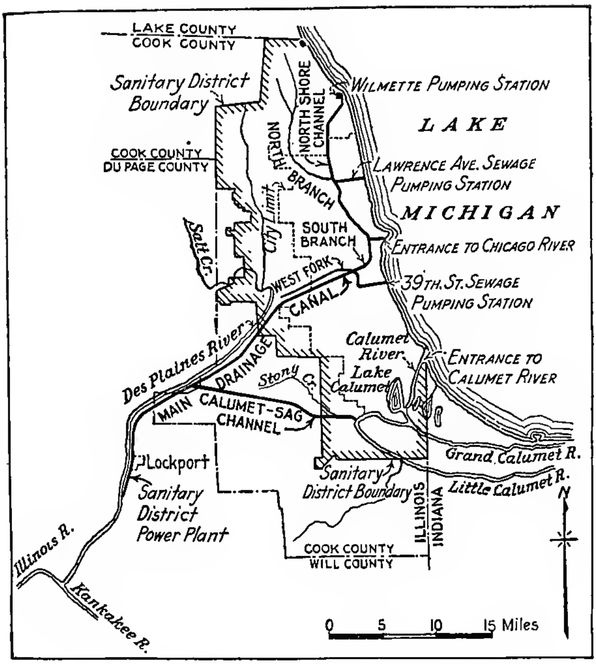


FIG. 45.—Location of rivers and canals, Chicago, Ill.

city, which is discharged into the canal. To reduce the quantity of diluting water required, as a result of the rapid growth of the district and the enormous development of its industries, treatment works fed by intercepting sewers and discharging into the canal are being put into operation.

Bibliography

1. Rept. of Mass. State Board of Health on Discharge of Sewage into Boston Harbor, 1900.
2. ALLEN, KENNETH: *Jour.*, Assoc. of Eng. Soc., 1911; **46**, 275.
3. *Jour.*, Assoc. of Eng. Soc., 1906; **36**, 272.
4. RAWN, A. M., and H. K. PALMER: *Trans.*, A. S. C. E., 1930; **94**, 1036.
5. WHIPPLE, G. C.: "Microscopy of Drinking Water," 4th ed., 1927.
6. FITZGERALD, DESMOND: *Trans.*, A. S. C. E., 1895; **34**, 67.
7. FISHER, E. A.: Rept. on Sewage Disposal System of Rochester, N. Y., 1913.
8. Rept. of Lake Michigan Water Comm., 1909.
9. WHEELER, W. H.: "Practical Manual of Tides and Waves," 1906.
10. *Monthly Weather Rev.*, 1902.
11. WHIPPLE, G. C.: Rept. on Quality of the Water Supply of the City of Cleveland, Ohio, 1905.

12. Ackerman, J. W.: *Proc.*, Am. Water Works Assoc., 1913, p. 291.
13. Sanborn, M. F.: *Eng. News*, 1910; **63**, 272.
14. *Prof. Paper* 31, Corps of Engrs., U. S. Army, 1904.
15. Prelim. Rept. of Chicago Drainage and Water Supply Comm., 1887.
16. Special Rept. of Mass. State Board of Health on Examination of Water Supplies, 1890.
17. Prelim. Rept. of an Investigation of Rivers, Ohio State Board of Health, 1897.
18. Ann. Rept., Mass. State Board of Health, 1902.
19. Evidence and Arguments before Committee on Charles River Dam, 1903.
20. Rept. of Eng. Board of Review, San. Dist. of Chicago, on the Lake Lowering Controversy and a Program of Remedial Measures, Part III, App. I, 1925.
21. *Pub. Health Bull.* 172, U. S. Pub. Health Service, 1927.
22. *Pub. Health Bull.* 193, U. S. Pub. Health Service, 1930.
23. *Pub. Health Repts*, 1925; **40**, 693.
24. *Pub. Health Bull.* 183, U. S. Pub. Health Service, 1928.
25. Black, W. M., and E. B. Phelps: Rept. to Bd. of Estimate and Apportionment on Discharge of Sewage into New York Harbor, 1911.
26. Winslow, C.-E. A. *Trans.*, Am. Inst. Chem. Eng., 1910; **3**, 392.
27. *Public Works*, 1921; **50**, 188.
28. Knowlton, W. T.: *Trans.*, A. S. C. E., 1928; **92**, 984.

CHAPTER IX

SEWAGE DISPOSAL BY IRRIGATION

The general features of sewage disposal by irrigation have been outlined in Chap. VI. It will be remembered that there are two general types of irrigation systems—sewage farming, or surface irrigation, and subsurface irrigation. *Sewage farming* is also known as broad irrigation and land treatment. In this process sewage is caused to flow over cultivated fields or to percolate through the ground until it joins the natural ground water or passes into underdrains, incidentally watering and to some extent fertilizing the growing crops. In *subsurface irrigation* sewage is distributed beneath the surface of the ground and penetrates into the soil from open-jointed pipes. While sewage farming has been employed for large communities as well as for smaller ones, subsurface irrigation is confined to small water-carriage systems, more particularly to those of isolated dwellings, hotels, country clubs and institutions. For this reason, discussion of subsurface irrigation is deferred to a later chapter, dealing with the treatment and disposal of residential and institutional sewage.

Purification Processes Operative in Soil.—As in dilution, the processes of purification operative in soil are physical, chemical and biological in nature. They are akin to the forces of self-purification encountered in water but are restricted to a narrower range of acitivity.

Physical Processes.—The physical processes of purification are greatly dependent upon the character of the soil. Two of them are particularly important, namely, *filtration* and *aeration.* Passage of sewage through the soil removes larger solid particles by straining and smaller ones either by sedimentation in the quiescent portions of the tortuous channels between the soil grains or by adsorption on the many contact surfaces. The quantity of sewage that can percolate through the ground is obviously dependent upon soil texture. A stiff clay will allow little water to pass, while a coarse sand will absorb a great quantity. For a discussion of the principles of filtration and for their formulation, see Chaps. XVIII and XXI.

The process of aeration which is active in irrigation is much the same as that taking place in sewage disposal by dilution. On the land surface, oxygen is absorbed from the free atmosphere, while within the soil, oxygen is taken up from the ground air. The sewage as it recedes from the surface, draws in behind it a fresh supply of air which serves to maintain aerobic conditions in the soil. This *breathing* of the soil is important in the economy of irrigation works just as re-aeration is in dilution. In order to permit a replenishing of the oxygen supply, sewage must be applied to land intermittently. A rest period must be provided. It stands to reason that aeration is most active in the upper strata of the soil.

An open, sandy soil would appear to be the most favorable type for sewage treatment by irrigation, because of its greater permeability to both water and air. In sewage farming, however, no matter what the character of the underlying strata, the loam and vegetation will hinder the passage of water and the circulation of air, so that sewage farms can successfully treat only a small percentage of the volume that can be handled on uncultivated sand areas by what is called *intermittent sand filtration.*

Chemical Processes.—Of the chemical processes which are active in irrigation, oxidation is the most important. Aerobic decomposition carries the cycle of organic matter to a point not only where the end products are inoffensive but where they become available as plant foods. This fertilizing value of sewage is discussed below, in connection with the action of sewage on crops.

Biological Processes.—Bacteria abound in moist soil, where they are responsible for the rotation of the elements which enter into the structure of organic matter, just as they are in water. Other organisms, too, are found in sewage-irrigated fields, but life as a whole is less varied than in sewage-polluted watercourses. As sewage matters become converted by biochemical processes into substances which will serve as plant foods, they are taken up by the roots of the irrigated crops.

Action of Sewage on Crops.—Sewage contains chemical constituents which, when present in suitable form, possess fertilizing value. 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, amounting to 99.9 per cent. Probably the chief agricultural value derived from sewage when used for irrigation is due in most cases to its water content. Only portions of the nitrogen, phosphorus and potassium compounds in the sewage are found in a form suitable for direct utilization by plants. Nitrogen should be present as nitrate to be of most value. Usually only a small part of it, or none at all, has arrived at that state of oxidation when the sewage is applied to land. 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 which can be converted into available form.

The fertilizing constituents of sewage are associated with fat and soap, which are injurious to land. They 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 in any of his fields. A precautionary measure against it is the removal of the coarse suspended matter from the raw sewage by racks and sedimentation tanks.

In agriculture, fertilizers should be applied at certain stages in the rotation and growth of crops and the appropriate fertilizers 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 a satisfactory effluent. The crops must be regarded merely as by-products. Evidence furnished by long experience in a number of countries under many conditions does not reveal that it is practicable to obtain much fertilizing effect from city sewage by the means that must be used to obtain successful treatment, but it indicates that where irrigation has been successful agriculturally, about the same results would have been produced with water.

Many varieties of crops have been grown on sewage farms. The general classes of vegetation cultivated are: grasses; coarse beets and other fodder; kitchen vegetables, especially cabbages; corn and wheat; and groves of walnut and orange trees.

Mitchell (1) has presented the information shown in Table 47 on the yield of crops from sewage-irrigated lands in comparison with that from lands not receiving sewage. The sewage farms in question were connected with state institutions.

Preliminary Treatment.—Although crude sewage has been employed successfully for irrigating sandy soils by filtration, it is generally preferable to settle the sewage prior to disposal. This reduces clogging of the irrigation areas and consequent nuisance. Septic tanks have been widely employed in this connection, particularly for small communities and institutions. More recently the effluents from trickling filters and activated-sludge plants have been disposed of by irrigation in those sections of this country and other parts of the world in which the value of water is substantial. From the agricultural standpoint the fertilizing value of the liquid sewage is enhanced by complete treatment, as the nitrogen is rendered more readily available.

Froehde (2) reports that the effluent from the Pomona, Cal., treatment works, consisting of Imhoff tanks and activated-sludge units, contains 21.7 p.p.m. of nitrogen and 9.4 p.p.m. of P_2O_5. Assuming that horse manure contains 2 per cent of nitrogen, he computes that the daily

TABLE 47.—CROPS OBTAINED WITH AND WITHOUT SEWAGE IRRIGATION AT CERTAIN STATE INSTITUTIONS

Institution	Population, 1922	Farm area, acres	Type of crop	Year	Crop yields per acre	
					Land irrigated with sewage	Land not irrigated
Training School for Feeble-Minded, Vineland, N. J.	600	6	Yellow shelled corn	1916	54 bu.	22½ bu.
				1917	70 bu.	10[1] bu.
				1919	60 bu.	37[2] bu.
			Silage corn	1920	13½ tons	12[2] tons
				1921	12 tons	11[3] tons
State Institution for Feeble-Minded, Vineland, N. J.	970[4]	15	Shelled corn for silage	1919	33 tons	12[5] tons
						23[6] tons
						28[7] tons
Lincoln University, Chester, Pa.	265	..	Potatoes, corn (estimated)	1920	140 bu.	87 bu.
				1922	80 bu.	50 bu.
Eastern Pennsylvania State Institution for Feeble-Minded.	...	..	Corn	1917	42 bu.	20 bu.
			Mangels	1917	25½ tons	16½ tons
			Sugar beets	1917	25.2 tons	20.3 tons

[1] No fertilizer.
[2] Six tons of stable manure per acre.
[3] Soil made fertile by good treatment for several years.
[4] About 140,000 gal. daily.
[5] Where fertilized only.
[6] Ten tons of manure per acre.
[7] Twenty tons of manure per acre.

nitrogen content of the effluent, with a flow of 960,000 gal. daily, is equivalent to that of 8750 lb. of horse manure. Both effluent and sludge are employed for agricultural purposes at Pomona.

Construction of Irrigation Areas.—Sewage is applied to land in various ways, some of which are illustrated in Fig. 46. If it is flowed over sloping fields from an upper level to a lower one, the process is called *ridge, 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, the process is called *filtration.* This variety of filtration must be carefully distinguished from *intermittent sand filtration* which is practiced on specially prepared areas that are not used for agricultural purposes, wherever this method of treatment is carried on most efficiently.

Ridge, surface or broad irrigation (Fig. 46*b*) 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 cross ditches 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.

Bed irrigation and land filtration are terms used in Europe 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 (Fig. 46*a*). This method is called *ridge-and-furrow irrigation* in England

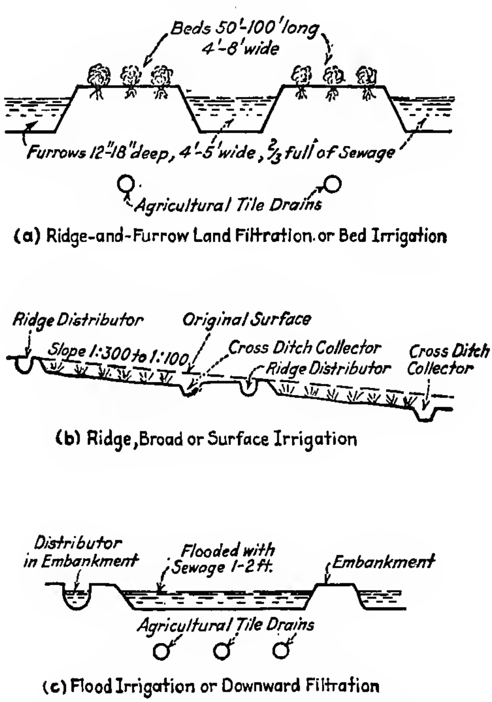


FIG. 46.—Construction of irrigation areas.

and is employed when it is desired to keep the sewage from contact with the crops growing on the beds.

In *flood irrigation* a plot of land surrounded by a bank is periodically covered with sewage to a depth of 1 or 2 ft. (Fig. 46*c*). In Germany, a method of spraying sewage upon the land from movable pipes attached to the sewage conduits has been employed.

To avoid pools of sewage dotted over the fields, the grading of the surface and of the ditches and channels must be carried out correctly. Much more care is paid in England to grading the land for filtration than for surface irrigation, although it is recognized that, even with surface irrigation, pooling of the sewage in detached places is likely to result in sewage sickness of the land at such points and in a poorer effluent. The cost of preparing the land for filtration is much greater than for surface irrigation, not only on account of the need for having the surface perfectly level, but also because it is frequently desirable to strip off the fine surface soil overlying a coarser subsoil.

Fig. 47.—Distribution system employed at sewage farm, Munday, Texas.

Distribution of Sewage.—The sewage carriers are commonly either earth ditches or open or closed conduits laid on the hydraulic grade line and constructed of masonry, concrete or vitrified tile. Force mains and pipe lines under slight pressure have also been employed. The design of the sewage carriers should be developed with a view to minimizing (*a*) the attention required in their operation and maintenance and (*b*) the injury to the system incident to the use of draft animals and agricultural implements in cultivating the land. Even where the land is not cropped, cultivation is considered necessary in order to promote

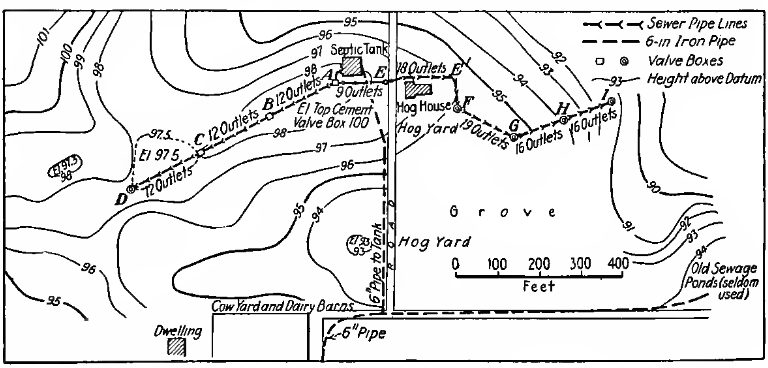


Fig. 48.—Layout of sewage farm, State Institution for Feeble-Minded, Vineland, N. J.

aeration and to check weeds. The distribution system employed at Munday, Texas (3), is illustrated in Fig. 47 and the layout of the sewage farm at the State Institution for Feeble-Minded at Vineland, N. J. (1), is shown in Fig. 48. An outlet developed by Mitchell for underground distributors is shown in Fig. 49. This outlet was employed at the State Reformatory for Women at Clinton, N. J., asphalt jointing compound being used for the pipes. The municipal sewage farm at Vineland, N. J., is provided with a tile main with three valve boxes controlling the flow of sewage into three distribution ditches. At certain points the liquid is diverted into groups of 30 or 40 furrows, 3 ft. apart, at right angles to the ditch (4). The farm covers about 50 acres and serves a population of 8000.

Underdrainage.—The underdrainage of land receiving sewage is regarded as having a dual object: the aeration of the soil and the removal of the effluent after it has percolated through the soil. Drainage is frequently carried out by deep ditches into which lines of 3- or 4-in. agricultural drain-tile discharge. It is held by some engineers that open ditches should be used, to permit inspection of the effluent, and that they should be deep enough to keep the level of the ground water below the level of the tile drains, so that the soil may be kept well aerated. Instead of open ditches, tile-pipe drains often are employed. The general opinion is that all land except clay needs underdrainage when the ground water lies within 4 ft. of the surface. Clay is difficult to underdrain because the tile lines seem to increase the number of cracks which open in dry, hot weather and permit raw sewage to enter the drains in an unchanged condition. The underdrains of English sewage farms, according to Kershaw (5), vary from 2 to 6 in. in size, from 12 to 66 ft. in spacing and from 3 to 9 ft. in depth. The main drains are 4 to 24 in. in diameter.

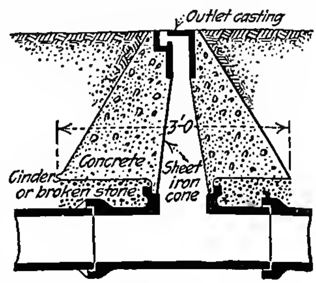


FIG. 49.—Outlet for underground distributors.

At the Fresno, Cal., sewage farm the ground-water level is maintained at least 6 ft. below the surface by drainage pumps drawing water from 16-in. wells, 150 to 320 ft. deep, arranged in 1800-ft. squares (6). The water pumped is clear and practically free from *B. coli.* The farm covers about 812 acres of land and disposes of 10 to 12 m.g.d. of clarified sewage. About 60 per cent of the filtered sewage is pumped up from the wells and delivered to the Fresno Irrigation District for re-use in irrigation.

Underdrains are naturally omitted in surface irrigation and frequently, too, in filtration through porous soils.

Area Required.—The area required for sewage farming depends upon many factors, among which may be listed the method of irrigation, the nature of the soil, climatological conditions—especially rainfall, temperature and evaporation—degree of preliminary treatment and skill of operation.

Common values of the quantity of sewage which can be treated on an acre of land vary from 2000 to 40,000 gal. daily, with an average of

8000 gal. Under European conditions these values correspond to a population of 50 to 1000, with an average of 200 persons per acre. Since irrigation is controlled in large measure by the water load, the population figures must be reduced about 40 per cent, to compare them with American conditions. From two to ten times as much land is required for surface irrigation as for filtration.

Mitchell (1) states that loadings of 50 to 100 persons to an acre have given satisfactory results, although a larger population per acre is permissible. Cohen (7) reports loadings per acre of 8000 to 20,000 gal. daily on 57 Texas farms and Gillespie (8) gives values of 10,000 gal. daily at Pasadena, 15,000 at Visalia and 20,000 at San Luis Obispo, Cal. At these three California plants the areas are cropped. For the uncropped fields at Fresno, Gillespie records a dosage of 60,000 gal. per acre daily and much higher values for Modesto, Kingsbury and Carmel.

Irrigation areas must rest about half of the time. Dosing periods vary from 12 hr. to 10 days at different farms. In addition to the short periods between doses, there must be provided longer *rest* or *drying-off periods*, to permit the land to recover and to allow for the harvesting of crops. In England 20 to 50 per cent of the area is under irrigation at one time. Longer resting cycles are required for surface irrigation than for filtration. American experience appears to be much the same.

Compared with the rainfall the quantities of sewage used for irrigation are often large. At Nottingham, England, for example, where filtration is practiced, the annual rainfall is 25 in., while the annual depth of sewage filtered is about 18 ft. At South Norwood, England, where surface irrigation is employed, the values are 24 in. and 5 ft., respectively. At Visalia, Cal., the annual depth of sewage passing on to the irrigated areas is over 9 ft., which approximates a rate of 8000 gal. per acre daily (9).

English Experience with Sewage Farms.—The operation of irrigation areas was kept under close observation by the Royal Commission on Sewage Disposal, appointed in Great Britain in 1901. A large number of farms were visited and eight were selected as typical of different conditions. These were:

Soil	Farm	Soil	Farm
Sand	Aldershot camp	Heavy loam and clay	South Norwood
Gravelly loam	Croydon (Beddington)	Heavy loam and clay	Leicester
Light loam	Nottingham	Peaty soil and sand	Altrincham
Light loam	Cambridge	Heavy loam	Rugby (high-level farm)

TABLE 48.—INFORMATION CONCERNING EIGHT ENGLISH SEWAGE FARMS, 1900 AND 1901

Place	Aldershot camp	Altrincham	Croydon (Beddington)	Cambridge	Leicester	Nottingham	Rugby (high level)	South Norwood
Date of construction	1864	1870	1861	1895	1891	1880	1867	1864
Total acreage	138	75	673	102	1,699	907	40	191
Irrigable acreage	120	35	420	74	1,350	651	35	152
Average acreage irrigated at one time	40	17	70	18	337	300	7	50
Population draining to farm	20,000	18,000	100,000	50,000	197,000	258,584	6,000	21,000
Population per irrigable acre	166	514	238	675	146	397	171	138
Sewage per capita daily, U. S. gal	60	53	48	54	44	32	60	34
Sewage per acre irrigated simultaneously, U. S. gal	30,000	55,200	68,500	149,900	25,800	28,000	51,400	14,400
Sewage per irrigable acre, U. S. gal	10,000	27,600	11,400	36,500	6,400	12,900	10,200	4.800
Composition of sewage, parts per million:								
Total nitrogen	133.0	35.6	72.0	57.0	81.0	77.0	97.0	52.0
Total organic nitrogen	51.0	22.9	21.0	19.0	23.0	32.0	33.0	15.0
Albuminoid nitrogen	16.2	6.2	9.0	9.0	12.0	18.0	17.0	7.0
Oxygen absorbed[1]	208.0	52.5	125.0	101.0	224.0	232.0	184.0	77.0
Chlorine	150.0	94.3	83.0	85.0	134.0	137.0	100.0	75.0
Solids in suspension	366.0	...	345.0	265.0	341.0	520.0	473.0	219.0
Dilution of effluent	6	3	12	15	1	160	130	⅛
Composition of effluent, parts per million:								
Total nitrogen	60	22.2	22.0	20.9	25.3	22.7	23.1	23.0
Total organic nitrogen	...	2.6	2.6	2.1	4.5			
Albuminoid nitrogen	2.6	1.3	1.4	0.9	2.0	0.3	1.8	1.0
Oxygen absorbed	27.2	17.5	14.1	8.1	25.0	1.9	14.1	14.4
Bacteria per cc. in effluent, averages:								
Gelatine at 20°C	183,260	363,400	1,413,200	711,476	532,777	[2]	637,133	778,322
Reduction, per cent	99	97	95	94	95	...	97	98
Agar at 37°C	37,308	7,275	112,000	78,327	70,500	[2]	81,526	35,157
Reduction, per cent	99	99	97	94	95	...	97	99
B. coli and *coli*-like microbes	1,000–10,000	100–1,000	...	1,000–10,000	1,000–10,000	[2]	1,000–10,000	100–1,000
Spores of *B. enter. spor*	10–100	1–10	10–100	10–100	10–100	[2]	10–100	10–100
Average annual rainfall, in	22	37	24	21	20	25	26	24
Net annual cost of treatment, per 1,000,000 gal., including loan charges, based on dry-weather flow.	no loan charges $7.73	$4.70	$22.51	$9.17	$22.73	...	$5.99	$62.43 including pumping
Net annual cost of treatment per head of population draining to farm, including loan charges	$0.16	$0 08	$0.38	$0.18	$0.36	...	$0.12	$0.78

[1] Oxygen absorbed from permanganate in 4 hr. at 80°F. [2] With few exceptions the effluents were remarkably good.

These farms were studied for more than two years and observations were made less regularly at other farms. The detailed work was under the direction of McGowan, Houston and Kershaw, and full accounts of it were published in four appendices to the fourth report of the Commission. These furnish the most complete information regarding the actual operation of sewage farms which is available. In using it elsewhere, however, careful attention should be paid to the effect of differences in soil and climatic conditions, for the generally moist, equable climate of England, even though the actual rainfall is not high, makes the maximum quantity of water which land can take up very much less than the maximum quantity in places having a warmer and drier climate.

General information concerning these farms, taken from the reports of the Royal Commission, is given in Table 48. It will be observed that from 12 to 53 per cent of the area was not irrigable, either because it was unsuited for the purpose or because it had not been prepared to receive sewage or was required for roads, sites for buildings and sludge beds, grazing and such purposes. The ratio between the total irrigable acreage and the acreage required to treat satisfactorily the normal flow of sewage indicates the actual reserve capacity of the farm, to permit the land to rest after receiving sewage and to provide for unusually heavy demands upon the farm. Theoretically, also, it should be proportional to the character of the land, a farm containing heavy soil

Table 49.—Average Areas of Land Required for Treating a Dry-weather Flow of 1,000,000 U. S. Gallons Daily (10)

Class of soil	Method of working	Volume of settled sewage which can be treated per acre per 24 hr., U. S. gal.	Total area of land required to treat a dry-weather flow of 1,000,000 U. S. gal. daily,[1] acres
I. Good soil and subsoil	Filtration with cropping	14,400	70
I. Good soil and subsoil	Filtration with little cropping	30,000	33
I. Good soil and subsoil	Surface irrigation with cropping	8,400	121
II. Heavy soil on clay	Surface irrigation with cropping	6,000	167
III. Stiff clayey soil on dense clay	Surface irrigation with cropping	3,600	278

[1] These areas are sufficient for the treatment in times of storm of three times the mean dry-weather flow.

requiring a larger reserve capacity than one with a light soil. In practice, this relation is difficult to observe, financial and topographical conditions obscuring its effect.

The Royal Commission's conclusion from its investigations was stated substantially as follows in its fifth report, issued in 1908:

Having regard to the volume of sewage dealt with and the purification effected at the farms kept under the Commission's observation, as well as the testimony given by various expert witnesses called by the Commission, it estimated that the several classes of soil and subsoil could deal effectively with the volumes of settled sewage given in Table 49, under the climatic conditions of England.

The Sewage Farms of Paris and Berlin.—The two largest cities in the world that dispose of their sewage mainly by irrigation are Paris and Berlin. A brief description of their sewage farms is given below.

Paris.—The sewage farms about Paris are the chief French examples of sewage irrigation. In 1923 the volume of the city's sewage, from a population of almost 3,000,000 in the districts connected with the farms, was more than 200 m.g.d., of which 120 m.g.d. were used for irrigation, the remainder being discharged into the River Seine. The area of the farms is given in Table 50.

The farms owned by the city had cost $7,220,000 in 1900 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 was 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 as he desires. The owners of land within the irrigation districts take the sewage as they desire it.

TABLE 50.—SEWAGE FARMS OF PARIS IN 1924

District	Area of farms, acres		
	Privately owned	Owned by city	Total
Gennevilliers	1,880	15	1,895
Achères	410	2,965	3,375
Méry-Pierrelaye	3,731	1,236	4,967
Carrières-Triel	2,137	210	2,347
	8,158	4,426	12,584

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.

Berlin.—The sewage farms of Berlin afford the largest opportunity for this method of treatment yet undertaken in any country. At the close of 1910 the area of these farms was as given in Table 51 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 7994 acres were used for broad irrigation, 12,250 acres for filtration, 502 acres for settling basins and 2105 acres for roads and buildings. The rate of filtration was about 3700 gal. daily per acre of prepared land. 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 $3200 annually. In 1926 the Berlin sewage farms were treating about 150 m.g.d. of sewage from 4 million persons on 27,250 acres.

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. The net cost of sewage disposal was therefore about $24 per mil. gal. Similar balances have been maintained during recent years.

TABLE 51.—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
Total	27,304	6,442	9,263	43,009

American Sewage Farms.—Sewage irrigation has not been employed in the United States on such a large scale as in Europe, but it has been practiced in a small way at institutional disposal works and in a number of communities, particularly in Southern California and Texas, for many years. The cities of Danbury, Conn., Fresno, Pasadena and

Pomona, Cal., and San Antonio, Tex., are among the larger places where it has been given long trials with more or less success. Cohen (7) reports its use in 57 Texas cities, excluding San Antonio, mostly located in the western and southwestern portions of the state, the largest city having a population of 20,000. Health officers report from time to time that sewage is treated by irrigation, when what actually 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. Cheyenne, sometimes 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 farm land with sewage, gave up the practice many years ago.

The more extended use of irrigation in Southern California and Texas than elsewhere in the United States is probably due to the suitable character of the soil for sewage farming, the relatively low rainfall and the scarcity of water. The first condition materially reduces the cost of this treatment, while the last two greatly facilitate using the sewage in a fairly satisfactory agricultural way and without decided sanitary defects.

A number of American sewage farms have been mentioned in previous sections of this chapter. Generally speaking, records of construction and operation are not adequate because of the very nature of the works.

The sewage of San Antonio, Tex., with a population of 231,542 in 1930, was conveyed 12 miles from the city to a 6700-acre, privately-owned tract, where it was used in irrigating 4000 acres. The sewage amounted to about 20 m.g.d., so that the land was dosed at the rate of 5000 gal. per acre daily. The irrigating season was from the middle of February to the middle of November and during the remaining three months of the year the sewage was discharged into an artificial lake, having an area of approximately 1000 acres. In recent years the sewage flow became too heavy for existing arrangements and an activated-sludge plant now treats the sewage before discharging it into Lake Mitchell. The lake waters either flow into the Medina and San Antonio Rivers or may be used for irrigation. There is a possibility that an irrigation district will be formed to utilize the plant effluent for irrigation purposes (11).

In 1926 Lubbock, Tex. (population 20,520 in 1930), began to apply settled sewage to 88 acres of land, to irrigate a variety of crops. The flow is equivalent to 4.7 in. per month on the cultivated land and it is estimated that 40 per cent is lost by evaporation and seepage, leaving

3 in. per month going to the crops. After the rainfall is added and evaporation from soil and transpiration are deducted, the net residual moisture going into the soil is said to be 3 in. per month. Roberts (12) states that "experience at Lubbock has indicated that conditions and results would be better if this residual were about one half this. This would mean that 1 acre of land would take care of 2200 gal. of settled sewage per day."

At Pomona, Cal., a city of 20,804 persons in 1930, the effluent from an Imhoff-tank, activated-sludge plant is sold to the Northside Water Company for irrigation purposes (2). The company pays ½ cent per inch-hour[1] of effluent during the winter and 1 cent during the summer, approximately equal to \$7.50 per mil. gal. and \$15.00 per mil. gal., respectively. The contract runs for 20 years. During 1928–1929 the return from the sale of effluent was \$4669 and from the sale of sludge \$414. The cost of operating the plant was \$9112, including \$1000 for chlorination.

At Vacaville, Cal., with a population of 1556 in 1930, the effluent from a septic tank passes into a header ditch, from which pipe ports, 15 to 20 ft. apart, lead to the irrigation area (13). The soil is tight and the sewage flows over the dense growth of grass covering the irrigating beds into storage ponds holding 3.5 mil. gal. Most of the sewage evaporates or is used by the vegetation. While 125,000 gal. of sewage are discharged daily on 8 acres of land, totaling 30 mil. gal. during the dry period, not more than 3 mil. gal. accumulate in the ponds. The effluent is well purified and produces a prolific growth of algae in the ponds.

The effluent from three septic tanks, which provide 14-hr. detention, is discharged on to 400 acres of irrigation beds used chiefly for row crops of cotton, corn, maize and cane, and also for pecan trees and grass meadows, at Abilene, Tex., a city of 23,175 persons in 1930 (14). The soil is well drained and no effluent reaches the nearby stream. The rate of application of the 650,000 gal. of sewage a day is about 1600 gal. daily per acre, or 22 in. a year. The average annual rainfall is 24 in., and the average annual evaporation from water surfaces is 54 in.

Sewage irrigation at Pasadena, Cal., began about 1887, when 300 acres were bought at \$125 an acre for a municipal farm. In 1905, 160 acres more were purchased at \$150 and later purchases brought the total up to 518 acres in 1914. The property was 4½ miles from the city, from which the sewage was carried in a 16- to 20-in. vitrified pipe. For some years the sewage was passed through settling basins, where the deposits were mainly rags, corks and coffee grounds, and was then delivered through pipes to outlets 400 to 500 ft. apart. These delivered the sewage into head ditches, from which furrows were run 2 to 6 ft. apart.

[1] One miner's inch equals 674 gal. per hour. Hence, 1 inch-hour = 674 gal.

A septic tank was constructed in 1910 and concrete pipes were laid to distribute the sewage about the farm. The standpipes were placed about 150 ft. apart. The sewage was not allowed to run continuously on any area of open ground longer than 4 to 10 days and, as soon as the land was dry, it was thoroughly cultivated and occasionally plowed. In the walnut groves sewage was formerly kept on the land from Dec. 1 to April 1, while the leaves were off the trees. During the remainder of the year, the sewage was used exclusively on the open fields. Alfalfa has been a profitable crop, but, as the plants collected the solids in the sewage, cultivation of this crop was abandoned for a time.

The condition of the farm in 1914 is outlined in the following notes from Smith, then city engineer:

> Alfalfa was raised on 122½ acres; walnuts on 112 acres; oranges on 70 acres; oat hay on 120 acres; corn on 40 acres; pumpkins on 4 acres; sweet corn on 3 acres; Kaffir corn on 2 acres. About 1,800,000 gal. of sewage were received daily, and after the septic tank was put in service there was no trouble caused by solids. Alfalfa is now one of the best crops and a cutting is made about once a month, the land being irrigated after each cutting. The walnuts are now irrigated about twice during the summer and twice during the winter.

Growth of population in the vicinity of the Pasadena farm caused dissatisfaction with this method of disposal and an activated-sludge plant was built and placed in operation in 1924. The effluent of the activated-sludge plant is at present used for irrigation at the farm. Hatch (15) reports that the irrigation season is from May to November and during this time about half the effluent is utilized. It is expected that the rest of the effluent will eventually be sold to private companies for irrigation purposes. It is said that an offer of $2,500,000 has been made for the present 576-acre farm.

Public Health Aspects of Sewage Farms.—From the hygienic standpoint there are two phases of sewage farming that must be considered, *i.e.*, its ability to produce a satisfactory effluent and the possible dangers from the sewage as it passes over or into the soil. In regard to the first point, Table 52 shows that the purification effected by land treatment is relatively great and comparison with the so-called artificial-treatment methods, discussed in succeeding chapters, will indicate that the effluent from well-managed sewage farms is as good as that produced by most of the treatment processes that make use of sedimentation followed by oxidation. This is true more particularly of filtration than of surface irrigation. Pollution of watercourses, therefore, may be greatly reduced by land treatment of the sewage prior to its discharge into them.

So far as the second point is concerned, the Royal Commission on Sewage Disposal expressed its opinion as follows in its fourth report:

TABLE 52.—ANALYSIS OF APPLIED SEWAGE AND EFFLUENTS OF EUROPEAN SEWAGE FARMS
(Results of chemical analyses in p.p.m.)

	Nottingham		South Norwood[1]		Paris[2]		Berlin[3]	
	Raw	Effluent	Raw	Effluent	Raw	Effluent	Raw	Effluent
Nitrogen:								
Total	76.9	22.7	52.0	23.0				
Organic	31.5	...	14.9	...	6.7	0.0		
Albuminoid	14.5	0.3	6.7	1.0	...	...	99.5	2.3
Ammonia	39.8	1.3	35.4	8.7	15 4	0.0–0.2		
Nitrite	...	20.6		3.9	...	...	...	146.6
Nitrate	...		...		0.8	19.1–21.0	...	
Oxygen consumed	232.1[4]	1.9[4]	77.1[4]	14.4[4]	...	...	333.7	33.6
Dissolved oxygen	...	7	...	3.7				
Carbon dioxide	...	120	...	55				
Solids:								
Total	...	...	...	...	...	...	978.4	987
Mineral	...	...	...	...	152	18.2–30.7	693.2	863
Organic	...	...	...	...	33.5	0.9–1.7	285.2	124
Suspended	519	...	219					
Bacteria per cc.								
Gelatin 20°C	23,350,000	21 to 1,540[5]	48,600,000	6,700 to 1,620,000	148,300,000	125 to 1,000		
Agar 37°C	5,150,000	3 to 547,000[5]	6,360,000	980 to 83,000				
B. coli	> 100,000	0 to 500[5]	> 100,000	10 to 5,000				

[1] Royal Commission on Sewage Disposal, 4th and 5th Reports, 1904–1908.
[2] Bechman, 1901. For results for the year 1922 see Diénert, *Rev. d'Hygiène*, 1924; **46,** 1117.
[3] Dunbar, 1908.
[4] Four hours at 80°F.
[5] With one exception.

As regards the likelihood of sewage farms being dangerous to health, we can do no more than tentatively express the opinion that no convincing proof has yet been furnished of direct or wide-spread injury to health in the case of well-managed farms.

It may be possible that the foul emanations from a badly-managed or over-sewaged farm constitute an indirect source of danger to health by lowering the vitality of weakly or susceptible individuals.

Possible channels of infection are flies and other insects carrying germs of disease mechanically from farms to human habitations; kitchen vegetables consumed without cooking; the milk of cows pastured on irrigated lands, and ground water polluted by sewage seeping into it from the filtration areas. In the absence of epidemiological experience, considerations of hygiene dictate that a safe policy be adopted. Dairying and the raising of produce to be consumed raw, even though sewage does not come into direct contact with it—as is true of some vegetables grown on ridge-and-furrow areas and of the fruits of some sewage-irrigated trees—should not be allowed on sewage farms. Ground-water supplies in the vicinity of sewage farms should be closely watched for evidences of contamination.

In general, the average layman has a strong prejudice against the use of foodstuffs grown on sewage farms and the odors arising from poorly managed irrigation areas are apt to strengthen his prejudice, in spite of the lack of scientific support. It is said that the 4000 people resident on the Berlin sewage farms have consistently had good health. The views of the California State Board of Health are expressed in a circular addressed to all cities, counties and state institutions in 1918. The following regulations are taken from this circular:

Resolved that the following regulations governing the use of raw sewage, septic or Imhoff tank effluents or similar sewage or water polluted by such sewages for the irrigation of garden truck, berries, fruit trees, and stock feed, be adopted:

Rule 1. Raw sewage, septic or Imhoff tank effluents, or similar sewages, or water polluted by such sewage shall under no circumstances, be used to water garden truck or berries intended to be eaten raw by human beings, *e.g.*, tomatoes, celery, lettuce.

Rule 2. Garden truck used or intended to be used for human consumption in the cooked state may be watered with sewage, provided that no sewage shall be applied to the soil or vegetable for at least one month prior to harvesting and consumption of the product. Such crops are green corn, cabbage, cauliflower, chili peppers, pimentoes, asparagus.

Exception: By special permit from the State Board of Health, fruits and vegetables included in Rule 1 and Rule 2 may be irrigated with sewage when said products are used exclusively for commercial canning purposes.

Rule 3. Vegetables harvested exclusively in the dry state may be watered with sewage; such crops are beans, late potatoes.

Rule 4. Trees bearing fruits or nuts may be watered with sewage. Provided, however, that windfalls and fruit lying on the ground shall not be used for human consumption.

Rule 5. Cantaloupes, watermelons and cucumbers may be irrigated with sewage, provided that the sewage be not allowed to come in contact with the vine or product.

Rule 6. Stock foods, such as alfalfa, fodder corn and cow beets, may be watered with sewage, provided that milk cows be not pastured on lands while moist with sewage.

Live Stock on Sewage Farms.—Kershaw (16) states that

cattle should rarely be permitted to roam loose on land used for sewage treatment, on account of the damage they do to land—especially heavy soils—by "pocketing" it with footprints, in which the sewage or tank liquor after application to the land, stagnates and smells when the field is dried off. Where surplus land is available on the farm, or where the cattle are stall-fed, this objection naturally does not arise, and heavy crops of rye grass can be disposed of regularly where cattle are kept.

At Melbourne, Australia, cattle and sheep are bred and fattened and horses are pastured on the municipal sewage farm (17). Altogether there were grazed and fattened on the 8084 acres under irrigation in 1926 from 6000 to 8000 head of cattle and from 5000 to 17,000 sheep, while from 500 to 1000 horses were on pasturage. Dairying is not permitted by the health authorities. The revenue from grazing more than covers the expenses of land treatment, apart from interest on capital. During drought years the revenue also covers interest. The population of Melborne is about 800,000 and the annual rainfall averages 18 in.

Present Status of Sewage Farming.—The fact that sewage farming is the oldest method of sewage treatment, if it be regarded as a treatment process, should not detract from its value. This view is supported by the fact that the municipality of Berlin is abandoning more recent and thoroughly efficient trickling-filter installations for sewage farms. With other methods of treatment available, however, sewage farming can compete with the so-called artificial treatment methods only when wide tracts of suitable land are available at low cost, efficient management both from a sanitary and agricultural viewpoint is assured, and water is scarce and hence valuable.

In America, therefore, sewage farming seems practically to be restricted to the semiarid regions of the Southwest. In England its use is decreasing rapidly. In France other processes are being studied with a view to abandoning some of the existing farms. In Australia, South Africa and countries with similar climatic and soil conditions, however, extensions to sewage farms are being made.

Bibliography

1. Mitchell, G. A.: *Eng. News-Rec.*, 1924; **92,** 284.
2. Froehde, F. H.: *Calif. Sewage Works Jour.*, 1929; **2,** 82.

3. Ehlers, V. M.: *Sewage Works Jour.*, 1930; **2**, 105.
4. Mitchell, G. A.: *Eng. News-Rec.*, 1930; **104**, 65.
5. Kershaw, G. B.: "Modern Methods of Sewage Purification," 1911.
6. Jensen, A. M.: *Calif. Sewage Works Jour.*, 1929; **2**, 37.
7. Cohen, Chester: *Proc.*, 11th Texas Water Works Short School, 1929, p. 126.
8. Rossi, F. J.: *Calif. Sewage Works Jour.*, 1929; **2**, 96.
9. Gadsby, L. H.: *Calif. Sewage Works Jour.*, 1929; **2**, 92.
10. Fifth Rept., Royal Comm. on Sewage Disposal, 1908.
11. Helland, H. R. F.: *Proc.*, A. S. C. E., 1929; **55**, 1415.
12. Thackwell, H. L.: *Proc.*, A. S. C. E., 1929; **55**, 1419.
13. Gillespie, C. G.: *Sewage Works Jour.*, 1928; **1**, 68.
14. Riney, W. A.: *Proc.*, 10th Texas Water Works Short School, 1928, p. 254.
15. Hatch, J. N.: *Public Works*, 1929; **60**, 129.
16. Kershaw, G. B.: "Sewage Purification and Disposal," 1915.
17. *The Engineer*, 1926; **142**, 519.

CHAPTER X

RACKS AND SCREENS

Municipal sewage contains floating and suspended matter, or trash, such as cloth, paper, kitchen refuse, pieces of wood, cork, hair and fiber, and uncomminuted fecal solids, all of which are removed readily by screening. In some cities with combined systems of sewers large street inlets are used and occasionally open brooks are tributary to sewerage systems. In such cases quantities of bulky trash may enter the sewers, such as barrels, boxes, cans, lumber, railway ties, brush and small trees.

Among the objects accomplished, at least in part, by screening the following are prominent:

A. In conjunction with appliances for conveying and pumping sewage and sewage sludge:
1. Protection of pumps from injury.
2. Protection of inverted siphons and force mains from clogging.
3. Protection of valves and gates to insure effective operation.

B. In conjunction with sewage disposal by dilution:
1. Removal of unsightly matters which would float on the surface of the waters receiving the sewage or become stranded on their shores.
2. Reduction in the quantity of sludge settling to the bottom of slow-moving bodies of water and likely to form sludge banks or cause offense by decomposing.

C. In conjunction with sewage-treatment processes:
1. Removal of floating matters that tend to form unsightly scum on settling and aeration tanks.
2. Removal of coarse solids which would tend to clog or otherwise interfere with the operation of mechanical sludge-moving equipment in sedimentation tanks.
3. Prevention of heavy and extremely tough floating scum on the surface of septic tanks and other sludge-digestion tanks.
4. Removal of solids likely to clog trickling-filter nozzles or the surface of filters or of irrigation areas.
5. Removal of larger solids that may settle to the bottom of aeration units, where they may interfere with air diffusion or may putrefy.
6. Removal of coarse solids and uncomminuted fecal matters that are not penetrated readily by chlorine when sewage is disinfected.
7. As a temporary expedient for service while developing more complete methods of treatment and building the plant for applying them.

Classification of Screening Devices.—The following definitions are introduced here to avoid confusion as to the meaning of certain terms (1):

A *screen* is a device with openings, generally of uniform size, used to retain coarse sewage solids. The screening element may consist of parallel bars, rods or wires, grating, wire mesh, or perforated plate and the openings may be of any shape, generally circular or rectangular slots. A screen composed of parallel bars or rods is called a *rack*. Although a rack is a screening device, the use of the term *screen* should be limited to the type employing wire cloth or perforated plates. However, the function performed by a rack is called *screening* and the material removed by it is known as *screenings*, although *rakings* is more convenient in some cases. If racks or screens are stationary, they are termed *fixed* and if they are capable of motion, they are termed *movable*. According to the method of cleaning, racks and screens are designated as *hand cleaned* or *machine cleaned.*

An important distinction is made, according to the purpose of screening, between *fine screening*, which has certain elements of a treatment process, and *coarse screening*, which is not primarily intended to be such a process. In a general way, the size of opening which is on the boundary between coarse and fine screening is taken by American engineers at ¼ in. By size of opening is meant the least dimension of the waterway. The relative terms used to indicate the various sizes of openings employed in racks and screens are shown in the following schedule:

	Size of Opening
Racks:	
Coarse	More than 2 in.
Medium	1 to 2 in.
Fine	Less than 1 in.
Screens:	
Coarse	¼ in. or more, sizes larger than ⅜ in. seldom being used.
Fine	Less than ¼ in.

Duplication of Racks and Screens.—Duplicate equipment is generally provided, so that at least one unit may be in service while others are being repaired or cleaned. In the case of racks this duplication is provided by placing them either in series in a single channel or in series or parallel in two or more channels, side by side. The preference is generally given to two channels, in order that accident to one rack may not interfere with the operation of another. In the case of movable racks, however, installations are made in pairs, one rack being placed a short distance upstream from the other, so that when the upstream rack is lifted for cleaning, the downstream rack may be in use. With such an installation there is the possibility that some large objects may be swept along the sewer, when the downstream rack is lifted.

RACKS

Racks are commonly used for the protection of appliances for conveying and pumping sewage and sludge, for the general protection of sewage-treatment plants and for removing unsightly objects where sewage is disposed of by dilution.

Location with Respect to Grit Chambers.—Where racks are used in conjunction with grit chambers, there may be some question as to whether it is better to place the racks upstream or downstream from the

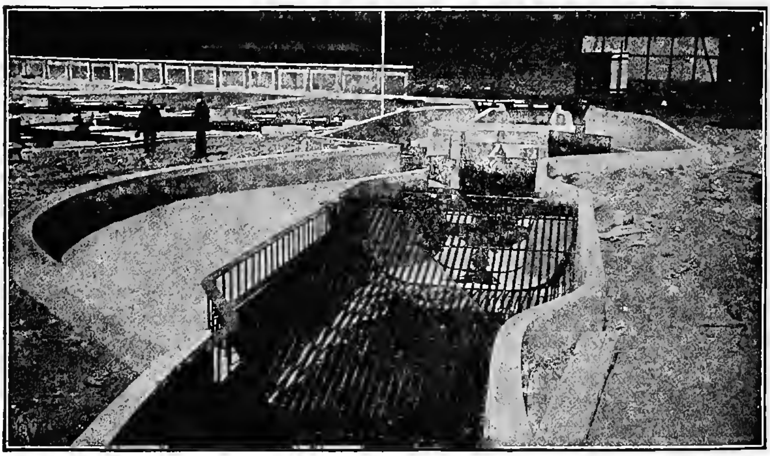


Fig. 50.—Waste-weir at rack of Emschergenossenschaft plant.

chambers. Prevailing practice seems to be to place the racks on the upstream side.

When partially clogged, racks placed upstream will cause the sewage to back up in the sewers, thus tending to form deposits there rather than in the grit chambers. On the other hand, if placed downstream the racks will tend to cause the deposition of organic matter in the chambers and thus increase the offensive properties of the grit. Where grit chambers are made abnormally large for the purpose of retaining coarse organic material, the cleaning of racks will be facilitated if they are placed downstream.

Area of Fixed Racks.—Racks generally are placed at right angles in plan to the axis of the channel they cross. Their area should depend upon the quantity and character of sewage to be screened, the frequency of cleaning, the size of the openings and the permissible loss of head.

Where necessary in order to provide adequate area, the channel is sometimes widened at the racks. In other cases the same purpose has been accomplished by the use of racks which are angular, curved or even round in horizontal cross section. The object is attained more commonly, however, by inclining the racks, as shown in Fig. 50. Boston experience indicates that with combined sewers the rack should have a clear area for the passage of sewage at least 50 per cent greater than that of the channel leading to it. Fuller and McClintock (2) state that for domestic sewage a rack area of 20 sq. ft. per mil. gal. daily is ordinarily satisfactory, where racks are cleaned three times during the day and are left clean at night.

Width of Opening of Fixed Racks.—Where racks are provided to protect centrifugal pumps, the width of openings depends in considerable measure upon the type and design of impellers. Clogging may be caused by relatively large objects, such as bodies of dead animals, or by stringy material like rags and rope. For small pumps with the older divided or shrouded impellers having small passageways, rather narrow spacing of racks was employed, or screens of rectangular mesh even were installed occasionally. With pumps of the modern nonclog type, larger objects will pass through the impellers and coarser racks are being employed. With the older type of pumps a clear waterway of ½ to ¾ in. between bars was employed commonly, while with the more modern impellers racks with clear openings of 2 to 3 in. frequently give satisfaction. In the case of some of the larger pumping installations, as at the Connors Creek storm-water pumping station at Detroit, racks have been omitted. With pumps of some designs, even of considerable size, it is necessary to use racks of narrow openings.

The size of the openings in racks guarding treatment works is gradually becoming standardized at ½ to 3 in. A 1-in. opening generally is employed in the Boston district for racks screening sewage at pumping stations and before its disposal by dilution. Sometimes two or three racks are used in series, openings of 1¼ to 1½ in. being provided in the first rack and of ¾ in. in the last one.

In the case of racks with openings smaller than ½ in., there are three features to be considered particularly: first, the danger of unduly raising the sewage level at the rack by the clogging of the openings with trash; second, the arrangements that must be made for the disposal of the large quantity of rakings from such fine racks; and, third, the cost of labor or machinery required for cleaning. The first point is important because, if screening devices become so clogged as to cause pooling of the sewage, troublesome deposits may be formed in the sewers. As to the second point, the rakings from fine racks, if not promptly disposed of, are more likely to cause objectionable odors than those from coarse racks.

At Plainfield, N. J., Lanphear found that racks with ½-in. openings became so clogged between 5 and 9 A.M., if unraked, that the loss of head was 6 to 9 in. This much exceeded the loss during the hours of regular cleaning.

There is considerable variation in the degree of importance attributed to keeping racks clean. In some small separate systems, where no storm water enters the sewers, little 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 occasionally an arduous task. There is apparently no way to foresee exactly what the conditions will be and experience indicates that the racks and their supports should be strong enough to prevent their failure, in case they become clogged and the sewage is dammed back by them.

If there is any likelihood of the development of a high head upon a rack, it may be desirable either to install a float-operated alarm system, which will automatically give notice when the sewage above the rack reaches a dangerous height, or to provide a by-pass with a screened waste-weir of sufficient length, as indicated in Fig. 50, to prevent the sewage from rising above a predetermined height. As a preliminary step in estimating the probable effect of a rack on the elevation of the sewage in the channel above it, a computation may be made by the following formula:

$$h = \frac{V^2 - v^2}{2g} \times \frac{1}{0.7}$$

where V and v represent, respectively, the velocity in feet per second through the openings of the rack and in the channel above the rack, and h is the head in feet due to the rack. The acceleration due to gravity is denoted by g and 0.7 is a coefficient suggested for this use by Frühling (3).

Slope of Fixed Racks.—Some types of movable racks are constructed in a vertical position, to simplify lifting them and to economize space, while other types are inclined. Stationary racks are inclined, to facilitate cleaning and afford a larger area for the passage of sewage, thus reducing the velocity of flow through the openings. There is no close agreement among engineers concerning the best slope for these inclined racks, but the usual inclination is somewhere between 30 and 45 deg. from the vertical. With hand-raked racks of moderate size, the greater the inclination, the easier will be the cleaning.

In a few German plants, where the inclination of the racks is great, their length has been reduced, in order to facilitate cleaning, by supporting their feet on a low transverse wall built up on the invert of the channel. Where such a plan is followed, it is manifest that care should be taken to design the wall so that as little sludge as possible will accumulate in front of it.

Mechanical rack-cleaning equipment, of the types manufactured by the Dorr, Link-Belt, Chain-Belt, Jeffrey and other companies, ordinarily is designed for fixed racks inclined at 30 deg. from the vertical.

Box or Basket Type of Racks.—Fixed racks in the form of baskets or boxes have been employed at some plants. Regarding an installation of this type at the Irwin Creek plant, Charlotte, N. C., McConnell (4) states:

> There are two compartments, each of which is provided with two box racks measuring 24 in. square by 20 in. deep, constructed of ¼-in. by 1¼-in. galvanized-steel bands with ⅜-in. clearances. The bottoms are solid and

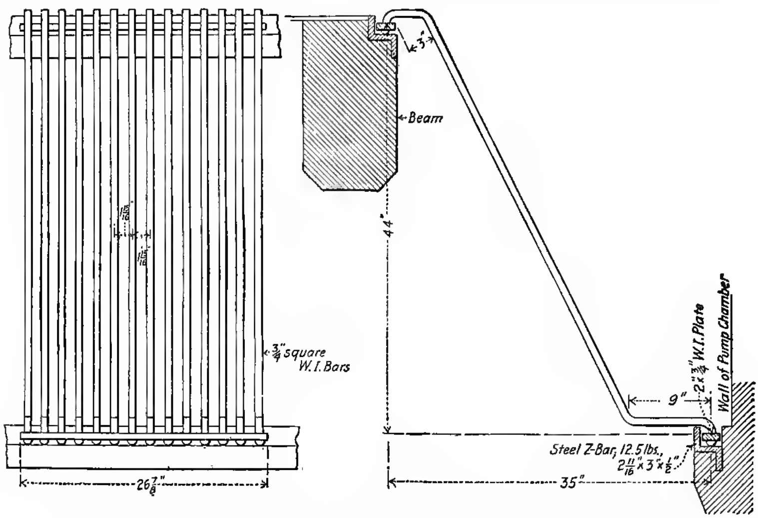


Fig. 51.—Rack used at pumping station, North Attleborough, Mass.

> operate as sluice gates, which permits of dumping the screenings directly into a wagon below. . . . Sluice gates are provided for changing the flow from one compartment to the other for cleaning purposes.

Hand-cleaned Fixed Racks.—Hand-cleaned racks are raked with long-handled rakes, with teeth spaced to fit into the rack openings. In well-designed plants the floors and receiving troughs are sloped and perforated in such a way as to permit the prompt escape of the water which drains out of the rakings, as well as that used for washing the floors, screens and tools. Cleanliness about the screen room and prompt removal of rakings are necessary to prevent objectionable odors and flies.

The rack illustrated in Fig. 51 permits of easily pulling the rakings back over the tops of the bars and allowing them to fall on a platform or into a trough.

Machine-cleaned Fixed Racks.—Cleaning racks of small and moderate size by hand is sometimes laborious, particularly during storms and in the fall when leaves are washed into combined sewers. The hand clean-

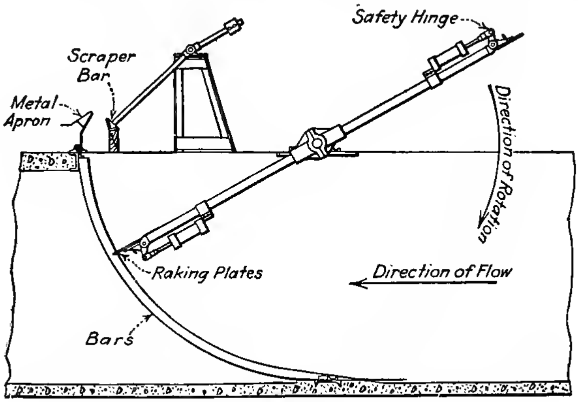


FIG. 52.—Cross-section of curved Dorrco bar screen.

ing of many large racks is impracticable because of their great depth and size. These conditions have led to the installation of machine-cleaning

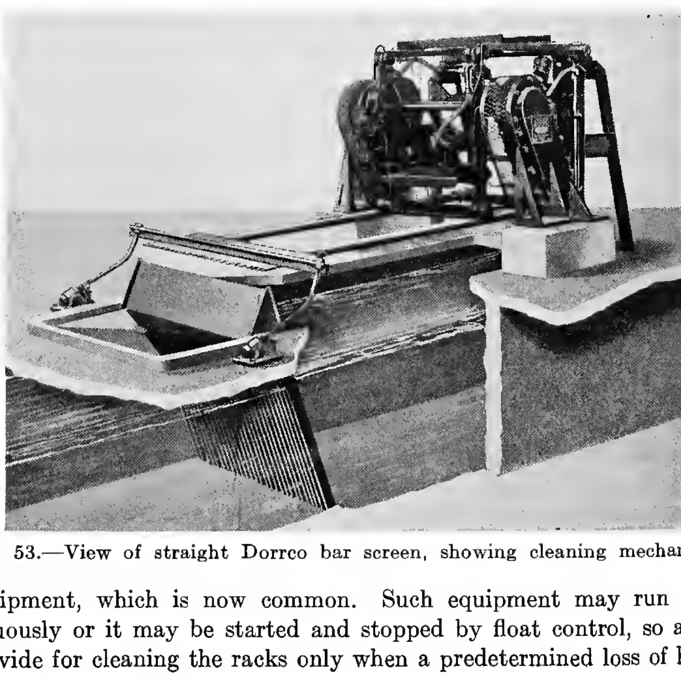


FIG. 53.—View of straight Dorrco bar screen, showing cleaning mechanism.

equipment, which is now common. Such equipment may run continuously or it may be started and stopped by float control, so as to provide for cleaning the racks only when a predetermined loss of head is reached. Intermittent operation results in a saving in power and in wear on the equipment.

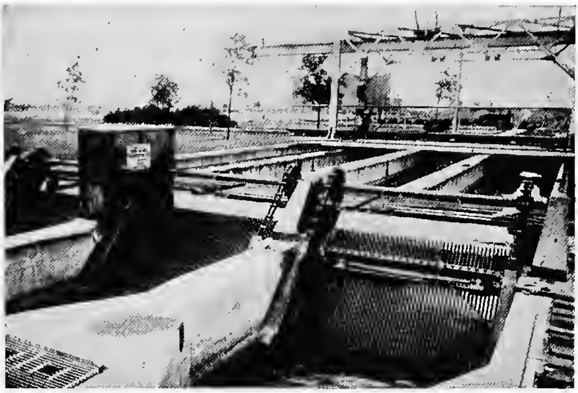


FIG. 54.—Dorrco bar screens at Calumet treatment plant, Chicago, Ill.

In one of the early forms of Dorr machines, shown in Fig. 52, the cleaning mechanism consists of a revolving arm carrying raking plates

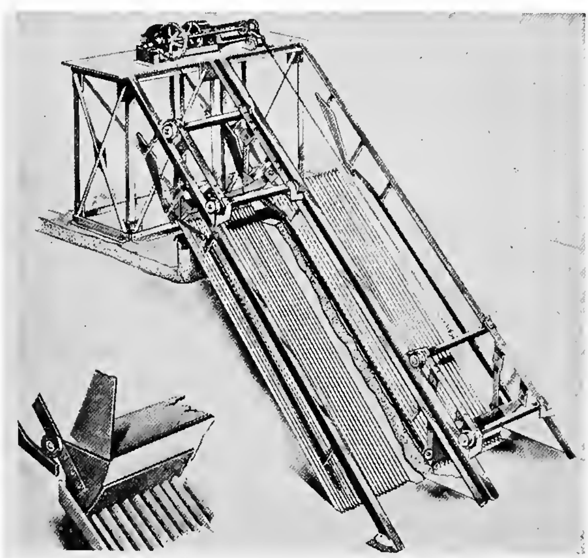


FIG. 55.—Link-Belt straight-bar rack rake.

at each end. For this type of equipment the rack bars must be curved in the shape of a quadrant, concave upstream. The raking plates are dentated to fit the rack and sweep the screenings up the bars. When the plates arrive at the top, a scraper is engaged, which removes the solids. The largest installation of equipment of this type is at the West Side plant in Chicago, where there are five racks, each 11 ft. deep and 10.5 ft. wide, with 1-in. clear openings.

Another form of Dorr machine for a stationary rack, which is usually set at an angle of 30 deg. from the vertical, is illustrated in Fig. 53. As the raking plate of this apparatus clears the top of the bars, it engages with the rake scraper, which moves the rakings across the raking plate and discharges them into whatever receptacle is provided. A typical installation of this equipment at Trenton, N. J., cleans a rack 6 ft. deep and 8 ft. wide, with 1½-in. clear openings.

Another type of Dorr machine, installed at the Calumet plant of the Chicago Sanitary District, is shown in Fig. 54.

A type of machine made by the Link-Belt Co., shown in Fig. 55, has the rake secured to a carriage which travels back and forth over the rack. On the upward travel the rake engages with the rack and carries the accumulated solids to the top. As the rake passes the top of the rack, it swings past a scraper, which scrapes the adhering solids into a trough or conveyor. The rake is then carried back, clear of the rack, to the bottom, where it is lowered again into engagement with the bars for a repetition of the operation. The speed of the rake is approximately 7 ft. per minute. By means of a time relay the rake is stopped at the highest and lowest positions for a period ranging from ½ to 30 min., dependent upon local conditions. Equipment of this type is provided at New Haven, Conn.

One of the older machines is at Toronto, Ont., where there are six racks, each 5 ft. 8½ in. wide, two in chambers 33 ft. deep and four in chambers 14 ft. 3 in. deep. The racks are made of bars 10½ ft. long and ½ in. thick, spaced to give ½-in. openings. 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 brushed off manually. The material drops to a traveling belt and is discharged into a hopper. Grit removed by the bucket elevator is also dropped directly into a hopper. A truck is run under the hopper to remove the material.

Movable Racks.—There are many forms of movable racks which may be classified in two groups, hand cleaned and machine cleaned. The first class is found more generally in the United States than in Europe, where racks of the second class have been commonly used for many years.

Hand-cleaned Movable Racks.—The only movable racks cleaned by hand are cage racks, which were designed first in America for the Calf Pasture pumping station of the Boston Main Drainage Works. They were planned for the protection of the pumps and went into operation

Jan. 1, 1884. Several similar racks have been installed in the Boston Metropolitan Sewerage District. These installations consist of a rack chamber, extending from the invert of the sewer to the surface of the ground and surmounted by a superstructure for housing the cleaning and hoisting apparatus. Usually the racks are in sets of two, one upstream of the other, with two sets in parallel, as in Fig. 56. Each rack consists of a cage, provided on three sides with vertical ¾-in. bars, with open spaces of 1 in. The front of the cage, facing the incoming sewage, is open and the steel bottom is perforated to allow the water to drain off as the cage is hoisted. Each cage is about 9 ft. wide by

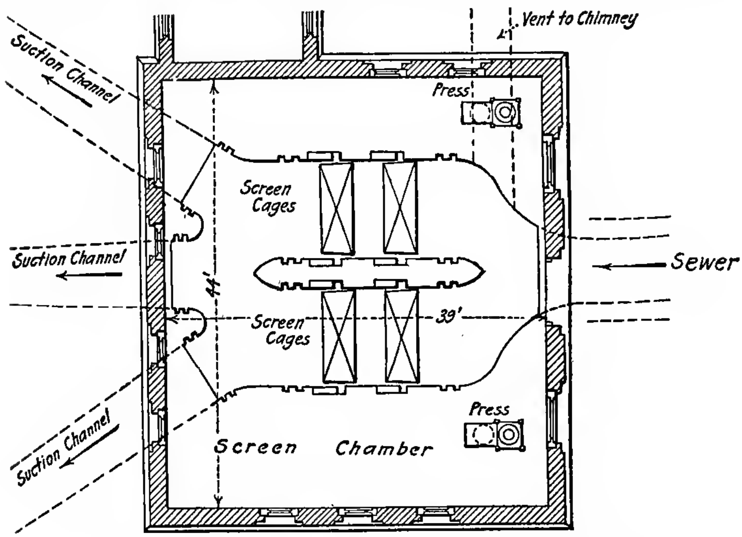


FIG. 56.—Rack chamber at Ward Street pumping station, Boston.

9 ft. high by 3 ft. 6 in. deep and is similar in appearance to a passenger elevator. After the works had been in operation for a short time and before the official acceptance tests of the pumps, the builders objected to the quantity of suspended matter coming through the racks. For this reason a second row of bars was staggered behind the first. These are placed so that the actual open space as measured diagonally between bars is still about 1 in. One of the racks at Nut Island also has a similar second row of bars. At Deer Island and East Boston, this second row of bars was omitted, as it was felt that with the centrifugal pump installations at these places such care in keeping out fine material was not necessary.

The cages are balanced by counterweights and are manipulated by small reversing engines attached to the frame supporting the cages. At the Calf Pasture station both the channels leading to the racks are provided with large hydraulic-operated sluice gates, so that either channel can be shut off, allowing but one set of racks to be used at a time, which has been the usual practice. At the Ward Street station, however, the gates are dispensed with and both channels are used regularly. Grooves are provided in the masonry for insertion of stop planks, in case it is necessary to shut off one channel. Considerable trouble

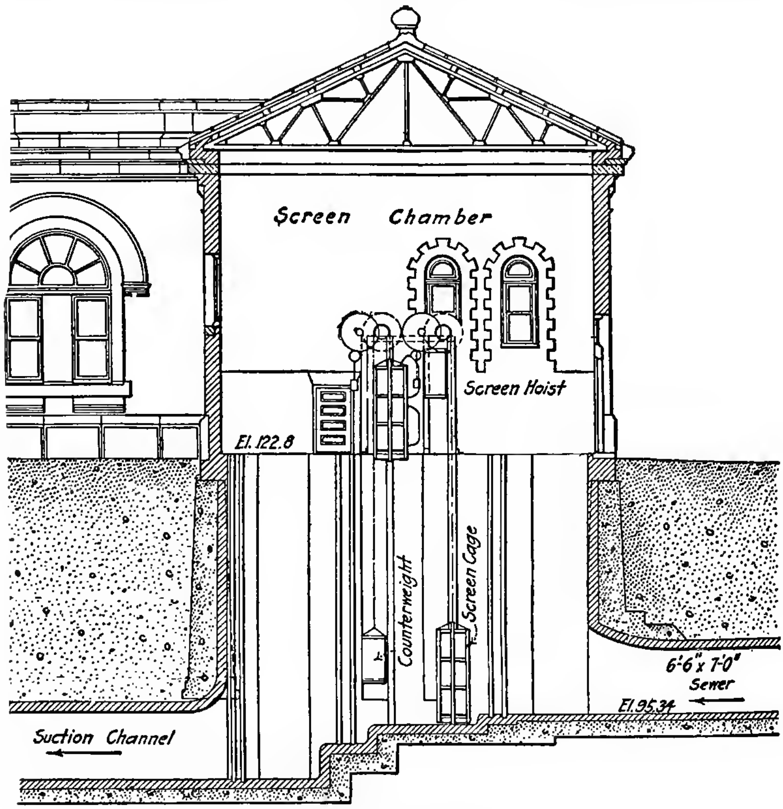


Fig. 57.—Sectional elevation of Ward Street rack chamber.

has been caused at the Calf Pasture station by debris forcing its way through the racks. This difficulty is attributed largely to the relatively small proportion of rack opening to sewer area, particularly when only one channel is being used. The debris clogs the racks rapidly and often before cleaning a head of water has been built up sufficient to force material through the bar openings. At the Calf Pasture station, with only one set of racks in use, the ratio of rack area to sewer area is 70.8 per cent and, with both sets in use, it is 141.6 per cent. At the Ward Street station the ratio of rack opening to sewer area is 162 per

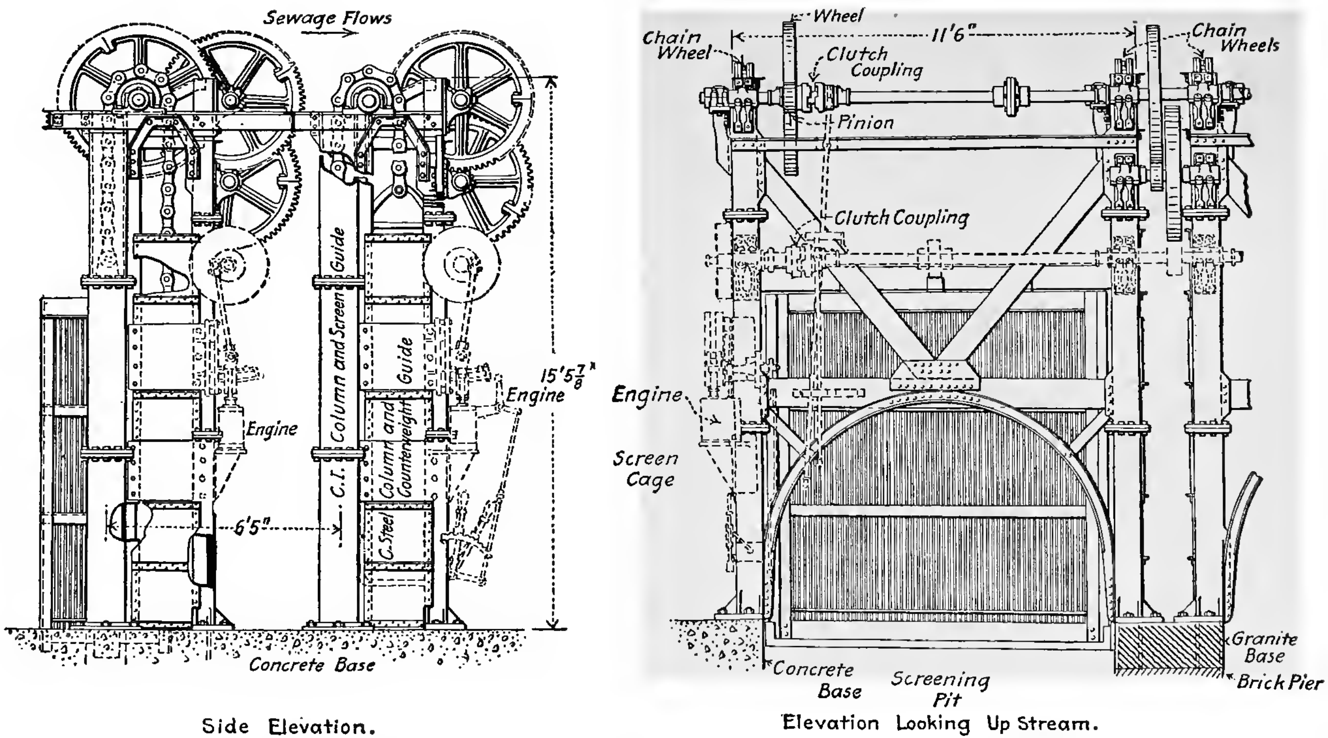
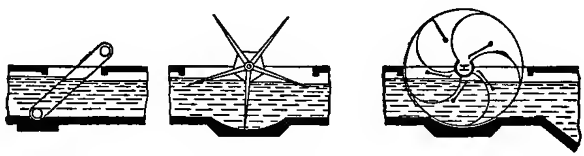


FIG. 58.—General features of rack machinery, Ward Street pumping station.

cent with one rack and 324 per cent with both racks in operation. Even under the latter conditions, it is said that in times of storm the racks need constant attendance and cleaning to keep them sufficiently free from clogging material.

A sectional elevation of the rack chamber at the Ward Street station is shown in Fig. 57. Each set of two racks is provided with two engines and by a system of clutches either or both engines may be used for hoisting either of the cages. By a system of reducing gears, the power is transmitted to chain wheels located directly over cast-iron guides on either side of the cage, as shown in Fig. 58. These wheels are rigidly attached to a horizontal shaft located over the center of the cage, thus insuring, during hoisting, a positive and equal movement for each side of the cage. There is a 2-in. clearance between the edge of the wall and the cage.



(*a*)-Band rack (*b*)-Wing rack (*c*)-Shovel-vane rack
FIG. 59.—Three types of European machine-cleaned racks.

The hoisting apparatus has been considerably simplified at the Deer Island and East Boston stations, by the substitution of a worm direct-connected to an engine or electric motor, which actuates a gear wheel fastened to the shaft carrying the chain wheels. Companion racks in the other channel have similar arrangements and the ends of the shafts are fitted with wheels for rope drive, by means of which all of the screens can be operated by one motor in case of accident to the other.

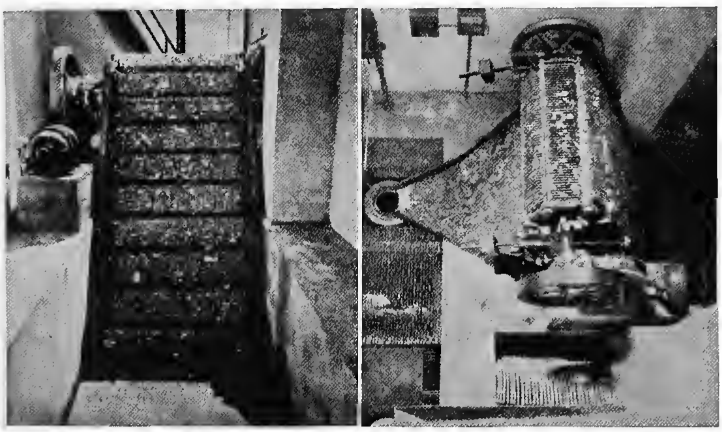
Among the installations of cage racks are those at Milwaukee, Toledo and Washington.

Machine-cleaned Movable Racks.—Three types of machine-cleaned movable racks are shown in Fig. 59, the *band* type being the only one used in this country.

A band rack of the Brunotte type (Fig. 60), furnished by the Link-Belt Co., has been installed at St. Petersburg, Fla. It is 5 ft. 10 in. wide with ¾-in. clear openings and has a capacity rating of 30 m.g.d. The speed may be either 7 or 10 ft. per minute.

The rack is cleaned by a weighted swinging rake, equipped with rubber fingers, which match the openings between rack bars. The solids removed by the fingers collect on the upper steel plate of the rake. After the bars of the tray have been cleaned, the rake is swung back and the accumulated rakings are removed by a brass scraper and dropped into a trough.

The *wing* or *Frankfort rack* is a type of movable rack developed with a view to keeping above the sewage all parts subject to frictional wear. 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. The racks are cleaned by a pendulum arm hinged at the top, which carries a brush and pushes the screenings from the inner edge of each rack toward the outer edge, finally delivering them upon a belt conveyor.



a. Front view. *b.* Top view.

FIG. 60.—Brunotte rack.

A modification of this machine, in which the racks are curved instead of flat, has been installed in a few European cities. It is called the *Geiger* or *shovel-vane* rack.

SCREENS

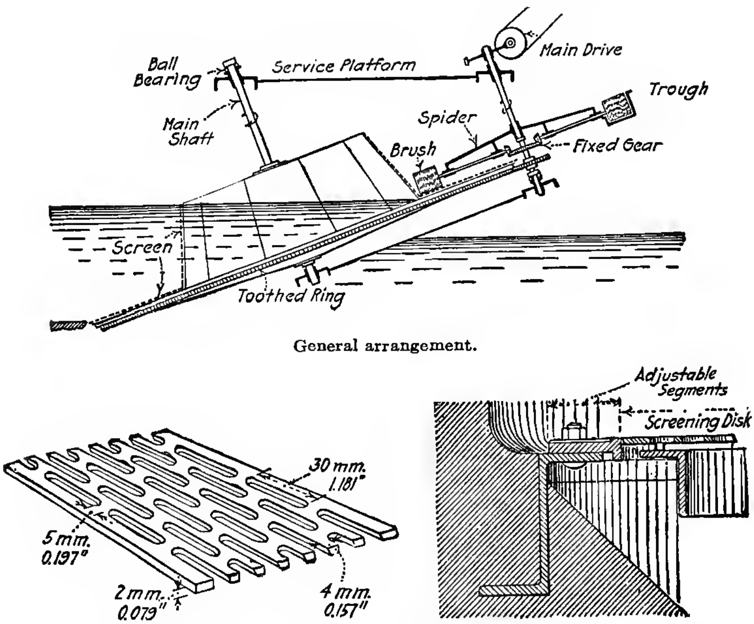
Screens are practically all of the movable type and are either self-cleaning or machine cleaned. The screening medium generally is made of manganese-bronze plates with perforations in the form of slots, commonly $\frac{1}{16}$ in. in width but ranging from $\frac{1}{32}$ to $\frac{3}{16}$ in. Wire cloth is seldom used, because of its tendency to become clogged with material which cannot be easily removed and because of its inferior wearing qualities.

Types of Screens.—The different types of screens have their distinctive features as to shape, operation, cleaning and handling of screenings. The Riensch[1]-Wurl and Tark screens are cleaned by brushes.

[1] Pronounced Reensh.

The Rex screen is cleaned by brushes, or by jets of water or compressed air. The Dorrco screen is self-cleaning. Where there is much grease in the sewage, the washing of brushes with kerosene has been found useful. The condition of the screen is also improved by this treatment or by blowing steam or hot water over the screen once or twice a day.

The head lost in screening may be appreciable. Wurl, who developed the screen known by his name, estimated the loss of head by the ordinary orifice formula, assuming, however, that a coefficient of discharge of 0.4 would be more conservative than the values ordinarily used for



General arrangement.

Screening plate.

Detail of seal.

FIG. 61.—Riensch-Wurl screen.

water and that from one-half to two-thirds of the submerged openings would be covered with screenings. For the Riensch-Wurl screen, he estimated that the area of submerged openings was equal to about 25 per cent of the submerged area of the screen. Hence from the formula:

$$Q = CA\sqrt{2gh}$$

$$h = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2 = 9.7\left(\frac{Q}{S}\right)^2$$

where Q = quantity of sewage, cubic feet per second

S = submerged surface, square feet

C = coefficient of discharge, assumed as 0.4
A = effective area of openings, square feet = say, $0.4 \times 0.25S = 0.1S$
h = head lost, feet
g = 32.2 ft. per second per second

Disk Screens.—The disk separator, or Riensch-Wurl screen, provides an adequate screening area by inclining the screening surface 10 to 25 deg. from the horizontal, as shown in Fig. 61. It was developed in Germany and has been used to some extent in the United States, notably in New York City. It consists of a disk 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 disk. One fourth to one third of the area of the disk is penetrated by perforations through which the sewage passes. The weight of the disks in screens up to 16 ft. in diameter is generally 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 either case the bearing is above the level of the sewage.

As the screenings are raised above the surface of the sewage, they are swept off by brushes on the ends of the arms of a large spider. The brushes revolve and the arms also revolve, the combined motion of the several parts being such that every portion of screening surface of the disk is passed over at least twice. This cannot be accomplished, however, with the conical screen in the center of the disk, which is cleaned by a vertical brush of cylindrical form. At some plants the cone plates are not perforated.

Screens of this type have been constructed with disks from 4 to 26 ft. in diameter and with perforations from 0.03 to 0.2 in. wide and commonly 2 in. long. The largest plants installed in this country are those of New York City.

The characteristics of the Riensch-Wurl screens at the Irondequoit plant, Rochester, are shown in Table 53.

Drum Screens.—The first drum screens used in the United States were of the Weand type, in which the sewage flowed outward through a conical drum covered with wire cloth. Installed at Reading, Brockton, and Baltimore, they have been superseded by drum screens covered with perforated metal sheets, through which the sewage passes inward. The use of the Weand screen at Reading was discontinued in 1912 and at Brockton in 1918.

The Dorrco screen, Fig. 62, was developed originally for screening industrial wastes. It consists of a rotating drum partly submerged in the sewage. The sewage passes into the drum from the outside and escapes through an opening at one end. The direction of rotation is such that the screen lifts the sewage within it on the side opposite the inlet, so that the sewage stands 6 to 8 in. above the level of the incoming sewage. This "head" automatically pushes the screenings off outside

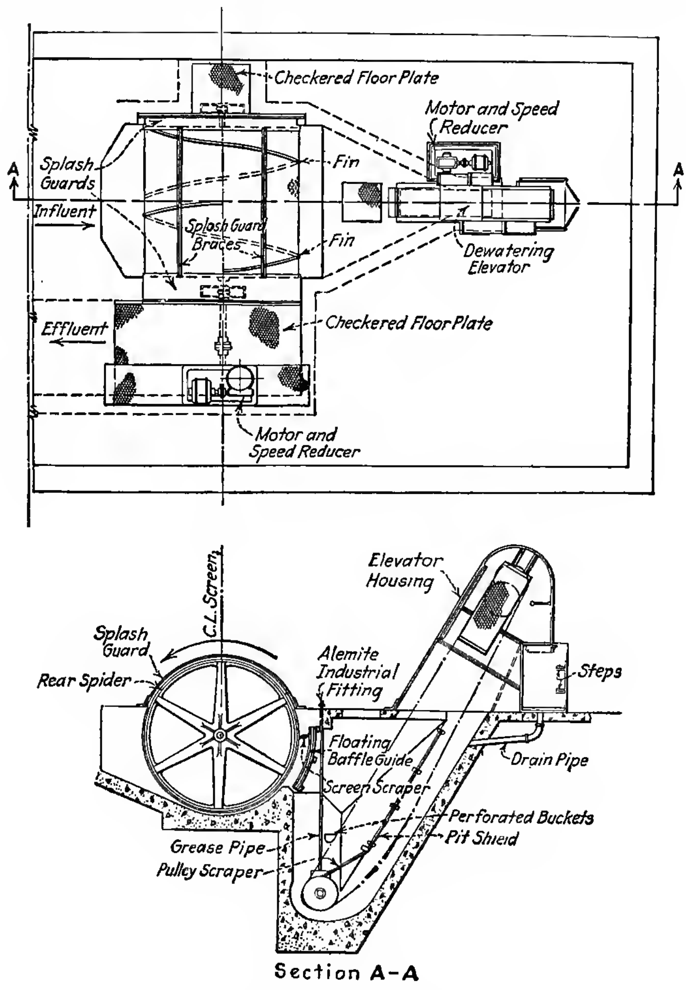


Fig. 62.—Dorrco screen.

of the plates and they settle into a screen pit adjacent to the drum. A clean surface is thus produced, to be re-immersed in the incoming sewage. This screen is therefore self-cleaning. The screenings collected in the screen pit are dredged continuously by a bucket elevator with perforated buckets.

The characteristics of the Dorrco screens at the South Hyperion plant at Los Angeles, are shown in Table 53.

In the Tark screen, manufactured by the Link-Belt Co. and shown in Fig. 63, the sewage passes from the outside through slotted plates

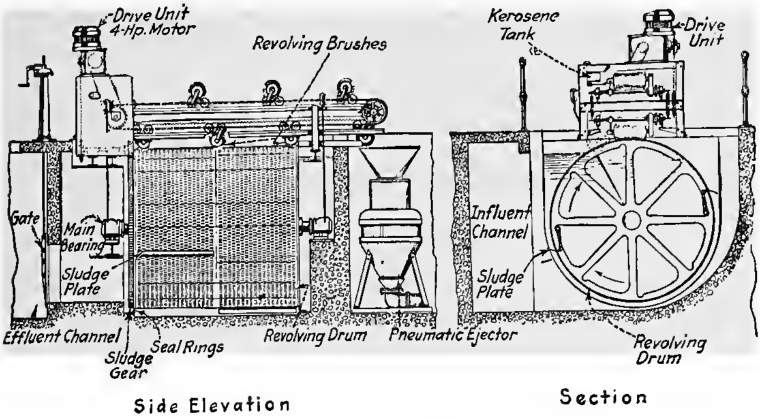


Fig. 63.—Tark screen.

into a revolving drum partly submerged in the sewage. The screened sewage escapes laterally and the screenings are swept off by brushes traveling on an endless chain over the top length of the screen. The

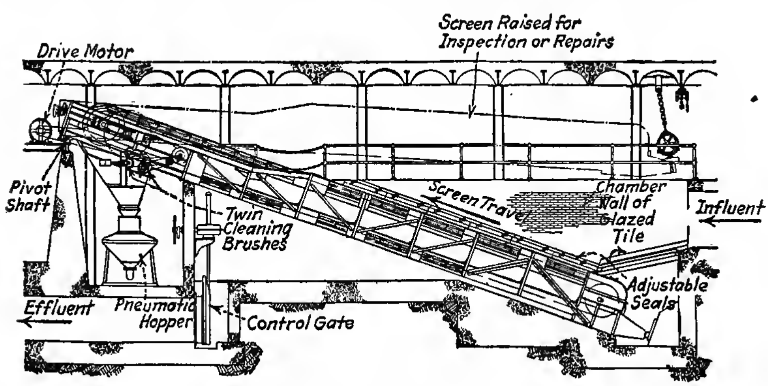


Fig. 64.—Rex screen.

brushes are supplied automatically with kerosene or other suitable liquid, to cut the grease from the screen.

Swinging plates project from the drum to scoop up any sludge that may settle in the bottom of the screen chamber. These plates are withdrawn inside the drum just before they reach the brushes.

The characteristics of the Tark screens at Milwaukee are shown in Table 53.

Band Screens.—Band screens or belt screens have been used for a long time in Great Britain. In the United States their use for municipal sewage is relatively new. The Rex screen, made by the Chain-Belt Co. and shown in Fig. 64, consists of sections of screening plates fastened to endless chains. The sewage passes through the plates which are cleaned by jets of water or compressed air, or by brushes.

The characteristics of the Rex screens at the Canal Street plant, New York City, are shown in Table 53.

TABLE 53.—DESIGN CHARACTERISTICS OF FINE SCREENS

	Rochester, N. Y., Irondequoit plant	Los Angeles, Cal., So. Hyperion plant	Milwaukee, Wis.	New York, N. Y., Canal St. plant
Type of screen	Riensch-Wurl	Dorrco	Tark	Rex
Year installed	1916–1929	1924	1925	1925
Rated capacity, m.g.d	120	80	150	120
Loss of head, in	9	3	8	
Number	6	8	8	3
Size:				
Diameter, ft	12	8	8	
Length, ft	.....	8	8	55
Width, ft	.....	.....	.....	6
Cone { Diameter, ft	6.5			
Cone { Height, ft	2.8			
Plates:				
Material	Manganese bronze	Manganese bronze	Manganese bronze	Manganese bronze
Thickness, in	3/16	3/16	3/16	3/16
Inclination, deg. from horizontal	30	.....	.....	20
Submerged area, per cent of surface	45	50	80	50
Slots:				
Size, in	1/8 × 2	1/16 × 2	3/32 × 2	3/64 × 2
Per cent of surface slotted	20.2	18.6	27.5	18.7
Operating speed { ft. per min	.....	300	5.5–11	6–12
Operating speed { r.p.m	0.7			
Brushes:				
Number	4	.....	8	2
Speed { r.p.m	38	.....	.....	11–23
Speed { ft. per min	.....	.....	100	
Power consumption, kw.-hr. per mill. gal	3	11	2.6	3.3

Efficiency of Screening.—It is not practicable to ascertain, even with approximate accuracy, the screen efficiency by determining the suspended solids in the sewage before and after screening, for most of the substances removed by screening are of such a nature that they cannot be sampled fairly by practicable methods. One way of overcoming this

TABLE 54.—EFFICIENCY OF SCREENING

	New York, N. Y., Dyckman St., 1919				Cleveland, Ohio, W. 58th St. Testing Station, 1916		Rochester, N. Y., 1928	New Britain, Conn., 1922	Pleasantville, N. J., 1921	New York, N. Y., Canal St., 1926
Type of screen	Riensch-Wurl				Riensch-Wurl		Riensch-Wurl	Dorrco	Tark.	Rex
Size of openings, in.	3/64		1/16		1/32		1/8	1/16	1/32	3/64
Time of test	Day	Night	Day	Night	Day	Night				
Crude sewage:										
Suspended solids, p.p.m.	175	48	151	55	314	150	188	153	245	226
Settling solids, p.p.m.	133	48	93	55						
Screenings:										
Suspended solids, p.p.m.	36	27	22	9	11	5	4	25	67	21
Settling solids, p.p.m.	36	27	22	9						
Moisture, per cent	78	78	82	83	87	87	92	80	.....	92
Volatile, dry basis, per cent	88	84	91	91	.....	.....	.....	....	.....	88
Volume, cu. ft. per mil. gal.	27.1	9.5	18.4	9.2	.....	.....	6.2	18.8	32.6	
Weight, lb. per mil. gal.	1,449	447	1,040	447	708	306	394	1,069	.....	2,180
Weight, lb. per cubic foot	53.4	47.0	56.5	48.6	.....	.....	59.8	56.9		
Efficiency of removal:										
Suspended solids, per cent	20.6	56.2	14.6	16.4	3.5	3.3	2.1	16.3	27.3	9.3
Settling solids, per cent	27.1	56.2	23.7	16.4						

difficulty is to measure the quantity of screenings collected and the quantity of suspended or settling matter in the screened sewage. Combining the two values will yield moderately reliable information. In recording quantities of screenings and efficiencies, it is desirable to state the dates of tests, for screens may be more effective in cold than in warm weather, because of a reduced tendency of the solids to disintegrate

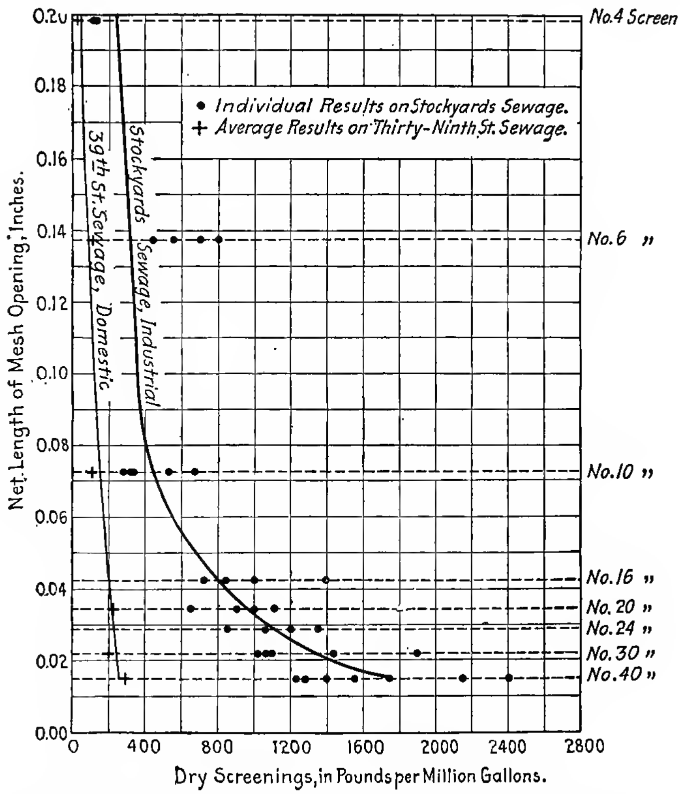


FIG. 65.—Removal of suspended matter by fine screens at Chicago.

at low temperatures, and more effective in the autumn than at other seasons, because of leaves washed into combined sewers.

Table 54 gives a few data reported for screens of various kinds.

The effect of fine screening on a weak domestic sewage and a strong industrial sewage was studied by Pearse (5) for the Sanitary District of Chicago. The results are summarized in Fig. 65. The screens were 4.2 sq. ft. in area and had openings of the sizes indicated on the diagram. At 39th Street they were under a constant head of 6 in. and at the

Stock Yards under a head of 4 ft. 6 in. It will be observed that screening was of appreciable influence only in the case of strong sewage and screens with finer mesh than about 0.08 in.

RAKINGS AND SCREENINGS

Character and Quantity of Screenings.—The character and quantity of screenings produced depend in a measure upon whether the sewers are on the separate or combined plan, on the fluctuations in sewage flow, on the slopes and lengths of sewers, on the time it takes the sewage to reach the screens, on the cleanness of the interior of the sewers, on the season of the year, on the habits of the population and on any pumping to which the sewage has been subjected prior to being screened. For any particular screen they depend especially upon the size of the openings and the frequency and manner of cleaning.

As stated by Allen (6),

> at one plant there will be found a large percentage of paper; at another fecal matter will predominate; at another vegetable wastes and at another sticks, twigs and leaves, depending on the season. With a normal domestic sewage the screenings will be composed largely of paper and feces, both of which will diminish in amount in stale sewage by disintegration.

Data on the quantity and character of material removed from racks and from screens of several kinds are given in Tables 55 and 56. The following values are common:

Quantity of Screenings:

Racks with openings of 0.5 to 2 in.: 0.5 to 6 cu. ft. per mil. gal.

Fine screens with openings of $\frac{3}{4}$ to $\frac{3}{32}$ in.: 5 to 30 cu. ft. per mil. gal.; 5 to 15 per cent removal of suspended matter.

Character of Screenings:

Weight per cubic foot: 40 to 60 lb.

Moisture: 80 to 90 per cent.

Volatile matter, dry basis: 80 to 90 per cent.

Handling and Disposal of Screenings.—The offensive character of screenings makes their prompt removal from the screens desirable. On this subject Gregory (5) has stated that the handling and removal of screenings from a screening plant are just as important as handling and removing sludge from settling tanks. His observations led him to the opinion that the simplest way to avoid offensive conditions was to keep the screenable materials in the sewage and handle them in settling tanks. Soper (5) stated that inspection of European fine screens convinced him that when they were operated without regard to the production of odors, they were the most offensive apparatus used in sewage treatment. According to Allen (6),

TABLE 55.—QUANTITY OF MATERIAL REMOVED FROM RACKS

Location	Size of opening, inches	Quantity of rakings		Density, lb. per cu. ft.
		Cu. ft. per mil. gal.	Lb. per mil. gal.	
Data from Report of Comm. on Sewage Disposal, Am. Pub. Health Assoc., 1930				
Aurora, Ill.	1¼	0.60–0.75		
Blackwell, Okla	1	1.0		
Bloomington, Ill.	1	0.75		
Canton, Ohio	1 and 2½	3.0		
Charlotte, N. C.	¾	1.7		
Chicago, Ill.				
Calumet plant	1	0.17		
Cleveland, Ohio				
Southerly plant	1½	1.08		
Westerly plant	1½ and 3	0.96		
Columbus, Ohio	½ and 1	0.67		
DeKalb, Ill.	1	4.0		
Durham, N. C.	1⅛	3.0		
Elgin, Ill.	1	0.77		
Fitchburg, Mass.	2½	0.54		
High Point, N. C.	1¼	0.9		
Marion, Ohio	1½	2.0		
Middletown, N. Y.	1¼ and 3	3.5		
Morris Plains, N. J.	2½	3.0		
Parsons, Kan.	1½	2.0		
Pontiac, Mich.	1¼	1.0		
Reading, Pa.	2	2.75		
Schenectady, N. Y.	1½	1.5		
Syracuse, N. Y.	½	0.73		
	2½	0.71		
Trenton, N. J.	1½	2.6		
Urbana-Champaign, Ill.	1 9/16	1.0		
Worcester, Mass.	2	2.37		
Data from other sources				
Atlanta, Ga.	1⅝	....	40	
Boston, Mass.				
No. Metropolitan	1	1.78		
So. Metropolitan	1	4.77		
Columbus, Ohio	⅜	4.6	300	65
Plainfield, N. J.	½	5.7	261	46
Toronto, Ont.				
Morley Ave. plant	½	2.8–3.7	127–171	38
No. Toronto plant	2	0.3	15	50
Washington, D. C.	1½	0.63	37	54

TABLE 56.—QUANTITY AND CHARACTER OF SCREENINGS

Location	Type of screen	Size of opening, in.	Quantity of screenings		Density, lb. per cu. ft.	Moisture, per cent	Volatile matter, dry basis, per cent
			Cu. ft. per mil. gal.	Lb. per mil. gal.			
Akron, Ohio	Dorrco	1/8	1.4				
Brockton, Mass.	Weand	0.0214	38	1990	52	71	
Long Beach, Cal.	Riensch-Wurl	1/32	27	1810	67	87	
	Dorrco	1/16	....	2485	...	87	
Los Angeles, Cal.							
No. Hyperion plant	Dorrco	1/16	16.5	....	....	87	
Milwaukee, Wis.	Tark	3/32	7–8	315	....	88	
New Britain, Conn.	Dorrco	1/16	18.8	1069	56.9	80	
New York, N. Y.:							
Canal St. plant	Rex	3/64	....	2180	....	92	88
Dyckman St. plant	Riensch-Wurl	3/64	27.1	1449	53.4	78	88
		1/16	18.4	1040	56.5	82	91
26th Ward plant	Riensch-Wurl	1/32	5.2	....	....	77	68
		3/64	5.7	....	....	87	94
		1/16	7.5	....	....	80	81
		5/64	8.8	....	....	82	83
Pasadena, Cal.	Dorrco	1/16	30	....	....	88	
Plainfield, N. J.	Riensch-Wurl	1/16	15.8	....	....	89	
Pleasantville, N. J.	Tark	1/32	32.6				
Reading, Pa.	Weand	0.014	25	1500	60	90	73
		1/30–1/36	31	....	....	90	
Rochester, N. Y.	Riensch-Wurl	1/8	6.2	394	59.8	92	

disposal of sewage screenings is not often a difficult problem in the case of the small installation and where there is land available where they can be buried in trenches, plowed under or merely covered, to prevent odors, fly breeding, etc.; but as the volume increases and land becomes more restricted, it presents difficulties that may be very perplexing.

Where racks are cleaned by hand, the rakings are handled with broom, shovel and wheelbarrow. Where racks are machine cleaned, rakings are either discharged directly into receptacles or conveyed to them by band or screw conveyors. Fine screens are commonly provided with receptacles into which the screenings are swept or delivered by bucket conveyors, or they are equipped with bands on which the screenings are transported to receptacles.

The screenings accumulating in the receptacles in some cases are ejected by compressed air into carts, which carry them to the point of disposal. If screenings are disposed of in the plant or its immediate vicinity, the compressed air may convey them through pipes to the point of disposal. At the North Side sewage-treatment works in Chicago the racks are provided with a pneumatic-ejector system for removing the rakings. The ejectors discharge into two steel storage tanks, each having a capacity of 1000 cu. ft., from which the rakings can be loaded into railroad cars or trucks by additional ejectors.

The prevailing methods of disposal of screenings include burying or plowing under, filling low land, incineration and disposal at sea. Prior to disposal, screenings are sometimes compressed to remove some of their moisture and thus to reduce their bulk and offensiveness.

Burial of Screenings.—If screenings are disposed of by burial, they should not be buried too deeply, as they decompose but slowly if removed from the upper layers of soil, known as the "zone of living earth," in which bacterial activity is greatest. At the same time, they should be covered sufficiently well to prevent odors and fly breeding. At Plainfield, N. J., screenings are spread upon neighboring farm lands and plowed under. At Rochester and Syracuse fine screenings are carried to a dump and covered with material removed from grit chambers.

Dewatering.—Prior to disposal, screenings are sometimes dewatered, particularly where the method of disposal is by incineration.

Aside from the draining of screenings incidental to their removal from sewage, water may be forced out of them by pressing and by centrifuging. Rakings have been pressed at a number of plants for many years. The presses have usually been of the "cider" type, operated by steam or hydraulic pressure. Screenings at Dayton are dewatered in a press made by the Hydraulic Press Mfg. Co. (Fig. 66). The press basket consists of a steel-slab curb 28 in. in diameter and 30 in. deep. The hydraulic ram has a capacity of 100 tons, making the pressure on the screenings 325 lb. per square inch. It takes about 20 min. to make one pressing. At this rate, the capacity of the press is about 250 cu. ft. of wet screenings in 8 hr. The screenings are reduced about 50 per cent in volume and after pressing contain about 35 per cent of solids, of which about 4 per cent is mineral matter.

During a test run at Baltimore, unpressed rakings averaging 7.75 cu. ft. per mil. gal. of sewage, weighing 60.17 lb. per cubic foot and containing 86.45 per cent moisture, were reduced by pressing to 4.04 cu. ft. per mil. gal., weighing 50.03 lb. per cubic foot and containing 72.19 per cent moisture.

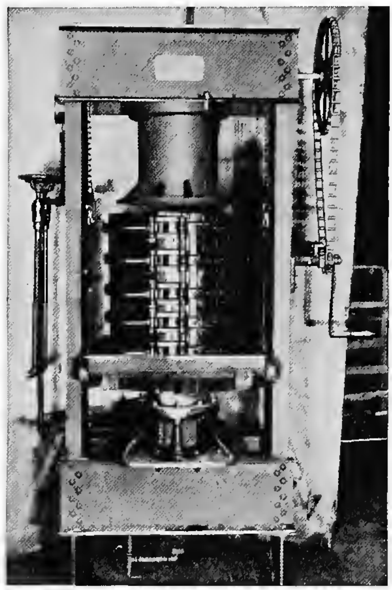


FIG. 66.—Press for dewatering screenings, made by Hydraulic Press Manufacturing Co.

The rakings at the Ward Street pumping station in Boston are dewatered to 65 to 75 per cent moisture.

It is probable that screenings can be dewatered practically by centrifuge, although this machine has been used but little. At Reading, Pa., screenings from a fine-mesh Weand screen were placed in canvas bags and dewatered by a centrifugal "hydro-extractor" about 6 ft. in diameter by 3.5 ft. high, which reduced a charge of 500 lb. from 85 to 90 per cent to 61 to 73 per cent moisture in 8 min. The dewatered screenings were burned under the boiler. At Brockton, Mass., screenings from a Weand screen weighed slightly more than 50 lb. per cubic foot after dewatering by centrifuge.

Experiments were conducted by Gascoigne at Cleveland, in which about 1¼ tons of fine screenings with 85 per cent moisture, after a brief period of draining, were dewatered in a centrifuge to 65 per cent moisture.

Following tests with a centrifugal machine of the laundry type, as manufactured by the Tolhurst Co., Milwaukee installed five machines of this type for dewatering mixed screenings and grit prior to incineration. The screenings are loaded into ejectors and forced by air pressure through pipes into the centrifuge baskets. Townsend (7) describes the installation as follows:

> The baskets of each of the centrifugal machines, which revolve at a speed of about 500 r.p.m., will hold 12 cu. ft. of wet screenings and grit.[1] After

[1] Grit is not dewatered by centrifuge at present.

the centrifuging cycle has been completed, the hinged cover of the unit is raised, and the basket is removed from the machine by means of small electric jib cranes. The basket is swung outward, then inverted, its contents falling into a hand-propelled buggy of suitable size to hold two such charges. The contents of the buggies are deposited on the floor above the incinerator near the charging holes, from which place they are fed by hand into the incinerators.

Incineration.—Dewatered screenings may be burned readily under steam boilers where available, as is done at some Boston plants. It is practicable also to dispose of such screenings by incineration where high-temperature incinerators are available.

Although a few incinerators have been installed in treatment plants, comparatively few data relative to the incineration of screenings are available. Such information is rather conflicting and unsatisfactory. At Long Beach, Cal., screenings are burned in an incinerator constructed at the plant (8). During the fiscal year ended June 30, 1926, a total of 80,220 cu. ft. of screenings, weighing 2507 tons, was burned. The ash removed from the incinerator, which was used for filling low land around

TABLE 57.—FUEL USED FOR INCINERATION OF SCREENINGS

Plant	Moisture in screenings, per cent	Fuel used to incinerate 1 lb. of screenings[1]	Assumed B.t.u. per unit of fuel	Estimated B.t.u. supplied per lb. of screenings[1] incinerated	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
Baltimore, Md.......	72	0.22 lb. coal	14,000	3080	Test run; screenings pressed.
Cleveland, Ohio......	65	0.25 lb. coal	14,000	3500	Experiments; screenings dewatered in centrifuge.
Dayton, Ohio........	70	1.98 cu. ft. Imhoff gas	700	1390	Year 1932; screenings pressed.
Long Beach, Cal.....	85–88	1.7 cu. ft. natural gas	1,150	1940	Year ended June 30, 1926; screenings partially dried.
Los Angeles, Cal., Hyperion plant.	68	0.0106 gal. fuel oil	145,200	1540	Year ending Jan. 31, 1935; screenings pressed.
New York, N. Y.....	69	0.13 lb. oil	19,000	2470	Test run of 3 hr.; screenings pressed.
South Yonkers, N. Y..	80	0.0125 gal. fuel oil	147,000	1840	Representative tests over year's time; screenings pressed.

[1] Moisture in screenings as indicated in column 2.

the plant, weighed 29 tons, or slightly more than 1 per cent of the screenings burned.

At Dayton pressed screenings, containing about 96 per cent volatile matter on a dry basis, are burned in an incinerator built at the treatment plant. They are reduced about 95 per cent in weight. The incinerator is fired by gas collected from the Imhoff tanks and stored in large pressure tanks underground. A similar screenings press and an incinerator are provided at the Allentown, Pa., sewage-treatment plant.

Data available upon incineration of screenings are given in Table 57.

The general opinion seems to be that a temperature of 1250°F. is the minimum at which screenings can be incinerated without producing objectionable odors. Average temperatures at Dayton are about 1600°F. Of 8760 temperature readings taken inside the combustion chamber of the Long Beach incinerator, the highest was 1690°F. and the lowest 1561°F. The available data indicate that 1500 to 3500 B.t.u. are required to incinerate 1 lb. of dewatered screenings.

Digestion of Screenings.—The digestion of screenings alone is seldom practiced. Large-scale experiments at Milwaukee on digestion of fine screenings alone are reported upon by Townsend (7) as follows:

> After a period approximating 1½ years, during which time a wide range of temperature and pH control were investigated and also during which time many changes in mechanical equipment and various other appurtenances were made, the experiment was abandoned, it having been concluded as far as the work had progressed that its application upon a large scale appeared to be impracticable.

In experiments upon the digestion of mixtures of screenings and activated sludge, it was found that while suitably seeded activated sludge digested alone at 70°F. in 30 days, replacement of 10 and 20 per cent of the sludge by screenings increased the digestion period to 35 and 40 days, respectively.

At Baltimore, for several years, rakings have been ground and returned to the sewage for digestion with primary sludge. Since the material removed from sewage by racks is a relatively small proportion of the material removed in sedimentation and activated-sludge units, it is probable that the rakings may be readily disposed of by digestion with the sludge. Laboratory experiments reported upon by Keefer and Kratz (9) indicate that garbage, which generally constitutes a considerable proportion of rakings, after being ground, digests rapidly with an equal quantity of raw sludge on a volatile-solids basis, when seeded with a suitable quantity of digested sewage sludge and kept at 28°C.

Boruff and Buswell (10) state that a special, drum-type screenings digester has been designed and operated, which has been found to overcome the difficulties encountered in other experiments with the digestion of rakings and screenings alone and to produce a nonoffensive material, which can be dried readily on sand beds. The normal operating capacity is reported to be 200 lb. dry weight daily per 1000 cu. ft. of tank.

During investigations made by Rawn (11) and by Rudolfs and Heisig (12), they were able to treat screenings by digestion at daily rates of only 112 lb. and 41 lb. dry weight, respectively, per 1000 cu. ft. of tank capacity.

Shredding Screenings.—At Baltimore, where racks are employed to protect pumping equipment, tests have been made on the grinding of sewage rakings in a macerator (13). After being ground, the material was returned to the sewage.

A 24- by 28-in. macerator was found to be capable of shredding 1.5 to 2 tons of rakings an hour, with a current consumption of about ¼ kwh. per cubic foot of ground material. The tests indicated that under certain conditions the rakings could be shredded, returned to the sewage, digested with the other solids removed from it and dried on sludge beds at a cost which would compare favorably with the cost of pressing and incinerating them.

Keefer further reported that immediately after the conclusion of the tests the macerator was permanently installed at the Eastern Avenue Sewage Pumping Station and since then has been used to grind all the rakings collected at this station (14). The daily quantity of material pulverized varies from 200 to 300 cu. ft. During 3½ years of successful service it was necessary to install one new set of hammers in the grinder; other than this, practically no repairs were required. The grinder pulverizes practically everything that is present in sewage. However, in pulverizing rags it is necessary to introduce the material into the machine at a slow rate. After this thorough test it was decided to install two machines for grinding rakings from coarse bars at the Baltimore sewage-treatment plant.

At sewage-treatment works, where it is desired to remove from the sewage as much coarse material as possible, partly to lighten the load on subsequent treatment units, the addition of macerated screenings to the sewage would defeat the purpose for which the screens were installed. Moreover, the possibility of increasing the scum on sedimentation and digestion tanks requires consideration.

Cost of Rack and Screen Installations.—The cost of hand-cleaned racks, which are commonly built up of simple structural-steel members, principally flats and rounds, is about 10 cents a pound. The cost of the machine-cleaning equipment at Syracuse, N. Y., in 1923 was about $18,200, or $330 per m.g.d. based on maximum plant capacity. The tenders received by the authors in 1929 for machine-cleaning equipment for a rack, 6 ft. wide by 11 ft. deep, ranged from $2500 to $3500, including installation. The cost of the Aurora, Ill., Dorr machine-cleaning equipment in a channel 5 ft. wide and 4 ft. deep was about $2500, not including installation. The cost of the Brunotte band rack at St. Petersburg, Fla., in 1927 was about $10,000, or about $333 per m.g.d. rated capacity.

The power required for operating mechanical rack equipment is relatively small, varying according to the size and number of racks and the frequency of operating the cleaning equipment. The cost of power at Trenton, N. J., for operating Dorr machine-cleaning equipment for a rack 8 ft. wide and 6 ft. deep is not more than 30 cents a day. The average cost of operating the racks at Baltimore, Md., for 8 years, 1921 to 1928, was 35 cents per mil. gal. of sewage treated.

Fine-screen costs vary with the size and number of units and range from $4500 for a small screen, taking care of flows up to 1 m.g.d., to $25,000 for a screen treating flows up to 35 m.g.d. The cost of the two Dorrco screens at Akron, Ohio, having a combined capacity of 10.4 m.g.d., was about $16,000, including erection, in 1927. The cost including erection of the Tark screen at Tenafly, N. J., having a maximum capacity of 3 m.g.d., was about $7000 in 1928. The cost in 1925 of the Milwaukee, Wis., Tark screens, having a maximum capacity of 150 m.g.d., was about $114,000 erected.

A hydraulic-screenings press, such as used at Allentown, Pa., and Dayton, Ohio, with a basket 28 in. in diameter and 30 in. deep, costs about $2500 installed. A machine for macerating screenings, like that in use at Baltimore, capable of shredding 1.5 to 2 tons of screenings per hour with a current consumption of ¼ kwh. per cubic foot of ground material, costs about $600. At 1.5 cents per kilowatt-hour the cost for power is about 37.5 cents per 100 cu. ft. of ground material. The cost of incineration with coal, assuming 0.25 lb. of coal per pound of screenings, with a density of 60 lb. per cubic foot, is $6 per 100 cu. ft. of screenings, with the price of coal at $8 a ton.

Operating costs of screens include supervision, renewals and repairs and power. At Baltimore, four revolving-drum screens, 10 ft. long and 12 ft. in diameter, are placed between the sedimentation tanks and trickling filters. The average cost of operating these screens for eight years, 1921 to 1928, was 54 cents per mil. gal. of sewage treated. The total operation and maintenance cost per million gallons of sewage treated at the Long Beach, Cal., Dorrco-screen plant, including the incineration of screenings, was $6.60 during the fiscal year ended June 30, 1926. The cost of incineration per ton of wet screenings was $0.69 for fuel and $0.60 for labor, a total of $1.29. The cost of fuel was based on an allowance of 20 cents per 1000 cu. ft. of gas, at 1150 B.t.u. per cubic foot, the gas being secured without charge from the municipal gas plant.

Bibliography

1. Manuals of Engineering Practice, A. S. C. E., No. 2, 1928.
2. Fuller, G. W., and J. R. McClintock: "Solving Sewage Problems," 1927.
3. Frühling, August: "Die Entwässerung der Städte," 1910.
4. McConnell, E. G.: *Public Works*, 1928; **59**, 268.
5. Allen, Kenneth: *Trans.*, A. S. C. E., 1915; **78**, 880.
6. Allen, Kenneth: *Canadian Eng.*, 1929; **57**, 750.
7. Townsend, D. W.: *Water Works and Sewerage*, 1933; **80**, 317.
8. Appleton, R. A.: *Eng. News-Rec.*, 1927; **99**, 500.
9. Keefer, C. E., and Hermann Kratz, Jr.: *Sewage Works Jour.*, 1934; **6**, 250.
10. Boruff, C. S., and A. M. Buswell: *Sewage Works Jour.*, 1932; **4**, 973.
11. Rawn, A. M.: *Sewage Works Jour.*, 1932; **4**, 982.
12. Rudolfs, Willem, and H. M. Heisig: *Sewage Works Jour.*, 1929; **1**, 519.
13. Keefer, C. E.: *Proc.*, A. S. C. E., 1929; **55**, 1759.
14. Keefer, C. E.: *Proc.*, A. S. C. E., 1930; **56**, 1021.

CHAPTER XI

GRIT CHAMBERS, SKIMMING TANKS AND STORM-WATER TANKS

Grit chambers and skimming tanks perform an essentially similar function, namely, the separation from sewage of readily removable matters that may interfere with the operation of sewage works or affect adversely streams or other bodies of water into which otherwise untreated sewage is discharged. Skimming-detritus tanks accomplish in part the work of both grit chambers and skimming tanks. All these types of preliminary treatment are finding increased use in American plants. The use of storm-water tanks for partial clarification of storm flows in excess of those that can readily be handled in the treatment plant proper is not common in this country. Such tanks are widely used in British and German practice.

GRIT CHAMBERS

Grit chambers are essential in combined sewerage systems, because of the large amount of heavy material washed into such systems during storm runoff. In sizable sewage-treatment plants, moreover, grit chambers may be of sufficient value to justify their inclusion in the works even when they serve separate sewerage systems. Appreciable quantities of grit may find their way into separate sewers from many sources, such as the waste water from bath-rooms and kitchens, the washing of floors and cellars, silt which enters sewers through openings and loose joints, illicit ground-water or storm-water connections, surreptitious drainage of trenches and other excavations, and wastes from manufacturing processes. The inclusion of excessive amounts of organic solids in grit-chamber deposits should be avoided as far as possible, because they render the grit offensive and make its disposal more difficult.

In combined systems of sewerage, the need for and load on grit chambers are greatly influenced by the character of the tributary drainage area, particularly by the extent to which streets are paved. Even with paved streets, however, the burden of grit may be considerable, because of the practice of sanding pavements that are made slippery by moisture, snow or ice and also because of the quantity of dirt washed into sewers when paved streets are cleaned. Catch basins installed on combined systems do not obviate the necessity for grit chambers but do reduce the load placed on them.

The extent to which storms affect the quantity of solids in sewage which can be removed by grit chambers was indicated by testimony before the Royal Commission on Sewage Disposal (1). During a 4-hr. storm at Burnley, for example, Pickles and Ross reported that during the first, second, third and fourth hours of the rainfall the suspended matter averaged 248, 800, 256 and 56 p.p.m. In dry weather, on the other hand, the suspended matter averaged 1190 p.p.m. At Heywood, the suspended matter during the first rush of a protracted storm was reported by Bolton to be 2380 p.p.m. and at the ends of successive hours it was: first hour, 1110; second hour, 690; third hour, 500; fourth hour, 380; fifth hour, 330; sixth hour, 280; seventh hour, 180. In dry weather the suspended matter averaged 290 p.p.m.

Design of Grit Chambers.—Before the design of grit chambers is undertaken, it is well to fix upon the desired degree of removal of heavy material from the sewage and freedom from putrescible organic matter in the grit. Removal of a large percentage of heavy solids is not always compatible with relative freedom from organic matter and it is necessary to weigh the factors involved before reaching a conclusion. In some instances, it may be less objectionable to allow some of the fine grit to pass through the grit chambers and be deposited elsewhere in the plant, whence eventually it must be removed, than to include in the grit-chamber deposits quantities of organic matter that would later cause annoyance. In other cases, it may be essential to retain as much fine grit as practicable in the grit chambers, because of its interference with satisfactory operation of succeeding processes. Since the removal of fine grit from the sewage is commonly associated with the inclusion of relatively large quantities of organic solids in the grit, special provisions may be required for the satisfactory disposal of the putrescible grit obtained in such cases.

Where sewage-treatment works are remote from human habitations and factories, the necessity of obtaining clean grit is not so great as where residences adjoin the plant or disposal of grit on dumps without creating annoyance becomes a problem.

At Cleveland, extensive experiments on developing a working grit chamber, especially one that would produce a grit suitable for disposal as fill in residential districts, led to the conclusion that, if the grit were to be relatively inoffensive, it must not contain on an average more than 15 per cent of volatile matter (2). To obtain this result, the grit-chamber velocity had to be maintained between 0.5 and 1.0 ft. per second, with an average detention period of 50 sec.

The selective efficiency of grit chambers is usually expressed in terms of the percentage of volatile matter in the grit. This figure may be misleading where materials such as coal and cinders, which are nonputrescible, and seeds, which are relatively nonputrescible, make up a large part of the volatile matter.

The factors to be considered in grit-chamber design are: velocity of flow and maintenance of uniform velocities with varying rates of sewage flow; detention period; shape of chamber; grit storage; and method of cleaning.

Velocity of Flow.—It is commonly assumed that a velocity of 1 ft. per second will permit the deposition of the bulk of the heavier mineral solids without including so much organic matter as to render the grit offensive. A range of 0.5 to 1.0 ft. per second generally is employed. The velocity fixes the cross-sectional area of the chambers. In order to keep the velocity within desired limits, it is usually necessary to provide two or more independent channels, to allow for fluctuations in sewage flow. A separate channel for the dry-weather flow is frequently considered desirable.

Other methods of controlling the velocity of flow in grit chambers are: the variation of the width of the chambers at different depths; the provision of either adjustable or fixed outlet weirs, which are designed to increase the depth of flow as the rate of sewage flow increases; and the control of the rate of pumping, in cases where sewage is lifted to the treatment works.

Gascoigne (2) concluded "that hopper-bottom grit chambers lend themselves more readily toward producing in practice the theoretical velocities used in design." He pointed out the difficulty of maintaining theoretical velocities in grit chambers of rectangular cross section, where the effective cross section changes as the deposits increase.

Influent and effluent conduits should be so designed that the sewage enters and leaves the chambers with a minimum of turbulence, while maintaining self-cleaning velocities. If it is desired to obtain in a grit chamber certain limiting velocities for a rather definite period of time, the entire length and width of the chamber should be made available. To accomplish this, it is commonly necessary to break up the rapid thread of the entering stream and cause an even distribution of flow through the chamber by flaring the inlet or providing baffles or other stilling devices. The type and location of the baffle are found best by trial. At the Cleveland demonstration plant good results were obtained by placing a set of bar gratings at the head of each chamber. This reduced the required length of inlet channels and caused an even distribution of sewage as it entered the grit chambers.

At Albany, N. Y., the velocity of flow through a grit chamber, which was built in 1915, is controlled by a proportional weir at the outlet end. The fixed opening above the crest of the weir is shaped in such a way that the discharge is in direct proportion to the head on the weir and the depth of flow above the level of the crest is proportional to the quantity entering the chamber. By this means the velocity of flow through the grit chamber is kept uniform, regardless of the quantity of sewage handled. Outlets of a similar type, illustrated in Fig. 67, have been recently installed in grit chambers at Springfield, Ill., and at the North Toronto sewage-treatment plant in Toronto, Ont.

Skinner (2) found that the deposits in the grit chamber at the Irondequoit plant in Rochester commenced 15 or 20 ft. downstream from the inlet. He suggested that a 12-in. plank baffle, inclined downstream, be set across the bottom of the chamber at its upstream end, in order to deflect the flow upward, thus obviating the scour on the bottom. This baffle, when installed, apparently added about 50 per cent to the efficiency of the grit chamber.

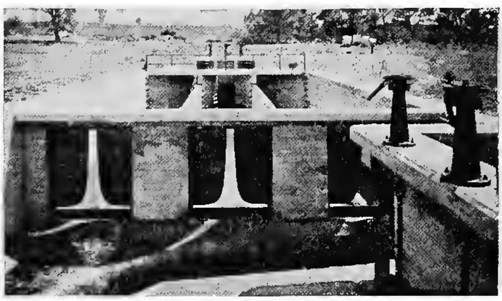


Fig. 67.—Outlet end of grit chambers at sewage-treatment plant, Springfield, Ill., showing proportional weirs.

Where the underflow from detritus tanks is removed at a constant rate to supplementary grit chambers for classification of the materials, as is done at Akron and Dayton, Ohio, and the West Side plant in Chicago, Ill., the design factors of the grit chambers become more definite.

Detention Period.—The period of detention commonly employed is 1 min. This fixes the length of the chambers and since, in accordance with Hazen's theory of sedimentation, discussed in Chap. XII, the ratio of length to depth must not be too great, the effective depth is delimited at the same time. Since sedimentation of granular solids is dependent to a large extent upon the surface area of the chambers, their width should be made as great as possible, while remaining consistent with other requirements. The separate units must not be made too wide for convenient cleaning or for the maintenance of a uniform velocity in the cross section.

Grit Storage.—Storage space for grit is provided either throughout the length of the chambers or by means of one or more pits deeper than the remainder of the basins. Observations on different types of chambers at the Cleveland demonstration plant and studies of the quality of grit collected by them led to the acceptance of hopper-bottom grit chambers for subsequent Cleveland installations. Flat-bottom chambers are simpler in design and less expensive than the hopper-bottom type. In large installations, where mechanical equipment for cleaning is justified, chambers including hoppers or sumps may be more advantageous than flat-bottom channels.

When allowance is made for grit storage, some uncertainty is introduced in estimating the velocity of flow through the chambers. Experiments have indicated the frequent existence, at or near the invert level of the influent conduits, of a plane below which stagnation occurs. In estimating velocities, some movement below this level should be provided for, if the velocity of flow through the chamber is to be high enough to prevent the settling of organic solids when the grit-storage space is clean. Concentration of grit storage in pits may be useful in this connection, as well as for cleaning purposes. Reduction of the cross section below the invert elevation and separation of the flowing-through section from the grit-collecting section by means similar to those employed in Imhoff tanks have also been resorted to.

Cleaning Grit Chambers.—In addition to giving the grit chamber a cross section which will result in suitable velocities, the designer must adopt a section which can be easily cleaned. Cleaning is facilitated by rounding the intersections of the walls and bottoms and avoiding all angles in which sludge can collect. Where the grit is to be removed by shovel, the bottom of the pit receiving it may be horizontal, but if mechanical cleaning is to be practiced, the bottom should slope to a valley or a hopper. In some of the early trough and hopper bottoms, the slopes were too flat for the grit to slide down them and there is some evidence to the effect that the inclination should be at least 45 deg.

There are a number of ways of cleaning grit chambers, the choice of cleaning devices depending largely upon the size of the plant and the frequency of cleaning required. The methods may be classed as hand cleaning, mechanical cleaning and hydraulic cleaning.

Hand Cleaning.—At small plants hand cleaning is commonly employed, the grit being shoveled into containers varying in size from buckets to industrial railway cars. The containers are lifted out of the chambers by hand hoists or by power-driven stationary or movable cranes. Hand cleaning requires the unwatering of the chambers. This is commonly accomplished either by gravity or by fixed or portable diaphragm or centrifugal pumps.

Mechanical Cleaning.—Devices for mechanical cleaning of grit chambers take the form of bucket elevators, scrapers and clamshell buckets on mounts of various types. These permit cleaning without unwatering and during operation, although chambers are often unwatered and by-passed during mechanical cleaning, where this is not strictly necessary. Where basins are cleaned by clamshell buckets, it may be desirable to prevent injury to the floors by protecting them with some hard covering, such as rails, granite-block pavement or steel

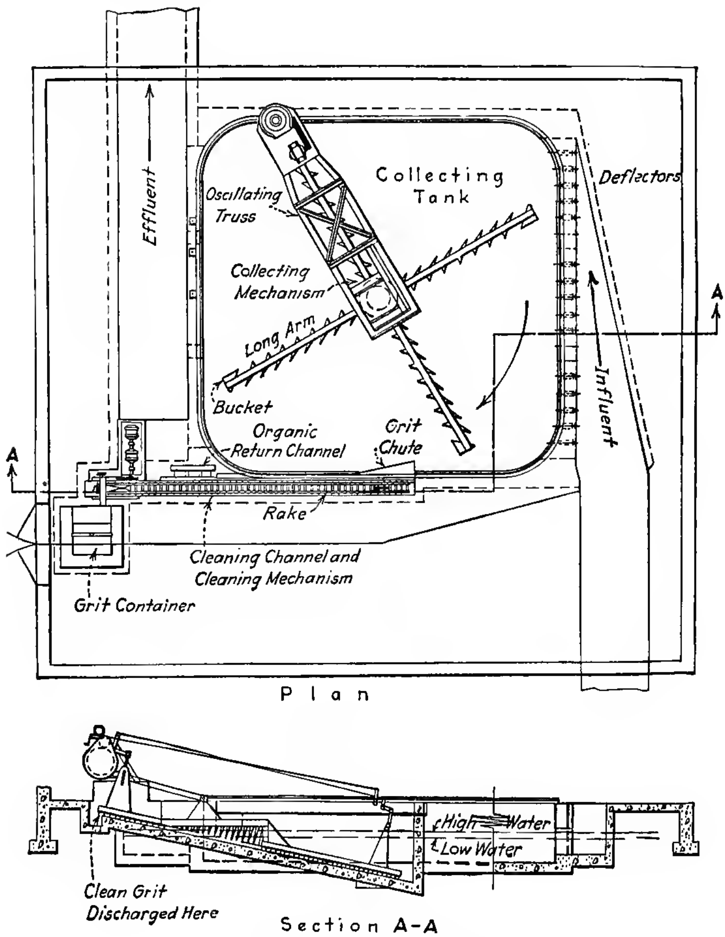


FIG. 68.—Dorr detritor.

plates. Continuous cleaning by mechanical devices, notably scrapers, is coming into use.

The Dorr "detritor," shown in Fig. 68, is designed as a square basin with rounded corners and equipped with a machine for the collection and removal of grit. Plows, mounted on four radial revolving arms, carry the grit to an inclined channel along one side of the basin. A series of reciprocating rakes move the grit up the inclined floor to the discharge point at the upper end. While passing up the incline, organic solids are separated from grit and carried back into the basin through a special opening. By this method a cleaner, dryer grit is said to be obtained than is possible by the ordinary methods of cleaning. Two units 55 ft. square, with a combined capacity of 150 m.g.d., have been provided at Providence, R. I. Two units at Erie, Pa., each 30 ft. square, are designed for a total flow of 100 m.g.d.

The Link-Belt grit collector and washer is shown in Fig. 69. The conventional rectangular shape of the grit chamber is retained and designed for a maximum velocity of 1 ft. per second. Collectors move the settled material from one end of the tank to the other, the flights,

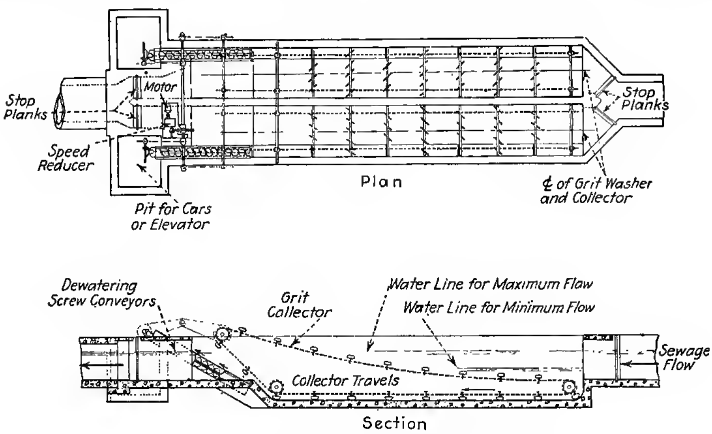


FIG. 69.—Link-Belt grit collector.

set at an angle to the direction of travel, pushing the detritus to one side of the tank and discharging it into a screw conveyor. The detritus is elevated and discharged into hoppers or containers. It is said that the heavy organic matter which settles with the grit is separated by the washing action of the screw and goes out with the effluent. Two units with a combined capacity of 75 m.g.d. are provided at Peoria, Ill., and four units with a total capacity of 172.5 m.g.d. have been installed at the joint sewerage works for twelve municipalities in Elizabeth, N. J.

Hydraulic Cleaning.—The principal devices for hydraulic cleaning of grit chambers are eductors, sand dredges and monitors. The first two devices lift the grit out of the chambers as a watery slush. Eductors are commonly connected to pits, while sand dredges may travel the length of the chambers. Monitors are fire nozzles that direct a stream of water from some central point into the chamber and flush the deposits out through pipes in the sidewalls or bottom of the chamber. They are particularly applicable to side-hill locations and one has been used in a grit chamber thus located at Fitchburg, Mass.

Sand ejectors were tried out at the Cleveland demonstration plant, but it was found that grit containing tar products and stones that varied in size from 0.5 to 1.5 in. could not be removed with the ejector installed. It was concluded that, if sand ejectors are to be used satisfactorily, the sewage must be fine-screened before entering the grit chamber.

Theorectical Study of Grit-chamber Performance.—As suggested by Skinner, the design of grit chambers can be studied on the basis of Hazen's theory of sedimentation (2). The latter is outlined in Chap. XII. To illustrate this approach, let it be required to investigate

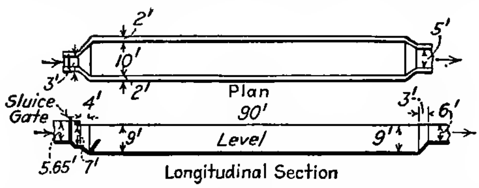


FIG. 70.—Grit chamber at Irondequoit plant, Rochester sewage works. (*After Skinner.*)

the theoretical performance of the grit chambers at the Irondequoit plant of the Rochester sewerage system. A plan and a longitudinal section of one of the six chambers are shown in Fig. 70.

If a velocity of 1 ft. per second, a freeboard of 0.5 ft. and a deposit of grit of 3 ft. are assumed, the characteristics of each chamber are as follows:

Area $b = 900$ sq. ft.

Effective depth $h = 5.5$ ft.

Length $l = 90$ ft.

Ratio of length to depth $\frac{l}{h} = \frac{90}{5.5} = 16.4$

Mean velocity of flow $V = 1$ ft. per second

Detention period $a = 90/1 = 90$ sec.

Rate of sewage flow $e = 1 \times 5.5 \times 10 = 55$ c.f.s.

$$\frac{a}{t} = \frac{bv}{e} = \frac{900 \times v}{55 \times 304.8^{1}} = 0.0537v$$

Since this is a continuous basin without baffling, the highest line in Fig. 77 applies. If the values of a/t corresponding to certain percentages of removal of suspended matter are taken from the curve or computed from the formula $x = \frac{1}{1 + a/t}$, the findings can be arranged in tabular form as below. Results of computations for $v = \frac{1}{0.0537}\frac{a}{t}$ are given in Column 3 and

[1] To convert to millimeters.

the diameters of particles removed, corresponding to the various hydraulic subsiding values as taken from Fig. 76, are shown in Columns 4 and 5.

Suspended matter removed, per cent	a/t	Hydraulic subsiding value v, mm. per sec.	Diameter of particle d, mm.		Suspended matter remaining, per cent
			Mineral, sp. gr. 2.65	Organic, sp. gr. 1.2	
10	0.11	2.0	0.05	0.2	90
20	0.25	4.7	0.08	0.4	80
30	0.43	8.0	0.12	0.7	70
40	0.67	12.5	0.14	1.2	60
50	1.00	18.6	0.18	2	50
60	1.50	28.0	0.27	4	40
70	2.33	43.4	0.43	>10	30
75	3.00	55.9	0.56	......	25
80	4.00	74.5	0.75	......	20
85	5.67	106	1.1	......	15
90	9.00	168	2.2	......	10
95	19.00	354	>10	......	5
99	99.00	1,850	....	......	1

If it is assumed that the theoretical results measure conditions with a fair degree of accuracy, the differential action of a grit chamber of this type is quite apparent. While more than 50 per cent of the mineral matter 0.2 mm. in diameter is removed, only 10 per cent of the organic matter of this size is deposited. While 90 per cent of the mineral matter 2 mm. in diameter settles, only 50 per cent of the organic matter of this size is removed.

Instead of approaching the problem in this way, it is possible to develop approximate formulas for use. From the equation $a/t = bv/e$ the following formulas are obtained by expressing v in terms of d, as on page 311:

1. For particles smaller than about 0.1 mm.

(a) Specific gravity 2.65; $d = 0.665\left(\frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}\right)^{1/2}$

(b) Specific gravity 1.2; $d = 1.90\left(\frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}\right)^{1/2}$

2. For particles between 0.1 and 1 mm.

(a) Specific gravity 2.65; $d = 3.05 \times \frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}$

(b) Specific gravity 1.2; $d = 25.2 \times \frac{a}{t} \times \frac{e}{b} \times \frac{60}{T + 10}$

In these formulas e/b represents the cubic foot per second of sewage treated per square foot of surface and the values of a/t for each type of sedimentation unit must be taken from the curves in Fig. 77 or from the corresponding equations.

Vertical-flow Grit Chambers.—Blunk (3) has designed a vertical-flow grit chamber for the Bochum plant in Germany, in which the sewage is led through a vertical pipe down into a tank and then flows vertically upward through one or more chambers, according to the variation of flow. From experiments, Blunk has found that the greatest settling velocity of individual particles of organic matter lies between 0.098 and 0.131 ft. per second and that, if the water in a grit chamber has an upward velocity of about 0.197 ft. per second, it is reasonable to expect that organic matter will pass through the grit chamber but sand of a size greater than 0.28 mm. will be retained, for the most part. The Bochum installation is designed for a minimum flow of 13.8 c.f.s. and a maximum of 35.3 c.f.s. The chamber is 9 ft. 8 in. in diameter and 33 ft. 7 in. deep, from the top of the wall to the bottom of the grit-collection pit, which has a storage capacity of 600 cu. ft.

Character and Quantity of Grit.—As determined by the loss-on-ignition test, the proportion of organic matter in grit may vary from 10 to 50 per cent. The higher values are commonly associated, however, with too low velocities. At Syracuse and Cleveland, grit containing 10 to 15 per cent volatile matter is said to be inoffensive. As previously stated, much of the volatile matter, such as coal, cinders and seeds, is nonputrescible or nearly so. Inclusion of such solids in grit may not be objectionable. From a sieve analysis of grit from the Irondequoit plant at Rochester, Skinner (2) concluded that

> Seeds, together with particles of coal, are the principal objects which give loss-on-ignition. Neither the seeds nor the particles of coal pass the 20-mesh sieve [size of opening, 0.83 mm. or 1/32 in.], which fact readily accounts for the 37 per cent loss on ignition above this size and only 15 per cent below it.

He stated further that the specific gravity of the finer material was 2.23, approaching that of silica, whereas that held on the 20-mesh sieve was 1.45.

Methods of determining how much of the volatile matter is putrescible have been described by Fischer (4). Working on grit from Salem, Ohio, Fischer found that, whereas the volatile matter in nine samples of grit varied from 8.18 to 46.30 per cent, the putrescible matter amounted to only 3.07 to 4.31 per cent.

The specific gravity of grit is about 2.0 and it contains about 35 per cent void space. Its weight is therefore about 85 lb. per cubic foot when dry and slightly more than 100 lb. per cubic foot when wet.

The quantity of grit is usually expressed in cubic feet, cubic yards or pounds per million gallons of sewage. The quantity of grit deposited depends not only upon the design and operation of the chamber but also upon a number of characteristics of the sewerage system. Among the latter are: separation of storm water and sewage; topography and surface of the sewered area, in particular the nature of street and yard paving; capacity of intercepters as compared with dry-weather and storm flows; sewer grades; efficiency of catch basins and their cleaning; tightness of sewers; and nature of industries.

With a velocity of 1 ft. per second and a detention period of 1 min., the average quantity of grit deposited is commonly from 2 to 3 cu. ft. per mil. gal. These values, while of service in deciding upon the quantity of grit to be handled and disposed of, cannot be used in fixing the storage capacity of grit chambers. For the latter, the quantity of grit deposited during storms between cleanings commonly controls. To allow for maximum storm demands between cleaning operations, grit storage of 10 to 30 cu. ft. per mil. gal. generally is provided, the particular allowance depending upon many conditions. With these values, periods between cleanings may drop from an annual average of two weeks to only a few days at times of successive storms. In northern climates a winter suspension of cleaning up to six weeks is common.

TABLE 58.—QUANTITY AND CHARACTER OF MATERIAL FROM GRIT CHAMBERS

Plant	Sewage flow, m.g.d.	Quantity of grit, cu. ft. per mil. gal.	Per cent volatile solids in grit	Character of grit	Authority
Cleveland, Ohio					
Easterly plant	83.7	2.07	....	Inoffensive	Gascoigne (2)
Upstream hopper	....	.....	13.7		
Downstream hopper	....	.....	27.7		
Westerly plant	14.8	2.00	....	Inoffensive	Gascoigne (2)
Upstream hopper	....	.....	9.9[1]		
Downstream hopper	....	.....	10.4[1]		
Guelph, Ont.	2.0	0.33	3	Inoffensive	Dallyn (2)
Kitchener, Ont.	0.8	12	....	Inoffensive	Dallyn (2)
Rochester, N. Y.					
Irondequoit plant	....	2.60[2]	....	....	Skinner (3)
St. Thomas, Ont.	1.5	3	25	....	Dallyn (2)
Stratford, Ont.	1.2	4	75	....	Dallyn (2)
Syracuse, N. Y.	21.2	2.16	5–10	Inoffensive	Holmes and Gyatt (5)
Toronto, Ont.	61.8	3.2	20	....	Dallyn (2)
Woodstock, Ont.	1.4	3.0	....	Very offensive	Dallyn (2)

[1] Discarding samples in which tar and mash were present.
[2] Nine-year average.

Table 59.—Results of Mechanical Analyses of Material Removed from Grit Chamber and of Lake Sand at Cleveland Testing Station

Date	Material	Per cent retained by sieve								Effective size, mm.	Uniformity coefficient	Remarks
		10	20	30	45	50	60	100	200	....	....	Commercial No.
		0.073	0.034	0.022	0.015	0.011	0.007	0.0055	0.0026	....	....	Size of opening, in.
12/12/13	Sample from inlet end of grit chamber.	1.2	12.2	31.3	52.0	65.0	68.5	94.7	97.8	0.15	3.07	Sample washed with water
1/14/14	Sample from inlet end of grit chamber.	5.0	31.8	56.3	73.6	83.1	84.1	95.6	96.0	0.16	4.82	Sample washed with dilute hydrochloric acid
2/ 2/15	Lake sand..........	13.5	26.8	43.2	61.5	72.7	96.0	96.6	98.0	0.20	5.18	

Where the grit is removed continuously, the chambers and equipment must be able to cope with the grit load caused by the maximum storm. The quantity and character of grit removed at some plants are given in Table 58.

The character of the grit deposited in the grit chamber at the Cleveland demonstration plant was studied by Gascoigne, who reported as follows (6):

> Two mechanical analyses were made of the deposited grit. In one the grit was thoroughly washed with water, while in the other the wash was with dilute hydrochloric acid. Both samples were originally black and contained a very small amount of organic matter. The sample washed with water remained black after washing, and although the sand grains of the grit which were washed with acid did not become clear, yet a large per cent of ferrous sulphide was removed as evidenced by the strong odor of hydrogen sulphide after the addition of the acid. Table 59 records the results of these mechanical analyses together with that of the lake sand which is washed upon the beach opposite the submerged outlet.

Disposal of Grit.—The method of grit disposal in any particular plant depends on the quantity and character of the grit as well as upon local conditions. It has been mentioned that at Cleveland, Ohio, the grit contains about 15 per cent volatile matter and is used for filling low-lying areas in a residential section. In the Emscher District, Germany, clean sand removed from grit chambers is used for resurfacing sludge-drying beds. At Syracuse, N. Y., the grit is used for resurfacing roadways and walks and for covering rakings removed by coarse and fine racks, with no noticeable odors from the deposits. According to Skinner (2), grit is of value for mixing with soil in the raising of certain garden crops, notably cucumbers, squashes, melons and tomatoes; like cinders, it is also useful as a top dressing for dirt or gravel paths and drives for light or occasional traffic. Washing the grit for the removal of offensive organic material prior to disposal has been undertaken, sometimes with indifferent success.

Filling is probably the most common method of disposal and where grit of good quality is obtained, the use of such grit for covering screenings, which are to be disposed of at the same time, is advantageous. As mentioned in Chap. X, an incinerator plant has been installed recently at Milwaukee to dispose of mixed grit and screenings.

American Grit Chambers.—The design characteristics of a few American grit chambers are given in Table 60. Among these installations, one of the Rochester chambers is illustrated in Fig. 70 and a plan and sections of the Cleveland chambers are shown in Fig. 71. A view of the North Toronto grit chambers is given in Fig. 72.

Gascoigne (2) gives the following additional information in regard to the grit chambers at the Easterly plant in Cleveland:

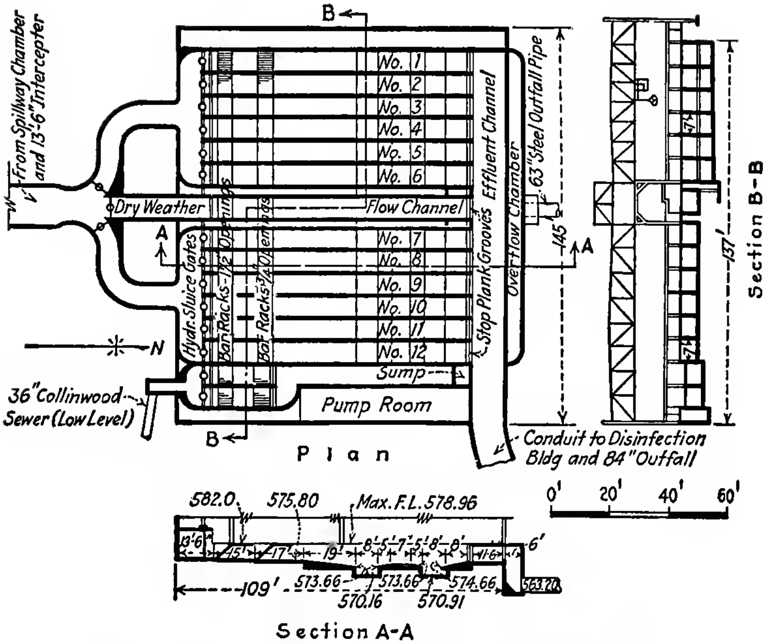


FIG. 71.—Grit chambers at Easterly sewage works, Cleveland.

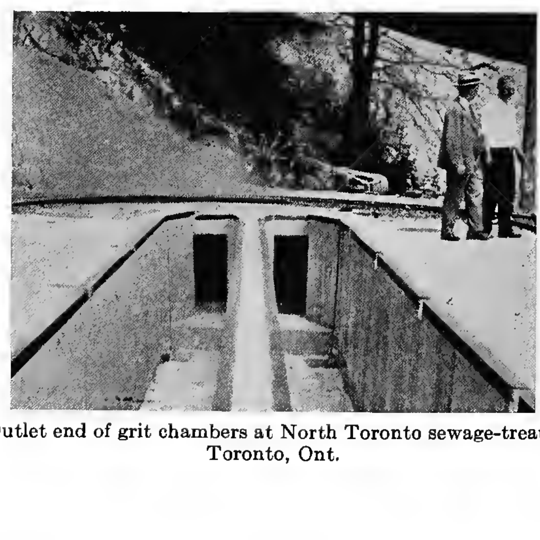


FIG. 72.—Outlet end of grit chambers at North Toronto sewage-treatment plant, Toronto, Ont.

TABLE 60.—DESIGN CHARACTERISTICS OF GRIT CHAMBERS

	Chicago, Ill., North Side plant	Cleveland, O., Easterly plant	Fitchburg, Mass.	Rochester, N. Y., Irondequoit plant	Syracuse, N. Y.	Toronto, Ont., North Toronto plant
Maximum sewage flow provided for, m.g.d.	420	288	7	180	55	12
Number of units	12	12[1]	2	6	3	4
Dimensions of unit, ft.:						
Length	80	60	50	90	40	45
Width	8	7	2	10	4	3
Over-all depth	6.5	8.8	6.5	9	..	5.5
Depth of grit-storage space, ft.	...	3	1	3	..	1
Average velocity of flow, ft. per sec.	¾–1	½–1	¾–1¼	...	1	1
Method of controlling flow	Sluice gates	Dry-weather channel and multiple chambers	Adjustable weirs	Sluice gates	Controls on pumps	Adjustable weirs
Method of removing grit	Grab bucket	Grab bucket and industrial cars	Flushing with monitor nozzle	By hand into industrial cars	Grab bucket	By hand into industrial cars
Disposal of grit	.....	Filling low land	Filling low land	Filling low land	Surfacing roads and walks; covering rakings	Filling low land

[1] Plus one dry-weather channel.

Population served: 575,000.
Average daily flow, 1923–1925: 83.6 million gal.
Average dry-weather flow, design value: 92 m.g.d.
Capacity of each chamber: 12 to 24 m.g.d.
Detention period: 60 to 120 sec.
Grit storage space: 13 cu. yd. per pair of hoppers = 156 cu. yd. in all 12 chambers. Additional space below flow line of outlet conduit, 243 cu. yd. Available capacity for maximum flow, 1.38 cu. yd. per mil. gal., and with 25 per cent ineffective, 1 cu. yd.
Frequency of cleaning, 1923–1925: 5 to 36 days, average 16 days
Method of cleaning: electrically operated grab bucket, traveling on a monorail system, loading into industrial cars
Racks: two bar racks, the first with 1½-in. openings, the second with ¾-in. openings.
Cost: $1245 per mil. gal. maximum daily capacity for grit chambers and essential appurtenances.

At the Syracuse plant, the number of grit-chamber compartments in service depends upon the rate of sewage flow, one compartment being used for 18.3 m.g.d. or less, two for 18.3 to 36.7 m.g.d. and three for flows of 36.7 to 55 m.g.d. (5). The compartments are cleaned, without unwatering, by a clamshell bucket, which fits the width of the chamber. For protection, rails are embedded in the bottom of the chamber. The bucket and traveling hoist are operated from an overhead-monorail track system and are controlled by an attendant in a cab, which is an integral part of the traveling mechanism. The chamber is ordinarily cleaned once a week. Daily removal of grit is usually necessary, however, during times of melting snow and following heavy precipitation.

At Rochester the grit chambers are unwatered and drained for a few days before excavating the grit, which is loaded by hand into the body of an industrial car, lowered into the chamber by a crane. After the car body has been filled, it is replaced on its truck and, when a train has been made up, an industrial locomotive hauls it away to a dump.

SKIMMING TANKS

A skimming tank is a chamber so arranged that floating matter rises and remains on the surface of the sewage until removed, while the liquid flows out continuously through deep outlets or under partitions, curtain walls or deep scum boards.

The object of skimming tanks is the separation from the sewage by flotation of the lighter, floating substances. The material collecting on the surface of skimming tanks, whence it can be removed, includes oil, grease, soap, pieces of cork and wood, vegetable debris and fruit skins, originating in household and industry.

The use of skimming tanks for the treatment of municipal sewage is a relatively new development, brought about by the increased use of oil for domestic and industrial purposes and the discharge of waste oil and grease into the public sewers from garages. In nonmanufacturing communities the ether-soluble matter is commonly estimated at 20 gm. per capita daily, or over 8 tons per 1000 population yearly. This is in itself a considerable quantity and is greatly increased by sudden discharges of oily and greasy wastes in relatively large quantities that may not appear in the usual analyses. Not all of this material, however, will rise to the surface of skimming tanks. Much of it is found as a coating on heavier particles and escapes with them.

Design of Skimming Tanks.—Most of the American skimming tanks in existence or under construction are elliptical or circular in shape and provide for a detention period of 1 to 15 min. The outlet, which is submerged, is situated opposite the inlet and at a lower elevation, to assist in flotation and to remove any solids that may settle. The characteristics of three American tanks are listed in Table 61.

TABLE 61.—DESIGN CHARACTERISTICS OF SKIMMIMG TANKS

	Washington, D. C.	Toledo, Ohio	Akron, Ohio
Quantity of sewage treated, m.g.d.:			
Average	85	21	33
Maximum	.....	101	94
Average detention period, min.	1	1.8	12.6
Number of tanks	1	1	2
Shape	Egg-shaped	Elliptical	Circular
Length, ft	22.75	21	55 (diameter)
Width, ft	16	13	
Maximum depth, ft	19	23	12
Inlets:			
Size, ft	5 (diameter)	6 (diameter)	4 × 3
Depth of invert below flow line, ft	6	12	5
Outlets:			
Size, ft	5 (diameter)	5 (diameter)	3.5 × 3.5
Depth of invert below flow line, ft	16	23	8

Aerated Grease-separating Tanks.—The removal of oil and grease from sewage appears to be facilitated by a short period of aeration. This may be due, in part, to causing the separation of oil and grease from heavier sewage particles to which they adhere, thus enabling the former to float to the surface; in part to promoting the coalescence of the oil particles, and in part to adhesion of oil and grease to the small, rising air bubbles which carry them to the surface.

Several systems have been developed in conjunction with the aeration, separation and collection of oil and grease. Imhoff has built combined aerating and skimming tanks at several plants in the Ruhr district in Germany. These tanks are rectangular in plan with trough-shaped bottoms. Detention periods of about 3 min. are employed. Each tank is divided into three sections by longitudinal vertical baffles. The central section is aerated, as by means of air diffused from porous plates set in the bottom. The scum collects in the compartments on either side of the aerated section and then is removed from the tank. Aerated skimming chambers of this type have been installed at Pasadena, Cal., and Springfield, Ill.

At Whittier, Cal., there is a pre-aerating chamber, 40 ft. long, 10 ft. wide and 9 ft. deep, followed by a skimming chamber 10 ft. square, where the scum is collected in a trough and the effluent is drawn from the bottom. The aeration system is of the ridge-and-furrow type, described in detail in Chap. XXVI.

Recently the tendency has been to provide a short period of aeration in tanks or channels preceding sedimentation tanks, which are equipped with skimming devices. At San Antonio, Tex., air-diffuser plates have been installed in the distribution channels leading to the preliminary-sedimentation tanks. The sewage is subjected to a short period of aeration and the oil, grease and other floating matters are collected by skimming arms on the Dorr traction mechanisms within the sedimentation tanks.

At Lancaster, Pa., diffuser plates have been placed in the influent channels of preliminary-sedimentation tanks. As reported by Laboon (7), aeration is provided for the purposes of keeping the solids in suspension, assuring suitable distribution of sewage between two tanks and assisting in the accumulation of grease. The scum is concentrated at the effluent end of each tank by a mechanism of the scraper-conveyor type and provision is made for skimming the grease to the side of the tank, whence it is conducted through an 8-in. pipe into a scum pit. The skimmings are removed from the pit into a can for transportation to an incinerator in the rack house.

The District of Columbia sewage-treatment plant, which is being designed in 1935, includes aerated grease-separating tanks giving a 4-min. detention period. The oil, grease and other floating matter will be skimmed mechanically from the surface of plain-sedimentation tanks.

In tanks where sludge-removal mechanisms are not readily adaptable for skimming, a system of water piping with small nozzles, as developed by Allen in the Fitchburg, Mass., Imhoff tanks, may be of value. The nozzles, which are horizontal hacksaw cuts in vertical pipes, discharge jets of water almost horizontally over the surface and drive the accumulation of oil and grease to collection points. An installation of this type in the preliminary-sedimentation tanks at the North Toronto plant, Toronto, Ont., illustrated in Fig. 73, has given good results.

Provisions for skimming the surfaces of sedimentation units are further described in Chap. XII.

Skimming-detritus Tanks.—In the tanks designed for Akron, Ohio, separation of the heavy, readily settling solids and of the light, floating materials is effected in a single unit called a skimming-detritus tank, which is illustrated in Fig. 74. The characteristics of this tank have been given in part in Table 61. Floating wooden booms skim the grease from the surface of the tank and concentrate it in front of an adjustable weir, over which it is pushed onto a draining bed. Dorrco plows, similar to those employed in mechanically cleaned settling tanks, move the grit and other solids settling from the sewage to a central hopper, whence it is pumped with about 20 per cent of the average sewage flow to grit chambers equipped with scraper plows. Here the real separation of grit takes place. After fine screening, this so-called "underflow" discharges into the main conduit of the plant and reunites with the main body of sewage that has passed directly through the skimming-detritus tanks. The plants at Allentown, Pa., and Dayton, Ohio, have skimming-detritus tanks of a similar type.

Fig. 73.—Skimming nozzles in preliminary-sedimentation tanks at North Toronto sewage treatment plant, Toronto, Ont.

At the West Side plant of Chicago, there are eight skimming tanks, each 105 ft. long, 66 ft. wide and with a water depth of 11 ft. These tanks provide a detention period of 15 min. at average flow and are designed to remove the oil and other scum-forming material, as well as to separate the grit from the sewage. The tanks are provided with mechanical equipment of the scraper-conveyor type, for removing the bottom sludge and surface scum. About 15 per cent of the average sewage flow is withdrawn with the bottom sludge and passed through grit chambers. The effluent of the grit chambers is pumped to the influent conduits leading to the Imhoff tanks. Five mechanically cleaned fine bar racks, each 10.5 ft. wide with 1-in. clear openings, are provided for screening the sewage before it enters the skimming tanks.

Removal and Disposal of Skimmings.—The materials collected on the surface of skimming tanks are commonly removed by hand. They are either dipped up or are collected on perforated trays that are dragged through the sewage until covered with floating materials. As previously noted, removal is greatly aided by driving scum to collecting points or into collecting bays or troughs by means of moving arms or jets of water. It is possible to draw the scum hydraulically from the bays or troughs. If the material has been dipped up or for some other reason contains

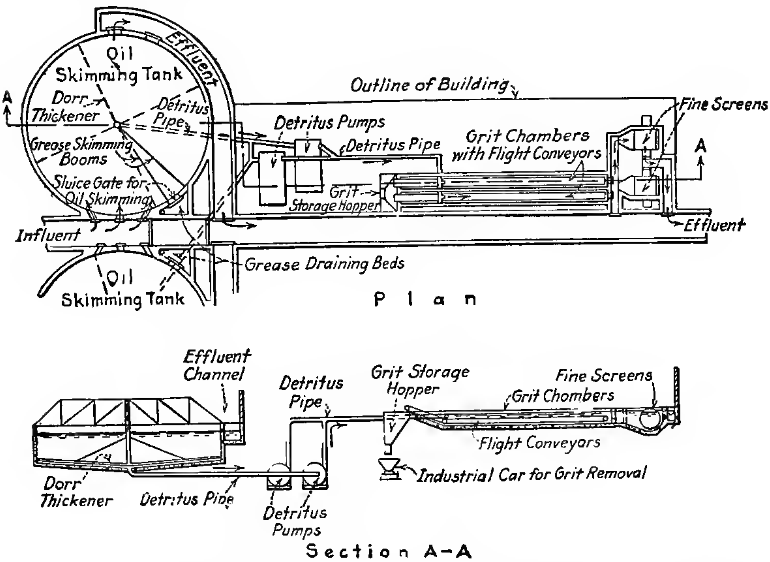


FIG. 74.—Diagrammatic sketch of skimming-detritus tanks at Akron, Ohio.

much water, it is sometimes placed in separating tanks, from which the water is drawn off, or in drainage boxes, or it is discharged on to drying beds similar to sludge-drying beds. The drained material may be buried, burned or mixed with dried sludge and used for filling.

Keefer and Kratz (8) have reported favorable results in tests on the digestion of scum collected from the preliminary-sedimentation tanks at Baltimore. They find that scum digests at least as rapidly as sludge and perhaps more so. At the North Toronto plant in Toronto, Ont., skimmings from the preliminary-sedimentation tanks have been disposed of by digestion with sludge for several years.

During 1930, the Committee on Sewage Disposal of the American Public Health Association investigated the operation of sewage-treatment plants from the standpoint of scum and floating solids and reported that "floating solids from the settling compartments, aside from oil, are sometimes collected and incinerated, but more frequently are placed in the digestion compartment" (9).

At Springfield, Ill., an attempt was made to dispose of skimmings in digestion tanks. The practice was abandoned after one month, because trouble was experienced with the digestion process, "probably due to the grease" (10).

Banta (11) reports that, at the joint disposal plant of the Los Angeles County Sanitation Districts, the disposal of skimmings by digestion in the main digestion tanks, together with primary sludge and excess activated sludge, resulted in increasing difficulty with drying digested sludge. He states:

> The sludge, when placed on the beds, had a slimy, mushy, dull appearance and failed to give up its water readily, either by percolation or by evaporation. Additional digestion time was provided in a lagoon with the belief that the digestion time was too short, but the added capacity made no appreciable improvement in the drying qualities of the sludge. The practice of digesting the skimmings in the same tanks with raw and activated sludge from the main plant was then discontinued. . . The drying quality of the digested sludge improved gradually as the skimmings were displaced from the digestion cycle.

The skimmings, which amounted to 4300 lb. daily and had a moisture content of 80 per cent, then were drained on sand beds and burned in an experimental incinerator at an estimated operating temperature of 1700°F. Banta states that, while the incineration process has not been odorless, the odors produced are not objectionable or persistent. He believes a higher operating standard could be maintained with a firebox built of firebrick and operating at a temperature in the vicinity of 2400°F.

At Lima, Ohio, according to O'Brien (12), an attempt was made to burn skimmings in an incinerator which had been installed to burn screenings. The temperatures of combustion were so high that they caused the chimney to crack. The skimmings then were hauled to a low place, ignited periodically and allowed to burn in the open.

Little information is available on the quantity of floating matter that readily can be removed from sewage. At Washington, D. C., about 100 lb. per day are obtained from 85 mil. gal. of sewage; at Worcester, Mass., about 100 cu. ft. of oil and grease per 1000 population were skimmed from the surface of Imhoff tanks during 1926. At North Toronto, Ont., the average monthly quantity varies from 0.5 to 2.5 cu. ft. per mil. gal. of sewage. The average quantity removed at Allentown, Pa., during the year 1930 was about 0.5 cu. ft. per mil. gal. The average quantity removed at Dayton, Ohio, from April to October, 1930, was 0.4 cu. ft. per mil. gal. of sewage flow, the maximum for one day being 3.4 cu. ft. per mil. gal.

The quantity of skimmings removed from tanks at various treatment plants, as reported in 1930 to the Committee on Sewage Disposal of the American Public Health Association, is given in Table 62 (9).

Table 62.—Quantity of Floating Matter Skimmed from Tanks at Various Treatment Plants

Location of Plant	Skimmings Removed, Cu. Ft. per M.G. of Sewage Flow Treated
Akron, Ohio	0.20[1]
Aurora, Ill.	1.25–1.50
Cleveland, Ohio:	
Southerly plant	0.3
Westerly plant	3.84
Fitchburg, Mass.	0.58
Sturgis, Mich.	5.0
Syracuse, N. Y.	0.34
Worcester, Mass.	0.1

[1] After drying to 33.8 per cent moisture.

Grease Traps.—Grease traps are in reality small skimming tanks connected with a house or industrial sewer. As they are situated close to the source of grease, which is often discharged with hot water, it is essential that they be given sufficient capacity to permit the sewage to cool and the grease to separate after congealing. Grease traps are employed at many manufacturing plants, garages, hospitals and hotels. A number of proprietary-tank patterns are in use. In most of them the inlet is situated below the surface and the outlet at the bottom. To be efficient they must be large enough to hold and, if necessary, cool the sudden discharges of oily or greasy wastes and they must be cleaned frequently and regularly. In many cases neither of these requirements is met. In the army cantonments constructed during the World War, a capacity of 300 gal. was found desirable to serve a 250-man kitchen.

STORM-WATER TANKS

In this country little provision is made for the treatment of more than two or three times the average dry-weather flow from combined sewers, although storm flows may reach many times this rate. Customarily, the excess overflows automatically at relief points and is discharged into natural waters without treatment. In England and Germany, where normal per capita flows are relatively small, provision is often made for partial treatment of a portion or all of the storm flow in excess of that passing through the main sewage-treatment plant, at least to the extent of the removal of grit and detritus, in so-called *storm-water stand-by tanks*. For ordinary storms a detention period of 20 to 30 min.

is frequently provided. A few similar tanks have been built in Canada and the United States.

Design of Storm-water Tanks.—The most important function of storm-water tanks is the retention of putrefactive solids which may accumulate on the bed of the diluting waters and continue to give trouble as they decompose. The tanks serve to catch the first flush of storm water which usually contains large amounts of suspended matter. During protracted storms, streams generally flow freely and thus are capable of carrying along much solid matter in suspension.

Temple (13) advises, for normal conditions, the provision of storm-water tanks 120 ft. long, of such cross section as to reduce the velocity

FIG. 75.—View of storm-water stand-by tank at North Toronto sewage-treatment plant, Toronto, Ont.

of all that passes the overflow weir, by-passing the main treatment plant, to 0.1 ft. per second.

North Toronto Storm-water Stand-by Tank.—The North Toronto sewage-treatment plant at Toronto, Ont., was designed for the complete treatment of a maximum rate of sewage flow of 12 million U. S. gal. daily. The capacity of the combined sewer to the plant is about 920 m.g.d. A storm-water stand-by tank, shown in Fig. 75, is provided for treating all flows in excess of 12 m.g.d. The tank is rectangular in shape, 70 ft. long by 103 ft. wide, and has an average water depth of 10 ft. The water capacity is about 66,700 cu. ft. The detritus channel, forming part of the tank, is 103 ft. long, 20 ft. wide and 11 ft. deep. This channel must be filled before the tank proper goes into operation. Much of the heavy detritus settles in this compartment.

After a storm, the supernatant liquor is drawn off and passed through the sewage-treatment plant. The detritus is agitated with streams of water from monitors and the lighter material washed out flows to a pump sump and is returned to the influent of the treatment plant. The heavy detritus is excavated with a clamshell bucket operated from a gantry crane and is transported over an industrial railway system to sludge dumps.

Columbus Storm-water Tanks.—An installation of storm-water tanks on each of two intercepters has been made at Columbus, Ohio, for the treatment by sedimentation of storm flows in excess of the maximum dry-weather rate of flow provided for at the main sewage-treatment plant (14). At the Whittier St. installation, when a storm occurs and the flow in the intercepting sewer, which at the storm tanks has a capacity of 680 c.f.s., exceeds the rate desired to pass down to the treatment plant, the excess flow discharges into one or more of the three storm-water tanks. A control house is provided for regulation of the quantity to be treated at the main treatment plant. If the excess storm flow continues for a sufficiently long period of time, the tanks fill up and overflow to the river. The overflow from the tanks is baffled to prevent floating material from passing out over the outlet weirs.

The arrangements are such that either one, two or three of the tanks can be opened to the intercepting sewer at one time. It is planned, however, normally to have only one tank connected. Under storm conditions, when this tank is nearly full, a second tank is automatically opened to the sewer. When this tank is nearly full, the third tank comes into service and, with the excess storm flow still continuing, all tanks overflow. The operation of the tanks can be controlled either automatically at the control house or manually at the tanks.

After the storm is over and when the rate of flow in the intercepting sewer falls to less than the maximum rate desired to pass to the sewage-treatment plant, the tanks are drained to the intercepting sewer. When the tanks have been drained, the deposited sludge is flushed back into the intercepting sewer and passed on to the treatment plant for segregation, treatment and disposal. The three units have an aggregate capacity of 4,011,000 gal. Each unit is 105 ft. long by 187 ft. wide, with provision for a water depth to overflow of 9.43 ft.

At Alum Creek there is a single unit, which performs a function similar to that of the three units at Whittier St. The tank is covered and is 106 ft. long by 181.25 ft. wide, with provision for a water depth to overflow of 10.25 ft. The effective capacity of the tank is 857,000 gal.

Cost of Construction, Operation and Maintenance.—The cost of grit chambers is dependent upon the type, size, elements of design, provision of mechanical equipment for cleaning, depth of excavation and locality. The unit cost may vary from $300 to $1250 per mil. gal. daily maximum capacity. The average cost of a number of installations, based on the average sewage flow provided for, was $1200 per mil. gal. daily.

The cost of operating grit chambers depends upon the quantity of grit deposited and the method of grit removal and disposal. The cost ranges from $0.50 to $1.00 per cubic yard of grit removed and disposed of. The cost at Worcester, Mass., during 1928 was $0.61 per cubic yard and $0.70 per mil. gal. of sewage treated.

The cost of skimming tanks does not differ materially from that of plain-sedimentation tanks. Unit costs of $0.60 to $1.00 per cubic foot of water capacity, including skimming equipment, are representative. The cost per mil. gal. daily average sewage flow provided for varies from $1500 to $2500. Based on the population of design, the cost per capita varies from 15 to 25 cents. The cost of the skimming-detritus plant at Dayton, Ohio, including skimming-detritus tanks, grit channels and equipment, pumps, screens, press, incinerator and building, was about $128,500, equivalent to some $5000 per mil. gal. daily average sewage flow and $0.50 per capita on the basis of design.

The elements of cost involved in the operation of skimming tanks and skimming-detritus plants may include the removal and disposal of skimmings, power for mechanical units, pumping of underflow, removal and disposal of grit, and pressing, incineration and disposal of screenings. The cost of disposal of grit and screenings has already been discussed. The power for mechanical equipment in skimming tanks is about the same as that required in plain-sedimentation tanks of equivalent sizes. When the underflow is pumped, as at Akron and Dayton, Ohio, and Allentown, Pa., the cost of raising about one-fourth of the sewage flow 6 to 10 ft. must be included. This may amount to $0.50 to $0.90 per mil. gal. pumped or $0.15 to $0.25 per mil. gal. of sewage flow. The skimmings are usually disposed of with the screenings and at similar unit costs.

There are only a few installations of storm-water tanks in this country. The total cost of the Columbus tanks built in 1930 was about $679,000. The cost of the North Toronto storm-water tank, built in 1928, was about $50,000. It was designed for a combined system ultimately serving 175,000 persons. On this basis the cost was about $0.29 per capita.

The operation of the North Toronto storm tank involves cleaning the tank and excavating the grit by means of a gantry crane. The grit is carried to the sludge dump in industrial cars. The cost of operation amounts to about $1 per cubic yard of grit removed. During the period from June to October, inclusive, 1930, the total quantity of detritus removed was 555 cu. yd.

Bibliography

1. Fifth Rept., Royal Comm. on Sewage Disposal, 1908.
2. *Trans.*, A. S. C. E., 1927; **91,** 496.

3. Blunk, H.: *Sewage Works Jour.*, 1933; **5**, 509.
4. *Sewage Works Jour.*, 1930; **2**, 347.
5. Holmes, G. D., and W. P. Gyatt: *Sewage Works Jour.*, 1929; **1**, 318.
6. Rept. on Tests at Sewage Testing Station, Cleveland, Ohio, 1914.
7. Laboon, J. F.: *Sewage Works Jour.*, 1933; **5**, 1021.
8. Keefer, C. E., and Hermann Kratz: *Sewage Works Jour.*, 1931; **3**, 50.
9. Am. Pub. Health Assoc. *Year Book*, 1930–1931, p. 140.
10. Ann. Operation Rept., Sewage Treatment Works, Springfield Sanitary Dist. for 1931 and 1932.
11. Banta, A. P.: *Sewage Works Jour.*, 1933; **5**, 659.
12. O'Brien, W. S.: *Sewage Works Jour.*, 1934; **6**, 42.
13. Temple, F. C.: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 190.
14. Gregory, J. H., R. H. Simpson, Orris Bonney and R. A. Allton: *Trans.*, A. S. C. E., 1934; **99**, 1295.

CHAPTER XII

PLAIN SEDIMENTATION

Sedimentation is the subsidence and deposition of suspended matter in a liquid by gravity (1). Applied to sewage treatment it is the process by which the suspended matter in sewage subsides and is deposited by gravity. The term *plain sedimentation* is used to distinguish the simplest form of this process, unaided by chemicals or other special means, such as contact aerators, to induce deposition of solids which otherwise would not settle. Generally it is taken also to mean the absence of provisions for the decomposition of deposited solids in contact with the sewage and is treated thus in this volume.

In general, distinction is made between the removal of heavy mineral solids, called grit, which is commonly accomplished in grit chambers or detritus tanks, and the settling as sludge of lighter, more organic, sewage solids in sedimentation tanks.

A sedimentation tank is defined as "a tank or basin in which sewage, partly treated sewage, or other liquid containing settleable solids is retained long enough and at velocity low enough to bring about sedimentation of a part of the suspended matter, but without a sufficient detention period to produce anaerobic decomposition" (1).

Objects of Sedimentation.—Plain-sedimentation tanks are generally provided for one or more of the following purposes:

A. As primary-sedimentation tanks for the removal of settling solids,

1. To prevent formation of offensive sludge deposits in waters into which the sewage is discharged.

2. To relieve the diluting waters of a portion of the burden placed upon them.

3. To reduce clogging of fine-grained filters.

4. To relieve coarse-grained filters of an unnecessary burden of solids.

5. To keep solids from settling to the bottom of aeration units, where they may clog air diffusers or by putrefaction interfere with the efficiency of the process.

6. To improve the efficiency of disinfection, since the coarser solids are not penetrated readily by disinfecting chemicals.

B. As final-sedimentation tanks following coarse filters, to remove settling solids and increase the overall efficiency of the process, as well as to prevent formation of offensive sludge deposits in waters into which the effluent is discharged.

C. As final-sedimentation tanks in the activated-sludge process, to settle out activated sludge, part of which is returned to the process and the remainder disposed of.

THEORIES OF SEDIMENTATION PROCESS

Sewage matters in suspension are found in this state either because their specific gravity is less than that of water or because the velocity of flow is such that they are carried along by the sewage. Since the transporting capacity varies as the sixth power of the velocity, it follows that a reduction in velocity will cause the sewage to deposit part of its burden of solid matter under the influence of gravity. Apart from substances that are lighter than water and will rise to the surface, such as oil, grease and floating matters, some of the suspended solids, sandy matters or grit, are deposited promptly when the velocity is reduced below that normally obtaining in sewers; others, called settling solids, require lower velocities and longer periods of time for settling; still others, colloidal and nonsettling solids, will not subside within a reasonable time even when the sewage is perfectly quiescent.

The processes governing sedimentation are extremely complex. In order to approach them it is necessary first to restrict discussion to some of the elementary considerations of the problem. Among these the subsiding characteristics of particles in still water must be given first attention.

Under the influence of gravity all particles heavier than water tend to settle. This tendency, however, is opposed by the frictional resistance of the liquid and its resistance to deformation or viscosity. While both are effective at the same time, experiments have shown that for larger particles friction controls, whereas for small ones viscosity controls.

The factors governing the rate of subsidence of particles in still water are: the specific gravity of the particles relative to the liquid; their size and shape, which together govern the ratio of volume to surface area, and the temperature of the liquid, which controls its viscosity as well as its density.

Assuming the particles to be spherical in shape, their weights and volumes vary as the cubes of their diameters, while their surfaces vary as the squares. Hence the relative surface presented by small particles is much greater than that presented by large ones and small particles settle much more slowly than do large ones.

Stokes's Law.—Stokes has formulated the relation that exists between the rate of subsidence of very small particles and the various factors influencing their sedimentation. If the values are expressed in the c.g.s. system, Stokes's law is formulated as follows:

$$v = \frac{2}{9}g\frac{s - s'}{e}r^2$$

where v = velocity of subsidence
g = acceleration due to gravity
s = density of the particle
s' = density of the liquid
e = viscosity of the liquid
r = radius of the particle

Since viscosity is governed by temperature, the viscosity factor may be replaced by a temperature factor. It will be found that the ratio of the viscosity at any temperature to that at 50°F., where the value of e is 0.01303, is approximately equal to $\frac{60}{T + 10}$ where T is the temperature in degrees Fahrenheit. When this substitution is made, with linear dimensions expressed in millimeters and the diameter d used instead of r, it is found that

$$v = 418(s - s')d^2\frac{T + 10}{60}$$

For quartz sand of specific gravity 2.65 the formula becomes:

$$v = 690d^2\frac{T + 10}{60}$$

For sewage solids of specific gravity 1.2:

$$v = 84d^2\frac{T + 10}{60}$$

Stokes's law holds for particles smaller than 0.085 mm. in diameter. Beyond this limit viscosity, while still active, no longer controls and friction comes more and more into play, the velocity varying as the square root of the diameter when friction controls. There is a transition space in between. This has been investigated by Hazen for quartz sands of specific gravity 2.65 with diameters of 0.1 to 1 mm. Hazen's values may be formulated approximately as follows:

$$v = 100d\frac{T + 10}{60}$$

On the assumption that the specific gravity of the material enters into the equation as in Stokes's law, the relation for particles of specific gravity 1.2 is $v = 12d\frac{T + 10}{60}$.

The rate of subsidence of particles expressed in millimeters per second is known as their *hydraulic subsiding value.* The hydraulic subsiding values of particles with specific gravity 2.65 and 1.20 are plotted in

Fig. 76. They hold for a temperature of 50°F. For other temperatures they must be multiplied by the temperature factor $\frac{T + 10}{60}$.

The marked effect of temperature upon subsidence should be noted carefully. Because of it much greater sedimentation efficiencies can be obtained in summer than in winter. At a temperature of 74°F., for example, the rate of settling is twice as great as at the freezing point.

Hazen's Theory of Sedimentation.—Hazen (2) has made a careful mathematical study of sedimentation. The following paragraphs are abstracted from the original paper, for the purpose of showing the various factors which enter into the problem. An example of the application of Hazen's theory of sedimentation to the design of grit chambers is given in Chap. XI.

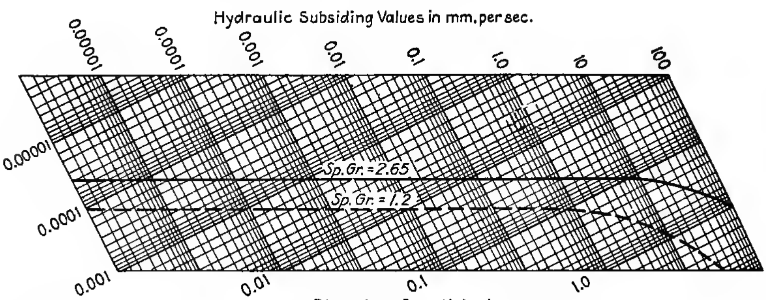


Fig. 76.—Hydraulic subsiding values of particles with specific gravity 2.65 and 1.20 in still water at 50°F.

In order to simplify the approach, Hazen made the following assumptions:

1. Whenever a particle of suspended matter strikes the bottom it remains where it strikes and is never carried forward on the bottom or picked up again.

2. All particles have the same hydraulic subsiding value, v.

Let t = time for a particle to fall from the surface to the bottom of the basin, the water meanwhile being absolutely still

a = time of sedimentation in case the action is intermittent; and, in case of continuous operation, let a be the quotient obtained by dividing the capacity c of the basin by e, the quantity of water entering or leaving it during each unit of time

n = number of basins, in case several basins are used in series

x = the proportion of sediment remaining at the end of the process, the quantity at the beginning being taken as unity, or 1

Proposition 1. *Intermittent Basin.*—On the assumption that the basin is full of water and absolutely quiet:

$$x = 1 - 1 \times \frac{a}{t}$$

The values obtained from this expression are plotted as "Theoretical maximum" in Fig. 77. They represent the theoretical maximum sedimentation that can be obtained in an intermittent basin; they cannot be reached in practice. Adverse factors are: the kinetic energy of the entering water which is still capable of producing vortex motion after long periods of apparent quiescence; the action of wind; and changes in temperature which, though slight, set up convection currents. All of these induce mixing of the liquid.

Proposition 2. *Intermittent Basin.*—On the assumption that the basin is full as in proposition 1, but that the water is kept mixed during the process of

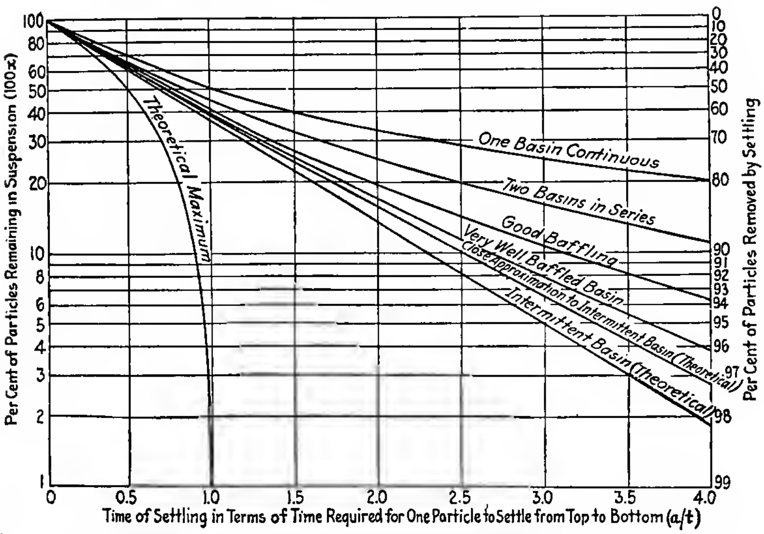


FIG. 77.—Sedimentation obtained in differently arranged settling units. (*After Hazen.*)

sedimentation to such an extent that the density of the suspended matter is the same in all parts of the basin, the quantity of suspended matter remaining after an interval of time da is:

$$x_1 = 1 - 1 \times \frac{da}{t} = \left(1 - \frac{da}{t}\right)$$

After another interval da:

$$x_2 = x_1 - x_1\frac{da}{t} = \left(1 - \frac{da}{t}\right)^2$$

Since there are $\frac{a}{da}$ intervals:

$$x = \left(1 - \frac{da}{t}\right)^{\frac{a}{da}}$$

If this expression is developed binomially and all values containing the first derivative da are neglected, $x = 0.367878$ for $\frac{a}{t} = 1$; other determinations can be made similarly. The values obtained are plotted as "Intermittent Basin (Theoretical)" in Fig. 77. They represent the theoretical minimum sedimentation for an intermittent basin. For n intervals of time, $x = \left(1 - \frac{a}{nt}\right)^n$.

Proposition 3. *Single Continuous Basin.*—On the assumption that the water enters and leaves the basin continuously and that the density of the suspended matter is kept the same in all parts of the basin by mixing, the quantity of sediment deposited in the time a is $(1 - x)$; in the time da it is $\frac{da}{a}(1 - x)$. Since the time required to deposit all the particles is t, the proportion deposited in the time da is $\frac{da}{t}$ and the quantity, $x\frac{da}{t}$. Hence

$$x\frac{da}{t} = \frac{da}{a}(1 - x)$$

$$x = \frac{1}{1 + \frac{a}{t}}$$

The values obtained from this expression are plotted as "One Basin Continuous" in Fig. 77.

Proposition 4. *Two Basins in Series.*—On the assumption that two basins are arranged so that the water from one enters the second, all other conditions remaining the same as in proposition 3, the time of sedimentation in each basin is $a/2$. Hence, from proposition 3, the quantity of suspended matter remaining after passage through the first basin is $x_1 = \frac{1}{1 + \frac{a}{2t}}$ and the quantity remaining after the second is $x = \frac{x_1}{1 + \frac{a}{2t}} = \frac{1}{\left(1 + \frac{a}{2t}\right)^2}$. The values obtained are plotted as "Two Basins in Series" in Fig. 77.

Proposition 5. *More than Two Basins in Series.*—If the reasoning of proposition 4 is extended to n basins in series, $x = \frac{1}{\left(1 + \frac{a}{nt}\right)^n}$. The values for $n = 4$, 8 and 16 are plotted as "Good Baffling," "Very Well Baffled Basin," and "Close Approximation to Intermittent Basin," respectively, in Fig. 77. For $n = \infty$, $x = \frac{1}{\left(1 + \frac{a}{nt}\right)^n} = \left(1 - \frac{a}{nt}\right)^n$, which is the same expression as that of Proposition 2, shown as "Intermittent Basin (Theoretical)" in Fig. 77. In other words, theoretically, a sedimentation basin operated on the continuous system with absolutely complete baffling would give the same results as a basin on the intermittent system kept absolutely mixed from top to bottom.

Proposition 6. *Very Shallow Basin.*—If a very shallow basin is assumed, with area b, depth h and capacity c, and e is the quantity of water treated in a unit of time, while v is the hydraulic subsiding value of the particles,

$$a = \frac{c}{e} = \frac{bh}{e}$$

$$t = \frac{h}{v}$$

$$\frac{a}{t} = \frac{bv}{e}$$

It will be noted that a/t, which is the only factor entering into the expression measuring sedimentation efficiencies of the different types of basins, is independent of the depth of the basin but varies directly with the area of the basin and the hydraulic subsiding value of the particles and inversely with the quantity of water treated in a unit of time. This fact that, within limits to be discussed later, sedimentation is independent of depth, has of late years been put to practical use by Imhoff in the construction of what he has called *shallow underdrained settling basins,* which act at the same time as sludge-drying beds.

Proposition 7. *Basin with Horizontal False Bottoms.*—On the assumption that a basin is divided by horizontal plates into two or more compartments one above the other, each horizontal subdivision will provide a surface to receive sediment. Since sedimentation is independent of depth and merely dependent upon surface area, the sedimentation efficiency will be increased in proportion to the increase in area. The most serious practical difficulty to be met in carrying out this idea is the increased cleaning necessitated. The *slate beds* of Dibdin operated on this principle; expansion of this principle explains in part the sedimentation efficiency of *contact beds* and *intermittent sand filters.*

Proposition 8. *Suspended Particles Having Unequal Hydraulic Subsiding Values.*—Since a is the same no matter what the size of the particle, while t varies for particles of different size and specific gravity, the per cent of particles of each size and weight removed in a given basin will vary, a greater proportion of small and light particles remaining in the effluent than of large and heavy ones. This permits of *differential sedimentation* as applied in grit chambers, in which the object is to remove the heavier mineral solids while permitting the lighter organic ones to escape, as explained in Chap. XI.

Limitations of Hazen's Theory.—In applying Hazen's theory of sedimentation to sedimentation tanks for sewage, a number of limitations of the theory must be kept in mind, most of which are pointed out in the original paper.

Bottom Velocities.—It was assumed that any particle striking the bottom remains there. This is not necessarily so. Both horizontal and vertical currents operate against it. The vertical currents are in the nature of vortex motion, wind-induced currents and temperature convection currents. The horizontal currents are chiefly due to the horizontal movement of the water through continuous basins. It would seem that, apart from adhesion, which may be a factor of considerable importance in the case of sewage solids, a bottom velocity equal to the hydraulic subsiding value would prevent deposition or lift the particle or move it along the bottom. Practically nothing is known about the magnitude of bottom velocities in relation to mean horizontal velocities. The data at hand seem to show that mean velocities of 20 to 40 times v are required to move sediment along the bottom. Since it appears from Fig. 77 that a/t must reach a value of about 1.5 to secure satisfactory sedimentation, say 60 to 80 per cent removal, it follows that the use of ratios of basin length to depth of less than 30:1 to 60:1 is indicated.[1] To this extent then sedimentation is not independent of depth as shown in Proposition 6.

Granular and Flocculent Solids.—Depth enters into the problem of sedimentation in another way also. Much of the suspended matter in sewage is flocculent in character, *i.e.*, the particles tend to gather together into aggregates. Sweeping down through the sewage they enmesh other particles, become larger in size and settle more rapidly. This is not so much the case with the granular solids found in turbid river waters. The process of flocculation explains the efficiency of some of the deep basins used in sewage treatment and of artificial aggregation by the addition of electrolytes or chemical coagulants in sedimentation combined with *chemical precipitation.* Salt water, too, induces flocculation. The rapidity of sedimentation observed when sewage is discharged into tidal water must be ascribed to this fact; for it cannot be supposed that an individual particle would settle more rapidly in salt water than in fresh water of lower density and viscosity.

Detention Period.—The time of sedimentation a is commonly called the detention period. In practice it is generally expressed in hours. If, for example, the rate of sewage flow is 2.4 m.g.d. and the tank capacity is 100,000 gal., the detention period is $\frac{0.1 \times 24}{2.4} = 1$ hr. The *flowing-through period*, on the other hand, is the time actually required for a unit volume of sewage to pass from inlet to outlet. Theoretically the detention period and the flowing-through period are one and the same measure. Actually they vary. Displacement of the sewage in the tank by the incoming sewage is seldom uniform, depending upon the distribution of the incoming sewage, the velocities in the tank, the method of baffling and the way in which the effluent is withdrawn. As a result part of the sewage reaches the outlet more rapidly than

[1] If l = length of basin and V = mean velocity,

$$V = \frac{l}{a};\ v = \frac{h}{t};\ \frac{l}{h} = \frac{aV}{tv}$$

does the remainder. The percentage ratio of the flowing-through period to the detention period is, therefore, a measure of the efficiency of distribution of the sewage. The best results are obtained where each unit of sewage flows through the basin with the least mixing with preceding and succeeding units.

The flowing-through period is readily determined by adding a solution of commercial salt to the inflowing sewage and titrating the effluent to determine the quantity of chlorides it contains. After the salt first reaches the effluent there will be a gradual increase in chlorides until a maximum is reached, after which there will be a gradual decrease. A typical curve obtained by Capen (3) for a tank at Phillipsburg, N. J., is shown in Fig. 78. The flowing-through period is fixed by the location of the vertical gravity axis of the area under the curve. In the case of

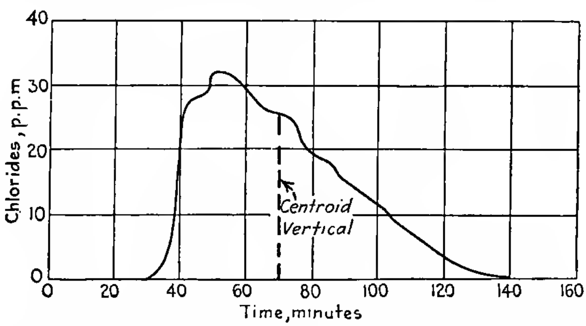


FIG. 78.—Flowing-through period of sedimentation tank at Phillipsburg, N. J.

Phillipsburg the flowing-through period was found to be 1.17 hr. against a detention period of 2.42 hr. The flowing-through period was therefore 48 per cent of the detention period. It should be noted that the peak concentration of chlorides occurred before the flowing-through period was completed and, since this peak commonly coincides with the maximum color produced by dyes added to the sewage, it is evident that the use of dyes to determine the flowing-through period is not reliable. Electrical resistance measurements can be substituted for chloride titration.

Currents in Tanks.—Besides the direct effect of the temperature on the viscosity of the water and consequent hydraulic subsiding value of small particles, changes of temperature produce disturbances interfering with the calculated velocities, which are discussed in a paper by Seddon (4).

Dunbar (5) says, in his "Principles of Sewage Treatment:"

The direction of the sewage currents at various depths in the tanks has been investigated by Bock and Schwarz. They employed small glass bottles, and sank these to depths varying from 1 to 6 ft. Their results showed that the sewage moved sometimes upward, sometimes downward, and sometimes toward the sides of the tanks, with a velocity two to three times as great as the calculated average velocity through the tank. These sources of error were later demonstrated in a very clear manner by Schmidt at Oppeln. By addition of a coloring matter (uranin) he showed that at the cooler periods of the year the warm sewage flowed on the top of the cooler contents of the tanks. Variations in temperature cause variations in the flow of the sewage, as depicted in Fig. 79. The dotted lines show the direction taken by the entering sewage, according as it is warmer or colder than the contents of the tank.

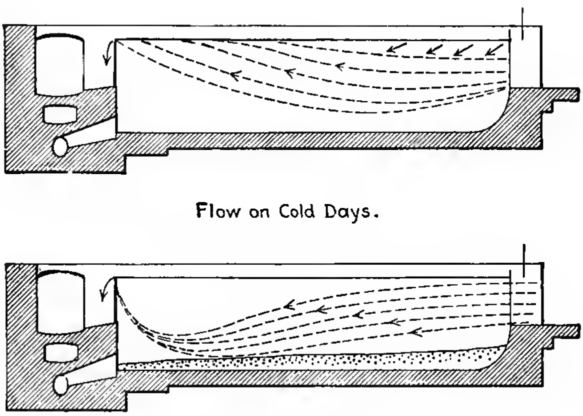


FIG. 79.—Effect of temperature on currents in tanks.

Johnson (6) states that stratification is a factor of slight importance.

Summary.—With present knowledge of the sedimentation characteristics of sewage solids, Hazen's theory can find only limited application. It holds in particular for the settling of heavy, gritty substances but is useful in other problems of sedimentation by showing that: within limits the area of a settling basin is more important than its depth; values of a/t of 1.5 or more should be obtained to secure good basin efficiency; the ratio of length to depth should be limited; good baffling is of prime importance and will increase the efficiency by about 25 per cent; temperature is of great significance; and wind action should be reduced to a minimum.

PLAIN-SEDIMENTATION TANKS

Sedimentation may be effected in either fill-and-draw or continuous-flow tanks. Discussion of the relative merits of these methods of take operation is deferred to a subsequent section. The fill-and-draw process

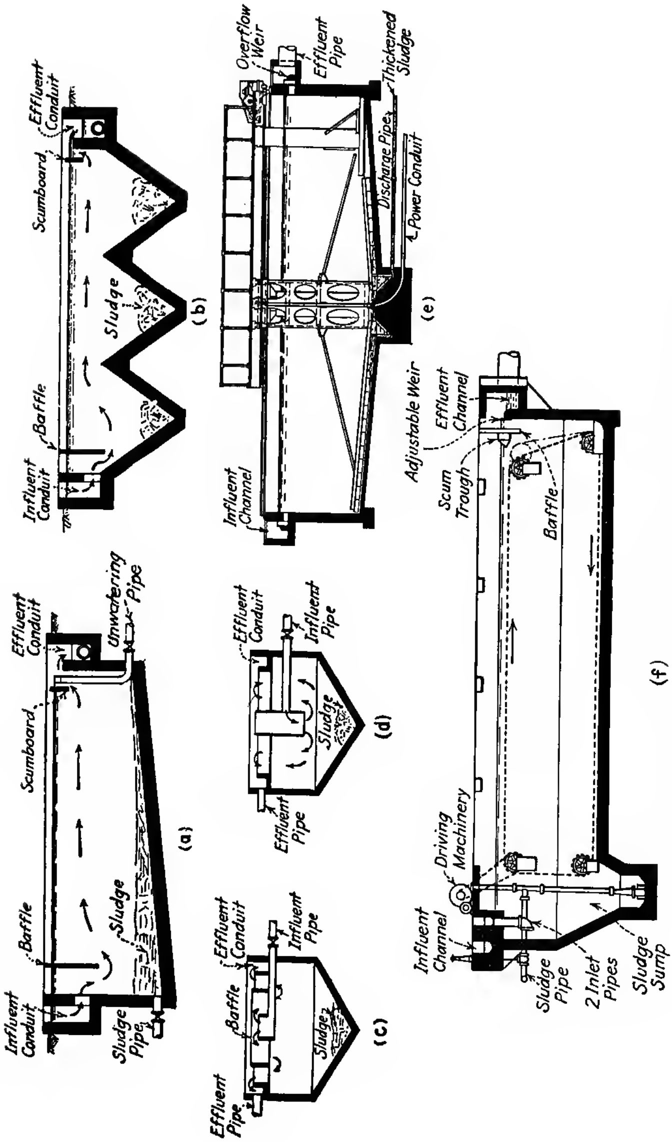


FIG. 80.—Various types of sedimentation tanks for sewage.

is seldom used. The types of tanks employed in the continuous-flow process may be classified as follows:

A. According to the direction of flow
  1. Horizontal-flow tanks
     a. With continuous longitudinal displacement of the flowing sewage from the influent to the effluent end, illustrated by Fig. 80*a*, *b*, *e* and *f*
     b. With continuous radial displacement of the flowing sewage from a central inlet to a peripheral outlet, illustrated by Fig. 80*c*
  2. Vertical-flow tanks with continuous vertical, and to a slight extent radial, displacement of the flowing sewage from a central downward directed inlet to a peripheral outlet, *Dortmund* tanks, Fig. 80*d*

B. According to the method of sludge collection and removal
  1. Flat-bottom tanks which must be emptied when the sludge is to be removed, shown in Fig. 80*a*
  2. Hopper-bottom tanks in which the sludge is withdrawn from the apex of the hopper, commonly during operation and under the influence of the hydrostatic head on the sludge. Tanks of this type are shown in Fig. 80*b*, *c* and *d*
  3. Flat-bottom tanks provided with mechanical cleaning devices, in the form of scrapers or plows which move the sludge to a collecting point whence it is generally withdrawn as under *B*2. Tanks of this type are illustrated in Fig. 80*e* and *f*

C. According to the function and position of the units in the treatment works
  1. Preliminary, primary, or presedimentation units, which precede oxidation processes or disposal without oxidation treatment
  2. Final or secondary-sedimentation units, which follow oxidation treatment. Succeeding trickling filters, these are known as humus tanks

The principles of operation and design of plain-sedimentation tanks will be treated in this chapter. Where plain-sedimentation tanks are utilized as final-sedimentation tanks following either trickling filters or aeration tanks in the activated-sludge process, further discussion is included in the chapter dealing with the particular process.

Mechanically cleaned settling tanks are also known by the type of mechanical equipment employed, such as Dorr, Link-Belt, Hardinge, Buchart and Mieder.

Fill-and-draw and Continuous Operation.—Formerly many plants were operated on the fill-and-draw method. Dunbar (5) states the disadvantages of this method as follows:

Theoretically, intermittent action in which the sewage is allowed to come to rest is more efficacious than continuous action in which the sewage is allowed to flow continuously through the tanks. Continuous action has many practical advantages over the intermittent method of working. At each emptying and filling of the tank there is danger of stirring up the sludge which should, therefore, be removed each time the tank is emptied. . . Intermittent action also causes a loss in available head of the sewage equal to the height in the tank, and the time of filling and emptying are not utilized in the purification process.

Nearly all the suspended matter which will settle out in a reasonable length of time when the sewage is held quiescent will also settle out when the sewage is allowed to flow slowly through the tank. Furthermore, it is practically impossible to hold sewage absolutely quiescent in large tanks, because of wind action, oscillations of the mass of sewage in the tanks and currents set up by differences of temperature of the sewage in different parts of the tanks. There is also generally considerable delay due to cleaning the tanks between emptying and filling, especially where they are of the shallow, horizontal-flow type, and if the sludge is not removed each time before refilling, it is mixed with the incoming sewage and tends to hasten decomposition, which may be objectionable because of the odors produced about the plant.

In the continuous-flow method, the sewage may leave the tank in a comparatively short time without having taken up dissolved matter present in the sludge and in some cases may contain some dissolved oxygen, an assurance that exceedingly offensive odors are not being given off.

The development of mechanical equipment ror removing sludge without emptying the tank has made the continuous-flow tank still more advantageous, so that it is now almost universal practice to allow sewage to flow continuously through sedimentation tanks, the sedimentation efficiency being substantially as good as by the intermittent system and the operation of the tanks being much more convenient and economical. Fill-and-draw operation, however, may still find useful application in the treatment of industrial wastes.

Direction of Flow.—Among the different types of plain-sedimentation tanks, *horizontal-flow* tanks with longitudinal displacement of the sewage are in most common use. They are employed for both primary- and final-sedimentation units and are commonly equipped with mechanical sludge-removal apparatus. *Vertical-flow* tanks have been used to some extent in this country for final-sedimentation tanks following trickling filters. In England they are used extensively as final tanks in the activated-sludge process. *Radial flow* is not employed much for primary sedimentation, but more for final sedimentation, as at the Easterly Plant in Cleveland, at Indianapolis and elsewhere.

Formerly plain-sedimentation tanks in some plants were arranged in series to provide a long route through which the sewage had to travel. A prolonged period of sedimentation was generally provided in such plants. The tanks are now generally arranged for operation in parallel.

Tark (7) states that

rectangular vertical-flow tanks give a slightly higher percentage of removal [of suspended solids] than the horizontal-flow tanks, but their cost of construction is higher and they can only be built in small units. They are therefore suitable only for small or medium-sized plants.

HORIZONTAL-FLOW TANKS

In the order of their development, horizontal-flow tanks may be classified as flat-bottom tanks, hopper-bottom tanks and tanks with sludge-removing mechanism. The distinguishing feature of these tanks is the method of sludge collection and removal.

The older type of horizontal-flow tank is commonly a shallow rectangular structure. The bottom rises toward the outlet and slopes transversely toward one or more drains running longitudinally through the tank which must be emptied for cleaning. A draw-off pipe or floating arm and valve are provided to remove the supernatant liquid, as indicated in Fig. 80*a* and Fig. 83. If this pipe or valve is situated at the outlet end, the greatest possible quantity of water will be withdrawn. Sludge sluices are provided at the deep end, where the heavier and greater part of the settling solids subside if the velocities are not too high. Part of the sludge moves to the sludge sluices by gravity; the remainder must be pushed down the sloping bottom by means of wooden scrapers and squeegees. Gates in the side walls of the tank are sometimes used to introduce flushing water from adjoining tanks and thus to supplement the work of the scrapers. Hose connections on water-supply lines may also be employed for this purpose.

To eliminate hand cleaning, hopper-bottom tanks were developed. In these tanks the bottom is shaped into a series of hoppers into which the sludge settles. The slopes of the hoppers are sufficiently great to permit the settling solids to slide to the bottom, where the sludge outlets are situated. The sludge is withdrawn from time to time either by gravity or by pumping. Two different arrangements, illustrated in Fig. 84, are commonly used; vertical or inclined riser pipes ending near the bottom of the hoppers and horizontal sludge pipes connected to the apices of the hoppers. Sludge may be withdrawn during operation.

The latest development in plain-sedimentation tanks is the provision of mechanical sludge-removing equipment. Several types of mechanism are now in use: the Dorr mechanism, Fig. 80*e*; the Fidler mechanism, Fig. 85; the Link-Belt mechanism, Fig. 80*f*; and the Mieder mechanism, Fig. 86. Dorr mechanisms have been designed for square or circular tanks and for square tanks with circular bottoms. Mechanisms of the Fidler type may be used either in circular tanks or in square tanks with circular bottoms. Mechanisms of the Link-Belt and Mieder types have been developed for use with rectangular tanks. Plows or scrapers are employed to move the deposited sludge toward the sludge outlet, whence it is withdrawn by gravity or pumping. Sludge may be removed continuously or intermittently, without placing the tank out of operation.

Sludge-removal mechanisms are employed chiefly in connection with primary sedimentation, the sludge being removed to separate digestion tanks, and final sedimentation of the effluent from activated-sludge aeration tanks, the sludge being returned to the process or otherwise disposed of. In both cases it is commonly desired to remove the sludge before septic decomposition sets in. Also, recently, they have been commonly employed for the removal of sludge from final-sedimentation tanks following trickling filters. They are used at Dearborn, Mich., for the removal of sludge from chemical-precipitation tanks.

Elements of Design.—The design of horizontal-flow sedimentation tanks is governed by the quantity and strength of sewage to be handled, the desired surface area, velocity and detention period, the method of sludge removal, and, where provided, the type of mechanical equipment for sludge removal. Among the elements to be considered are: the number and arrangement of units; tank dimensions, length, width and depth; arrangement of inlets and outlets with possible baffling; possible skimming equipment; provision for sludge storage and removal; and freeboard. These items are considered in detail in subsequent paragraphs.

Tark (7) summarizes the essential features to be observed in the design of a tank as follows:

1. Means for decreasing the velocity of flow so that the solids can settle out.
2. A zone of quietness, where the settled solids can accumulate.
3. Means for removing the accumulated solids or sludge.

Number and Arrangement of Tanks.—It is customary even in small plants to build duplicate settling tanks, so that there may be no complete interruption in the treatment of the sewage. In larger plants the number of units provided depends upon structural considerations, operating conditions and facilities for removing sludge. Often several relatively small or narrow tanks are provided, rather than one or two large or wide tanks, to minimize the effect of wind and to aid in establishing uniform rates of flow throughout the width of the tanks. The number of tanks is usually sufficient to afford the desired width of unit and flexibility of operation by cutting in or out one or more units.

Detention Periods.—The detention periods employed in the design of horizontal-flow sedimentation tanks vary according to the requirements for which the tanks are provided, the efficiency of sedimentation required and the character of the sewage treated.

The sedimentation of settling solids is most rapid during the first hour and an increase in detention period beyond 2 hr. does not ordinarily effect a material further removal of solids. The results of experiments at Syracuse, shown in Fig. 81, indicate how little may be accomplished by way of sedimentation during prolonged detention periods (8).

Where primary-sedimentation tanks are provided prior to activated-sludge treatment, the detention period ordinarily varies from ½ to 1½ hr. Preceding trickling filters the detention period ordinarily provided is 1½ to 6 hr., the average being about 3 hr. in earlier installations and 2 to 2½ hr. in more recent plants. Sedimentation tanks following aeration units in the activated-sludge process usually provide

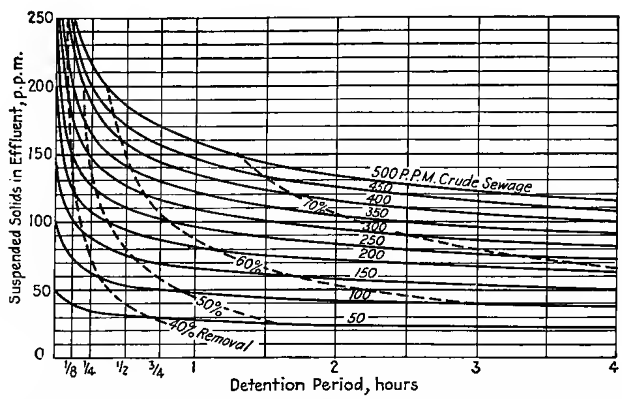


FIG. 81.—Effect of detention period on removal of suspended solids in sedimentation tank at Syracuse, N. Y.

a detention period of 1½ to 2 hr. Following trickling filters, final-sedimentation tanks generally have a detention period of 1 to 1½ hr.

For most small or medium-sized plants, the detention period used as a basis for design takes into consideration available data on the strength of sewage and the desired removal of solids. For the design of larger plants the quantity of suspended matter precipitated in various periods of time is often ascertained and that period of sedimentation is chosen which will cause the removal of the largest quantity required by the conditions.

The portion of the tank reserved for sludge storage is not commonly included in the calculation of effective detention period. In hopper-bottom tanks, for example, the volume of the hoppers is not included in the detention capacity.

Detention periods are commonly based upon average dry-weather flows. In combined systems 1.5 to 3 times such flows are commonly handled; excess quantities of storm flow are generally by-passed, but in some cases separate storm-water tanks are provided. Since fluctuations in sewage flow are more pronounced in small plants than in large ones, detention periods for small plants are sometimes 50 to 100 per cent greater than for large ones.

Limiting Velocities.—It is apparent from a consideration of the theory of sedimentation that the velocities ordinarily encountered in sedimentation tanks are of small moment as long as they are not sufficiently great to lift particles from the bottom or to move them along the bottom toward the outlet. Where sludge is in contact with the flowing sewage, lower limiting velocities are required than where the solids settle out of the sedimentation compartment into a lower sludge compartment. Velocities of 45 to 150 ft. per hour are common in the case of preliminary-sedimentation tanks and humus tanks. On the other hand, final settling tanks in the activated-sludge process often provide for velocities of only 30 to 40 ft. per hour.

Area.—The tank area, as pointed out in previous sections of this chapter, is an important consideration. For granular solids, for which area is the controlling factor, Imhoff (9) has suggested a value of 1 sq. ft. for each 300 gal. daily. For sewage solids and depths greater than 5 ft. he considers that 1 sq. ft. for each 600 gal. daily will suffice, because sedimentation is due in part to the aggregation of the flocculent solids into masses of larger bulk, while they settle through the sewage. For final-sedimentation tanks in the activated-sludge process, values of 800 to 1000 gal. of tank effluent daily per square foot of surface area are common.

Tank Dimensions.—The tank dimensions to be considered are width, length and depth, which are frequently limited by local subsoil conditions, especially by ground water. These conditions affect the depth directly and the other tank dimensions indirectly. Construction of deep tanks may not be economical unless the ground-water table is low and ledge is not encountered near the surface.

For longitudinal displacement of the sewage it is customary to build tanks from 6 to 16 ft. in depth. Moreover, unless tanks are made round or square, in order to facilitate the removal of sludge by certain types of mechanical equipment, they are usually built with a ratio of width to length ranging from 1:2 to 1:8. 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 tank. The desired width may be secured either by constructing a number of parallel tanks or by providing longitudinal baffle walls. In large tanks uniformity of flow is affected by winds and by differences in the 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 land area available for the tanks.

In present designs the tank dimensions are governed somewhat by the type of mechanical equipment to be provided. As previously stated, mechanisms of the Link-Belt and Mieder types are adapted to rectangular tanks. The plain-sedimentation tanks of the Joint Meeting sewage-treatment plant in Elizabeth, N. J., equipped with Mieder apparatus, are 75 ft. wide, 280 ft. long and have an average depth below maximum high water of 13.85 ft. Mechanisms of the Link-Belt type have been installed in tanks varying in width from 8 to 68 ft. and in length from 30 to 179 ft. The two preliminary-sedimentation tanks at Barrington, N. J., are 8 ft. wide, 32 ft. long and 7 ft. deep. The final-sedimentation tanks at the Wards Island plant in New York are 42.5 ft. wide, 179 ft. long and 12.5 ft. deep.

The following examples are given to illustrate the range in size of tanks to which revolving sludge-removal mechanisms have been adapted. At Kiel, Wis., there is a primary-sedimentation tank 20 ft. square. At Aurora, Ill., the four sedimentation tanks preceding trickling filters are 50 ft. square and the side-wall water depth is 12 ft. Eight primary-sedimentation tanks, 90 ft. square, with a side-wall water depth of 10 ft., are provided at Toledo, Ohio. The eight preliminary-sedimentation tanks at the Wards Island plant are 98.5 ft. square. At Baltimore, Md., there are two new preliminary-sedimentation tanks, each 140 ft. square, with a side-wall water depth of 15.5 ft.

The characteristics of a number of plain-sedimentation tanks are given in Table 64 and those of several final tanks in the activated-sludge process are shown in Tables 109 and 113.

Inlets and Outlets.—The most obvious way to induce relatively uniform distribution of the sewage entering and leaving horizontal-flow tanks is by means of inlet and outlet weirs extending entirely across the ends of the basin. However, 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 or providing gated openings in the side of the influent channel, but these arrangements are generally unsatisfactory. The prevailing 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 suitable baffling devices placed in front of and close to the influent openings. Figure 82 illustrates such a device installed in a covered sedimentation tank. It is equally applicable to tanks with straight walls.

Care is required in the design of inlet and outlet channels to insure equal distribution of sewage and suspended matter among the different tanks. At Baltimore, thin longitudinal walls are provided in the inlet channel and the individual channels thus created lead to separate groups of tanks. At Boonton, N. J., the inlet chamber provides a small stilling chamber with 12 weirs. Each weir connects with a separate pipe, leading to one of the 12 settling tanks.

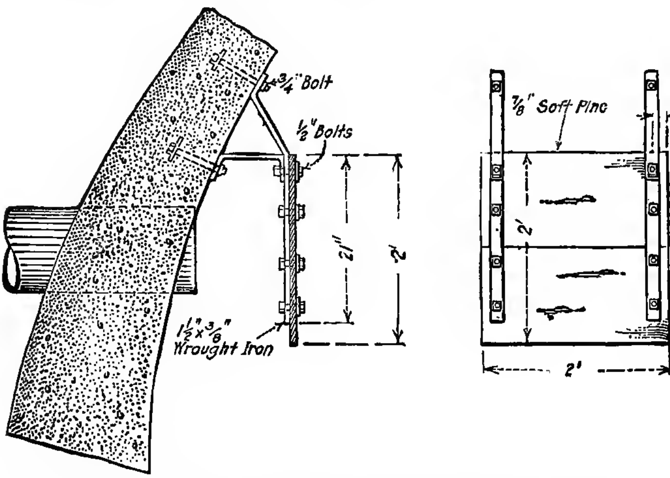


Fig. 82.—Baffle plate in front of inlet pipe of sedimentation tank.

At the Joint Meeting sewage-treatment works in Elizabeth N. J., the flow to each of the four sedimentation tanks will be regulated by motor-operated sluice gates. Sewage flows through the influent flumes into individual distributing channels, each controlled by motor-operated gates, and then through open ports and through a perforated diffusion wall designed to distribute the flow entering the tanks. The diffusion wall is perforated with holes, 8 in. in diameter, spaced 10 3/8 in. center to center. Steel-plate baffles, having a submergence of 8 in. at minimum water level, are located at the influent end of the tanks and the effluent weirs have a net width of 54 ft. in each tank.

In the Dorr "Sifeed" clarifier, described later in this chapter, the influent is brought under the tank to a central feed pipe, from which it is diffused beneath the surface through a number of ports and by means of a perforated circular baffle. The flow is radial and the effluent discharges over a peripheral weir.

At Lancaster, Pa., and San Antonio, Tex., equal distribution of sewage and suspended matter among the preliminary-sedimentation tanks is promoted by the diffusion of air from porous plates set in the distribution channels.

Scum Boards and Baffles.—To facilitate the collection of floating matter on plain-sedimentation tanks and prevent its passage over the effluent weirs, scum boards may be provided. These may take the form of either relatively shallow baffles, which are fixed in position and rise above the sewage surface, or floating boards, which rise and fall with changes in the sewage level. Scum boards are commonly placed near the outlets and sometimes also near the inlets.

Where inlet weirs are used they are often guarded by scum boards 2 to 4 in. in front of them, or more in some cases, and extending to a depth of at least 2 ft. The narrow openings between boards and weirs are easily kept free from scum and the sewage is compelled by

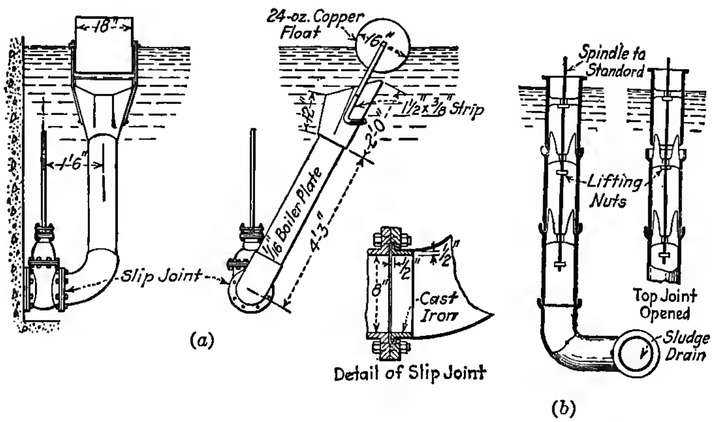


FIG. 83.—Equipment for unwatering flat-bottom tanks.

the depth of the boards to take the same course as it would follow with submerged openings.

Long plain-sedimentation tanks are sometimes equipped with transverse baffles, placed at equidistant points throughout the length of the tanks, in order to foster continued good distribution of the sewage as it flows through the tanks and to minimize the disturbances due to wind action, vortex motion and other adverse conditions. Over-and-under baffles have been employed. Increasing the rate of flow by too much baffling is likely to result in agitating the sludge and mixing it with the sewage. Capen (3) states that baffles can be helpful but are more often detrimental than useful.

On the other hand, tests made by Stevenson (10) on baffled and unbaffled tanks at the Philadelphia experiment station have shown that baffled tanks are markedly superior. On this subject the Bureau of Surveys reports as follows:

Another matter not shown in the analyses, but noted during the operation of all the tanks, was that in unbaffled tanks the presence of visible trade wastes at the influent end was soon followed by the same waste at the outlet, showing a current of high velocity through quiet portions of the tank, whereas after proper baffling of the tanks the entire cross section of the tank was placed in service.

At Boonton, N. J., surface baffles are provided at both inlet and outlet ends of the sedimentation tanks. Sewage enters each tank through two vertical 12-in. pipes with quarter bends pointing toward the end of the tank against the direction of flow. Two weirs at the outlet end collect the flow over the entire width of the tank. By raising or lowering these weirs with stop planks, the sewage level in the tanks may be maintained either above or below the under surface of the roof. By submerging the roof, scum drying is prevented.

Bottom Slopes.—The bottom slopes of sedimentation tanks differ widely, depending upon the character of sludge to be expected as well as upon the method of sludge removal for which the tanks are designed. In flat-bottom tanks, sludge free from grit can be moved with scrapers and squeegees without difficulty on slopes of 1.5 to 3 per cent. 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 may be used in cleaning adjoining tanks. Tanks with mechanical sludge removal are generally given bottom slopes of 8 to 15 per cent, except for Link-Belt and Mieder collectors, where a slope longitudinally of 1 per cent is usual.

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 1.75 vertical on 1 horizontal, or better 2 on 1, in order to insure the feeding of sludge into the sludge pipes while it is being drawn. Sludge which contains gelatinous floc, often called colloidal matter by plant operators, has a tendency at times to adhere to smooth surfaces, as well as to those of a rougher character, and wherever two surfaces form an angle the sludge tends to remain. It must be remembered that the slope of the valley at the intersection of two inclined surfaces is less than that of the surfaces themselves. 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 submerged sludge in causing motion is less, by the weight of the displaced sewage, than that of exposed sludge. As a result, in drawing sludge from tanks operated continuously, there is 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 formed by steep sides in such a way that the sludge is inevitably concentrated in it.

To avoid excessive depths in large hoppers, the slope is reduced commonly to 1 vertical on 1 horizontal, or even to 1 vertical on 2 horizontal.

Volume Variations of Sewage Sludge.—The volume of sludge depends mainly upon its water content and only slightly upon the character of the solid matter. The water content is commonly expressed by weight. A "90 per cent sludge," for example, contains 90 per cent of water by weight. The solid particles, in the interstices of which the water is held, vary in composition, shape and size. If one third is composed of mineral matter with specific gravity 2.0 and two thirds of organic matter with specific gravity 1.0, the specific gravity of the whole s is found as follows: $\frac{1}{s} = \frac{1}{3} \div 2 + \frac{2}{3} \div 1 = \frac{5}{6}$; $s = 1.2$. If, furthermore, the specific gravity of the water is taken as 1.0, as it can be without appreciable error, the specific gravity of a 90 per cent sludge of this character s_1 is found thus: $\frac{s_1}{1} = \frac{0.1}{1.2} + \frac{0.9}{1} = 0.983$; $s_1 = 1.017$. This value is a common one for average sewage sludge from strictly separate systems. If one half of the sludge is composed of mineral matter and the other half of organic matter, computations similar to the preceding lead to a specific gravity for the solids of 1.33 and for a 90 per cent sludge of 1.025. The latter is a common value for sludge derived from combined sewage and storm water.

The volume of a 90 per cent sludge of specific gravity 1.017 containing 1 lb. of solid matter is $\frac{1}{8.33}\left(\frac{1}{1.2} + 9\right) = 1.18$ gal. If, however, the water content were 95 per cent, the volume of the sludge would be $\frac{1}{8.33}\left(\frac{1}{1.2} + 19\right) = 2.38$ gal., or slightly more than twice as much as that of a 90 per cent sludge. If the sludge composition is considered in terms of solid matter rather than water content, this fact is more apparent. A 90 per cent sludge contains 10 per cent of solid matter, whereas a 95 per cent sludge contains only 5 per cent. The volume of a 95 per cent sludge is therefore approximately twice as great as that of a 90 per cent sludge. If V_1 and V_2 are the volumes of the sludge, P_1 and P_2 the percentages of water, and p_1 and p_2 the percentages of solid matter, the following relationships hold approximately:[1]

$$\frac{V_1}{V_2} = \frac{100 - P_2}{100 - P_1} = \frac{p_2}{p_1}$$

If, for example, V_1 is the volume of the 90 per cent sludge and V_2 that of the 95 per cent sludge:

[1] Let s = density of the solids
w = weight of the solids, grams
W = weight of water, grams

Then

$$\frac{V_1}{V_2} = \frac{100 - 95}{100 - 90} = \frac{5}{10} = 0.5$$

For approximate calculations it is simple to remember that the volume varies inversely as the percentage of solid matter contained in the sludge.

The moisture content of sludge from plain-sedimentation tanks at a number of plants is given in Table 63. At these plants primary sludge has a water content of 94 to 96 per cent and, where excess activated sludge is discharged into the preliminary tanks and withdrawn with the primary sludge, the moisture is about 96 per cent.

TABLE 63.—MOISTURE IN SLUDGE FROM PLAIN-SEDIMENTATION TANKS

Plant	Period	Moisture in sludge, per cent
Primary sludge:		
Baltimore, Md.	1926–30	94.7
Grand Rapids, Mich.	1932–33	95.9
High Point, N. C.	1929–30	93.8
Average	....	94.8
Combined primary sludge and excess activated sludge:		
Elyria, Ohio	1931	96.1
Peoria, Ill.	1932	96.5
San Antonio, Tex.	1931	95.7
Springfield, Ill.	1931–32	96.1
Toronto, Ont., North Toronto plant	1930–32	95.7
Average	....	96.0

Sludge Storage and Volume of Sludge Produced.—The volume of sludge for which provision is made in horizontal-flow sedimentation

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$$V = \frac{w}{s} + W$$

$$P = 100\frac{W}{w + W}$$

$$W = \frac{Pw}{100 - P}$$

$$V = \frac{w}{s} + \frac{Pw}{100 - P}$$

$$V = \frac{w}{s}\left[\frac{100 + P(s - 1)}{100 - P}\right]$$

$$\frac{V_1}{V_2} = \frac{[100 + P_1(s - 1)](100 - P_2)}{[100 + P_2(s - 1)](100 - P_1)}$$

$$\frac{V_1}{V_2} = \frac{100 - P_2}{100 - P_1} = \frac{p_2}{p_1} \text{ approximately}$$

tanks depends upon the following factors: the character of the sewage received by the tanks—its strength, freshness and preliminary treatment; the period of sedimentation and the degree of purification to be effected in the tanks; the condition of the deposited solids—their specific gravity, water content and changes in volume under the influence of tank depth or mechanical sludge-removal devices; and the period of sludge storage between drawing operations The following discussion shows how these factors enter into the problem of calculating the required storage capacity:

In the hypothetical analysis of a typical sewage of medium strength (Fig. 1) it was assumed that the suspended solids amount to 300 p.p.m., of which 150 p.p.m. are capable of settling in tanks affording a 2-hr. detention period. Still larger portions are capable of settling in longer periods.

Primary-sedimentation tanks yield so-called *fresh* sludge. The specific gravity of the deposited solids varies with the type of the sewerage system and the strength of the sewage about as follows:

Type of sewerage system	Strength of sewage	Specific gravity
Strictly domestic....................	Weak[1]	1.02
Strictly separate....................	Average	1.03
Combined..........................	Average	1.05
Combined..........................	Strong	1.07

[1] Due to infiltration or high water consumption.

On the assumption that 150 p.p.m. of suspended matter are capable of settling in 2 hr., as an average, in accordance with the preceding statement, the weight of the dry solids deposited per million gallons is $150 \times 8.33 = 1250$ lb. If the sludge contains 95 per cent water and its specific gravity is 1.03, the volume will be $\frac{1250 \times 100}{8.33 \times 1.03 \times 5} = 2910$ gal. of sludge per mil. gal. of sewage. On the basis of a daily per capita sewage flow of 100 gal., the sludge storage capacity must be 38 cu. ft. per 1000 population for each day elapsing between sludge removals. These calculations give also the volume of sludge that must be digested, dewatered or disposed of, as the case may be.

The foregoing calculation is directly applicable to the design of storage facilities for fresh sludge in primary-sedimentation tanks. In preliminary-sedimentation tanks of activated-sludge plants, provision may be required for the excess activated sludge which may be discharged into the influent of the preliminary tanks for settlement and consolidation with the fresh sludge. Based on a removal of 95 per cent of the suspended solids by the complete treatment plant, the weight of solids, on a dry basis, deposited in the preliminary-sedimentation tanks is $0.95 \times 300 \times 8.33 = 2380$ lb. per mil. gal. of sewage. If the sludge contains 96 per cent moisture and its specific gravity is 1.03, the volume will be $\frac{2380 \times 100}{8.33 \times 1.03 \times 4} = 6940$ gal. per mil. gal. of sewage.

For humus tanks provision may be required for the "unloading" of trickling filters and for the storage of sludge for longer periods than ordinarily employed in primary-sedimentation tanks, unless mechanical equipment for sludge removal is provided. Where the sedimentation units are to be used as final-sedimentation tanks in the activated-sludge process, provision must be made for light, flocculent sludge of 98 to 99.5 per cent moisture and for quantities of sludge ranging from 2500

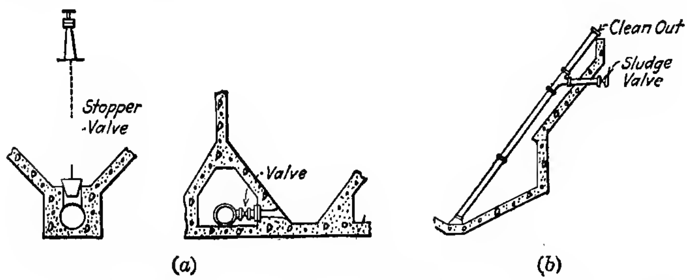


FIG. 84.—Details of piping for drawing sludge from hopper-bottom tanks.

to 7000 p.p.m. in the influent mixed liquor. Further consideration of the requirements for final-sedimentation tanks following trickling filters and aeration units is included in the chapters dealing with these oxidation processes.

Sludge Removal.—Methods of sludge removal vary with the different types of tanks and the means of sludge disposal. As drawn from settling tanks, sewage sludge will flow readily through pipes and open channels and can be pumped or caused to flow by gravity. The method of sludge removal from flat-bottom tanks has been described previously.

Hopper-bottom Tanks.—Sludge is commonly withdrawn from hopper-bottom tanks during operation. The sludge is drawn through pipes reaching from above through the tank to the bottom of the hoppers, as in Fig. 84*b*, or attached to them from below, as in Fig. 84*a*. On account of the hydrostatic head operative on the sludge inlets, the sludge can be discharged at elevations lower than the flow line of the tank, allowances being made for the losses of head at entrance and the friction losses in the pipes. The minimum diameter of sludge pipes is usually 8 in., in order to avoid clogging. If the sludge is discharged through a branch at the top of the sludge pipe, the outlet of the branch commonly is 4 to 6 ft. below the level of the sewage in the tank. If the sludge flows through greater lengths of pipe, a slope in the hydraulic grade line of 2 to 3 per cent is often provided.

Sludge is usually withdrawn from tanks at a slow rate, in order that the whole mass of sludge may settle and no conical depression may exist in the center. In the latter case sewage would be withdrawn instead of sludge. This would be disadvantageous in most cases, as it is usually desirable to secure as concentrated a sludge as possible and thereby reduce the burden on sludge-disposal facilities. Where gravity flow cannot be secured, the sludge is pumped.

Mechanical Removal of Sludge and Scum.—Sludge-removal mechanisms move the deposited solids to the sludge outlets during operation of the tanks. Such mechanisms may be operated either continuously or intermittently.

The Dorr mechanism (Fig. 80*e*) consists essentially of a group of arms, to which blades are attached in such manner that, as the arms revolve, the sludge is moved to a central draw-off sump. In one of the earlier forms, designed for circular-bottom tanks, the arms are attached to a central, motor-driven shaft, the motor being stationary and the mechanism being supported by an overhead bridge spanning the tank. In a later development, designed for square-bottom tanks, counterweighted swinging arms are attached to two of the four revolving arms in each tank. As the mechanism revolves, the swinging arms extend out into the corners and scrape the sludge into the circular zone. In a further development, known as the *traction clarifier,* the mechanism is supported by a bridge extending from a central fixed column to the top of the tank wall. The bridge and mechanism are revolved by motor-driven wheels resting on rails set on the tank wall. When installed in square tanks, a telescoping bridge is provided, which extends out to the corners as the mechanism revolves. The telescoping section is equipped with plows which scrape the sludge from the corners into the circular zone. In another design the mechanism is supported by a central column and revolved by a motor set on a turntable over the column. In conjunction with this latest development and with the traction clarifier, provision may be made for bringing the sewage under the tank to a central feed pipe within the column. This type of tank is known as the "Sifeed" clarifier. Means for the automatic skimming of sedimentation tanks have been developed in conjunction with the traction type of mechanism.

Scraper-conveyor mechanisms of the Link-Belt type (Fig. 80*f*) consist of a series of scrapers attached to endless chains. The chains are driven by a motor through a system of gears and sprockets, so as to drag the scrapers toward sumps or hoppers, which are usually placed at the influent end of the tanks. Where skimming is desired, as on the surface of preliminary-sedimentation tanks, the return flights of the scrapers are carried at the water surface in such manner as to accumulate the scum at one end of the tanks. When skimming is not desired, the return flights are carried back beneath the water surface. The width of a single flight is seldom more than 16 ft. For a tank wider than that, two or more series of flights may be provided. In such a case the sludge may be scraped to a trough at one end by longitudinal flights and cross flights may be employed in the trough to scrape the sludge to a hopper or sump at one corner of the tank.

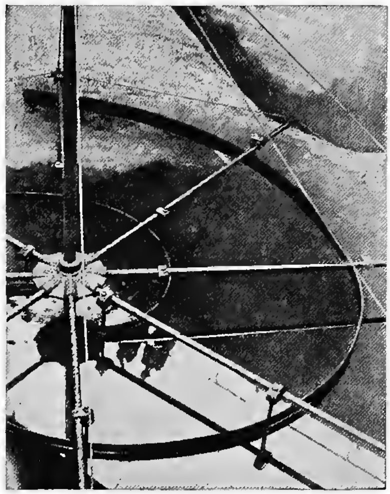


FIG. 85.—Fidler mechanism for sludge removal.

The Fidler type of mechanism (Fig. 85) consists of one or two spiral blades attached to a central motor-driven shaft, the motor being stationary. As the blades revolve, the sludge is moved to the center of circular-bottom tanks. The Buchart and Hardinge spiral sludge-removal mechanisms are similar to the Fidler type.

The Mieder mechanism (Fig. 86) consists of a traveling bridge carried on rails along the longitudinal walls of the tank, with a scraper attached to the bridge by hinged arms. The scraper is used to force the sludge into hoppers at the influent end of the tank and, when raised to the surface of the sewage, it is used to skim the tank. A mechanism similar to the Mieder is manufactured by the Hardinge Co.

The Mieder installation at Leipzig, Germany, with a 32.8-ft. span operates on about 2 hp. at low speed. The Dorr mechanisms at the North Toronto plant, Toronto, Ont., operating in 50-ft. circular-bottom preliminary-sedimentation tanks, take on the average about 1.2 hp. per unit. The Fidler mechanisms at the same plant in 65-ft. circular-bottom activated-sludge, final-sedimentation tanks, also operate at about 1.2 hp. per unit. Dorr mechanisms commonly are driven by 2.5- or 3-hp. motors.

Concerning mechanically equipped tanks, Tark (7) states:

The same underlying principles of design apply to these tanks as to other types. The removal of the sludge must be positive and take place with a minimum of agitation. Therefore, the speed should be uniform and low—not more than 5 ft. per minute and preferably 3 ft. per minute. Intermittent operation once a day in cool weather, twice a day in warm weather, will give much better results than continuous operation. Even slight agitation will cause the fine settled solids to go into suspension again. Tests have shown that a mechanical speed of 5 ft. per minute for a period of 30 min. caused a loss of efficiency of 10 per cent for 90 min.

The above quotation refers to primary-sedimentation tanks. In settling activated sludge, continuous removal from the tanks is desirable on account of the large volume of sludge deposited. Continuous operation also may be desirable in preliminary-sedimentation tanks, when the fresh sludge is mixed with excess activated sludge prior to discharge into separate sludge-digestion tanks.

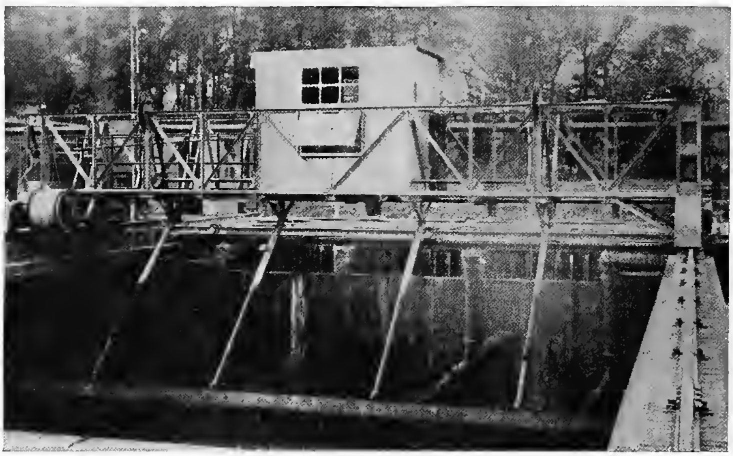


Fig. 86.—Mieder sludge and scum collector.

Aeration of sewage in tanks or channels preceding preliminary-sedimentation units, to promote the removal of grease, and additional provisions made for skimming the surface of sedimentation tanks are described in Chap. XI.

Freeboard.—The walls of open tanks commonly extend at least 6 in. above the surface of the liquid, in order to prevent sewage from being blown over the coping by winds. The height of the wall above the sewage surface is called the *freeboard.* A height of 12 in. is ordinarily employed, but heights of 18 in. to 2 ft. are common, especially for large tanks, so as to protect the sedimentation process from the disturbing influence of currents set up by winds.

Roofs.—In many small and medium-size plants the sedimentation basins have been roofed over, 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 tanks caused by roofing them, but in most cases the increase has been found insufficient to justify the expense of the covering.

Open tanks are sometimes surrounded by fences to protect trespassers from danger and children from falling into the tanks.

Statistics of Horizontal-flow Tanks.—The features of design of a number of existing horizontal-flow tanks, presented in Table 64, will serve to illustrate the principles set forth in the preceding sections of this chapter. In comparing the data on these tanks it should be borne in mind that the objects to be attained by sedimentation may differ widely at different plants. For example, the tanks at the North Side plant in Chicago, which provide a ½-hr. detention period, were designed

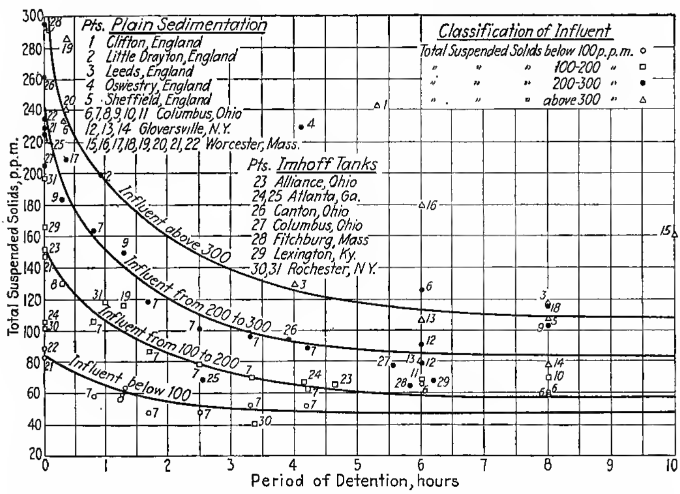


FIG. 87.—Removal of suspended solids from sewage by sedimentation.

to perform a function similar to that of grit chambers. On the other hand, the tanks at Marlborough, Mass., in which the sewage is settled for 5.8 hr., were designed to remove as large a proportion of suspended solids as practicable, in order to minimize the clogging of intermittent sand filters.

Efficiency of Horizontal-flow Tanks.—The many factors affecting the efficiency of sedimentation tanks have been discussed in the first part of this chapter. Foremost among them is the detention period, the practical significance of which can be gaged by a comparison of the results obtained from settling tanks in cities with sewage of different characteristics. Figure 87 shows the removal of suspended solids for a number of different sewages and different tanks during a given detention period. Similar curves, showing the effect of detention period on the efficiency of an experimental tank at Syracuse, have been presented in Fig. 81. A modified and often more convenient way of presenting the information obtained in sedimentation studies is shown in Fig. 88.

A study of Figs. 81, 87 and 88 shows, among other things, the following: the greatest proportion of sedimentation takes place during the first hour and little added clarification is obtained beyond the second and third hours; the stronger the sewage the greater is the proportionate relative removal of suspended matter; to reduce the suspended matter in the effluent to the same value, however, longer detention periods are required for strong sewage than for weak sewage; and by sedimentation it is possible to remove 40 to 75 per cent of the suspended matter from sewage of average strength, containing 300 p.p.m. suspended matter, in

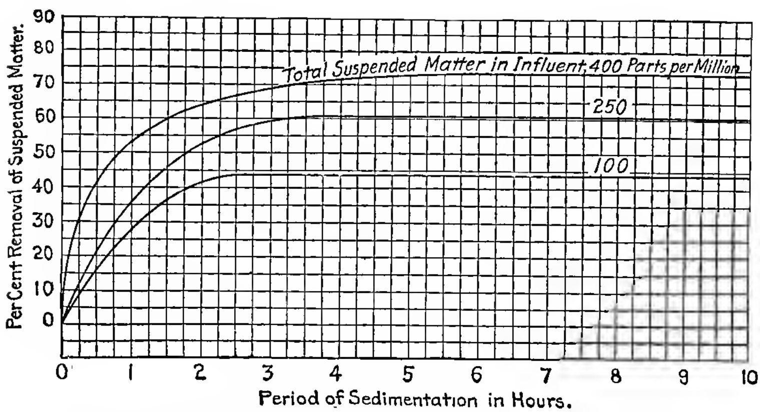


FIG. 88.—Percentage removal of suspended matter by sedimentation.

1 to 4 hr. All these conclusions refer to sedimentation under average conditions. At any particular plant, wide variations from the general average may occur, as evidenced by the results at Clifton and Oswestry, England, plotted in Fig. 87.

Other observations show that, in general, the fresher the sewage, the more complete is its clarification, because the suspended solids in fresh sewage have not been comminuted. In well-designed tanks, from 80 to more than 95 per cent of the settling solids may be deposited. Bacterial removal often approximates that of suspended matter, but during warm weather multiplication of bacteria in the liquid and nonsettling portions of the sewage may offset the removal by deposition. Pathogenic organisms, it is believed, do not multiply. The removal of total organic matter may be about half that of the suspended solids.

In comparing analyses of influent and effluent in order to determine the efficiency of sedimentation tanks, it is well to bear in mind that much grease and heavy suspended matter in sewage is not included in samples

TABLE 64.—COMPARISON OF HORIZONTAL-FLOW PLAIN-SEDIMENTATION TANKS

	Antigo, Wis.	Aurora, Ill.	Boonton, N. J.	Charlotte, N. C., Sugar Creek plant	Chicago, Ill., North Side plant	Joint Meeting plant	Fostoria, Ohio	Lakewood, N. J.	Marlborough, Mass.	Newark, N. J., Passaic Valley plant	Salem, Ohio	Syracuse, N. Y.	Toronto, Ont., No. Toronto plant
Population served	10,000[1]	67,000[1]	20,000[1]	50,000[1]	800,000[1]	467,000[1]	15,000[1]	....	15,000	979,000	15,000[1]	200,000[1]	100,000[1]
Quantity of sewage, m.g.d	0.9[1]	6.5	....	5[1]	175[1]	50	2[1]	1	0.6	100	....	27.5[1]	9[1]
Number of tanks	1	4	12	1	8	4	2	3	2	12	2	4	4
Type of tanks	Dorr	Dorr	Flat-bottom	Dorr	Dorr	Mieder	Dorr	Link-Belt	Flat-bottom	Hopper-bottom	Dorr	Dorr	Dorr
Tank dimensions, ft.:													
Length	35	50	100	100	80	280	30	60	30 62	225	35	71	50
Width	35	50	12	100	80	75	30	12	30 30	25	35	71	50
Maximum depth	....	12	13	....	13	14.6	12.5	16	8 7	33	9.5	10.5	14
No. of hoppers per tank	1	1	....	1	1	1	1	1	....	9	1	1	1
Depth of hoppers, ft	1.5	2	....	4	3.3	8.5	1.2	5.7	....	13	1.5	3	3.3
Slope of floor	1 on 12	1 on 12	7 per cent	1 on 12	1 on 12	1 on 270	1 on 12	0	0 3 per cent	1.3 on 1	1 on 12	1 on 12	15 per cent
Detention period, hr	2[1]	3	12[2]	2.5[1]	0.5[1]	4[1]	2[1]	4	5.8	2.7	0.5–3	1[1]	2[1]
Frequency of sludge removal	Twice daily	6 times daily	....	Intermittent daily	Intermittent daily	Daily	Intermittent daily	Continuous	6 times yearly	3 times weekly	....	Intermittent daily	Continuous
Quantity of sludge removed, cu. ft. per mil. gal	....	....	....	....	....	....	....	....	363	182	....	950[3]	
Moisture of sludge, per cent	88–90	....	....	....	....	....	....	....	....	90			

[1] Basis of design.
[2] Assuming 100 gal. per capita daily.
[3] In 1929.

taken for analysis. This matter has been discussed in connection with the efficiency of screening.

VERTICAL-FLOW TANKS

Elements of Design.—In vertical-flow or Dortmund[1] tanks (Fig. 80*d*) 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 usually 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, so as to take advantage of any influence the latter may have in promoting precipitation by attraction and coagulation. This 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 sewage passes this zone of equilibrium the coarser solids are mechanically filtered out by the suspended 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 to the sludge at the bottom.

Sludge is drawn from the conical bottom, as in hopper-bottom tanks, without requiring the emptying of the tank and may be delivered by gravity at a considerable elevation above the bottom of the tank, because of the hydrostatic pressure of the sewage above it.

Vertical-flow tanks are particularly adapted to the clarification of the effluent from trickling filters, because it is 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. If shallow tanks are used, the suspended matter in such effluents, accumulating on the sides and bottom, will reduce the quantity of dissolved oxygen in the liquid passing through them. Vertical-flow tanks are also frequently used for the settling of the effluent from activated-sludge units, because the deposited sludge is readily withdrawn and thus permits aerobic conditions to be maintained in sewage and sludge.

Limiting Velocity.—In well-designed vertical-flow tanks, the upward-directed velocity of the sewage is not greater than the hydraulic subsiding value of the suspended solids that are to be removed. A limiting value of 0.03 in. per second, or 9 ft. per hour, is generally employed.

Tank Dimensions.—As previously pointed out, vertical-flow tanks are restricted more or less to employment following trickling filters and activated-sludge aeration tanks. Their use for preliminary clarification

[1] A town in Germany where this type of tank originated.

is limited to such an extent that standards of design have not been evolved. The tanks at Gloversville, N. Y., are about 49 ft. deep and 36 ft. in diameter and those at Fitchburg, illustrated in Fig. 89, are 30 ft. in diameter and 22 ft. 6 in. deep.

Cost of Construction, Operation and Maintenance of Sedimentation Tanks.—The cost of construction of sedimentation tanks is dependent

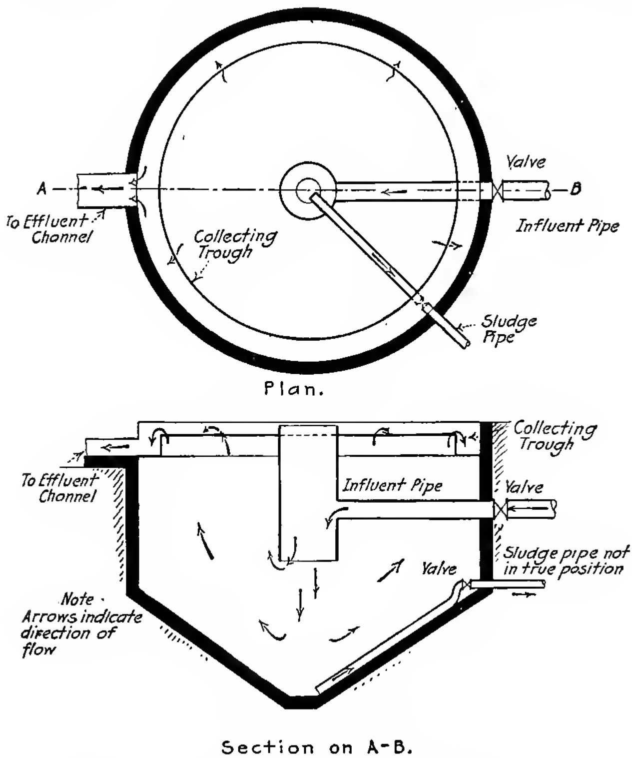


FIG. 89.—Final-sedimentation tank, Fitchburg, Mass.

upon the type and characteristics of tank, type of mechanical equipment provided, foundation conditions, prevailing unit prices for labor and materials and possibly other factors. The cost in 1926 of the humus tanks at Akron, which are rectangular tanks with fairly flat bottoms and no mechanical sludge-removal equipment, was about 61 cents per cubic foot of water capacity, not including engineering. Of this amount about 14 cents per cubic foot represented the cost of piles provided to counterbalance upward water pressure with the tanks empty. In 1918 the cost of Dortmund tanks at Fitchburg, not including engineering, was about 29 cents per cubic foot of water capacity. Plain-sedimentation tanks without mechanical sludge-removal equipment commonly cost 40 to 65 cents per cubic foot of water capacity. Including mechanical sludge-removal equipment the cost may be $1 or more per cubic foot.

The cost of operation is dependent upon the method of sludge removal provided and upon the quantity of sludge removed. The power consumption of mechanical sludge-removal equipment is relatively low. For tanks 40 to 60 ft. in diameter at the bottom the average power consumption may be about 1 hp. per unit. The costs of removing skimmings, of cleaning tank walls, of removing sludge and of other items of operation are so variable and are so seldom separated from other items in the cost of operating a sewage-treatment plant, that average figures, if available, would be of little value.

Maintenance costs of tanks with mechanical equipment are higher than for tanks not so equipped. Steel work requires painting at frequent intervals, timber needs to be painted, repaired or replaced, and worn-out machinery and parts require replacement. Maintenance of concrete work ordinarily does not involve large annual charges. However, in the event of disintegration, excessive cracking or other failure, the cost of maintenance and repair may be considerable.

Bibliography

1. Manuals of Engineering Practice, A. S. C. E., No. 2, 1928.
2. Hazen, Allen: *Trans.*, A. S. C. E., 1904; **53**, 45.
3. Capen, C. H., Jr.: *Eng. News-Rec.*, 1927; **99**, 833.
4. Seddon, J. A.: *Jour.*, Assoc. Eng. Societies, 1889; **8**, 477.
5. Dunbar, W. P.: "Principles of Sewage Treatment," 1908.
6. Johnson, G. A.: Rept. on Sewage Purification at Columbus, Ohio, 1905.
7. Tark, M. B.: *Proc.*, Am. Soc. Munic. Improvements, 1928; **34**, 225.
8. Holmes, G. D., and W. P. Gyatt: *Sewage Works Jour.*, 1929; **1**, 318.
9. Imhoff, Karl: "Taschenbuch der Stadtentwässerung," 1928.
10. Rept. of Bureau of Surveys, Comprising Work at Sewage Experiment Station, Philadelphia, 1910.

CHAPTER XIII

CHEMICAL PRECIPITATION

Chemical precipitation is a method of increasing the deposition of suspended matter and inducing that of colloidal matter by the addition to sewage of chemicals that in one way or another form *floc* in the liquid, the floc drawing to itself the substances that it is desired to remove from the sewage or being produced by these substances. In water purification this process is called *coagulation*.

Many different substances have been used as precipitants. The most common ones are calcium oxide or lime, aluminum sulfate or alum, lime and ferrous sulfate or copperas, ferric salts, sulfuric acid and sulfur dioxide. The degree of clarification obtained depends upon the kind and quantity of chemicals used and the care with which the process is controlled. It is practicable by chemical precipitation to remove 80 to 90 per cent of the total suspended matter, 70 to 80 per cent of the biochemical-oxygen demand and 80 to 90 per cent of the bacteria.

The handling and disposal of the sludge resulting from chemical precipitation is one of the greatest difficulties of this method of treatment. Sludge is produced in great volume, often reaching 5000 gal. for each million gallons of sewage treated. Furthermore, although the effluent may have a moderately satisfactory appearance, it is ordinarily putrescible and not comparable with the effluents produced by oxidation processes. These drawbacks, together with the expense of the chemicals, have curtailed the use of chemical precipitation and have led to its abandonment in most of the American, and many of the foreign, municipal plants formerly employing the process. Lime treatment is an important feature of the "direct-oxidation process" described in Chap. VI.

In 1925, an Imhoff-tank, trickling-filter plant displaced the chemical-precipitation works at Worcester, Mass. At Providence, R. I., works are under construction (1935) for an activated-sludge plant to replace the chemical-precipitation plant (1). A number of works in Great Britain, however, still employ this process.

A patent, obtained by Rudolfs in 1931 and assigned to the National Aluminate Corp., covers a process of treating sewage which comprises adding to it soluble iron salts in the proportion of 3.5 to 5.0 p.p.m., or 29.1 to 41.6 lb. per mil. gal. During the years 1932 to 1935 experiments with chemical-precipitation processes of sewage treatment have been conducted in a number of places in the United States, and construction of a few new plants employing chemical precipitation has been undertaken. These recent developments will be discussed later in this chapter.

In the treatment of municipal sewage, chemical precipitation may prove useful under some conditions where the sewage either requires treatment for brief periods more complete than plain sedimentation or receives unusual quantities of industrial wastes. This process may be applicable also, in some cases, as an adjunct to other methods of sewage treatment.

The use of sulfuric acid at Bradford, England, for the precipitation of grease from the municipal sewage, which is rich in wool-scouring wastes, provides one of the few cases in which a substantial return is obtained from the sludge resulting from chemical precipitation.

An important field for the application of chemical precipitation is in the separate treatment of industrial wastes, to render them suitable for independent disposal into natural bodies of water or for discharge into public sewers for further treatment or disposal with the municipal sewage. Such treatment of wastes may simply consist of the utilization of different wastes to produce the necessary reaction upon each other, or it may consist of the addition of chemicals to produce the desired floc.

Reactions Involved in Chemical Precipitation.—The coagulants added to sewage in chemical precipitation react either with substances normally present in the sewage or with substances added for this purpose. The quantity of chemical employed is expressed in parts per million, grains per gallon or pounds per million gallons. One grain per gallon equals 17.1 p.p.m. and, since 1 p.p.m. equals 8.33 lb. per mil. gal., 1 grain per gallon is equivalent to 142.5 lb. per mil. gal.

The reactions involved in chemical precipitation will be taken up in the following order: alum; copperas and lime; ferric sulfate; ferric chloride; lime; sulfuric acid and sulfur dioxide.

Alum.—When alum is added to sewage containing in solution calcium and magnesium bicarbonate alkalinity, the reaction that occurs may be illustrated as follows:

$$\underset{\substack{\textit{Aluminum sulfate}\\ \text{Soluble}}}{\overset{666.7}{Al_2(SO_4)_3 18H_2O}} + \underset{\substack{\textit{Calcium bicarbonate}\\ \text{Soluble}}}{\overset{3 \times 162 = 3 \times 100(CaCO_3)}{3Ca(HCO_3)_2}} \rightleftarrows \underset{\substack{\textit{Calcium sulfate}\\ \text{Soluble}}}{\overset{3 \times 136}{3CaSO_4}} +$$

$$\underset{\substack{\textit{Aluminum hydroxide}\\ \text{Insoluble}}}{\overset{2 \times 78}{2Al(OH)_3}} + \underset{\substack{\textit{Carbon dioxide}\\ \text{Soluble}}}{\overset{6 \times 44}{6CO_2}}$$

The insoluble aluminum hydroxide is formed as a bulky, gelatinous floc which settles slowly through the sewage, sweeping out suspended matter and producing other changes which will be described later. The reaction is exactly analogous when magnesium bicarbonate is substituted for the calcium salt. The numbers above the chemical formulas are the combining weights of the different substances and denote therefore what quantity of each is involved. Since alkalinity is reported in terms of calcium carbonate, $CaCO_3$, the molecular weight of which is 100, the quantity of alkalinity required to react with 1 grain per gallon of alum is $17.1 \times \frac{3 \times 100}{666.7} = 7.7$ p.p.m. If less than this quantity is available for decomposing the alum, artificial alkalinity must be added. This is seldom required in sewage treatment. Lime is commonly used for this purpose where necessary.

Copperas and Lime.—In ordinary sewages, copperas cannot be employed alone as a precipitant and lime must be added at the same time, as will appear from the reactions involved. The process is commonly spoken of as the *iron and lime process.*

$$\underset{\substack{\textit{Ferrous sulfate}\\ \text{Soluble}}}{\overset{278}{FeSO_4 7H_2O}} + \underset{\substack{\textit{Calcium bicarbonate}\\ \text{Soluble}}}{\overset{100(CaCO_3)}{Ca(HCO_3)_2}} \rightleftarrows \underset{\substack{\textit{Ferrous bicarbonate}\\ \text{Slightly soluble}}}{\overset{178}{Fe(HCO_3)_2}} + \underset{\substack{\textit{Calcium sulfate}\\ \text{Soluble}}}{\overset{136}{CaSO_4}} + \overset{7 \times 18}{7H_2O}$$

If lime, CaO, is now added in the form of *milk of lime*, $Ca(OH)_2$, [$CaO + H_2O \rightleftarrows Ca\ (OH)_2$] the reaction that takes place is:

$$\underset{\substack{\textit{Ferrous bicarbonate}\\ \text{Slightly soluble}}}{\overset{178}{Fe(HCO_3)_2}} + \underset{\substack{\textit{Calcium hydroxide}\\ \text{Slightly soluble}}}{\overset{2 \times 56(CaO)}{2Ca(OH)_2}} \rightleftarrows \underset{\substack{\textit{Ferrous hydroxide}\\ \text{Very slightly soluble}}}{\overset{89.9}{Fe(OH)_2}} + \underset{\substack{\textit{Calcium carbonate}\\ \text{Somewhat soluble}}}{\overset{2 \times 100}{2CaCO_3}} + \overset{2 \times 18}{2H_2O}$$

The ferrous hydroxide next may be oxidized to ferric hydroxide, the final form preferred, by oxygen dissolved in the sewage:

$$\underset{\substack{\textit{Ferrous hydroxide}\\ \text{Very slightly soluble}}}{\overset{4 \times 89.9}{4Fe(OH)_2}} + \underset{\substack{\textit{Oxygen}\\ \text{Soluble}}}{\overset{32}{O_2}} + \overset{2 \times 18}{2H_2O} \rightleftarrows \underset{\substack{\textit{Ferric hydroxide}\\ \text{Insoluble}}}{\overset{4 \times 106.9}{4Fe(OH)_3}}$$

The insoluble ferric hydroxide is formed as a bulky, gelatinous floc similar to the alum floc. One grain per gallon of copperas requires $17.1 \times \frac{100}{278} = 6.2$ p.p.m. of alkalinity; $17.1 \times \frac{2 \times 56}{278} = 6.9$ p.p.m., or 0.40 grain per gallon, of lime; and $17.1 \times \frac{32}{4 \times 278} = 0.49$ p.p.m. of oxygen. Oxidation is favored by a high pH value which is established in some degree by the lime.

In sewage treatment, lime commonly is added in excess of the quantity required to complete the "iron-and-lime" reaction, the excess lime being relied upon as an additional clarifying agent in accordance with the principles set forth later. Experience has shown that the best results are obtained when sufficient lime is added to produce a pink color when phenolphthalein is used as an indicator. This is equivalent to a pH of approximately 8.3. At Worcester a large excess of ferrous sulfate generally was present in the sewage and ferrous hydroxide was formed in great quantities, when lime was added to produce phenolphthalein or carbonate alkalinity. Under these conditions much of the ferrous hydroxide was not oxidized but a remarkably clear, colorless effluent could be produced.

Hazen found that ferric salts have an advantage over ferrous salts, because the ferric hydroxide formed is more readily precipitated and is more nearly insoluble than ferrous hydroxide (2). Furthermore, the use of lime may not be required with ferric salts, which are precipitated by bicarbonates in the sewage even in the presence of free carbonic acid. Ferric salts have not been generally employed in the past, because they were not readily available as a commercial product of low price. Recently, however, they have been placed on the market at much more favorable prices.

Ferric Sulfate.—When ferric sulfate solution is added to sewage which normally contains calcium and magnesium bicarbonate alkalinity, the reaction that takes place may be illustrated as follows:

$$\begin{array}{cccc} & 399.9 & & 3 \times 100(CaCO_3) \\ & Fe_2(SO_4)_3 & + & 3Ca(HCO_3)_2 \\ & \textit{Ferric sulfate} & & \textit{Calcium bicarbonate} \\ & \text{Soluble} & & \text{Soluble} \end{array}$$

$$\begin{array}{cccccc} & 2 \times 106.9 & & 3 \times 136 & & 6 \times 44 \\ \rightleftarrows & 2Fe(OH)_3 & + & 3CaSO_4 & + & 6CO_2 \\ & \textit{Ferric hydroxide} & & \textit{Calcium sulfate} & & \textit{Carbon dioxide} \\ & \text{Insoluble} & & \text{Soluble} & & \text{Soluble} \end{array}$$

If milk of lime is added to supplement the alkalinity of the sewage, the reaction may be assumed to be as follows:

$$\begin{array}{ccccccc} 399.9 & & 3 \times 56(CaO) & & 2 \times 106.9 & & 3 \times 136 \\ Fe_2(SO_4)_3 & + & 3Ca(OH)_2 & \rightleftarrows & 2Fe(OH)_3 & + & 3CaSO_4 \\ \textit{Ferric sulfate} & & \textit{Calcium hydroxide} & & \textit{Ferric hydroxide} & & \textit{Calcium sulfate} \\ \text{Soluble} & & \text{Slightly soluble} & & \text{Insoluble} & & \text{Soluble} \end{array}$$

Ferric Chloride.—When ferric chloride is added to sewage the reaction that takes place may be expressed as follows:

$$\begin{array}{ccccccccc} 2 \times 162.2 & & 3 \times 100(CaCO_3) & & 2 \times 106.9 & & 3 \times 111 & & 6 \times 44 \\ 2FeCl_3 & + & 3Ca(HCO_3)_2 & \rightleftarrows & 2Fe(OH)_3 & + & 3CaCl_2 & + & 6CO_2 \\ \textit{Ferric chloride} & & \textit{Calcium bicarbonate} & & \textit{Ferric hydroxide} & & \textit{Calcium chloride} & & \textit{Carbon dioxide} \\ \text{Soluble} & & \text{Soluble} & & \text{Insoluble} & & \text{Soluble} & & \text{Soluble} \end{array}$$

If milk of lime is added to supplement the alkalinity of the sewage, the reaction may be assumed to be as follows:

$$\begin{array}{ccccccc} 2 \times 162.2 & & 3 \times 56(CaO) & & 2 \times 106.9 & & 3 \times 111 \\ 2FeCl_3 & + & 3Ca(OH)_2 & \rightleftarrows & 2Fe(OH)_3 & + & 3CaCl_2 \\ \textit{Ferric chloride} & & \textit{Calcium hydroxide} & & \textit{Ferric hydroxide} & & \textit{Calcium chloride} \\ \text{Soluble} & & \text{Slightly soluble} & & \text{Insoluble} & & \text{Soluble} \end{array}$$

Lime.—When lime alone is added as a precipitant or is used in excess of the quantity required for the precipitation of the iron in the iron and lime process, the principles of clarification are explained by the following reactions:

56(CaO)		44(CO_2)		100		2 × 18
$Ca(OH)_2$	+	H_2CO_3	$\rightleftarrows$	$CaCO_3$	+	$2H_2O$
Calcium hydroxide		*Carbonic acid*		*Calcium carbonate*		
Slightly soluble		Soluble		Somewhat soluble		

56(CaO)		100($CaCO_3$)		2 × 100		2 × 18
$Ca(OH)_2$	+	$Ca(HCO_3)_2$	$\rightleftarrows$	$2CaCO_3$	+	$2H_2O$
Calcium hydroxide		*Calcium bicarbonate*		*Calcium carbonate*		
Slightly soluble		Soluble		Somewhat soluble		

A sufficient quantity of lime, therefore, must be added to combine with all the free carbonic acid and with the carbonic acid of the bicarbonates, called half-bound carbonic acid, to produce calcium carbonate, which acts as the coagulant. Much more lime is generally required when it is used alone than when sulfate of iron also is employed. Where industrial wastes introduce mineral acids or acid salts into the sewage, these must be neutralized before precipitation can take place.

If sufficient lime is added to give the sewage a distinct hydroxide alkalinity, a large proportion of the bacteria will be killed, thus producing the double effect of clarification and disinfection. This treatment is employed in the "direct oxidation process."

If too much lime is used in the treatment of sewage, some of the suspended organic matter will be dissolved by the caustic calcium hydroxide and escape in the effluent. If insufficient quantities are added, the effluent will not be well clarified.

Sulfuric Acid and Sulfur Dioxide.—In the Miles acid process for precipitation of fats, outlined in Chap. VI, either sulfuric acid or sulfur dioxide is used as a precipitant. Neither of these substances forms floc in the same manner as the precipitants previously mentioned. When sulfuric acid is added to sewage it neutralizes the alkalinity as follows:

98		100($CaCO_3$)		136		2 × 44		2 × 18
H_2SO_4	+	$Ca(HCO_3)_2$	$\rightleftarrows$	$CaSO_4$	+	$2CO_2$	+	$2H_2O$
Sulfuric acid		*Calcium bicarbonate*		*Calcium sulfate*		*Carbon dioxide*		
Soluble		Soluble		Soluble		Soluble		

Any excess remains in solution as sulfuric acid. Only soluble substances are formed in this reaction and the mechanism of precipitation, therefore, is not due to the production of a chemical precipitate as in the previous instances. Precipitation must be laid to other causes, which will be described later.

When sulfur dioxide is used, it hydrolizes to form sulfurous acid and reacts with the alkalinity of the sewage to form bisulfites. Both of these compounds are oxidized in the presence of dissolved oxygen. The reactions may be stated as follows:

2 × 82		100($CaCO_3$)		202		2 × 44		2 × 18
$2H_2SO_3$	+	$Ca(HCO_3)_2$	$\rightleftarrows$	$Ca(HSO_3)_2$	+	$2CO_2$	+	$2H_2O$
Sulfurous acid		*Calcium bicarbonate*		*Calcium bisulfite*		*Carbon dioxide*		
Soluble		Soluble		Soluble		Soluble		

$$\underset{\substack{\textit{Sulfur dioxide} \\ \text{Soluble}}}{\overset{64}{SO_2}} + \overset{18}{H_2O} \rightleftarrows \underset{\substack{\textit{Sulfurous acid} \\ \text{Soluble}}}{\overset{82}{H_2SO_3}}$$

$$\underset{\substack{\textit{Sulfurous acid} \\ \text{Soluble}}}{\overset{2 \times 82}{2H_2SO_3}} + \underset{\substack{\textit{Oxygen} \\ \text{Soluble}}}{\overset{32}{O_2}} \rightleftarrows \underset{\substack{\textit{Sulfuric acid} \\ \text{Soluble}}}{\overset{2 \times 98}{2H_2SO_4}}$$

$$\underset{\substack{\textit{Calcium bisulfite} \\ \text{Soluble}}}{\overset{202}{Ca(HSO_3)_2}} + \underset{\substack{\textit{Oxygen} \\ \text{Soluble}}}{\overset{32}{O_2}} \rightleftarrows \underset{\substack{\textit{Calcium sulfate} \\ \text{Soluble}}}{\overset{136}{CaSO_4}} + \underset{\substack{\textit{Sulfuric acid} \\ \text{Soluble}}}{\overset{98}{H_2SO_4}}$$

Here again only soluble substances are formed.

How Chemical Precipitation Acts.—The reactions which take place in chemical precipitation are complex. The typical reactions noted explain the changes only in part and they do not proceed, necessarily, exactly as indicated. They often are incomplete and side reactions with other substances in the sewage may take place. The chemical structure of the different substances and changes therein, however, will serve as a useful guide in the interpretation of the way in which precipitation acts.

With the knowledge now available, precipitation reactions may be ascribed to the following properties of the precipitant, which it may possess in part or as a whole: the formation by chemical reaction of insoluble or very slightly soluble precipitates, which in a mechanical way enmesh suspended matters and carry them down; adsorption of dissolved or colloidal matter on the large surfaces presented by the precipitate; ionization of the precipitant to yield high-valent ions, which may neutralize the electrical charges on colloidal particles and cause their coagulation and deposition, positive ions precipitating negative colloids and negative ions removing positive colloids; ionization of the precipitant to yield hydrogen ions or hydroxyl ions, thus rendering the sewage more acid or more alkaline.

The floc-forming chemicals possess of themselves all these attributes; the others possess only the last two, but may give rise to the first two phenomena listed by causing the formation of flocs of sewage matter. The first two properties are readily understandable. A brief explanation of the last two, however, may be helpful.

The properties of alum for this purpose may be considered as follows: The precipitate formed is aluminum hydroxide. The solubility of this substance depends upon the hydrogen ion concentration of the liquid. At low pH values it dissolves as aluminum ion, Al^{+++}, while at high ones it goes into solution as aluminate ion, $Al(OH)_2O^-$. Somewhere in between, it is practically insoluble. As far as floc formation is concerned, therefore, it is important to have the liquid at the pH of greatest insolubility. In pure water this has been found to be in the vicinity of pH 5.5. In sewage it probably varies considerably with the nature of the sewage matters present. It has been observed, however, that the presence of high-valent positive ions causes the precipitation of negative colloids and, conversely, that the presence of high-valent negative ions produces the settling of positive colloids. Hence it is seen that the best clarification is not necessarily obtained at the pH of greatest insolubility of the floc. What are the best conditions for clarification will depend upon the state of the matters to be removed. Ordinarily, it may be assumed that the mechanical and adsorptive removal of solids is brought about best by the formation of an insoluble floc, while colloidal materials are affected particularly by the presence of high-valent ions.

The importance of the hydrogen ion concentration of the liquid is apparent and studies to determine the optimum pH for clarification are helpful. Control of the hydrogen ion concentration, beyond the effects of necessary quantities of the coagulant itself, is commonly maintained by the use of sulfuric acid or lime.

Volume of Sludge Produced.—The quantity of sludge deposited in chemical precipitation tanks is greater than that obtained by plain sedimentation, because of the coagulating effect of the chemicals on the suspended matter and because of the presence in the sludge of the insoluble products of reaction of the chemicals used.

The following assumptions are illustrative:

1. Sewage solids removed by chemical precipitation in 8 hr., 210 p.p.m. = $210 \times 8.33 = 1750$ lb. per mil. gal.

2. Copperas added, 70[1] lb. per mil. gal. forming $70 \times \frac{106.9}{278} = 27$ lb. ferric hydroxide.

3. Lime added, 600[1] lb. per mil. gal., enough to satisfy the requirements of the copperas and combine with the free CO_2 and bicarbonates present to form calcium carbonate.

a. From reaction with copperas, $70 \times \frac{112}{278} \times \frac{100}{56} = 50$ lb. $CaCO_3$ per mil. gal.

b. From reaction with CO_2 and bicarbonates, $\frac{300}{112} \times \left(600 - 50 \times \frac{56}{100}\right) =$ 1530 lb. $CaCO_3$ per mil. gal.

c. Solubility of $CaCO_3$, 11 p.p.m. = $11 \times 8.33 = 92$ lb. $CaCO_3$ per mil. gal.

d. Total $CaCO_3$ in sludge, $(50 + 1530) - 92 = 1488$ lb. per mil. gal.

4. Total solids in sludge on a dry basis = $1750 + 27 + 1488 = 3265$ lb. per mil. gal.

On the assumption that the specific gravity of the sewage solids is 1.3, much of the light material being precipitated, and that the specific gravities of ferric hydroxide and calcium carbonate are 3.4 and 2.7, respectively, the specific gravity of the solids becomes 1.7 and, since sludge of this type commonly contains 92.5 per cent water, the specific

[1] These are quantities used for many years at Providence, R. I.

gravity of the sludge may be taken as 1.03. The volume will be $\frac{3265}{8.33 \times 0.075 \times 1.03} = 5060$ gal. per mil. gal. of sewage.

Elements of Design.—As chemical precipitation has not been generally employed for municipal plants lately, there is comparatively little American practice which is worthy of note. In cases where chemical precipitation may be employed in the future, for treatment of municipal sewage, it is probable that the plants will be designed along lines quite different from those employed in the old plants now abandoned and more like the plants used in water purification.

The treatment of industrial wastes varies so greatly in the conditions encountered that there is comparatively little uniformity in the designs thus far employed. The tendency in this field, as in that of the treatment of municipal sewage, is toward the utilization of more mechanical appliances than was common practice when chemical precipitation was more in vogue.

Recent Developments in Chemical Precipitation.—During the past 10 or 15 years mechanical equipment has been introduced into sewage-treatment practice in this country on a far more extensive scale than was prevalent during the preceding thirty years. This use of mechanical equipment, together with advances in the chemistry of coagulation and improvements in the manufacture of chemicals, which have made possible the production of certain precipitants at a fraction of their former cost, has led to renewed interest in chemical precipitation of sewage during the past few years.

Chemical-precipitation plants have been built or are under construction at Dearborn, Mich., Perth Amboy, N. J., Ashland, Ohio, and Birmingham, Ala. Plans for a 35-m.g.d. plant to be built by the city of New York at Coney Island contemplate the use of iron salts and lime as coagulants to aid sedimentation during the four summer months (3). The Sanitary District of Minneapolis-St. Paul has adopted a plan for treating a sewage flow of 134 m.g.d. by plain sedimentation, supplemented by chemical precipitation with ferric chloride during two months of the year (4). In addition, experiments with chemical-precipitation processes of sewage treatment have recently been conducted at a number of places in this country.

Laughlin Process.—In 1932 a sewage-treatment plant using a chemical-precipitation process developed by Laughlin was put into operation at Dearborn, Mich. (5). It is known as the West Side plant. Here the substances added to the screened sewage have been varied from time to time, but in general have been pulped waste paper, lime and ferric chloride. The sewage, after receiving the dose of chemicals, is subjected to a short period of vigorous mixing, followed by flocculation and sedimentation for a period of 1 to 3 hr. The effluent from the two precipitation tanks undergoes rapid upward filtration through a thin layer of magnetite sand, which is cleaned periodically by a traveling magnetic device. The final effluent is chlorinated and the sludge is dewatered by vacuum filtration. In 1935 an incinerator was installed to dispose of the dewatered sludge. Ordinarily, wet sludge from plain-sedimentation tanks at the East Side sewage-treatment plant is discharged into the sewage entering one of the chemical-precipitation tanks.

From October, 1932, to April, 1933, the West Side plant treated an average of 2.61 m.g.d. of sewage, together with 0.21 m.g.d. of sludge from the East Side plant (6). During this period the plant effected reductions of 95 per cent in suspended solids and 88 per cent in B.O.D., based on the composite analysis of incoming sewage and sludge. From November, 1933, to April, 1934, the method of operation was modified so that West Side sewage and East Side sludge were treated separately. During this period, the average quantity of sewage treated was 1.74 m.g.d. and the reductions in suspended solids and B.O.D. in the sewage alone averaged 92 per cent and 78 per cent, respectively. In the latter period, the quantities of precipitants used in treating the sewage alone were as follows: lime, 147 lb. per mil. gal.; ferric chloride, 306 lb. per mil. gal.; and paper, 73 lb. per mil. gal.

In 1935 a sewage-treatment plant designed to utilize the Laughlin process of chemical precipitation is under construction at Perth Amboy, N. J.

Guggenheim Process.—A small chemical-precipitation plant, demonstrating a process developed at the Guggenheim Laboratories, was operated for six months in 1933 at the North Side sewage-treatment works of the Sanitary District of Chicago (7). In this process the sewage is subjected to two operations: removal of suspended matter and nonbasic dissolved matter by coagulation and precipitation with an iron salt and an alkali, such as ferric sulfate and lime, and removal of soluble basic compounds by passing the clarified sewage through a bed of base-exchange zeolite. Disposal of the sludge is effected by filtration and incineration, the resulting ash being treated for the recovery of iron in the form of ferric sulfate. The zeolite is regenerated by means of a brine solution, from which ammonia is subsequently recovered.

The results obtained while operating this plant at Chicago from March 6 to Sept. 17, 1933, are reported by Gleason and Loonam (8) of the Guggenheim Laboratories. The average quantity of sewage treated was about 27,000 gal. daily. Iron and lime were added at average rates of 237 and 383 lb. per mil. gal., respectively. The plant effluent showed an average reduction of 91.2 per cent in the B.O.D. and a removal of suspended solids amounting to 97.4 per cent. The moisture in the sludge from the clarifier was reduced by a thickener and a vacuum filter from 95.2 to 76.4 per cent prior to disposal of the sludge in a rotary dryer and incinerator.

Relative to the Guggenheim process, Pearse (7) expresses the opinion that

> of the various so-called chemical processes it appears to be the most complete, offering an entire process, capable of delivering a high-grade effluent and disposing of the sludge. . . It would seem as though there may be situations where the process may be adaptable, especially on more concentrated sewage and in soft-water regions.

Recently the Guggenheim process has received favorable consideration for Raleigh, N. C.

Travers-marl Process.—In 1931 the Ohio Department of Health conducted a test of the Travers-marl process of sewage treatment at Circleville, Ohio (9). The main features of the demonstration plant consisted of chemical-dosing devices, a chemical-precipitation tank and a cascade aerator. The chemicals added to the sewage were hydrated lime, marl and ferrous sulfate. The precipitation tank was divided into two compartments, each provided with a shallow, trapezoidal sludge pit. The secondary compartment of this tank contained an upward-flow, shallow, sand-and-gravel filter, resting on a false bottom, above the sludge pit. The effluent from the chemical-precipitation tank passed over the aerator before being discharged into the Scioto River.

The average rate of sewage flow during the two months of the test was about 169,000 gal. daily. Based upon observations at Circleville, Waring states that this process may be expected to remove from municipal sewage nearly 92 per cent of the suspended solids, 15 to 30 per cent of the colloidal matter and nearly 70 per cent of the organic matter, as measured by the B.O.D. test. The water content of the sludge produced during the test run was somewhat less than 90 per cent, on the average.

In 1932 the Travers-marl process was installed at the sewage-treatment plant at Ashland, Ohio, existing septic tanks being utilized for chemical precipitation. Mohlman (10) states that the chemicals used at Ashland are ferrous sulfate, lime, marl and alum. He reports that tests made at Ashland over a period of 12 days in 1933, with an average flow of 500,000 gal. daily, gave approximately 85 per cent reduction of the suspended solids and 50 per cent reduction of the B.O.D.

Treatment with Chlorinated Copperas.—At the Shades Valley plant in Birmingham, Ala., chlorinated-copperas solution is used in the treatment of sewage by chemical precipitation (11). This solution consists largely of ferric chloride and chlorine. The plant, which is designed to treat a sewage flow of 2 m.g.d., contains a "flash mixer," a flocculator, two sedimentation tanks, two sludge-digestion tanks and five sludge-drying beds, in addition to a bar screen, two chlorinators and two dry-feed machines. There is a division wall in the flocculator which allows it to be used as two separate units. In normal operation, the screened sewage flows first into one side of the flocculator, where the chemical sludge, pumped from the secondary-sedimentation tank, is mixed with it continuously. This mixture then flows into the primary-sedimentation tank. The settled sewage then flows to the mixer, where chlorinated copperas and free chlorine are added. It then flows through the other half of the flocculator and into the secondary-sedimentation tank. This process seems to be practically the same as one for which Stevenson (12) applied for letters patent.

The sludge from the primary-sedimentation tank at the Shades Valley plant is digested in two stages, the first stage taking place in a heated tank, equipped with a mechanical stirring device, and the second in a plain, unheated tank. Gas is to be collected from both tanks and utilized in the plant. The digested sludge is to be dried on open drying beds.

The plant was placed in regular chemical operation in September, 1934, and the chemical process had been utilized for about two months up to Dec. 1, when chemical treatment was discontinued for the winter season and plain sedimentation was substituted (13). Treating an average flow of about 1 m.g.d. by chemical precipitation, the plant reduced the B.O.D. of the sewage from 80 to 10 p.p.m., on the average. The suspended-solids content of the sewage was reduced from 100 to 9 p.p.m. These results were accomplished by adding 70 lb. of chlorine and 440 lb. of ferrous sulfate to each million gallons of sewage treated, a dosage equivalent to 290 lb. of ferric chloride, computed on an iron-content basis.

Other Processes.—Other chemical-precipitation processes on which development work has been conducted recently include the following (10, 14): the Cabrera process, using aluminum sulfate and chlorine, with recirculation of sludge; the Diamond Alkali process, using chlorine, ferric chloride, and/or lime; the Lewis process, using ferric chloride, lime, cement dust and ferrous sulfate; the Miller-Koller process, using lime, copper sulfate, sodium carbonate, sodium aluminate and chlorine; the Putnam process, using ferric chloride, lime and charred sludge; the Scott-Darcey process, using iron and chlorine; the Stevenson process, using ferric chloride, together with chemical sludge which has been regenerated by chlorine; the Streander process, using ferrous sulfate, lime and air; and the Wright process, using paper pulp with or without lime, followed by vacuum filtration. A proprietary method of treatment called the "bio-reduction process" has recently come to the attention of the authors. This method consists essentially of chemical precipitation of the sewage solids and aerobic digestion of the sludge after concentration by centrifuges. The novelty of the process appears to lie in the return of part of the digested sludge to the incoming sewage, for the purpose of assisting coagulation, and in the aerobic rather than the anaerobic digestion of the sludge. The process is still in the developmental stage, although a 0.5-m.g.d. plant is being constructed in the South.

Bibliography

1. *Eng. News-Rec.*, 1931; **107,** 181.
2. Rept. of Mass. State Board of Health on Water Supply and Sewerage, Part II, 1890.
3. *Eng. News-Rec.*, 1934: **113,** 670.
4. *Eng. News-Rec.*, 1934; **113,** 601 and 838.
5. *Eng. News-Rec.*, 1932; **108,** 252.
6. Owen, M. B.: *Sewage Works Jour.*, 1934; **6,** 742.
7. Gleason, G. H., and A. C. Loonam: *Sewage Works Jour.*, 1934; **6,** 450.
8. Gleason, G. H., and A. C. Loonam: "Operation of Chemical Sewage Purification Plant—28,000 Gallons per Day Capacity—Chicago Sanitary District," Sept. 11, 1933.
9. Fifth Ann. Rept., Ohio Conference on Sewage Treatment, 1931.
10. Mohlman, F. W.: *Am. Jour. Pub. Health,* 1934; **24,** 25.
11. Hendon, H. H.: *Water Works and Sewerage,* 1933; **80,** 427.
12. Stevenson, R. A., and H. O. Banks: *Public Works,* 1932; **63,** 11.
13. Hendon, H. H.: Paper presented before N. Y. State Sewage Works Assoc., Jan. 15, 1935.
14. Eddy, H. P.: *Sewage Works Jour.*, 1934: **6,** 262.

CHAPTER XIV

SLUDGE DIGESTION

Apart from gritty, mineral matter settling in grit chambers, the solids removed from sewage by sedimentation and chemical precipitation settle to the bottom of clarification tanks and accumulate in a loose, honeycombed structure, the voids of which are filled with water containing more or less dissolved solids. This sludge is large in bulk, owing to its high water content, and putrescible, owing to its high organic content. Sludge resulting from plain sedimentation often contains 90 to 95 per cent water by weight and about two thirds of the solids are organic in nature.

Floating solids that are removed by screens or skimming tanks and heavy mineral solids deposited as grit in grit chambers or detritus tanks are not usually classed as sludge, but when no preliminary separation of these solids is made, the sludge will contain both these elements. The term *sludge* does apply in a wider sense, however, to the deposits from sedimentation of the effluent from trickling filters and from activated-sludge aeration units.

The storage, removal and disposal of the solids deposited in clarification units is an integral part of sedimentation and chemical-precipitation processes, although a part which, in many cases, has been neglected or handled inadequately. In a number of clarification devices the accumulating solids are stored for a sufficient length of time before removal from the clarification units, as in septic and Imhoff tanks, or in immediate connection with them, as in separate sludge-digestion tanks, to permit the sludge to undergo anaerobic decomposition. This is called *septicization* and, if carried on with a view to the relative completion of the processes of decomposition, it is termed *sludge digestion*.

There are several reasons for the digestion of sewage solids, among which the following may be mentioned: the breaking down of the putrescible organic matter, thereby rendering the sludge inoffensive and thus facilitating its disposal; the liquefaction, gasification and compacting of the sludge incidental to the destruction of the organic solids, which result in a reduced bulk of material to be handled; the physical conditioning of the sludge during digestion, which increases the readiness with which the sludge is dewatered and dried; and utilization of the combustible gases of decomposition.

Sewage sludge constitutes a rich culture medium in which hosts of microorganisms, notably bacteria, find an abundant food supply, providing them with energy for life, growth and reproduction. Being complex in character, the sludge is able to support a variety of groups of organisms capable of utilizing the different types of food substances, either as they occur originally in the solids or as they are modified by the activities of the various organisms developing in the sludge. In large measure the products of one group of organisms become available to other groups, until the bulk of the nutritive elements is consumed and the sludge is rendered stable. In this final state it is said to be well digested. The end products of digestion are gases, liquids, mineral compounds and nondigestible organic matter. Generally the latter is called "humus."

While protozoa may be seen to ingest visible particles of organic matter, most of the other unicellular microorganisms of decomposition probably absorb their food substances through their cell membranes. To obtain nourishment from solid matter, therefore, they must peptize, or liquefy, particles that are too large to be absorbed through their cell walls. This they do by means of enzymes, catalytic[1] agents that are carried in or secreted by living cells. Certain enzymes decompose nitrogenous substances, while others act on carbohydrates and still others on fats (1).

In treatment works, sewage solids are submitted to digestion either while remaining in contact with the flowing sewage, as in septic tanks, or after separation from the flowing sewage, as in Imhoff tanks and separate sludge-digestion tanks. The principles of digestion, however, are the same in both cases. Sludge digestion is an anaerobic process and, even though the flowing sewage be in immediate contact with the sludge, diffusion of oxygen into the sludge mass is so slow that, after the free oxygen in the sludge has been exhausted, anaerobic conditions prevail in the sludge mass.

It is quite important that digestion proceed rapidly, without interfering with the sedimentation processes, without adversely affecting the nature of the flowing sewage and without giving rise to offensive odors. All these requirements can be met by appropriate design and operation of clarification plants.

Materials commonly digested include the suspended and colloidal solids removed from the sewage by the several available processes of clarification, in some cases together with organisms which develop in large quantities in certain treatment units, such as trickling filters, as well as other substances which are products of biological changes. Coarse suspended matter and floating substances are digested with the

[1] Catalysts are substances that promote chemical reactions without themselves entering into the reactions.

other solids, unless separated therefrom by preliminary processes, such as screening and skimming, in which case they may be digested either by themselves or in conjunction with some sludge. In the case of sludge from chemical precipitation of sewage, which contains, in addition to the sewage solids, a chemical precipitate, little or no experience has been obtained with its digestion.

Course of Sludge Digestion.—While the way in which sludge digestion operates is still understood incompletely, much has been learned about it during recent times. Three major stages may be distinguished in the course of digestion of freshly deposited sewage solids: intensive acid production; acid regression or acid digestion; and intensive digestion of more resistant materials. The various changes manifested during these three stages have been outlined schematically by Rudolfs (2) as follows:

DIGESTION OF FRESH SEWAGE SOLIDS

(Neither well-digested sludge nor lime being added)

I. Period of intensive acid production
- *A.* Materials attacked
  1. Easily available carbohydrates (sugars, soluble starches, cellulose)
  2. Soluble nitrogenous compounds.
- *B.* Organisms responsible: *B. coli* group, spore-forming anaerobes.
- *C.* Characteristics
  1. Increase in acidity
  2. Solids: gray, less than half on top
  3. Odors: putrefactive, H_2S
  4. Liquid: fairly clear to slightly turbid
  5. Slight coagulation of colloidal material with heat and alcohol
  6. Disappearance of protozoa
  7. Increasing B.O.D. values.
- *D.* Products
  1. Organic acids, H_2S
  2. Gas: comparatively large volume with high percentage of CO_2 and N_2
  3. Acid carbonates.
- *E.* pH range: 6.8 to 5.1.
- *F.* Results
  1. Reduction of colon organisms
  2. Retardation of proteolysis[1]

II. Period of acid regression or acid digestion
- *A.* Materials attacked
  1. Organic acids
  2. Nitrogenous compounds.
- *B.* Organisms responsible; not definitely determined.

[1] *Proteolysis* is the digestion of proteid matter.

C. Characteristics
   1. Prolongation of the low level of pH values followed by a slow rise
   2. Solids: gray to yellowish-brown, half to four-fifths at top
   3. Odors: H_2S, indol, onion (mercaptans)
   4. Liquid: slightly turbid (milky) to yellow
   5. Some to considerable coagulation of colloidal material with heat and alcohol
   6. High B.O.D. values.

D. Products
   1. Gas: small volume, with decreasing percentage of CO_2 and N_2; hydrogen formed
   2. Ammonia compounds (amines, etc.)
   3. Acid carbonates.

E. pH range: 5.1 to 6.6 or 6.8.

F. Results
   1. Gradual rise of pH curve
   2. Acceleration of digestion
   3. Foaming.

III. Period of intensive digestion of more resistant materials

A. Materials attacked
   1. Nitrogenous materials
      (*a*) Proteins
      (*b*) Amino acids, etc.
   2. Lignocellulose (?)

B. Organisms responsible: spore-forming anaerobes, and fat-splitting organisms.

C. Characteristics
   1. Decreased acidity
   2. Increased alkalinity
   3. Slowly rising pH curve
   4. Solids: dark brown to black, half to none at top.
   5. Odors: odor associated with methane; tarry, rubber
   6. Liquid: slightly turbid to clear
   7. Some to no coagulation with heat, slight to no coagulation with alcohol
   8. Reappearance of protozoa
   9. Rapidly decreasing B.O.D. values.

D. Products
   1. Ammonia and other protein degradation products
   2. Organic acids
   3. Gas: large volume with high percentage of CH_4, low CO_2 and N_2, and no H_2.

E. pH range: 6.9 to 7.4.

F. Results: sludge stable enough for disposal.

It is evident that the progress of digestion can be measured in a number of ways. Many of the tests required to establish the changes that take place, however, are quite complicated. The volume and composition of the gases produced, taken together with the pH of the digesting sludge, present probably the simplest gage of the progress of digestion as a whole and are often used as such. Reduction in the organic content of the sludge or increase in mineralization is another simple and valuable criterion. The variations in these three parameters, together with certain other characteristic changes occurring during digestion, as given by Rudolfs (1), are shown in Fig. 90.

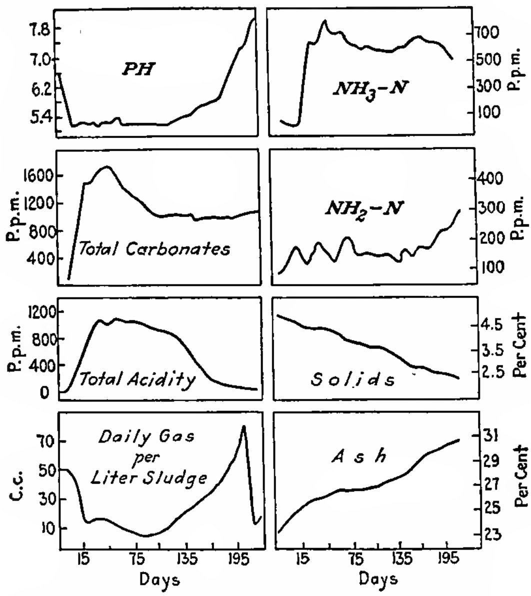


FIG. 90.—Characteristics of digesting sewage sludge.[1]

In sludge-digestion units all three stages of digestion are operative at one and the same time. Fresh solids are continually being added and digested solids are being removed from time to time. This presence of material in the various stages of digestion leads under suitable conditions to the establishment of a physical, chemical and biological balance such that digestion progresses rapidly and without the production of offensive conditions. Attainment of this balance is an important factor in the design and operation of digestion units.

Factors Influencing Digestion.—Among the factors that affect digestion and are readily subject to technical control are seeding, temperature and reaction.

Seeding.—By seeding is meant the addition to fresh solids of sludge already in process of digestion, some of it being fully digested. Seeding

[1] *NH_2-N* represents amino nitrogen.

is important in two ways. First, it inoculates the fresh solids with the desired organisms and enzymes of decomposition which have established themselves in the digesting material. Second, when properly controlled, it provides a balanced environment in which all organisms needful to carry the solids through the three stages of digestion find nutriment and in which reaction conditions prevail[1] which permit them to carry on their life processes and thus to digest the solids expeditiously and well. The difference in the rate of digestion of seeded and unseeded sewage solids, as presented by Rudolfs, is shown in Fig. 91 (1).

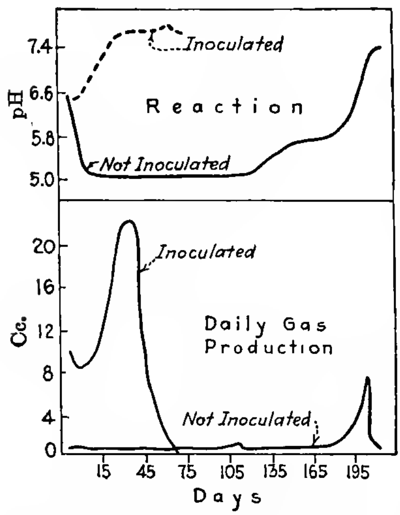


FIG. 91.—Effect of seeding upon the rate of digestion of sludge.

If sludge is submitted to anaerobic decomposition without seeding, as is done when a digestion tank is placed in operation after construction or after cleaning or repairs, it passes through the three stages but slowly. At the common sewage temperature of 60°F., for example, Rudolfs (3) found that about 8 months are required before the sludge can be drawn for satisfactory disposal, whereas 2 months are ordinarily adequate when the sludge is well seeded. This ratio of about 4:1 is observed, too, at other temperatures. The period required for balanced conditions to become established is called the *ripening* period. Once equilibrium is reached, fresh solids can be added continuously in definite quantities without destroying the balance.

Rudolfs (4) found that when about 2 per cent of fresh solids by weight, on a dry basis, or about 10 per cent by volume, on a wet basis, are added daily to sludge digesting normally at about 70°F., there is an increase in the ash or mineral content of the sludge proportional to the quantity of fresh solids added and the rate of digestion is therefore presumably equal to the rate of addition of fresh material. In other words, the time required for digestion is about 50 days. The digestion schedule naturally varies with the other factors discussed in this section.

Heukelekian (5) has concluded that sludge reaches an optimum condition for seeding purposes when decomposition or digestion is just

[1] The reaction of well-digested sludge is stable and not easily altered.

complete and that the condition is maintained for about a month thereafter.

Temperature.—Since digestion is a biological process, decomposition proceeds most rapidly at temperatures at which life processes of the organisms of decay are most active. As shown in Fig. 92, the optimum conditions for normal or readily maintainable temperatures are encountered between 80 and 90°F. Below this range, there is a rapid falling off in the rate of digestion. According to this diagram, if the digestion time at 60°F. is 4 months, the time at 72° is 2 months and at 85°, 1 month. In the report of the Committee on Sludge Digestion, Sanitary

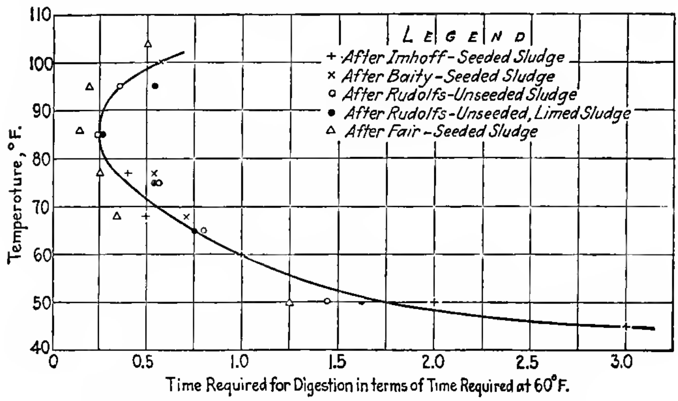


Fig. 92.—Influence of temperature on sludge digestion.[1]

Engineering Division, American Society of Civil Engineers, the effect of temperature on the time of sludge digestion was summarized as follows (6):

Temperature, °F.	Number of Days Required for Digestion, before Sludge Is Ready for Withdrawal
55	120
68	42
82	30
120–140	12
170	11

In septic and Imhoff tanks, the temperature of the sludge lags only slightly behind that of the sewage. From Tables 26 and 27 it will be seen that the average annual temperature of sewages in the northern part of the United States is about 60°F. and that temperatures favoring rapid digestion are restricted to the summer and autumn of the year.

[1] For additional information, see Fair and Moore (9).

Low temperature retards digestion, necessitating storage of the sludge in the digestion units for longer periods of time. A knowledge of prevailing sewage temperatures is, therefore, essential in determining upon the sludge-storage space to be provided in septic and Imhoff tanks.

In unheated separate sludge-digestion tanks the temperature of the sludge follows more closely the air temperature and averages lower than for septic and Imhoff tanks. Utilization of the gases of decomposition for the purpose of heating separate sludge-digestion tanks has recently come into vogue. The temperatures commonly maintained range from 70 to 90°F.

Thermophilic Digestion.—Digestion at the higher ranges of temperature is still under investigation. Present indications are that at temperatures between 125 and 145°F. digestion proceeds rapidly, owing to the development of heat-loving, or thermophilic, organisms.

Experiments with thermophilic digestion by Rudolfs and Heukelekian (7, 8) have indicated that the rate of digestion proceeds most rapidly at 55°C., or 131°F. At 50°C. it was found possible to secure a reduction of 73.3 per cent of the volatile matter in fresh solids in a theoretical digestion time of 2.1 days. Moreover, it was believed that the time might be reduced, as 88.5 per cent of the total gas produced was evolved within 24 hr. after charging. Similar results were secured in the digestion of activated sludge. It was noted that, presumably owing to the high temperature, a considerable portion of the sludge introduced had a tendency to rise to the surface. The practical significance of sludge digestion under thermophilic conditions has not as yet been established, although it has been stated that application for U. S. patent on this process has been filed by Heukelekian and that his interest in the prospective patent has been acquired by the Pacific Flush-Tank Co.

After studying the data of various authors as to the effect of temperature on the time necessary for digestion, Fair and Moore (9) find that thermophilic digestion seems to come into play in the vicinity of 125°F. and that the most favorable temperature for thermophilic digestion is near 131°F. Their figures indicate that the optimum temperature for nonthermophilic digestion appears to be between 86 and 99°F. The range between 99 and 125°F. seems to represent a region in which both thermophilic and nonthermophilic organisms work at a disadvantage.

Reaction.—As previously pointed out, the three stages of digestion are associated with marked differences in the reaction of the sludge, the activities of the microorganisms resulting during the first stage in acid conditions which are overcome but slowly during the second and third stages. During the period of intensive acid production, organic acids are formed faster than they are broken down; in the succeeding periods the reverse is true, as indicated in Fig. 90.

In sludge-digestion units the changes in reaction and in the accompanying rates of digestion are most pronounced under the following conditions: during the ripening period; when excessive quantities of fresh sludge are added to digesting solids; when large volumes of solids that have accumulated during periods of cold weather begin to "work," as the sludge is warmed by increasing temperatures; when the sludge contains acid or acid-forming industrial wastes; and when fresh sludge is allowed to accumulate or remain unseeded before passing into the digestion tank. Ordinarily in a well-designed and well-operated digestion tank it is possible, in the absence of interfering industrial wastes, to maintain a balance between fresh and digested solids such that the reaction will remain favorable. This is due to the alkaline condition of the digested material and its buffer action, discussed in Chap. III.

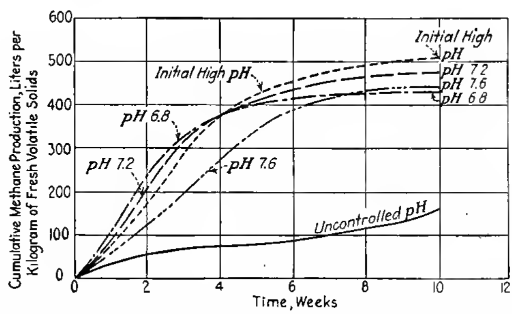


FIG. 93.—Effect of reaction control on sludge digestion.[1]

Artificial adjustment of the reaction can also be resorted to. Fair (10) has shown that the salts of alkaline-earth metals, but not of alkaline metals, can be used for the purpose of raising the pH of digesting sludge. Lime is most commonly employed. The effect of increasing the pH of sludge from Brockton, Mass., a sludge digestible with difficulty owing to the presence of industrial wastes, is illustrated in Fig. 93. The optimum pH is apparently in the vicinity of 7.2 or 7.3, but any value above 6.8 seems to produce good results.

Lime, added to digesting sludge in suitable quantities, affects digestion, not only by neutralizing acid materials, but also by stimulating the growth of organisms responsible for the digestion of nitrogenous substances and by causing the flocculation and precipitation of acid colloidal materials, particularly in the scum. At the same time liming permits gas bubbles held in the scum to escape, owing in part probably to changes in surface tension that take place.

[1] Curve marked "Initial High pH" represents sludge adjusted to an initial pH of 8.0 and then allowed to take its own course. During its period of greatest methane production this sludge registered a pH of 6.8 to 7.0.

The quantity of lime required to raise the pH of fresh solids or partly decomposed matter to 7.3 has been determined by Rudolfs (2) to be as shown in Fig. 94.

Both activated sludge and fine screenings will digest in much the same way as fresh sewage solids. Naturally there are quantitative and qualitative differences, as appears in the chapters devoted to these methods of sewage treatment.

Certain industrial wastes affect digestion adversely. Acids and alkalies may interfere. Mineral oils seem to have but little effect.

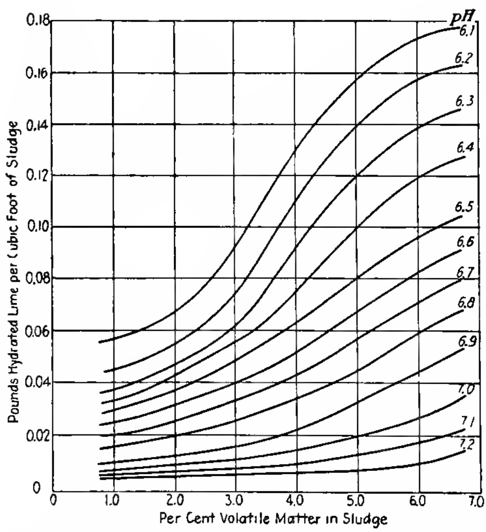


FIG. 94.—Quantity of lime required to raise the pH of fresh or partly decomposed sludge to 7.3.

Rudolfs (1) found that grease decomposes but slowly and may retard digestion, but that small quantities of iron wastes appear to accelerate the digestion processes, while increasing sedimentation and reducing odors, especially hydrogen sulfide odors.

On the other hand, Keefer and Kratz (11) report, as a result of small-scale experiments, that scum from primary-sedimentation tanks, containing 84 per cent ether-soluble matter on a dry basis, digests at least as rapidly as primary sludge, if not more so.

Two-stage Digestion.—In some plants, particularly in Germany, sludge digestion is carried on in two stages, by using two sets of separate digestion tanks in series.

Observations by Buswell, Symons and Pearson (12) on two-stage sludge digestion lead them to believe that there is at first a relatively rapid fermentation which results in the decomposition of the simpler compounds and the production of a large quantity of gas, including most of the hydrogen sulfide. This fermentation is apparently complete in a very few days. This observation is in accord with that of Hatfield (13) and others, who have observed that 50 per cent of the gas is evolved in the first 24 hr. After this fermentation period, it is necessary to allow the sludge to undergo some sort of a ripening process. The exact nature of this is not understood, but the net result is that the sludge loses its water-binding properties and can then be drained on sand beds.

The complete digestion process could best be carried out in a separate sludge digestion plant consisting of a relatively small primary tank designed to allow 6 or 8 days detention and equipped with the necessary circulating devices to prevent scum and foam formation, followed by a secondary tank or even a lagoon of sufficient size to allow for the necessary ripening of the sludge to a state where it will drain on sand beds. This would result in the following economies: (1) cost of cover would be reduced to approximately one tenth; (2) since no scum is observed in the secondary stage of digestion, no special measures would have to be taken to prevent scum formation; (3) it is possible that only the primary tank would need to be heated, and (4) the secondary tank might be replaced by lagoons.

At the joint disposal plant of the Los Angeles County Sanitation Districts, constructed in 1931, sludge from plain-sedimentation tanks is digested in four stages, a separate pair of tanks being provided for each stage of digestion (14).

A test of two-stage digestion at 90°F. at Peoria, Ill., is reported upon by Kraus (15) as follows:

Due to the difficulty of maintaining a suitable supernatant liquor using single stage digestion, two stage digestion was undertaken with the idea of employing short periods in each tank; the first tank being given over to digestion only, and the second acting to complete the digestion and to allow the separation of supernatant liquor and sludge.

The sludge treated during the period of the above operation was a mixture of activated sludge, fresh solids and digested "beer slop" solids. The data on this operation have been corrected where possible for the digested "beer-slop" solids present. ["Beer slop" is the waste fermented mash from the Commercial Solvents plant.]

Primary tank sludge was pumped into the first stage of digestion and simultaneously sludge was removed from the bottom of the tank at the same rate and for the same period of time to the second stage. Practically no supernatant liquor was wasted from the first tank. Both sludge and supernatant liquor were wasted from the second tank.

The amount of gasification in the first stage was sufficient to make the sludge homogeneous throughout. This great agitation afforded excellent seeding for the raw sludge added to the tank and at no time was it necessary to circulate the sludge from bottom to top.

The gas production in the first stage of digestion, using a 10.7-day period, was 84.4 per cent of the total gas obtained from both stages. The degree of digestion, as measured by the reduction in volatile solids, was 82.8 per cent of the total digestion.

The two tanks used for the experiment at Peoria are 85 ft. in diameter and have a side-wall depth of 29 ft. The capacity of each tank is about 167,500 cu. ft. The charging rate was 27,900 lb. daily, on the basis of dry solids, of which 20,000 lb. were volatile solids. The daily gas production was 94,400 cu. ft. from the first stage and 17,500 cu. ft. from the second. The solids in the sludge averaged 3.02 per cent as added to the first stage, 2.47 per cent as added to the second stage and 3.34 per cent as drawn. The volatile solids in the sludge were reduced from 72.20 per cent as added to the first stage to 63.56 per cent at the end of the first stage and 61.52 per cent as drawn from the second stage.

As a result of the experiment Kraus drew the following conclusions:

That digestion at 90°F. with a 10-day period will result in a 40 per cent reduction of volatile solids.

That digestion tanks should be operated at such a charging rate as will result in sufficient agitation by gasification to make the entire tank contents homogeneous. This results in excellent seeding for the undigested solids and enables tanks to be operated at high rates.

That separation of supernatant liquor and sludge can best be obtained in secondary digestion tanks.

That, since the amount of gas produced in secondary tanks is small, these tanks may be uncovered.

GAS PRODUCTION AND UTILIZATION

Until recently few attempts were made to utilize the gases given off during digestion of sewage solids, although their value has long been recognized. When the advantage of heating sludge-digestion tanks to reduce the digestion period was demonstrated, the possible economy of collecting and utilizing the gases of decomposition for heating the sludge was quickly appreciated. The facility with which such gases could be collected in roofed, separate sludge-digestion tanks was a contributing factor in the spread of their utilization. At some plants, the gases are advantageously used as fuel in gas engines for pumping sewage or sludge or compressing air. The collection and burning of such gas may be worth while to prevent the escape of offensive odors, even though the energy of the gas is largely wasted.

Quantity of Gas.—The volume of gas produced depends largely upon the quantity and character of sewage solids digested. Between 550 and 700 liters of gas are commonly produced during complete digestion by a kilogram of volatile, or organic, fresh sewage solids settling from domestic sewage, equivalent to 8.0 to 11.2 cu. ft. per pound. Between 70 and 80 per cent of this gas is combustible, being largely methane.

TABLE 65.—DATA ON GAS PRODUCTION

Imhoff Tank at Calumet Treatment Works; Average Results for 1 Year, 1926–1927

Volatile solids removed, p.p.m.	37
Volatile solids removed, lb. per day	430
Gas produced, cu. ft. per day	1924
Gas produced, cu. ft. per lb. volatile matter added.	4.5
Gas produced, cu. ft. per capita per day	0.44
Weight of 1 cu. ft. of gas, lb	0.054
Gas produced by weight from volatile matter added, per cent	24.3

TABLE 66.—QUANTITY OF GAS PRODUCED BY DIGESTION OF SLUDGE

Location of plant	Year	Gas produced	
		Cu. ft. per lb. volatile matter added	Cu. ft. per capita daily
Primary sludge:			
Antigo, Wis.	....	.....	0.66
Aurora, Ill.	1933	13.6	0.56
Chicago, Ill., Calumet plant[1]	1930	4.8	0.44
Chico, Cal.	1929	.....	2.1
Dayton, Ohio[1]	....	4.6	0.33
Fond du Lac, Wis.	....	.....	0.71
Grand Rapids, Mich.	1933	7.46	0.88
Sturgis, Mich.	....	.....	0.41
Primary sludge and humus sludge:			
DeKalb, Ill.	1930	11.3	1.10
Forth Worth, Tex.	....	5.8	0.57
High Point, N. C.	1929	3.5	0.21
Plainfield, N. J.	1930	6.0	0.32
Waukesha, Wis.	1930	.....	1.08
Primary sludge and activated sludge:			
Charlotte, N. C.	1929–1930	.....	0.50
Elyria, Ohio	1931	7.2	0.50
Peoria, Ill.	1932	8.3	1.17
Salem, Ohio	....	8.22	1.38
Springfield, Ill.	1933	8.52	0.60
Toronto, Ont., No. Toronto plant	1931	8.2	0.88

[1] Imhoff tanks; all others are separate sludge-digestion tanks.

The remainder is chiefly carbon dioxide and nitrogen. Under ordinary conditions of operation, digestion is not carried to completion and only about 75 per cent of the total gas evolution takes place. On this basis an average figure for per capita gas production from primary sludge may be about 0.4 cu. ft. daily.[1] In the case of sludge from complete treatment processes, such as activated sludge, or in the presence of large quantities of organic industrial wastes, the average value may be exceeded twofold or more.

A test of gas production at Chicago is summarized in Table 65 (16).

The average flow of sewage through the Calumet tank was 1.4 m.g.d. and the contributing population was 4400. The storage capacity for sludge was 2.3 cu. ft. per capita. The Calumet sewage contains 116 p.p.m. of suspended matter, of which 55 per cent is volatile. The tank was operated without heat or pH control. The gas produced during the maximum month, September, and maximum day was, respectively,

TABLE 67.—ANALYSES OF SLUDGE GAS

Municipality	Per cent by volume						Net heating value, B.t.u. per cu. ft.
	CO	H_2	CH_4	CO_2	O_2	N_2	
Imhoff tanks:							
Chicago, Ill., Calumet plant	...	...	76.6	14.7	0.5	8.2	696
Dayton, Ohio	...	0.0	76.6	14.0	0.4	9.0	695
Decatur, Ill	...	...	69.0	16.8	0.1	14.1	628
Stuttgart, Germany	...	4.7	75.5	14.0	....	4.7	700
Separate sludge-digestion tanks:							
Antigo, Wis	...	2.6	62.0	31.4	0.6	3.4	564
Aurora, Ill	2.1	0.0	51.8	32.3	0.4	13.4	471
Baltimore, Md	...	...	70.5	26.5	0.2	2.8	641
Birmingham, England	...	...	77.0	18.1	0.4	3.2	700
Elyria, Ohio	...	...	69.4	30.0	0.5	....	630
Grand Rapdis, Mich	...	2.4	63.5	30.5	0.14	3.4	580
Halle, Germany	...	...	72.9	24.0	0.6	1.6	663
Milwaukee, Wis	0.6	...	67.5	30.0	0.2	1.7	614
Peoria, Ill	...	...	67.5	27.8	....	4.7	615
Plainfield, N. J	...	0.0	65.8	30.6	0.0	3.6	599
Springfield, Ill	...	1.7	64.5	31.0	....	3.2	591
Toronto, Ont., North Toronto plant	...	3.7	58.5	28.0	1.8	8.0	541

[1] On the basis of a daily sewage flow of 100 gal. per capita and removal of 70 p.p.m. of organic settling solids, the daily per capita gas production, 75 per cent complete, equals

$$\frac{0.75 \times 600 \times 70 \times 100 \times 3.78}{1{,}000{,}000} = 12 \text{ liters} = 0.42 \text{ cu. ft.}$$

2.52 and 3.3 times the average daily production. During the minimum month, March, and minimum day gas production fell, respectively, to 0.37 and 0.08 times the average daily quantity.

The volume of gas produced at a number of American sewage treatment plants is given in Table 66.

Character of Gas.—The chemical composition of gas reported for Imhoff and separate sludge-digestion tanks is presented in Table 67.

The gas at any plant is constantly changing in character, but the analyses given in Table 67 may be considered typical. The net heating value has been calculated from the data given, on the basis of 908.5 net B.t.u. per cubic foot for CH_4 and 271.8 net B.t.u. per cubic foot for H_2.

As a rule, the gases contain 60 to 80 per cent methane, 15 to 30 per cent carbon dioxide, and 5 to 15 per cent nitrogen and miscellaneous gases. The net heating value is usually 500 to 700 B.t.u. per cubic foot. The value of the gas at 700 B.t.u. per cubic foot is about as follows:

One cubic foot has sufficient heat value to evaporate 0.72 lb. of water from and at 212°F., without allowance for any losses, and 40,000 cu. ft. of gas are equivalent to 1 ton of coal at 14,000 B.t.u. per pound. At 80 per cent efficiency, 60 cu. ft. of gas are equivalent to 1 boiler-hp. In a commercial gas engine, a flow of 14 to 15 cu. ft. of gas hourly may develop 1 brake hp.

Utilization of Gas.—Among the possible uses for the gas produced by digesting sludge are: the heating of separate sludge-digestion tanks to maintain a temperature favorable to rapid digestion; the heating of buildings about the plant and possibly sludge-bed glass-overs; the incineration of skimmings and screenings; utilization as fuel in internal-combustion engines; and sale to gas works.

Heating Purposes.—The value of heating separate sludge-digestion tanks to increase their effective capacity has been mentioned on page 362, together with the fact that the gases produced by sludge-digestion are sometimes utilized for heating the tanks. For this purpose the gases are commonly burned under hot-water or steam boilers. The hot water is either discharged directly into the digestion tanks, as at Essen-Rellinghausen, Germany, or circulated in pipe coils within the tanks, as at Peoria, Ill. Steam is either discharged directly into the digestion tanks or used in heating coils within them.

Because of the large percentage of carbon dioxide present in the gas, burners of the type used with manufactured gas are not suitable. The orifices are too small, causing the flame to blow away from the burners which are adjusted for much faster burning mixtures. Burners of the type used for natural gas are suitable for the gas from sludge digestion.

As shown by Walraven (17) at Springfield, Ill., the gas may be utilized economically for fuel in gas engines and about 55 per cent of the total heat units may be recovered from the water jacket and exhaust gases for heating purposes.

The gas is admirably suited for use in conjunction with the *incineration of skimmings and screenings.* The screenings incineration plant at Long Beach, Cal., utilizes natural gas, which is similar to the gas produced by sludge digestion. Gas collected from Imhoff tanks at Dayton, Ohio, is used for incinerating the screenings. The volumes of gas required for the incineration of screenings at Long Beach and at Dayton are given in Chap. X.

The gas may be *utilized for fuel in internal-combustion engines.* The average gas engine develops 1 brake hp. on 10,500 B.t.u. per hour. At 700 B.t.u. per cubic foot the average requirements would be 15 cu. ft. per hour per brake hp. A 55-hp. gas engine operating on sludge gas is provided at the Sugar Creek plant, Charlotte, N. C., for driving a 3500-g.p.m. sewage pump under a 12-ft. head. A 225-hp. gas engine, which may be adapted to the burning of sludge gas, is provided at this plant for driving a 3500-c.f.m. centrifugal air compressor. It is reported that "the gas engines operate exceptionally well on the gas, and this source of power materially reduces the power costs for plant operation" (18).

At Springfield, Ill., sludge gas from digestion tanks is collected and utilized for fuel in a gas engine, rated at 177 i.hp. at 514 r.p.m., direct-connected to a positive-pressure blower of the Connersville type, designed to deliver 3300 cu. ft. of air a minute against a discharge pressure of 8.5 lb. per square inch. Water for cooling the engine is circulated by centrifugal pumps through the engine water jacket, through a heat exchanger, where it is heated by the exhaust gases from the engine, and thence through coils within the digestion tanks, where the excess heat is absorbed before the water returns to the engine.

Walraven (17) has reported as a result of tests that the engine develops 1 brake hp. on 10,085 B.t.u. per hour. The heat recovered and put into the digestion-tank heating system is 133,000 B.t.u. daily per brake horsepower, in addition to the useful work. The total heat recovery is 80 per cent, divided as follows:

As useful work	25.2 per cent
From water jacket	36.8 per cent
From exhaust heat exchanger	18.0 per cent

The gas is not purified and no evidence of damaging corrosion has been found after operation for more than a year. Walraven estimates that 67 per cent of all the power required at an activated-sludge plant can be generated from the gas, provided that complete treatment and adequate sludge-digestion and gas-recovery units are installed.

During 1933, the gas engine at Springfield was operated at full or part capacity on 355 days, producing about 65 per cent of the power required for air compression. During January, 1934, 94.3 per cent of the power required for air compression was supplied by the gas engine. These results were obtained with an average air consumption of 1.02 cu. ft. per gallon of sewage, a tributary population of 70,000, an average sewage flow of 6.7 m.g.d. and an average gas production of 0.6 cu. ft. per capita daily (19).

At Newark, N. Y., a 15-hp. gas engine is direct-connected to a positive-displacement blower, having a capacity of 360 cu. ft. of free air a minute at 350 r.p.m. against a pressure of 5 lb. per square inch. The engine was guaranteed to have a fuel consumption not greater than 11,000 B.t.u. per brake hp. at full load, 12,000 B.t.u. at three quarters load and 14,000 B.t.u. at half load. Holmes (20) reports upon the operation of this gas engine as follows:

> The principal trouble with the gas engine has been due to pitting of the exhaust valves causing loss of compression. The gas has been unusually high in H_2S which burns slowly and is not fully consumed within the cylinder causing trouble in the exhaust piping and the heat exchanger. These troubles have been at least partially corrected by substituting new exhaust valves of higher heat resistance, increasing the temperature of the circulating water in the cylinder jacket from 120 to 150°F., and changing the exhaust hook-up to the exchanger.

There are two gas engines at the sewage-treatment plant in Rockville Center, N. Y. One engine is direct-connected to a positive-pressure blower with a capacity of 400 to 800 cu. ft. a minute, depending upon speed. A second engine, operating at 1200 r.p.m., is direct-connected to a 25-kva. generator. Water jackets on the engines and exhaust pipes are connected to the system for heating the sludge-digestion tanks. Gavett (21) states that the gas engines have operated very satisfactorily, requiring about 20 to 30 cu. ft. of gas per horsepower-hour.

Hazeltine (22) has stated that not more than 0.1 per cent of hydrogen sulfide should be present in gas used for power and that variations in methane content make advisable frequent and careful adjustment of the gas-air mixture, to insure the most efficient engine operation.

Use for City Gas.—Fulweiler (23) has made a study of the possibilities of using the gas produced by sludge digestion for city gas. He concludes that the quantity of gas produced will rarely exceed 1 per cent of that required within the municipality, that it cannot be expected to make any important contribution to the gas supply and that it will not permit of any saving of investment or operating labor in the gas plant.

As far as is known to the authors, no American plant has as yet sold gas produced by digestion of sewage sludge for use as city gas.

Collection and Burning of Gas to Prevent Offensive Odors.—The collection and burning of gases from the decomposition of sludge may be of value in preventing the escape of offensive odors, even though the gases cannot be economically utilized. The burning of gases at Austin, Tex., where operation of Imhoff tanks resulted in litigation against the city on account of offensive odors, has been described by Leonard (24).

Gas is burned in waste-gas incinerators at North Toronto, Ont., and at Dayton, Ohio, when not required for heating purposes, so as to preclude the possibility of the escape of offensive odors.

Gas Holders.—In a few plants gas holders have been installed in conjunction with the collection and utilization of gas. They are said to afford the advantages of stabilizing the flow of gas to the burners and maintaining gas pressure more nearly constant. They have the further advantage of supplying gas during the time when sludge is being withdrawn from digestion compartments of fixed-roof tanks or while repairs are being made to gas domes. It is noteworthy that, where gas is utilized in gas engines, holders generally have been provided, as at Charlotte, N. C., Newark and Rockville Center, N. Y., and Springfield, Ill.

The gas holder at Springfield has a capacity of about 20,000 cu. ft. The gas holder at the Sugar Creek plant in Charlotte has a capacity of 30,000 cu. ft. and operates with 6-in. water pressure. The capacity of the gas holder at Newark, N. Y., is 5000 cu. ft., as compared with an average daily gas production of about 8000 cu. ft. The gas is stored under 6-in. water pressure. The well is placed below the ground surface and the lift, or dome, is completely enclosed. Installation of gas holders within digestion tanks, as at Lancaster, Pa., or within storage tanks, as at Springfield, Ill., is a recent development.

At Dayton, Ohio, the gas collected from Imhoff tanks is compressed to a maximum of 50 lb. per sq. in. and stored in two pressure tanks, from which it is drawn through pressure reducers. The volume of gas which can be stored in the tanks is equivalent to 15,600 cu. ft. at atmospheric pressure.

In 1931 a patent was granted to Laird, covering a gas holder floating on the pipe line which conducts gas out of a gas-producing tank.

Measurement of Gas.—Measuring gas produced by the digestion of sludge is important as a guide in the operation of the sludge-digestion process and in the operation of gas-utilization or disposal equipment. Various methods of measuring the flow of gas are in use. In small plants the bellows type of meter similar to the standard house gas meter is commonly used. In larger plants orifice plates and venturi tubes, in conjunction with registering and recording meters, and rotary displacement meters of the Connersville type are in use.

Gas Hazards.—Sludge gas normally contains a mixture of gases, some of which form explosive mixtures with air and others are poisonous. It is important that collection, utilization and disposal of sludge gas be effected with due regard to the hazards involved.

Carbon dioxide, which is usually present in sludge gas in proportions of 15 to 30 per cent, will not support human life. A concentration of 30 per cent may be fatal in 30 min. or less (25). The maximum safe percentage of the gas in air is considered to be 2 to 3 per cent. Carbon monoxide, sometimes reported in sludge-gas analyses in proportions of 1 to 2 per cent, is very poisonous. It is considered fatal in 30 min. or less in proportions of 0.5 to 1.0 per cent in the gas-air mixture and the maximum safe concentration for long exposure is 0.01 per cent. Methane, which is present in sludge gas in proportions of 60 to 80 per cent, is not a poisonous gas but may dilute the oxygen of the air to such an extent as to cause asphyxia. A gas-air mixture in which the oxygen falls below 10 per cent may cause unconsciousness.

Sludge gas, principally because of its methane content, forms explosive mixtures with air. The limits for explosive mixtures of methane and air are from 6.8 to 15.4 parts of air to 1 part of methane.

Hazeltine (22) gives the explosive limits of sludge-digestion gas as 6.0 to 15.1 cu. ft. of air per cubic foot of gas containing 80 per cent methane, 15 per cent carbon dioxide and 5 per cent nitrogen. For sludge gas containing 60 per cent methane, 30 per cent carbon dioxide and 10 per cent nitrogen, he reports the explosive limits as 5.1 to 11.1 cu. ft. of air per cubic foot of sludge gas.

The explosion of a mixture of sludge gas and air may do considerable damage. This was demonstrated by a violent explosion which wrecked three sludge-digestion tanks at Woonsocket, R. I., in 1931, with the loss of two lives. In this case a man entered a tank, which had been ventilated only by open manholes and within which a considerable quantity of sludge still remained, and is believed to have caused the explosion by striking a match.

Protection against Gas Hazards.—In conjunction with the sludge-digestion process it is wise to provide protective measures against the hazards of asphyxia, poisoning and explosion, which may be caused by gases. These hazards commonly are minimized by providing a tight gas-collection system between the digestion compartments and the burners, with means for preventing flashback from the burners. Manholes, pits and tanks where there is opportunity for gas to collect or where sludge has collected are sometimes ventilated, as with a portable blower during the time required for men to work in them, on inspection, renewals or repairs. Notices may be posted about the plant warning of explosion hazards and plant employees should be instructed in safety measures to be maintained while working in and about the structures where gas may be present.

Unless provision is made to the contrary, there is danger of flashback from gas burners to digestion tanks. This is usually prevented by inserting a water seal on the gas line to the burners, as illustrated in

Fig. 106, which shows diagrammatically the gas piping and water-heating apparatus at the Antigo sewage-treatment plant (7). An explosion vent located in the open is usually provided, connected to the pipe line between the water seal and the burners. A paper gasket on a standard flange is sometimes used. Provision is also commonly made for the discharge of excess gas through pressure-relief valves. Pressure-reducing valves generally are employed to insure suitable pressures at the burners. Facilities for measuring the quantity of gas produced are often included in the equipment. A drip pot, placed on the gas line for the collection of moisture, may be advantageous.

At the North Toronto plant, fire checks or flame arresters of the Kemp type are installed on the gas main between the digestion tanks and the boilers. Each fire check consists of a combination of perforated refractory disks and tightly rolled copper gauze, assembled in a container. The purpose of such a check is to prevent the propagation of flame past the fire check, irrespective of the proportions of air and gas in the mixture which may reach it, and regardless of the velocity of flow of gas in the pipe. Provision is made for cleaning each check by blowing steam through it to the atmosphere.

Bibliography

1. Rudolfs, Willem: *Jour.*, Boston S. C. E., 1929; **16**, 37.
2. Rept. of Sewage Substation, N. J. Agricultural Experiment Sta., 1926.
3. Rudolfs, Willem: *Ind. Eng. Chem.*, 1927; **19**, 241.
4. Rudolfs, Willem, H. Heukelekian and P. J. A. Zeller: *Am. Jour. Pub. Health*, 1926; **16**, 365.
5. Heukelekian, H.: *Sewage Works Jour.*, 1930; **2**, 313.
6. Rept. of Comm. on Sludge Digestion, Sanitary Eng. Div., Am. Soc. C. E., 1931.
7. Rudolfs, Willem, and H. Heukelekian: *Ind. Eng. Chem.*, 1930; **22**, 96.
8. Rudolfs, Willem, and H. Heukelekian: *Ind. Eng. Chem.*, 1931; **23**, 67.
9. Fair, G. M., and E. W. Moore: *Sewage Works Jour.*, 1934; **6**, 3.
10. Fair, G. M., and C. L. Carlson: *Jour.*, Boston S. C. E., 1927; **14**, 82.
11. Keefer, C. E., and Herman Kratz: *Sewage Works Jour.*, 1931; **3**, 50.
12. Buswell, A. M., G. E. Symons and E. L. Pearson: *Sewage Works Jour.*, 1930; **2**, 214.
13. Hatfield, W. D., G. E. Symons and R. R. Mills: *Ind. Eng. Chem.*, 1928; **20**, 174.
14. Rawn, A. M.: *Civil Eng.*, 1931; **1**, 997.
15. Kraus, L. S.: *Sewage Works Jour.*, 1933; **5**, 623.
16. Zack, S. I., and G. P. Edwards: *Sewage Works Jour.*, 1929; **1**, 160.
17. Walraven, W. B.: Paper delivered before Ill. Soc. of Eng., Jan. 31, 1934.
18. McConnell, E. G.: *Public Works*, 1928; **59**, 438.
19. *Sewage Works Jour.*, 1934; **6**, 169.

20. Holmes, G. D.: *Sewage Works Jour.*, 1933; **5**, 793.
21. Gavett, Weston: *Sewage Works Jour.*, 1931; **3**, 615.
22. Hazeltine, T. R.: *Sewage Works Jour.*, 1933; **5**, 482.
23. Fulweiler, W. H.: *Sewage Works Jour.*, 1930; **2**, 424.
24. Leonard, C. E.: *Eng. News-Rec.*, 1922; **88**, 565.
25. Sayers, R. R.: Sixth Ann. Rept., Ohio Conference on Sewage Treatment, 1932; p. 11.

CHAPTER XV

SEPTIC TANKS

A septic tank is a sedimentation tank intended to retain the sludge in immediate contact with the sewage flowing through the tank for a sufficient period of time to secure a satisfactory decomposition of organic solids by anaerobic bacterial action (1).

Except for small, usually private installations, in which single-story septic tanks find wide application, as discussed in Chap. XXXI, this type of tank has been displaced largely by Imhoff tanks or by plain-sedimentation tanks combined with separate sludge-digestion units. There are still in use, however, many single-story septic tanks and occasionally such tanks are installed for the smaller municipalities.

Some of the factors unfavorable to the use of septic tanks are: the escape of solids in the effluent, which may at times increase the suspended solids content of the effluent above that of the influent and may cause clogging of contact beds or filters which receive the effluent; the septic character of the effluent, which often increases its oxygen avidity, is likely to render it offensive, and sometimes unfits it for further biological treatment; the nature of the sludge removed, which may contain a large proportion of insufficiently digested solids, unless tanks are operated in rotation and each tank, after the sludge-storage capacity has been reached, is allowed to remain idle for a relatively long period of time before the tank is cleaned; the difficulty of cleaning the tanks, especially in breaking up the scum layer, which is sometimes tough and several feet thick; the greater prevalence of objectionable odors; the better economy of other treatment methods, especially as regards the required tank capacity.

Characteristics of the Septic Process.—The septic process, aside from the physical sedimentation of the suspended solids, depends upon anaerobic bacterial action. Formerly it was considered essential to the most vigorous development of this action that it be carried out in the absence of atmospheric oxygen and in the dark. Experience has shown that this is not so. The bacterial action is similar to, and a continuation of, that going on in the sewers. The solids which accumulate at the bottom of the tank as sludge are disintegrated, coarse matter being transformed into finely divided particles. This action converts some of the organic solids into gases, as carbon dioxide, methane, nitrogen and hydrogen, or into soluble substances which pass out of the tank dissolved in the effluent. By gasification some of the sludge is lifted to the surface and a floating scum layer is formed, in which other floating solids are enmeshed. Sludge particles which are freed from gases at the surface lose their buoyancy and settle back to the bottom. An upward and downward movement of sludge is thus maintained. In an effort to prevent the escape of sludge particles in the effluent, much longer detention periods are provided than in plain-sedimentation tanks and carefully designed baffles are installed in some cases.

The scum which forms at the surface of the sewage consists chiefly of the coarse suspended matter which tends to float. Its quantity depends mainly on the character of this suspended matter. If the sewage is fresh and the suspended matter is not much disintegrated, large quantities of scum are probable. The suspended matter brought to the surface of the sewage sometimes forms such a compact mass that the entrained gases can be liberated but slowly. Meanwhile 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 cover its surface. Much trouble has been experienced under some conditions, particularly in the South, from the breeding of flies in this mat, necessitating screening and treatment with oil or other insecticides. Digestion of solids is greatly retarded in the scum.

Elements of Tank Design.—Septic tanks serve a twofold purpose, by providing for the deposition of the settling solids by sedimentation and for the partial or complete digestion of the sludge prior to its disposal. Horizontal-flow, flat-bottom or hopper-bottom tanks are employed almost exclusively. Relatively large sludge-storage capacity is provided, in order that the deposited sludge may remain in the tank for a sufficient length of time to undergo decomposition or digestion before being withdrawn. Such tanks are usually provided with concrete roofs.

In so far as a septic tank acts as a sedimentation tank, the principles of design outlined in Chap. XII apply to it. They are modified, however, by the special conditions introduced by the digestion of the sludge and the resulting characteristics of the sewage in the tank. The leading factors to be considered in design are:

1. Character of sewage reaching the tanks
2. Effect of septicized sewage on subsequent treatment processes
3. Number of tanks necessary to give the desired time of detention of the sewage over the sludge

4. Covering the tanks
5. Inlets and outlets which will not be affected by scum
6. Provision for sludge storage and removal.

Influence of Character of Sewage Reaching Septic Tanks.—Inasmuch as a septic tank is intended to improve both chemical and physical qualities of sewage, it is manifestly important to consider the possibility of the sewage reaching the works in different conditions at different times. In some cities industrial wastes are discharged into the sewers at certain hours and reach the treatment plant without much dilution. If there is ample tank capacity to receive and dilute these wastes, they will be not only unlikely to cause fluctuations in the results accomplished by the septic tanks, but also of little significance during the stages of treatment carried on in contact beds or filters. The equalizing effect of ample storage facilities on the variable character of sewage reaching the works may be an important feature for the designer to consider. In 1898 investigations were made at the Lawrence Experiment Station of the Massachusetts State Board of Health, which indicated that sewage which has traveled for any considerable distance in sewers will be practically devoid of dissolved oxygen when it reaches its point of disposal and need not remain in the septic tank longer than is necessary for precipitation of suspended organic matter and accumulation of fats upon the surface of the sewage. The report for 1899 modified this statement by saying that, to obtain the best results, sufficient time for the installation of the desired bacterial life is necessary and this time varies with different kinds of sewage and at different seasons.

Effect of Septicized Sewage on Filtration.—If sewage remains too long in the presence of sludge undergoing anaerobic decomposition, it becomes offensive and more difficult to treat by subsequent oxidation processes. Sewage having such properties is termed "oversepticized." It is desirable, in cases where septic tanks are to be used for preliminary treatment, to provide a sufficient number of small tanks to allow the number of units in use to be varied in the most advantageous way.

The danger of keeping sewage too long in the tanks was explained by the Massachusetts State Board of Health as follows (2):

> It seems probable—although this is not yet proved—that the anaerobic action has been carried so far before the sewage reaches the filters that various bodies have been generated in the sewage which prevent the development of the nitrifying bacteria in the filters. The sewage has an odor more nearly resembling that of the wastes from a cesspool than that of ordinary fresh, stale or septic sewage. It is certain that, if the anaerobic process is carried too far, there may be a formation of distinctly poisonous bodies, which might prevent nitrification.

Further investigations showed that, when a small volume of sewage is applied to a sand filter, nitrification will take place, no matter what degree of putrefaction this sewage has attained at the time of its application, if there is an abundance of air to come into contact with the sewage (3). When, however, sewage in an advanced state of putrefaction is applied to a contact filter and the entire open space of the filter is filled with this sewage, Clark and Gage consider it possible that oxidation may be so rapid that the supply of oxygen within the filter will be exhausted before the process of nitrification has had time to begin. The slow absorption of oxygen by fresh sewage and its rapid absorption by stale or septic sewage have been proved by a number of experiments. The application of a small volume of oversepticized sewage to an intermittent sand filter, where there is an opportunity for considerable oxidation on the surface of the bed and there is a large quantity of air in the filtering medium, presents far different conditions from those of a contact bed having less opportunity for nitrification. In 1901 it was found that by aerating an oversepticized sewage it could be treated satisfactorily in a contact bed, which was impossible without the aeration.

Number of Tanks and Detention Period.—The size of tanks depends on the necessary detention period, character of sewage, space desired for sludge accumulations in the tanks and local topography. The best size having been decided upon, the number of tanks depends upon the quantity of sewage to be treated. It is customary to construct several tanks, sometimes of the same size and sometimes of different sizes, even in the case of small treatment plants.

Septic tanks ordinarily have a capacity for 8 to 24 hours' dry-weather flow, in addition to the sludge-storage space. Experience has shown that detention periods of 8 to 12 hr. may lead to the discharge of gas-lifted solids with the effluent or to the necessity of removing the sludge from the tank before digestion has been completed. In plants where septic tanks are operated in rotation, a given tank being put out of service as a settling tank when the effluent begins to contain solids sufficient to clog filters or nozzles, the detention periods are commonly compared to those employed in plain-sedimentation tanks.

The number and size of septic tanks in several plants are given in Table 68.

Roofs.—Generally septic tanks are provided with roofs, to curtail the drying out of scum and the escape of odors. 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 tank. Roofs are also utilized to keep the sewage warm and out of sight of the public and, in small plants without attendants constantly on duty, to keep off children and irresponsible persons.

Occasionally the roofs are constructed of wood, but more often reinforced concrete slabs carried by reinforced concrete beams are employed. The roofs of the septic tanks at Saratoga Springs are groined arches. The sewage is somewhat warmer than the outside air during a part of the year and the inner surfaces of the walls and roof are likely to become covered with moisture. This water may absorb hydrogen sulfide given off by the decomposing sludge, the result being deterioration of the concrete.[1] Surfaces which may be exposed to such action are commonly made as dense and smooth as practicable, to prevent not only external deterioration but also the penetration of acid moisture to the steel reinforcement.

Where a roof is employed, vent pipes are provided in order that the rapid entrance or escape of a large quantity of sewage may not cause damage on account of the difference in air pressure above and below the roof. As the gases which escape from a septic tank are sometimes explosive, it may be desirable to post a notice to this effect near the entrance to a roofed tank.

Inlets and Outlets.—Scum on the surface of the sewage in septic tanks makes it desirable to provide submerged inlets and outlets, or their equivalents. Another reason for submerged openings is that the action of the sewage tank, in liquefying organic matter, is apparently weaker at mid-depth than at the level of the sludge or just below the scum and it is therefore desirable to admit and draw off the sewage near mid-depth. Among the experiments leading to this conclusion are some made at Plainfield, N. J., by Lanphear (4).

Although submerged openings are preferred by some engineers for the reasons stated, weirs are also employed in many septic tanks, but they are usually guarded by scum boards ½ to 2 ft. in front of them, extending to a depth of at least 2 ft. The narrow openings between the boards and the weirs are easily kept free from scum and the sewage is compelled by the depths of the scum boards to take about the same course it would follow with submerged openings.

Sludge Storage and Volume of Sludge Produced.—In computing the operating capacity of a septic tank, the space occupied by the sludge is usually estimated by the methods followed in the case of a plain-sedimentation tank, but the result is modified by the reduction in volume of the sludge stored, brought about by anaerobic decomposition.

Provision is commonly made for storing 60 to 70 per cent of the suspended matter not removed by racks and grit chambers, plus its accompanying water. The total storage space to be provided is fixed by the assumed lapse of time between tank cleanings, which depends in turn upon the condition of the sewage reaching the works, the temperature and the degree of digestion required.

[1] Some American experience of this nature is reported in *Eng. Rec.*, May 16, 1908, and *Eng. News*, Dec. 12, 1912.

There is always a probability that septic tanks built for small American towns will not receive proper attention and usually such tanks are given large sludge-storage capacity in consequence. With the fresh, weak sewage likely to be received at such plants and with small tanks, there is more danger of large quantities of suspended matter in the effluent, due to high velocities of flow, than of producing a septic effluent. Consequently it is well to provide storage capacity 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 weight, assuming that 30 per cent by weight of the volatile solids are digested. In plants for larger cities, where fairly competent supervision is likely to be given, the space provided for sludge is sometimes reduced by designing the tanks so that a small quantity 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 may be higher than when a long period of digestion is permitted.

If suitable allowances are made for differences in conditions, computations following those outlined for Imhoff tanks in Chap. XVI may be applied to single-story septic tanks.

Removal of Sludge from Tanks.—As in the case of plain-sedimentation tanks, sludge is removed from septic tanks either by gravity or by pumping. Sludge is commonly withdrawn from hopper-bottom tanks during operation. However, tanks operated on the principle of rotation are usually emptied for cleaning.

In drawing sludge from hopper-bottom septic tanks it is essential that the rate of withdrawal be slow enough to allow the whole mass of sludge to settle and to prevent a conical depression from forming in the center. If such a depression forms, a considerable quantity of relatively fresh sludge may be withdrawn, leaving some of the older, well-rotted material, whereas it is desirable that all the sludge which still is undergoing active decomposition remain in the tank, in order to maintain without interruption the changes taking place there.

Ordinarily the slopes of the bottoms of septic tanks do not differ from those employed in plain-sedimentation tanks. 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 the tank is drained, must be broken up, either by jets of water or by shovels or other implements, before it can be easily handled on such slopes. In Great Britain a slope of about 6 per cent is considered desirable for the discharge of septic sludge by gravity. 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

TABLE 68.—COMPARISON OF SEPTIC TANKS

	Fort Meyer, Va.	Greenwich, Conn., Cos Cob plant	Ithaca, N. Y.	Mansfield, Ohio	Mt. Kisco, N. Y.	Reading, Pa.	Saratoga Springs, N. Y.	Washington, Pa.
Volume of sewage treated, m.g.d.	0.1	0.6	9[1]	1.0	0.33	3.5 to 5.2	1.25–2.50	2
Number of tanks	1	5	1	4 covered	1	1	4 covered	4
Tank dimensions, ft.:								
Length	36	90	117	100	53	250	91	100
Width	12	8.8	88	50	54	50	52	25
Depth, max.	10.3	8.5	14	8	10.2	16	8.2	12
Total tank capacity, gal.	26,000	201,000	1,100,000[1]	1,000,000	......	......	250,000	200,000
No. of hoppers per tank	3	4	......	......	2	5	None	8
Depth of hoppers, ft.	3.3	1.5	......	......	0.5	4	......	0.5
Slope of floor	0.055	0.15	0.01–0.02	......	......	......	......	0.02
Detention period, hr.	7	8	8	24	......	5–10	10–15	10
Frequency of sludge removal	......	......	......	4 yr.[2]	Annually	6–24 wk.	18 mo.[3]	
Volume of sludge removed, cu. ft. per mil. gal.	......	......	......	22.2[2]	......	3.5		
Moisture of sludge, per cent	......	......	......	81	......	91	94[3]	

[1] Computed from tank dimensions and detention period.
[2] 1902–1906.
[3] 1903–1904.

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 to rely upon hand cleaning for the complete removal of sludge.

Examples of Septic Tanks.—A number of single-story, hopper-bottom septic tanks were built at army cantonments during the World War after the designs of Doten (5). The bottom of each tank had several hoppers and bottom baffle walls extended between hoppers nearly to the surface. Top baffles were provided at inlet and outlet and upstream of the bottom baffle walls between hoppers. A pipe in each hopper served for removing the sludge either by hydrostatic pressure or by pumping. Few of these tanks have been built since the war.

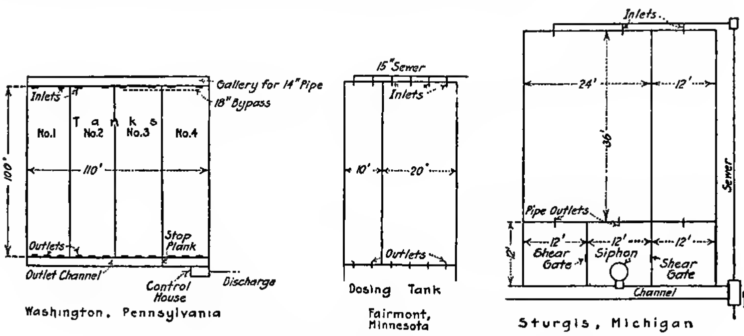


Fig. 95.—Different arrangements of septic tanks.

Table 68 gives the characteristics of several American septic-tank installations.

The treatment plant at Mount Kisco, N. Y., consisting of a septic tank followed by contact beds and intermittent sand filters, was built to prevent pollution of the New York water supply.

The Cos Cob treatment plant at Greenwich, Conn., designed by Potts, was put into operation in 1931. The septic-tank effluent is chlorinated before being discharged into Greenwich Harbor. A unique feature of this plant is the location of the glass-covered sludge-drying beds directly over the septic tanks. The sludge-bed underdrains discharge into the septic tanks and provision is made for returning ripe sludge to the tanks, for the purpose of seeding the freshly deposited solids.

The septic tanks built at a number of plants are shown in Fig. 95.

Efficiency of Septic Tanks.—Apparently in many cases there is little difference between sedimentation and septic tanks in the removal of suspended solids. Johnson has reported that parallel experiments with the two kinds of tanks resulted in an average reduction in suspended matter of about 63 per cent in sedimentation tanks with 6-hr. detention and 66 per cent with 8-hr. detention, while in septic tanks with detention periods of 8, 16 and 24 hr. the removal of suspended matter averaged 61, 66 and 67 per cent, respectively (6). These experimental figures agree well with the results of the operation of septic tanks subsequently built at Columbus. In the annual report on these operations for 1913, the removal of suspended matter during an average detention period of 4 hr. is given as 60 per cent.

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 is 10 to 40 per cent, the average being about 30 per cent. A considerable part of the reduction in the volume of sludge is due to an increase in its density, following its disintegration and compacting, resulting from prolonged stay in the tank. The sludge removed from a septic tank may be much less than 50 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, owing 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 little odor, but often it is decidedly offensive.

The reduction of solids by digestion in a closed septic tank at Worcester, Mass., was estimated in the following manner by Kinnicutt and Eddy (7). During a period of 2 years 3 months, the difference between the results of analyses of the influent and effluent showed that 1194 lb. of suspended matter had been held back in the tank. The sludge at the end of this period contained 729 lb. of dry solids, indicating that 465 lb. had been liquefied. This was a reduction of nearly 39 per cent. The liquefaction during the first 15 months was 28 per cent and during the last year nearly 48 per cent.

The efficiency of sedimentation in the septic tanks at Plainfield, N.J., during 1909, 1910 and 1911 is shown in Table 69. The average detention period was 6 to 9 hr.

Table 69.—Efficiency of Septic Tanks at Plainfield, N. J.
Percentage Reduction

	1909	1910	1911
Suspended solids	59.5	64.0	64.3
Oxygen consumed	31.7	29.1	30.8
Bacteria	54	39	31

Cost of Construction, Operation and Maintenance of Septic Tanks.—The cost of septic tanks per unit of capacity is comparable with that of plain-sedimentation tanks without mechanical sludge-removal equipment. There is no information readily available on the cost of operation and maintenance of septic tanks during recent years. Elements entering into such costs include regulation of flow, breaking up and removal of scum and removal of sludge.

Bibliography

1. Manuals of Engineering Practice, A. S. C. E., No. 2, 1928.
2. Ann. Rept., Mass. State Board of Health, 1899.
3. Ann. Rept., Mass. State Board of Health, 1900.
4. Lanphear, R. S.: *Eng. Rec.*, 1912; **65**, 47.
5. Doten, L. S.: *Trans.*, A. S. C. E., 1919–20; **83**, 337.
6. Johnson, G. A.: Rept. on Sewage Purification at Columbus, O., 1905.
7. Ann. Rept., Conn. Sewerage Com., 1902.

CHAPTER XVI

IMHOFF TANKS

The *Imhoff tank* is a deep two-story tank, consisting of an upper or continuous-flow sedimentation chamber and a lower or sludge-digestion chamber. The floor of the upper chamber slopes steeply to a trapped slot, through which solids may settle into the lower chamber. The latter receives no fresh sewage directly but is provided with gas vents and with means for drawing digested sludge from near the bottom.

Characteristics of Imhoff Tanks.—Theoretically, the sewage flows through the sedimentation chamber only and the sole function of this chamber is the removal of the settling solids. These solids drop to the inclined surfaces of the floor of the sedimentation chamber and slide through a slot into the 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 during decomposition rise through the portions of the sludge chamber which are extended above the top of the tank, called gas vents or scum spaces, and scum collects on the surface of the sewage in these vents, just as it does on the surface of a septic tank, and often to a greater depth. On account of the depth of the sludge chamber and the length of time during which the sludge is allowed to digest, sludge from tanks of this type which are operating normally is denser 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 sludge-digestion tanks.

To secure uniform distribution of settled solids throughout the length of the digestion compartment and thus to utilize the storage capacity in greatest measure, provision for reversing the direction of flow through the tanks is commonly made and transverse obstructions are avoided, as far as structurally practicable.

One of the most important advantages of this form of tank is its production, when operating efficiently, of sludge which is easily disposed of. Sludge is removed in much the same way as from other hopper-bottom tanks.

In order to prevent particles of sludge or scum from penetrating into the sedimentation or flowing-through compartment, the sludge and scum must be kept some distance below and above the slots, respectively. The clear zone is called a "neutral zone."

A peculiar feature of operation at some plants from time to time has been the production of a light voluminous foam, due to a tenacious film enclosing the gas bubbles rising in the vents at times when large quantities of gas are being produced. This is called "foaming." At times this foam has accumulated in considerable masses, filling the gas vents, overflowing their walls, spreading out and covering walks and the sewage in the sedimentation compartment.

Under normal operating conditions, the gases of decomposition which pass into the gas vents either are permitted to escape to the atmosphere or are captured by covering the vents and providing them with gas

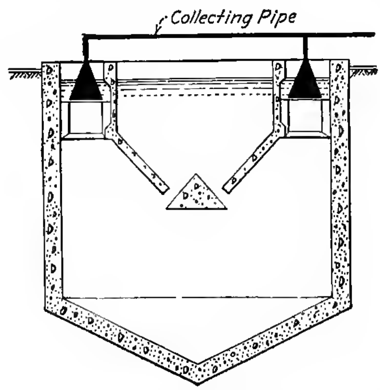


FIG. 96.—Gas-collecting equipment for Imhoff tanks.

hoods, which commonly collect the gas under water, as shown in Fig. 96. The gases, which consist chiefly of methane and carbon dioxide, may be utilized for heating or power purposes. Only the combustible gases, of course, are of value in this connection.

The digestion of the organic matter in the deposited solids results in the production of a practically inoffensive residue, or digested sludge, which is generally black, somewhat flocculent, and well filled with gas, as drawn from the tank under ordinary conditions, and which drains and dries readily when spread on porous beds of sand or similar material. To secure digested sludge of this character, provision for storage of solids deposited during the late fall, winter and early spring months is required in northern climates.

Most of the Imhoff tanks in this country are of the horizontal-flow type and rectangular in plan. To facilitate sludge withdrawal, the bottom of the tank is constructed in the form of hoppers or troughs and the sludge is withdrawn through pipes, which extend within a short distance of the hopper or trough bottoms.

Elements of Tank Design.—The design of Imhoff tanks involves the design of the sedimentation compartment and of the sludge-digestion compartment, including provision for scum space and gas vents.

The elements of design are governed by the theory of sedimentation as described in Chap. XII and the theory of sludge digestion as described in Chap. XIV, subject to the modifications imposed by the type of construction and method of operation peculiar to the Imhoff design.

Inlets and Outlets.—The design of the inlet and outlet channels is more important in the case of Imhoff tanks than in other types of sedimentation units. An economical design provides for full utilization of the space in the sludge chamber of these tanks. This is accomplished in horizontal-flow tanks with more than one well only by using the inlet and outlet chambers interchangeably and thus reversing the direction of flow through the tanks. When this is done at least once a month, accumulation of the heavier parts of the suspended matter is enabled to proceed equally in both of the end wells. Imhoff usually employs only two wells in series, in order to avoid the unequal distribution of sludge which attends the use of more than this number. The importance of the inlet and outlet channels also is increased in the case of Imhoff tanks by the desirability of keeping the surface of the sewage in the sedimentation chamber at the same elevation under all conditions, in order to prevent septic sewage from rising through the slot, which may occur with fluctuations of surface level.

Where reversal of flow is provided for, either the channels serving the tanks are so constructed as to act alternately as influent and effluent channels or both influent and effluent channels are provided at each end of the tanks. In the former case, as at the Akron and Chicago West Side plants, inlets and outlets are designed alike. At these plants weirs, which act alternately as inflow distributors and effluent weirs, are provided at the ends of the tanks. Where influent and effluent conduits are provided at both ends of the tanks, as at Allentown, Dayton and Fitchburg, the channels are commonly superimposed, the upper one acting as an effluent channel and the lower as an influent channel. Sewage is usually passed from the influent channel to the sedimentation compartment through submerged, gated ports and from the sedimentation compartment to the effluent channel over weirs.

The crests of the weirs are set above the maximum flow line in the effluent channel, in order to prevent variations in water level in the tank and consequent exchange of contents between settling and digestion chambers, which may be sufficient to produce disturbances in the digestion of the sludge. The depth and width of the channel usually are designed so as to provide a velocity of 2 ft. per second in the influent channel during normal flows. Inlets and outlets are generally designed to give the appropriate proportion of flow to each sedimentation chamber. Where operating conditions are such as to require careful attention from the superintendent, the sedimentation chambers are often connected at each end of the tank, so as to maintain the same surface elevation in all of them.

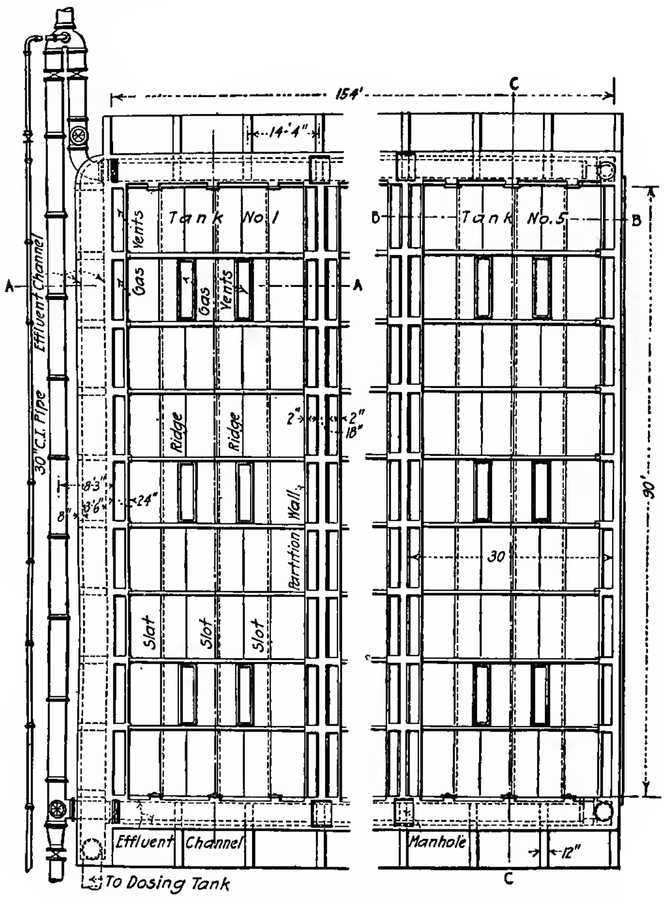


FIG. 97.—Partial plan of Imhoff tanks, Fitchburg, Mass.

The inlet and outlet arrangements of the Imhoff tanks at Fitchburg are shown in Figs. 97 and 98.

Sedimentation Compartment.—There are three types of settling compartments for Imhoff tanks, longitudinal horizontal flow, vertical flow, and radial flow, the first-named being by far the most common. There are few, if any, vertical-flow sedimentation compartments in Imhoff tanks in this country.

Concerning the *radial-flow chamber*, Frank and Fries (1) have made the following statement:

> A possible difficulty is that the radial direction of flow and consequent limitations of length of flow line may result in this flow line being too short to give the upper, rapidly moving currents sufficient opportunity to unload their sediment into the "slow-motion zone." It is obvious that the near surface velocities are not decreased in direct ratio to the decrease in length of tank, and hence these upper rapidly moving currents might have a detention period much too short for efficient sedimentation. However, the

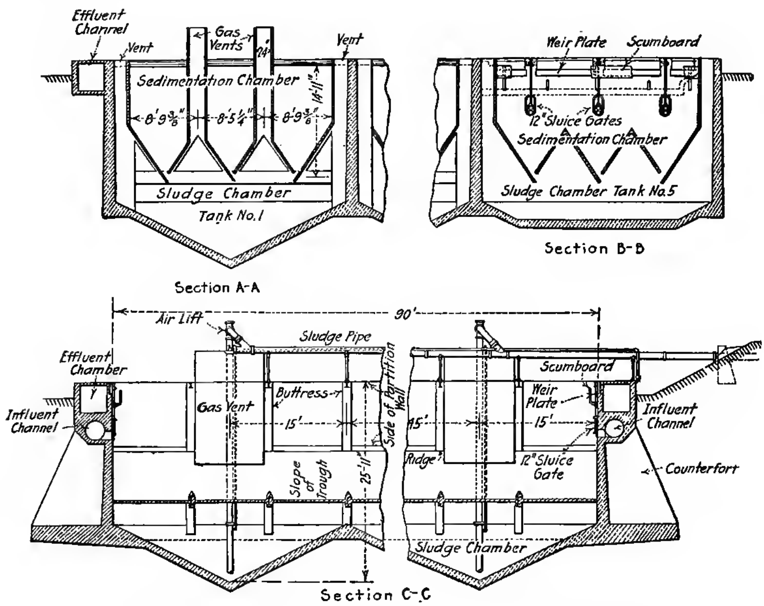


FIG. 98.—Sections of Imhoff tanks, Fitchburg, Mass.

> writers believe that if this radial-flow line is at least 14 or 15 ft. there will be no danger from this source.
>
> A real advantage in the radial-horizontal-flow type is that here the velocity of flow, in order to provide the proper detention period, must necessarily be very low, and this fact will probably permit a greater amount of overloading than in the case of long-flow-line tanks, before critical velocities are reached.

Radial-flow tanks have been constructed at several plants. Expense and construction difficulties have been largely responsible for their decreased use.

Four radial-flow Imhoff tanks, having a combined capacity of 16 m.g.d. to serve a population of 160,000 persons, were built in Flint, Mich., and placed in operation in 1927 (2). Each tank consists of 18 hexagonal cells with a sedimentation chamber, sludge pocket and gas vent in each cell. In operation, sewage enters the tank at the center, divides and flows radially in every direction, with a decreased velocity toward the outside wall, and is drawn off through 12 orifices placed around the outside wall. Each orifice consists of a baffled 3- by 8-in. opening in a steel plate, placed over the outlet of a pipe tee in a vertical riser pipe. These are adjusted so that each orifice will draw the same quantity of settled sewage and a uniform flow from the center

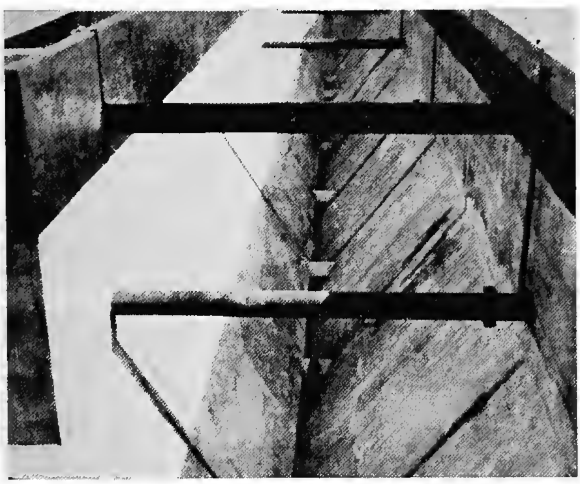


Fig. 99.—Sedimentation chamber in Imhoff tank at Trenton, N. J.

of the tank will be maintained in all directions. The designed detention period is 4 hr. at the average rate of flow and the sludge capacity is based on a unit volume of 2.75 cu. ft. per capita.

Horizontal-flow sedimentation chambers are commonly rectangular in plan. The cross section is usually rectangular at the top with a V-shaped trough bottom, as shown in Fig. 99. The design of the sedimentation chamber is dependent upon the various factors governing sedimentation in plain settling tanks, such as character of sewage, detention period and velocity. The effect of the character of sewage has been discussed in Chap. XII.

Experience has shown that a sedimentation period of 1½ to 2 hr. usually secures the deposition of such a large percentage of settleable solids that a longer period is seldom employed. There may be cases in which it is desirable to provide somewhat longer detention periods, so as to relieve succeeding oxidation processes of a greater portion of their loads, and many of the earlier installations provide longer periods. The detention periods for several American plants are given in Table 72. Some engineers do not include the V-shaped section of the sedimentation compartment in their allowances for detention period. Folwell (3) states that "a channel is not effective for sedimentation to a depth greater than 6 to 8 ft., and only this depth should be used in calculating its capacity."

Since the flowing sewage is not in contact with the settled solids, the limiting velocities are not so low as would otherwise be required. Imhoff places the limiting velocity at 2 in. a second for Imhoff tanks and velocities of ¼ to ½ in. a second are common in American practice.

If the velocity is kept within these limits, the length of tank and area of cross section are governed by the time required for sedimentation and by considerations of adequate distribution of solids in the sludge compartment and of flow in the channel. Tank lengths commonly vary from 25 to 100 ft. If the cross section is so shallow that the flow encroaches upon the desired slow-motion zone above the slot, the maximum possible percentage removal of settleable solids will not be secured. On the other hand, if the chamber is so deep as to inhibit thorough vertical distribution of flow, part of the volume of the tank will be ineffective.

As it is essential for the settling solids to move steadily to and through the slots of an Imhoff tank, the inclined bottom slabs of the sedimentation chamber generally have steep slopes and hard, smooth surfaces. Slopes of 1.5 vertical on 1 horizontal commonly are used. Even with care in construction, it may be necessary to clean the slopes from time to time with rubber squeegees, for soft, sticky material accumulates on them, which may 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 arranged so that this cleaning can be done readily and thoroughly.

The slot at the bottom of the sedimentation chamber is often 6 to 8 in. wide, depending upon the size of the chamber and the character of the sewage. The horizontal overlap to prevent gases from passing through the slot is commonly 8 in.

The sludge chamber of an Imhoff tank 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 tank so that no material fluctuations in sewage level will occur, 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 slots exist to such an extent that considerable fresh sewage enters the sludge chamber, there is a possibility of increased evolution of hydrogen sulfide, as sulfates in the fresh sewage are reduced to this gas.

With two or more sedimentation chambers over a single sludge chamber, as in Fig. 100, inlets and outlets are generally designed to give the appropriate proportion of flow to each sedimentation chamber, thus reducing the danger of diffusion between sludge chamber and sedimenta-

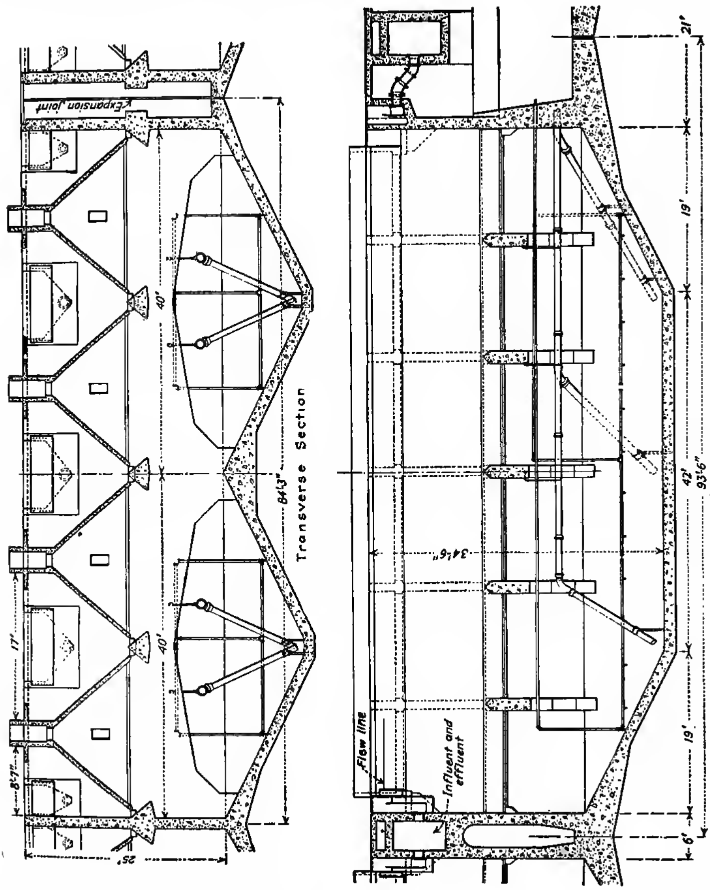


FIG. 100.—Sections of Imhoff-tank structure at West Side treatment plant, Chicago.

tion chambers. Notwithstanding all precautions, however, there is an inevitable interchange of liquids between the digestion and sedimentation chambers, frequently due in large measure to differences in temperature.

The concrete slabs used for the sides and bottoms of sedimentation chambers are built either on forms, like most slabs reinforced with bars, or on metal lath by plastering it with cement mortar. The baffle walls at the West Side plant in Chicago, illustrated in Fig. 101, were built of precast 12½- by 13-ft. slabs, 5 in. thick, weighing 5 tons (4).

The slabs in the Imhoff tanks at Akron were gunited and those at Allentown and Dayton were cast in place. At Fitchburg certain 3-in. slabs, which were plastered, have not proven satisfactory.

Inasmuch as the inclined slabs may collect solids which must be pushed down them, they require greater strength than is needed simply to support their own weight and that of a small quantity of submerged solids, as well as sufficient stiffness to avoid cracking.

The baffle walls at the Decatur, Ill., plant were made 2½ in. thick, reinforced with 3- by 3-in. mesh, No. 4 gage galvanized wire, with small I-beams imbedded at the supporting shoulders where there was greater

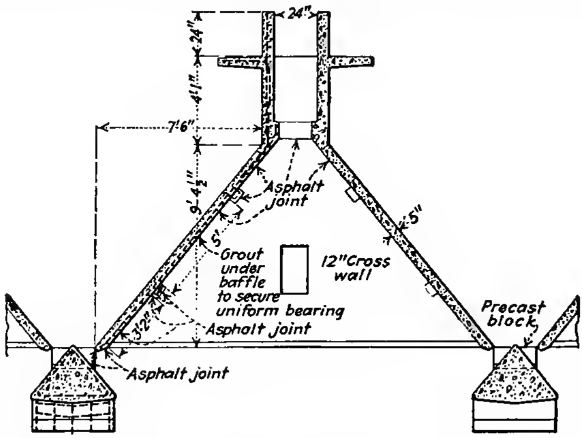


FIG. 101.—Precast-slab baffle walls in Imhoff tanks at West Side treatment plant, Chicago.

thickness. Forms were placed 1¼ in. back of the mesh and gunite was shot through the mesh against the forms, building up the thickness to 2½ in. with the wire in the center. The surface was scraped flat, flushed and smoothed, so that the maximum density, combined with smoothness, was obtained on the inner, or hopper, side. Gunite baffle walls have also been built at Elgin and Bloomington, Ill.

Scum Boards and Baffles.—Scum boards are commonly provided at both ends of the tank, the one at the inlet end serving to collect the larger floating solids and to distribute the flow and the one at the outlet end serving to hold back the smaller floating particles. The scum boards are commonly made to penetrate 12 to 16 in. below the flow line and they usually project about 12 in. above it. If they extend to too great a depth, they may render ineffective part of the upper volume of the chamber or immoderately increase the velocity in the lower slow-motion zone.

Frank and Fries (1) have stated:

> Baffles are not considered advisable either in the upper or lower part of the sedimentation chamber. They have been tried, both from the standpoint of better flow distribution and in order to promote the production of the slow-motion zone. The conclusion has been that instead of this they decrease the cross section of flow and tend to produce subcurrents, thereby hindering sedimentation rather than promoting it.

It is desirable to have the horizontal velocity gradually decrease toward the lower depths of the sedimentation chamber, in order that there may be no currents to interfere with the settling solids passing through the slots into the sludge chamber.

Removal of Skimmings.—Appreciable quantities of floating matter, such as oil, grease and other scum-forming material, may collect on the surface of sedimentation chambers in Imhoff tanks. The removal of this material may be materially assisted by providing for skimming the tanks with water jets, as has been done by Allen at Fitchburg (5). The water jets are placed at strategic points and directed so as to drive the scum to one corner of the tank, from which it may be removed and disposed of as described in Chap. XI.

Sludge-digestion Compartment.—Regarding the physical conditions in the sludge chamber, Frank and Fries (6) state:

> A universal conception of the physical part of the process is that there is normally a quantity of sludge lying at the bottom generating gas within its "live" particles, and that continually emerging from this lower sludge body and rising and falling in the liquid above it are other particles of sludge which have generated sufficient gas to give them the required buoyancy, and which are either in the act of rising to give up this gas or have given it up and are again settling. At irregular intervals part of the mass of the sludge may rise to the top in a body and remain there till it has given up sufficient of its gas to settle again. Hence, whether to accommodate the rising and falling particles or to provide room for the temporary floating position of a mass of sludge, it is necessary to have above the plane of the slot in the sludge chamber a certain amount of "transition" space.

The sludge chamber may be regarded as composed of three parts, the sludge-storage space, the space above the slot to provide for storage of gas-lifted or other solids which collect above the plane of the slot, and a neutral zone above and below the plane of the slot, between the sludge- and scum-storage spaces.

The variation in the conditions governing the required capacity of the digestion compartment is so great that the design should be based upon an intelligent analysis of the conditions of the particular problem rather than on a general assumption of a definite number of cubic feet per capita adopted at some other place or places. Among the items which need to be considered are the following:

1. The prevailing temperature of the sewage and duration of the period of low temperatures. In Imhoff tanks the temperature of the sludge lags slightly behind that of the sewage but follows it fairly closely.

2. The character and quantity of sewage solids deposited and stored in the tank, including the effect of industrial wastes. Provision may be required in the sludge chamber not only for the fresh solids settling in the sedimentation chamber but also for sludge from final-sedimentation tanks employed with oxidation processes.

3. The changes taking place in the volume and density of the sludge during digestion.

4. The quantity of seeding material to be left in the tank at the beginning of the period of low temperatures, during which no sludge is drawn.

5. The space required for the storage of scum.

6. The space required for the "neutral zone."

Other things to be considered in the design of the sludge chamber are the following:

1. The uniform distribution of the deposited solids throughout the digestion compartment. For this purpose a relatively short tank, with the sludge chamber subdivided as little as practicable, may be advantageous.

2. The depth of tank and its effect on sludge digestion and scum formation.

3. Provision for breaking up gas-lifted solids, as by impact on the partition between sedimentation and digestion compartments.

4. Provision for the control and disposal of scum. For this purpose it is desirable for the sludge compartment to have relatively small exposed surface areas, so that the sludge may be kept submerged as much as practicable. It may be advisable to provide means for discharging liquor from the neutral zone over the surface of the scum and for drawing off scum to the sludge-disposal area.

5. Provision for the liberation or collection of gases resulting from the process of digestion.

6. Provision for the withdrawal of the digested sludge and possibly for the distribution and circulation of sludge in the chamber.

Depth of Tank.—Deep tanks have apparently many advantages over shallow tanks. In a discussion of reasons for differences in behavior of Imhoff tanks, one of the authors (7) made the following statements:

The influence of depth on tank action is not however entirely clear. One theory is that in the deeper tanks there is a longer path through which the gas-lifted solids must ascend in order to form an accumulation of scum at the top and that this prolonged travel affords opportunity for the escape of the gases after which the solids may return to the bottom, thus avoiding their accumulation as scum. . . . In deeper tanks, the gas is generated under greater pressure and the bubbles entrained in the sludge are subject to considerable expansion on rising, thus possibly aiding in their escape from the solids; likewise there is a deeper accumulation of sludge, which tends toward greater density.

Imhoff (7) states, "There is no doubt whatever that digestion chambers of great depth do more work than shallow ones of the same size."

Frank and Fries (6) report as follows:

> The depth of the sludge chamber seems to have an influence upon the sludge digestion process. This influence has been but little understood, but it has been continually maintained that shallow tanks yield a poorer quality of sludge than deep tanks. A possible explanation of this may lie in the theory that the more quickly the entering fresh sludge particles come into contact with decomposed sludge particles the more quickly will they themselves decompose, and it is obvious that the deeper the tank the more intense the gas development, and hence the more intense the stirring action. This should be conceived on the principle of "inoculation," which assumes that an organic particle, to be decomposed by the quickest possible odorless route, must be inoculated as quickly as possible with the proper bacteria; these bacteria it can best obtain from the other sludge particles already properly decomposing. However, to bring about this condition, intimate contact, particle for particle, is necessary, and this does not occur easily when fresh sludge is allowed to rest "dead" in large masses. This may be the reason why the violent stirring action of deep Imhoff tanks results in good sludge and odorless decomposition. A depth of tank which has most been used in the Emscher District is about 30 ft. from water surface to bottom of tank. A few have been made 24 and 27 ft. deep. In some cases, however, it may be advisable, for economic reasons, to adopt tanks as shallow as 20 ft., deliberately preparing to accept a somewhat poorer grade of sludge. It has not been proved that a tank of 20 ft. or even less in depth will not in many cases yield a sufficiently good sludge for all practical purposes, but it is now reasonably clear that a 30-ft. depth is sufficient.

Scum Space and Gas Vents.—It is customary to make the scum-storage space about one half as large as the sludge-storage space.

Imhoff (7) reports: "Digestion chambers having small gas vents show the greatest security against foaming and floating sludge." As important aids to proper functioning, Skinner (7) suggests that gas vents should be wide, but not necessarily of large area, and that scum chambers should be ample in width and connect freely with the gas vents.

Gas vents or collecting chimneys form integral parts of the sludge-digestion compartments of two-story tanks. The design of such vents or chimneys depends upon whether or not the gases are to be collected and utilized. Vents are commonly so designed that workmen can enter the sludge chamber through them, when the tank has been emptied. The volume of the digestion compartment above the slots commonly becomes equivalent to ½ to 1½ cu. ft. per capita. When the digestion compartments are vented to the air, the vents or chimneys are usually given an area ranging from 12 to 25 per cent of the tank area. Where the vents are small, the scum rapidly becomes thick and, if it is not broken up, it may rise over the edge of the vents. Imhoff has recorded an instance of scum rising 6 ft. above the level of the sewage, because the gases in it could not escape. Where the gas is collected, rather small chimneys leading to the gas bells have been built.

In the case of certain industrial wastes, such as those from packing houses, the total quantity 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 and to necessitate radical changes in their design as well as departure from the usual proportions of Imhoff tanks.

Volume of Sludge.—As stated in connection with the design of the sludge-digestion compartment, the volume of sludge for which provision is made depends, among other things, upon the temperature which prevails during digestion. In northern climates there is great variation from season to season in the temperature of air, sewage and, most important of all, sludge in the digestion compartment. In summer, when the temperature in the sludge compartment may be 65 to 70°F., digestion is rapid, as indicated by vigorous generation of gas, but in winter it is retarded greatly.

In northern climates digestion compartments are designed of sufficient size to provide storage for the solids deposited during the late fall, winter, and early spring, for during this period sludge is not usually digested thoroughly enough to be free from objectionable odor and to drain and dry readily.

The solids as first deposited in the sludge chamber form a thin, voluminous sludge. As decomposition progresses, the percentage of mineral matter in the sludge increases and the relatively inert digested material gravitates to the bottom. As the depth of sludge increases, the solids in the bottom tend to become compacted. The upper layers of sludge, where active decomposition is going on, are continually disturbed by the increments of fresh sludge settling in the sludge compartment and by the rising gas and gas-lifted solids. As a result of these conditions, the mass varies in consistency from a thin, watery material at the top to a comparatively thick sludge at the bottom.

The organic matter in the sludge of Imhoff tanks undergoes changes similar to those which occur in single-story septic tanks. A theoretical method of arriving at the quantity of sludge accumulating in the sludge-digestion chamber is shown in Table 70. Here it is assumed, for purposes of illustration, that a hypothetical Imhoff tank situated in the northerly United States receives from combined sewage 100 lb. of deposited solids a month, of which 60 lb. are assumed to be organic and 40 lb. mineral matter. It is assumed also that 30 lb., or 50 per cent of the organic matter, must be digested in order to produce an inoffensive sludge. This is equivalent to 30 per cent of the total solids. For convenience this portion of the organic matter is designated *digestible solids*, although digestion is not confined to it. With separate sewage,

TABLE 70.—THEORETICAL COMPUTATION OF SLUDGE ACCUMULATION IN IMHOFF TANK AT TEMPERATURES RANGING FROM 45 TO 64°F.

	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.
Temperature, °F	60	54	52	49	48	45	51	54	59	64	64	63
Digestion, per cent, from Fig. 92	50	36	32	27	25	17	30	36	50	63	63	59
Digestible Solids												
Solids deposited during month, lb	30	30	30	30	30	30	30	30	30	30	30	30
Rate of digestion, lb. a month												
Solids of stated month	7.5	5.4	4.8	4.0	3.7	2.5	4.5	5.4	7.5	9.4	9.4	8.8
Solids from previous months	15	10.8	9.6	8.1	7.5	5.1	9.0	10.8	15.0	18.9	18.9	17.7
Solids remaining at end of month, lb.:												
From stated month	22.5	24.6	25.2	26.0	26.3	27.5	25.5	24.6	22.5	20.6	20.6	21.2
From 1 month before	6.2	11.7	15.0	17.1	18.5	21.2	18.5	14.7	9.6	3.6	1.7	2.9
From 2 months before	0	0	2.1	6.9	9.6	13.4	12.2	7.7	0	0	0	0
From 3 months before	0	0	0	0	0	4.5	4.4	1.4	0	0	0	0
From 4 months before	0	0	0	0	0	0	0	0	0	0	0	0
Total	28.7	36.3	42.3	50.0	54.4	66.6	60.6	48.4	32.1	24.2	22.3	24.1
Nondigestible Solids												
Solids deposited during month, lb	70	70	70	70	70	70	70	70	70	70	70	70
Solids drawn during month, lb	0	0	0	0	0	0	140	140	140	140	140	140
Solids remaining at end of month, lb	70	140	210	280	350	420	350	280	210	140	70	0
Total Solids												
Solids remaining at end of month, lb	99	176	252	330	404	487	411	328	242	164	92	24
Solids remaining at end of month, cu. ft.[1]	13	23	32	42	52	63	53	42	31	21	12	3
Solids remaining at end of month, cu. ft. per capita	0.39	0.69	0.96	1.26	1.56	1.89	1.59	1.26	0.93	0.63	0.36	0.09

[1] Average solids, 12.5 per cent

the proportion of digestible solids would approximate 40 per cent of the total solids.

The temperatures of the digestion compartment are assumed to be those observed for the sewage at Schenectady, N. Y., in 1927. The rate of digestion is taken from Fig. 92, with the assumptions that digestion is completed in 2 months at 60°F. and that the rate is constant. The latter is not strictly true, but errs on the safe side, as shown by Fig. 90. Since all the solids freshly deposited in any month are not subjected to digestion for the whole month, one half of the prevailing rate of digestion is assumed for these solids.

The sludge remaining in the tank at any time comprises the undigested solids contributed, plus the residue of digestible solids at the end of the month, deduction being made for sludge drawn during six months, May to October, inclusive, at a uniform rate per month, sufficient to remove all the digestible material at the end of October, except a small quantity remaining undigested and another portion required for seeding. The solids drawn amount to 140 lb. a month, or 840 lb. a year, 360 lb. having disappeared as the result of digestion.

The volume of sludge remaining in the tank has been computed on the assumption that the sludge will contain 15 per cent of solids at the bottom and 10 per cent at the surface and will average 12.5 per cent.

The hypothetical data are reduced to terms in common use by assuming that the solids deposited in the tank correspond to 3 lb.[1] per capita monthly, equivalent to 120 p.p.m. on the basis of 100 gal. per capita daily sewage flow. The population contributing 100 lb. of solids per month is 100/3. This computation indicates a required capacity for sludge storage of $63 \times \frac{3}{100} = 1.89$ cu. ft. per capita, the maximum being stored at the end of April. To this must be added an allowance for the neutral zone, which is not to be filled with sludge. The neutral zone is commonly assumed to occupy 18 in. of tank depth below the slots, say 20 per cent of the digestion-compartment volume below the slots, making a total of 2.36 cu. ft. per capita. The total sludge drawn during the year, 840 lb., on the assumption of 15 per cent solids is equivalent to 2.7 cu. ft. per capita or 550 gal. per million gallons of sewage.

It is evident that an adequate knowledge of temperature conditions is essential to determining the sludge-storage capacity of Imhoff tanks, as well as single-story septic tanks and separate sludge-digestion tanks. For rough computations the following approach is sometimes taken.

From analyses of the sewage, or by a study of the sources of the sewage when analytical records are lacking, the quantity 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

[1] For domestic sewage this would approximate 2.2 lb. per capita monthly, or 88 p.p.m.

systems. This quantity is equivalent to 1460 lb. of suspended matter per 1,000,000 gal. of sewage. It will be assumed that a detention period of 2 hours is selected and that in this time 50 per cent of the suspended matter passes from the sedimentation chamber to the sludge chamber. On this basis 730 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[1] per cent, leaving 548 lb. of solids to go into the sludge. The water content of the sludge will be taken as 87.5 per cent. Sludge of this water content and with a specific gravity of its solids of 1.4, because of the digestion of the lighter organic solids, measures 0.452 cu. yd. per 100 lb. of suspended matter. Hence $5.48 \times 0.452 = 2.48$ 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 527 cu. yd. of space are required for a daily flow of 1,000,000 gal., or 1.42 cu. ft. per capita on a basis of sewage flow of 100 gal. per capita daily. No attempt should be made to compare this value with that of the preceding example, which was found for combined sewage; on the basis of separate sewage the preceding analysis would have yielded a value of 1.40 cu. ft. per capita.

Another method of estimating the volume of sludge space has been developed by Allen, who has proposed the following formulas (8):

Storage, cubic feet, for combined sewage = $10.5\,PD$

Storage, cubic feet, for separate sewage = $5.25\,PD$

where P is the population in thousands and D is the sludge-detention period in days. The formulas are based on fresh sludge with 90 per cent moisture and digested sludge with 80 per cent moisture.

It has been estimated that the digestion compartments of the Imhoff tanks at Fitchburg, Mass., provided 18.6 cu. ft. per pound of suspended solids removed from the sewage daily during 1924. The total capacity of sludge and scum compartments was equivalent to 34.5 cu. ft. per pound of suspended solids treated daily. At Worcester, Mass., during 1928 the digestion capacity was equivalent to 20.1 cu. ft. per pound of suspended solids settling out daily in the Imhoff tanks. At Marion, Ohio, during 1925 and 1926 the digestion capacity was equivalent to 45 cu. ft. per pound of suspended solids settling daily in the Imhoff tanks.

Digestion of Humus Sludge in Imhoff Tanks.—At some Imhoff-tank, trickling-filter plants, the humus sludge from the secondary-sedimentation tanks is digested with sewage solids in the Imhoff tanks.

[1] Forty per cent of the sludge may actually be assumed to be digestible, but allowance must be made for the fact that the sludge deposited in the months preceding sludge-drawing is only partially digested.

At Fitchburg, the humus sludge was pumped at first into the influent pipe of the Imhoff tanks, in the expectation that the sludge solids would settle into the digestion compartments together with the solids from the sewage. According to Hartwell (7),

> This method delivered secondary tank sludge into the sedimentation compartments of all the tanks while they were in normal operation, which procedure was objectionable on account of (1) the light flocculent foam that covered all of the tanks for a number of days, (2) the reduced efficiency of the trickling filter and (3) the unsatisfactory final effluent.

Subsequently, piping was installed to discharge the humus sludge directly into the digestion compartment of one of the five Imhoff tanks at a depth of 18 in. below the slots. The humus sludge so discharged into the Imhoff tank caused no foaming and did not affect the quality of the tank effluent. The combined sludges digested well, although the digested sludge from this tank did not dry quite so rapidly on the sludge beds as that from the other Imhoff tanks. The introduction of all the humus sludge into one Imhoff tank greatly increased the load of sludge in this tank, so that piping was later installed for pumping part of the humus sludge into other Imhoff tanks. This method of disposal of the humus sludge at Fitchburg has been entirely satisfactory.

The solids in the humus sludge to be digested may be estimated from the suspended solids in the sewage and the effluent from the primary-sedimentation tank. It may be assumed that 50 per cent of the suspended solids in the sewage will be removed by the primary-sedimentation tanks. Of the sewage solids passing through the primary-sedimentation tanks to the trickling filters, a portion will be removed by digestion within the filters. From experience, it appears reasonable to assume that digestion will result in 20 per cent less suspended solids in the trickling-filter effluent than in the primary-sedimentation-tank effluent. It may also be assumed that 50 per cent of the suspended solids in the trickling-filter effluent will be removed in the secondary, or humus, tanks. On these assumptions, the humus-sludge solids to be digested will be equivalent to $100 \times 0.8 \times 0.5 = 40$ per cent of the primary sludge solids.

Digestion of Excess Activated Sludge in Imhoff Tanks.—Digestion of excess activated sludge with fresh sewage solids in Imhoff-tank digestion compartments apparently can be accomplished satisfactorily. As removed from final-sedimentation tanks, activated sludge has a high water content, 97 to 99.5 per cent, and is extremely unstable, becoming septic in a short time. Digestion increases the sludge density and anaerobic decomposition produces a sludge which is similar in some respects to digested sludge from Imhoff tanks. When activated sludge is digested together with fresh solids, the ratio of activated to fresh solids enters into the problem. Tests by the Sanitary District of Chicago indicate that a lower total yield of gas results from mixtures containing more than 33 per cent by weight of activated sludge on a basis of total solids. If it is assumed that 50 per cent of the solids in the raw sewage are removed by preliminary sedimentation and that in the final-sedimentation tanks 90 per cent of the solids remaining after preliminary sedimentation are removed, the proportion of activated solids in the resulting mixture of activated and fresh solids will be

$$\frac{0.5 \times 0.9}{0.5 \times (1 + 0.9)} = 47 \text{ per cent.}$$

The capacity requirements of Imhoff-tank digestion compartments for handling a mixture of primary and activated sludges have not been established. They may be estimated using the method outlined for primary sludge alone, with allowances for the increase in weight of sludge solids and for a probable greater volume of sludge, due to higher moisture content. An average moisture content of 95 per cent during digestion may be representative of probable conditions within digestion compartments of Imhoff tanks handling mixed sludge. Under such conditions, the digestion-compartment capacity requirements may be several times that for primary sludge alone. The gas yield will also be increased, but perhaps not in the same proportion as the weight of solids digested. Tests at the Des Plaines plant at Chicago have shown that the digestion period of mixed activated and fresh solids is somewhat increased over that expected for fresh solids alone.

The digestion of primary sludge and excess activated sludge was studied on a large scale at the Calumet plant in tests extending over a period of three years. The results were reported by Goodman and Wheeler (9) as follows:

An average of 1.75 m.g.d. of sewage was settled in an Imhoff tank and the effluent was treated by the activated sludge process. The waste activated sludge was returned to the incoming sewage of the Imhoff tank.

After six months' operation it was apparent that additional sludge-digestion capacity was needed to provide storage for the bulkier sludge from the activated sludge process. An additional Imhoff tank was used to supply the extra capacity required. This was operated as a separate sludge-digestion tank. The partially digested sludge was pumped over into this second Imhoff tank where the digestion was allowed to continue, until the sludge was in a satisfactory condition for drying.

The activated sludge was wasted as slowly and as gradually as possible in order to obtain as nearly as possible continuous mixing of activated with fresh sewage solids in the Imhoff tank. Occasionally the flow of activated sludge had to be shut off so as not to withdraw too much from the activated-sludge process. The combined sludge after drying had all the characteristics of Imhoff sludge produced at the plant except that it averaged about 2 per cent higher in moisture content as drawn, contained somewhat higher total nitrogen, and cracked into much smaller sections when drying.

Average figures for the entire period of experiment gave the following results: 9.4 cu. yd. per mil. gal. of activated sludge at 98.5 per cent moisture mixed with 2.2 cu. yd. per mil. gal. of fresh sewage solids at 90 per cent moisture (est.) gave 2.4 cu. yd. per mil. gal. of combined (digested) sludge at 89.9 per cent moisture. This amounts to an 80 per cent reduction in volume. The digestion of dry solids was based on determinations of suspended solids in the influent and effluent of the Imhoff tank and on the actual quantity of activated sludge wasted. These data over a long period indicated 32.5 per cent digestion of suspended solids.

This experiment at the Calumet Sewage Treatment Works indicated the practicability of satisfactory digestion of Imhoff and activated sludges. This procedure was adopted in the design of the West Side Treatment Works, where the waste activated sludge of the North Side Treatment Works will be mixed with the fresh incoming sewage of the West Side plant and the mixed settled solids digested.

Subsequently, experience with the digestion of waste activated sludge from the North Side plant together with primary sludge in the Imhoff tanks at the West Side plant has shown that a much larger digestion capacity is required than indicated by the tests at the Calumet plant, mainly because of the failure of the combined sludge to reduce in volume by consolidation to the extent obtained during the tests.

If excess activated sludge is to be introduced into the influent to two-story tanks, the bottom slopes of the flowing-through compartments are commonly made steeper and the sludge slots wider than they are where no activated sludge is introduced. If the activated-sludge floc is broken up seriously, as by moderate-speed centrifugal pumps, during the process of discharging it into the influent, most of the floc may not settle out in the Imhoff tanks, thus reducing their efficiency. If the sludge is pumped directly to the digestion compartment, excessive amounts of scum may be formed.

In relating experiences with the digestion of excess activated sludge in an Imhoff tank at Pomona, Cal., Froehde (10) states that pumping activated sludge into the digestion chamber did not work, because a large quantity of sludge came up in the sedimentation compartment and was discharged with the tank effluent into the activated-sludge units. Fairly good results, although not entirely satisfactory, were obtained when the excess activated sludge was discharged into the inflow and settled out with the fresh solids in the sedimentation compartment. Reinke (11) has reported the Imhoff tanks at Pomona as being overloaded.

The digestion of activated sludge within the digestion compartments of Imhoff tanks has seldom been adopted. Where the activated-sludge solids are to be digested, the method generally in vogue is to provide preliminary sedimentation and separate digestion, the excess activated sludge being discharged into the influent of the preliminary-sedimentation tanks for settlement and disposal with the primary sludge.

Sludge Withdrawal.—In most Imhoff tanks gravity is the only force relied upon to make the sludge slip down the slopes to the inlet of the sludge drawoff pipe. Experience has shown that slopes of at least 1.75 vertical on 1 horizontal, or better 2 on 1, are advisable. To avoid excessive depths in large-size hoppers, the slope is commonly reduced to 1 vertical on 1 horizontal, or even to 1 vertical on 2 horizontal. In the latter cases, provision is often made for agitating and flushing the sludge with water. The water is admitted through a 1½ to 2-in. pipe, at or near the top of the slopes, and occasionally through another ring or single jet in the sump at the bottom of the hopper. The pipe perforations are commonly about 5⁄32 in. in diameter and about 20 in. apart. Although air has occasionally been used in place of water to secure agitation, it is not used in tanks where gas is collected, because of the danger of producing an explosive mixture.

Sludge is usually removed from the tank through an 8-in. pipe. In some cases the pipe is extended vertically until the top is above the surface of the sewage, so that, in case it becomes clogged with sludge, either the material can be stirred by a jet of water, introduced through a cock trapped into the cap on the end of the pipe, or the pipe can be cleared by rodding.

When a straight pipe is used, the sludge is not discharged from the end but through a curved branch with a valve, which is commonly located so that the outlet of the branch is 5 to 9 ft. below the level of the sewage. This head of sewage is enough to force the sludge up through the pipe and its branch, after the sludge has been loosened by jets of water from the perforated pipes.

According to Frank and Fries (6), a gravity pipe line for Imhoff sludge should have a slope of at least 12 per cent, in order to avoid danger of stoppage. When the available head between the sewage surface in the tank and the sludge-drying-bed drain is less than necessary to discharge the sludge by gravity, 8.5 ft. commonly being considered the minimum head for this purpose, some method of pumping the sludge is provided.

Frank and Fries (6) comment as follows:

> The important thing in sludge pumping seems to be the retention of the gases of decomposition. As has often been stated in articles upon the drying of decomposed sludge, the reason for the rapidity of the drying is that the expansion of the minute gas bubbles, upon being removed from a pressure of, say, 2 atmospheres at the bottom of the tank to a pressure of 1 atmosphere at the surface causes the sludge to be lighter than water.

When the sludge is placed upon the sludge-drying bed, therefore, it soon floats on its own water content, and this water, being free, quickly passes away through the porous drying-bed material, leaving the sludge spadable in a few days. Therefore, whatever method of sludge lifting is employed it should be such as not to remove any of these gases of decomposition.

Among the types of pumps which have been used are: plunger pumps, centrifugal pumps, air lifts, ejectors and hand membrane pumps.

Plunger Pumps.—Plunger pumps have proved to be well adapted to lifting sludge, where they are placed sufficiently low so that the sludge from the tanks can enter the pumps by gravity and does not have to be sucked in as this suction tends to remove the gases. If the pump is simple in construction, it will insure minimum operation difficulties from the passage of solid particles of reasonable size.

Centrifugal Pumps.—Centrifugal pumps are efficient and easy to operate, but they seem to have the disadvantage of stirring the sludge too violently and tending to disengage prematurely from each other and segregate the gas and sludge particles, thus, as above indicated, retarding drying.

Air Lifts.—Air lifts have been successfully used for pumping Imhoff sludge at Akron and Cleveland, Ohio, Fitchburg, Mass., and Flint, Mich. An advantage claimed for air lifts is the possible mingling of the compressed-air particles with the sludge, increasing the quantity of entrained gas and thus promoting the drying operation.

Ejectors.—Another method of lifting sludge, which is utilized at Allentown, Pa., at the West Side plant in Chicago, Ill., and at Dayton and Marion, Ohio, is by means of compression chambers. When the sludge from one or more sludge chambers is collected by gravity in an airtight drum, the admission of compressed air to the drum is a simple means of forcing the sludge to the drying beds. The great advantage of such a method is the practical elimination of clogging and the non-interference with the gas content of the sludge.

Hand Membrane Pumps.—Hand membrane pumps are sometimes used in small Imhoff tanks. If the pipes used in lifting the sludge are of appropriate diameter, the velocity will be sufficiently great to avoid sedimentation.

Gas Collection.—Two-story tanks lend themselves readily to the incorporation of gas-collecting devices. In modern installations gas-collection is frequently made part of the structural design, for either immediate or anticipated use. Older plants, too, can easily be modified to include this feature. The partitions separating the sedimentation and digestion compartments form part of the gas-collection system and little additional equipment is needed. To prevent the clogging of the gas bells a scum barrier, consisting of tongued and grooved boards set loosely, or of porous plates, is sometimes provided. Foaming, which may render inoperative the gas-collection system, has been effectively controlled by recirculating liquor from beneath the scum over the scum surface for a period of 5 or 10 min. a day.

Hatch (12) states that at the San Bernardino, Cal., Imhoff-tank, trickling-filter plant, the gas-vent areas in the Imhoff tanks are

covered with concrete roofs, 12 in. below the water line, so that the gas is all diverted into 4-ft. gas collectors. The gas collectors have artificial water seals so designed that the gas pressure can be varied from 4 to 10 in. of water. The gas is piped from the collectors to two 750-ft. capacity lift-type gas holders. From the gas holders it is piped back to the control house, where it is used to operate a 40-hp. gas engine direct connected to a 28-kva., 50-cycle, 3-phase generator, which provides all power and lighting for the plant.

Hatfield and Morkert (13) have described the gas-collection system on the Imhoff tanks at Decatur, Ill., as follows:

The gas vents which were originally open to the air were reconstructed in 1926 to collect the digestion gases, by building a submerged arch roof in each vent which terminated into 80-in. tile between each walk way. Over these tile were placed galvanized metal bells for collecting the gases. The collecting bells were originally connected by insulated 1-in. pipes, but these small pipes froze badly in subzero weather and have been replaced by 2-in. galvanized pipes. The 2-in. pipes frost up in subzero weather and insulate themselves with cellular ice crystals, and at 17° below zero have not frozen due to the insulating properties of the frost lining.

Scum formation in the gas collectors has not been a serious item of maintenance. The majority of the gas domes are not taken off and cleaned more than once or twice a year.

Practically all the gas is collected and measured through a Foxboro orifice meter. The volumes are corrected to 60°F. and 29.90 in. of mercury, which is standard industrial gas practice. Before adding the excess plant sludges the gas volume was about 50,000 cu. ft. per day, but during 1930 and 1931, although the added solids increased to 12,000 and 14,000 lb. per day, the daily gas volume dropped 5 to 7 per cent (or to about 47,000 cu. ft. per day).

The volume of gas collected varied from 3.3 cu. ft. per pound of solids added to the digestion compartments in 1931 to 5.7 cu. ft. per pound during 1929.

The gas vents of the Imhoff tanks at Dayton, Ohio, are provided with gas-collecting domes, as shown in Fig. 102. These cast-iron domes are intended to operate under pressures varying from 12 to 16 in. of water. If the pressure is excessive, it may be relieved by the tilting of the dome. The top of the dome is removable for inspection and cleaning. A wooden baffle prevents the entrance of scum into the gas dome.

From the collection domes the gas is piped into a lateral leading through a water seal to the gas main. The water seal is provided as a protection against flashback, or disturbance at one dome spreading to the gas main or domes on other laterals. Provision is made at each dome for gas sampling.

The gas main leads through an orifice meter and condensate tank to a compressor. Connection is made between the meter and the compressor to a waste-gas incinerator, which is provided to consume gas not utilized for other purposes. This is also provided with a water seal for safety purposes.

The gas is compressed to a maximum of 50 lb. per square inch and stored in two pressure tanks, from which it is drawn through pressure

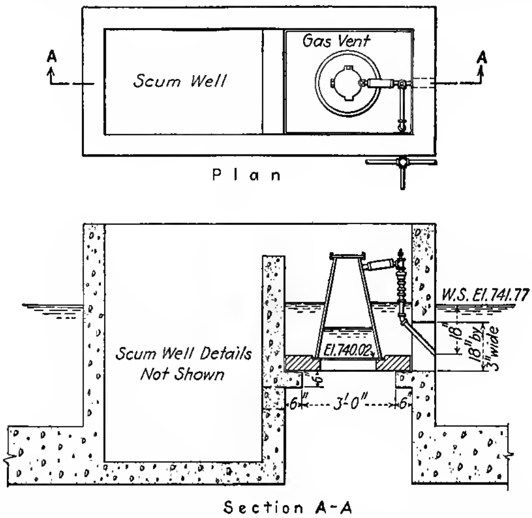


FIG. 102.—Gas-collecting dome in Imhoff tank at Dayton, Ohio.

reducers for use in the screenings incinerator, plant furnaces and laboratory, and for use in the clubhouse on a near-by golf course. Only a portion of the total gas produced has been required for these purposes and the remainder has been disposed of in the waste-gas incinerator.

During the first 24 weeks of 1931 the gas production at Dayton was as shown in Table 71.

TABLE 71.—GAS PRODUCTION IN IMHOFF TANKS AT DAYTON, OHIO, JAN. 4 TO JUNE 20, 1931

Quantity of gas produced:	
Cu. ft. a day	68,200
Cu. ft. per capita daily	0.34
Cu. ft. per mil. gal. of sewage	3,459
Cu. ft. per lb. solids deposited	3.25
Proportion of gas:	
Compressed for use, per cent	61
Burned to waste, per cent	39

TABLE 72.—COMPARISON OF IMHOFF TANKS

	Framingham, Mass.	Atlanta, Ga., Proctor Creek plant	Fitchburg, Mass.	Allentown, Pa.	Dayton, Ohio	Cleveland, Ohio, Southerly plant	Chicago, Ill., West Side plant
Population of design	10,000	30,000	55,000	128,000	262,000	400,000	1,850,000
Date of contract for construction	1924	1910	1913	1927	1927	1928	1927
Average sewage flow, m.g.d., design	1	4.2	5.5	17.3	26.2	....	400
Separate or combined sewers	Separate	Combined	Combined	Separate	Both	....	Combined
Preliminary treatment:							
Racks	No	....	Yes	Yes	Yes	....	Yes
Grit chambers	No	Yes	Yes	No	Part of flow	Yes	Part of flow
Fine screens	No	Yes	No	Part of flow	Part of flow	Yes	No
Detritus tanks	No	....	No	Yes	Yes	....	Yes
Type of tank	Horizontal	Horizontal	Horizontal	Horizontal	Horizontal	Horizontal	Horizontal
Number of units built	2	4	5	6	6	12	108
Inside length and width of unit, ft.	47 × 23	96.17 × 25	90 × 30	75 × 66	88 × 73.75	105 × 60	80 × 80
Maximum water depth, ft.	25.8	24.8	25.17	33.58	28.5	25.17	32.8
Total water capacity, cu. ft. per capita	3.98	4.4	4.6	5.91	3.34		
Sedimentation compartment:							
Maximum water depth, ft.	10.8	13.5	14.67	13.5	13.08	11.0	11.7
Length and width of compartment, ft.	47 × 7.83	96.17 × 20	90 × 26	75 × 11.5	88 × 9.33	107 × 11	80 × 17
Number of slots in tank width	2	2	3	4	6	4	5
Clear width of slot, in.	6	9 ±	6	8	8	8	7
Horiz. overhang of partitions, in.	7.5 ±	6 ±	4 ±	8	8	6	9
Slope of partitions, vertical on horizontal	1½ on 1	1½ on 1 ±	1½ on 1	1½ on 1	1½ on 1	1½ on 1	1¼ on 1
Sedimentation period, hr., at average design flow	2.2	3.0	3.75	2.0	2.2	....	2.53
Velocity of flow, theoretical, ft. per hr.	21.3	32	24	37.5	40	....	31.6
Volume for sedimentation, cu. ft. per capita	1.23	2.2	2.2	1.50	1.24	1.41	3.04
Sludge compartment:							
Depth below plane of slots, ft.	15.0	11.3	10.5	20.08	15.42	14.17	21.1
Type of bottom, each tank	2 hoppers	3 conical hoppers	3 hoppers	2 troughs	3 troughs	troughs	2 troughs
Slope of bottom, vertical on horizontal	1 on 2	1 on 2	1 on 2	1 on 2	1 on 2	....	1 on 2
Period of sludge storage, mo.	....	5	6	6	6		
Storage capacity, cu. ft. per capita[1]	2.0	1.44	1.30	3.40	1.64	1.80	5.46[3]
Scum compartment:							
Gas vent area, per cent of total tank surface	21	1.3	15	1.0	2.3	18.5	5.8
Capacity, cu. ft. per capita[2]	0.75	0.76	1.10	1.01	0.46		
Method of sludge removal	Gravity	Gravity	Air lift	Gravity and ejector	Gravity and ejector	Air-lift	Gravity and ejector
Provision for reversal of flow	Yes	Yes	Yes	Yes	Yes	Yes	Yes

[1] Measured below intersection of sloping sides at slots.
[2] Measured above intersection of sloping sides at slots, excluding sedimentation capacity.
[3] Includes provision for digestion of excess activated sludge from North Side plant.

Additional data on the quantity of gas produced in Imhoff tanks are included in Chap. XIV, together with a discussion of other phases of the production and utilization of the gases from sludge digestion.

Statistics of Imhoff Tanks.—The comparison of Imhoff tanks presented in Table 72 illustrates the variations in design of some existing installations.

Special features of the Imhoff tanks at the Westerly treatment plant in Cleveland are described by Gascoigne (14) as follows:

> The 16 Imhoff tanks, which are built on natural foundations without piles, occupy a total area of about an acre, the entire tank structure being built in eight independent units so that should trouble develop in any of the units a large percentage of the plant can be kept in operation during repairs. Between the tanks there are constructed two galleries for removing sludge, operating the perforated water lines around the hoppers and emptying the tanks, should this be found necessary, through special drains in the bottom. Skimming chambers are provided at the entrance to each flow compartment of the tanks. The gas vents are closed with hinged wooden covers.

The design of the Westerly works is based upon an estimated population of 288,000 and an average dry-weather flow of 36 m.g.d.

Efficiency of Imhoff Tanks.—As previously stated, the Imhoff tank performs two functions, the sedimentation of the readily settleable solids and the digestion of the settled solids, so that they may be readily dewatered and dried satisfactorily. The efficiency of the Imhoff tank as a sedimentation device is substantially the same as that of plain-sedimentation tanks as described in Chap. XII. The efficiency of Imhoff-tank digestion compartments has been discussed in the present chapter, in connection with the volume of sludge produced.

In general, it may be said that 40 to 60 per cent of the suspended solids are removed from the sewage in the sedimentation compartment and, if digestion is allowed to proceed to a suitable extent, the solids deposited will be reduced by 30 to 40 per cent by digestion and may be drawn from the tank with a moisture content of approximately 90 per cent.

Starting Operation of Imhoff Tanks.—In northern climates some operating difficulties may be experienced when Imhoff tanks are placed in service for the first time. This is particularly true when plants are completed and placed in operation in the late fall, as is often the case. Sludge solids accumulate during the following long spell of cold weather, while digestion proceeds very slowly, if at all. With the arrival of warmer weather active digestion is established in the large accumulations of sludge, which frequently results in excessive foaming. Usually there is adequate tank capacity during initial operation so that the tanks so affected may be taken out of service when difficulties develop and possibly also some of the sludge may be transferred from them to spare units.

To avoid such difficulties it is desirable, in the northern portion of this country, to start operation of the tanks during the late spring or the summer season. Active digestion may then become established prior to the cold season and the tanks may be seeded with a sufficient quantity of ripe sludge to continue the process, even though at a greatly reduced rate during the winter months. The practice of filling the tanks with water or weak sewage before starting operation appears to be helpful.

Rudolfs (15) summarizes his experiments on seeding new tanks as follows:

> Experiments on seeding of fresh-sewage solids with horse manure, cow manure and muck, in comparison with ripe Imhoff sludge, as aids in digestion tend to show that neither manure nor muck is as effective for seeding as ripe sludge. Muck is about half as good as ripe sludge and horse and cow manure still less. If sludge from a polluted stream is available for seeding, it is to be favored; second choice would be horse manure (without straw) and finally cow manure. Either of these substances is better than nothing, but poorer seeding material than ripe Imhoff sludge. . . . Additions of lime to fresh solids when ripe sludge is present for seeding keeps floating solids down.

Operation of Imhoff Tanks.—Imhoff tanks should be operated with a view to promoting their two important functions, namely, the removal from sewage of suspended solids and the digestion of such solids to a degree sufficient to render them inoffensive when spread on drying beds, used for filling or fertilizer, or otherwise disposed of. Furthermore, the tanks should receive the necessary attention and cleaning required to prevent the production of offensive odors. Beaumont (16) states:

> The satisfactory operation of Imhoff tanks requires intelligent and conscientious supervision founded on a thorough understanding of the principles involved. Well-designed tanks improperly operated have failed to produce satisfactory results, while others of less favorable design have by skillful manipulation been made to give results which have been regarded as excellent. Even in the case of well-designed and properly operated tanks, difficulties are likely to arise at times which will tax the ingenuity of the most skillful operator.

Among the difficulties encountered in the operation of Imhoff tanks are: excessive accumulation of scum, foaming, production of odors, production of sour or acid sludge, choking of slots and sludge pipes, collection of solids on sloping bottoms of sedimentation chambers, and failure of sludge to move to the apex of hopper bottoms.

As in the case of single-story septic tanks, in drawing sludge from the tanks the rate of withdrawal should be slow, in order that the whole mass of sludge may settle and no conical depression may be formed over the orifice. If the latter were the case, relatively undigested sludge might be withdrawn and some of the older, well-rotted material might be left. Poorly digested sludge, if drawn, would be offensive and difficult to dispose of. All the sludge which still is undergoing active decomposition should remain in the chamber to keep up, without interruption, the changes taking place there.

Odor Control.—Frequent flushing and cleaning of tank walls and removal of skimmings will do much to prevent the escape of odors from the tanks. The frequent use of a squeegee in the sedimentation compartment insures the removal of such solid material as may collect on the sloping bottom or in the slots. Influent and effluent channels also should be kept free from deposits and growths.

The control of odors from Imhoff tanks has been summarized by Cohn (17) as follows:

> Care and cleanliness will do much to maintain Imhoff tanks in nonodorous condition, given a fresh sewage and flow conditions through the tanks that do not result in depletion of dissolved oxygen. Where such conditions do not exist, the use of chlorine applied in the raw sewage, either in the trunk sewer as described above, or at the inlet to the tanks, can be depended upon to effect remarkable odor control. At Schenectady, the writer has controlled odors in the Imhoff tanks by prechlorination of the raw sewage at the tank inlet at rates of 4 p.p.m. and over. At rates of 6 p.p.m. and over the tanks gave off a clean chlorosubstitution odor that was apparent to both laymen and sewage experts who visited the plant. The tanks took on a grey color during the morning and a coppery brown tinge in the afternoon. Residual chlorine was not present in the effluent or at any point in the tanks and some hydrogen sulphide was found by repeated tests to be present in the tank effluent. It would appear that odor control does not require the neutralization of all hydrogen sulphide in the sewage. Experience at Schenectady this year has indicated that tank odors are most marked during the early morning hours and the evening hours just prior to the fall of darkness. The prechlorination of the crude sewage flow at these hours is resulting in excellent odor control at the tanks. Without a doubt, the practice of the three c's (care, cleanliness and chlorine) will control odors from Imhoff tanks.

Fair and Carlson (18) and Keefer and Kratz (19) have found that chlorination does not affect digestion.

Scum and Foam Control.—Excessive accumulation of gas-lifted solids, or scum, in the gas vents may be a serious obstacle to successful operation. In some plants the scum has risen above the tops of gas vents and in others it has extended downward practically to the elevation of the slots. Under the latter condition, solids have been carried upward through the slots into the sedimentation compartment, from which they were carried out with the settled sewage, greatly decreasing the efficiency of sedimentation.

Under some conditions the proportion of solids collecting in the bottom of the tank has been so small that it has been practically impossible to remove any solids in the form of sludge without first driving down the scum by breaking it with paddles or streams of water to liberate the gas. If scum is thus broken and the solids are allowed to settle, they may rise again in a comparatively short time.

The reduction of scum by hand methods or by hose streams involves a comparatively heavy operating expense. Learned (20) states that mechanical agitators have been successfully used for breaking up scum at Newton, Kan., and at Enid, Okla.

Foaming seems to be associated with acid conditions in the digesting sludge, such as are observed when the tank is overloaded, when rapid digestion of the sludge accumulated during the cold winter months sets in as the weather becomes warmer, or when acid or acid-forming industrial wastes are present in the sewage. Excessive growths of certain protozoa are said to be associated with foaming at times. Occasionally such foam has accumulated in considerable masses, filling the gas vents, overflowing their walls, and spreading out and covering walks and the sewage in the sedimentation compartments.

Imhoff reports that in the 40 plants in the Ruhr and Emscher Districts foaming is unknown.

Buswell (21) found that scum could be kept soft and disintegrated, a condition favoring its digestion and settling to the bottom of the tank as sludge, and that foaming could be controlled, by routine circulation of liquor from beneath the scum in a stream over the scum blanket for a period of 5 to 10 min. per day. In carrying out a large-scale experiment at Buswell's suggestion, Hatfield reported the installation of the recirculating device

> in four of the eighteen gas collectors of a badly foaming Imhoff tank. During six weeks' operation the foaming seemed to be so well controlled that connections have been placed in all the 108 gas collectors of the six-tank plant. Circulation for scum and foam control is now in use on one tank (18 collectors) and permanent connections to all gas holders are contemplated.

Results of experiments on a plant scale in the control of foaming by liming are reported by Rudolfs (22) as follows:

> A comparison of two Imhoff tanks, one treated with lime to adjust the reaction of its contents, and the other untreated, showed that the treated tank gave no sign of foaming and was free from scum for several months in spite of the fact that it was continuously operating, whereas the untreated tank had to rest and could not be put into operation for a long time on account of heavy foaming.

Production of Acid Sludge.—If the digestion processes do not proceed under normal and favorable conditions, the sludge may be poorly digested and may be in a sour or acid condition. In this connection Gillespie (23) reported as follows: "For acid sludge, milk of lime is an uncertain remedy. In many plants time has eventually accomplished the change from an acid to an alkaline condition."

Cost of Construction, Operation and Maintenance of Imhoff Tanks. The cost of Imhoff tanks, exclusive of land and engineering, based on prices prevailing from 1925 to 1929, varies from $15,000 to $35,000 per mil. gal. daily of sewage treated. An average figure is about $20,000. On the basis of the population for which they were designed, the per capita cost varies from $1.50 to $3.75, a common figure being $2.50. The large variations in unit costs per mil. gal. daily and per capita are partly due to the differences between various installations in allowances for settling and sludge-storage compartments. The principal items of cost are concrete and excavation. Owing to the hopper bottoms, numerous division walls, thin partitions, gas chimneys, baffles and conduits, all of which require steel reinforcement and difficult form work, the unit cost of concrete is relatively high. Excavation costs may be affected by deep cuts requiring shoring, pumping and rock excavation. An average cost of tank per cubic foot of water capacity below the normal operating water line is about 60 cents. Costs vary from 45 to 75 cents in various installations.

The principal work involved in the operation of Imhoff tanks is cleaning, removal of sludge, scum and skimmings, and control of flow. Where pumping is required to remove the sludge, this adds to the operating costs. The tanks are usually operated in conjunction with other treatment processes and operating costs for the tanks alone are not generally available. Such costs as are published range from $1.50 to $5.00 per mil. gal. of sewage treated. The annual operating cost for plants serving 50,000 persons, or less, may approximate 15 cents per capita, although for larger installations the per capita cost may be only half as much.

Bibliography

1. Frank, L. C., and Franz Fries: *Eng. Rec.*, 1913; **68**, 501.
2. Drury, W. R.: *Eng. News-Rec.*, 1928; **101**, 357.
3. Folwell, A. P.: "Sewerage," 10th ed., 1929.
4. *Eng. News-Rec.*, 1928; **100**, 344.
5. *Public Works*, 1926; **57**, 343.
6. Frank, L. C., and Franz Fries: *Eng. Rec.*, 1913; **68**, 519.
7. Eddy, H. P.: *Trans.*, A. S. C. E., 1925; **88**, 465.
8. Elsner, A., F. Spillner and Kenneth Allen: "Sewage Sludge," 1912.
9. Goodman, A. H., and C. E. Wheeler, Jr.: *Sewage Works Jour.*, 1929; **1**, 444.
10. Froehde, F. H.: *Cal. Sewage Works Jour.*, 1929; **2**, 82.
11. Reinke, E. A.: *Sewage Works Jour.*, 1932; **4**, 83.
12. Hatch, J. N.: *Public Works*, 1929; **60**, 255.
13. Hatfield, W. D., and K. Morkert.: *Sewage Works Jour.*, 1932; **4**, 790.

14. Gascoigne, G. B.: *Eng. News-Rec.*, 1922; **89**, 528.
15. Rept. of Dept. of Sewage Disposal, N. J. Agricultural Experiment Sta., 1927.
16. Beaumont, H. M.: *Sewage Works Jour.*, 1929; **1**, 211.
17. Cohn, M. M.: *Sewage Works Jour.*, 1929; **1**, 568.
18. Fair, G. M., and C. L. Carlson: *Eng. News-Rec.*, 1927; **99**, 1030.
19. Keefer, C. E., and Hermann Kratz, Jr.: *Sewage Works Jour.*, 1930; **2**, 338.
20. *Eng. News-Rec.*, 1923; **91**, 301.
21. Buswell, A. M.: *Jour. Ind. Eng. Chem.*, 1929; **21**, 322.
22. Rept. of Sewage Substation, N. J. Agricultural Experiment Sta., 1926.
23. Gillespie, Peter: "The Imhoff Sewage Tank in America," 1922.

CHAPTER XVII

SEPARATE SLUDGE-DIGESTION TANKS

Instead of placing the digestion unit below the sedimentation unit, as is done in an Imhoff tank, the units may be constructed side by side or at some distance from one another, in which case the digestion unit is known as a *separate sludge-digestion tank.* The expense and construction difficulties sometimes incident to the great depth of two-story tanks are thus reduced. Any one of the common forms of settling tanks may serve as the sedimentation unit, the deposited solids being conveyed thence continuously or at regular intervals to the digestion unit. The sludge is commonly pumped to the digestion unit, which therefore can be placed at the most advantageous location and elevation in conformity with other plant structures and subject to foundation conditions. In modern separate sludge-digestion tanks, the incoming solids are thoroughly mixed with digesting sludge and pocketing of fresh solids is thus avoided; the settled solids are introduced while still fresh, *i.e.*, before acid fermentation has set in; and the digestion tank is maintained as warm as practicable. Since the digested solids drawn from the tank contain less water than the incoming fresh solids, provision is made to remove the excess sludge liquor accumulating in the tank. This liquor either is discharged into the influent of the sedimentation tank or is passed through a small sand filter before joining the effluent from the plant. The tanks are usually covered and gas is collected and utilized in maintaining favorable temperatures for digestion.

Most of the recent installations are circular tanks 10 to 40 ft. in depth. Sludge is commonly added within the middle third of the tank depth and withdrawn from the bottom at the center. Where sludge is drawn without the aid of mechanical sludge-removal equipment, the tank bottom is usually given a slope of 1 vertical to 3 horizontal or steeper. Where mechanical stirrers and sludge-removal equipment are provided, the bottom slopes are made relatively flat, as in the case of mechanically equipped plain-sedimentation tanks.

Types of Tanks.—Several types of sludge-digestion tanks have come into use. The older installations consist of either rectangular earth basins, sometimes called digestion pits, or flat- or hopper-bottom tanks, similar to plain-sedimentation tanks. Some of the newer designs have been developed for the collection of gas and for the utilization of mechanical equipment for stirring and mixing the sludge, breaking up scum and facilitating sludge removal.

The newer designs are exemplified by the Emscher, Springfield, Dorr and Kremer-Kusch separate digestion tanks, shown in Fig. 103. The Emscher digester (Fig. 103*a*) is circular in plan with a hopper bottom. Fresh sludge is introduced at the top; excess sludge liquor can be drawn off at different elevations and digested sludge is removed from the

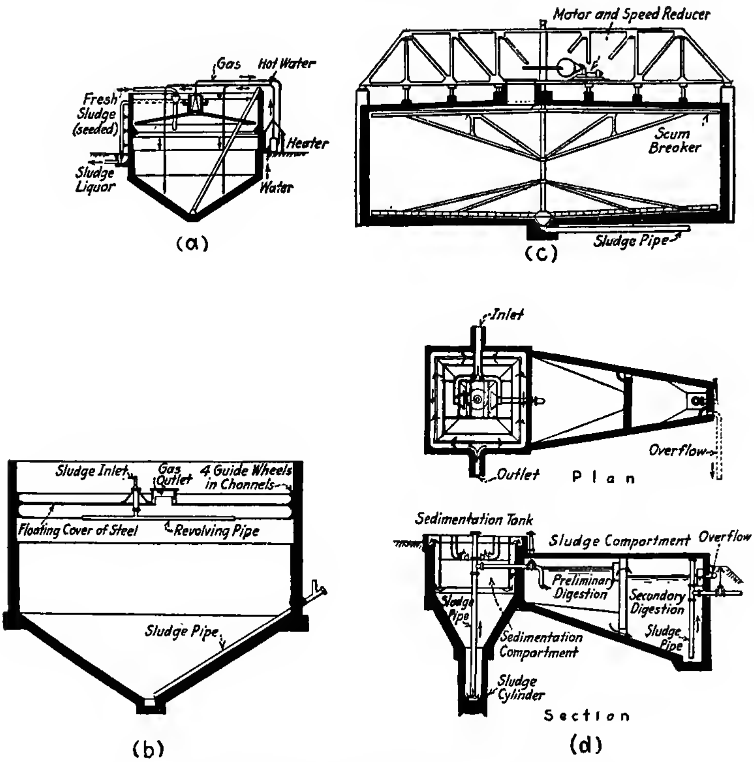


FIG. 103.—Types of separate sludge-digestion tanks.

bottom. The tank illustrated has a fixed cover and is heated by its own gases. The Springfield tank (Fig. 103*b*) is also circular in plan with a hopper bottom and is equipped with a floating gas cover and a special means for distributing the fresh solids. The Dorr digester (Fig. 103*c*) is equipped with a sludge-removal mechanism which also acts as a scum breaker and distributes the incoming solids. In the case of the Kremer-Kusch tank (Fig. 103*d*) which is chiefly employed in Germany, the digestion tank is divided into two compartments which are used in series for two-stage digestion.

Two types of sludge-digestion tanks at the Baltimore sewage-treatment plant are illustrated in Fig. 104.

Elements of Design.—The design of separate sludge-digestion tanks is dependent upon the factors governing the digestion process, local conditions and requirements, the type of tank and equipment to be provided, heating and conditioning requirements and various other elements. Summarized, the more important considerations are: quantity and character of sewage solids to be digested; period of sludge digestion, which is in turn dependent upon temperatures to be maintained and whether or not provision is made for the storage of digested sludge; allowances for freeboard, clear liquor zone and factor of safety;

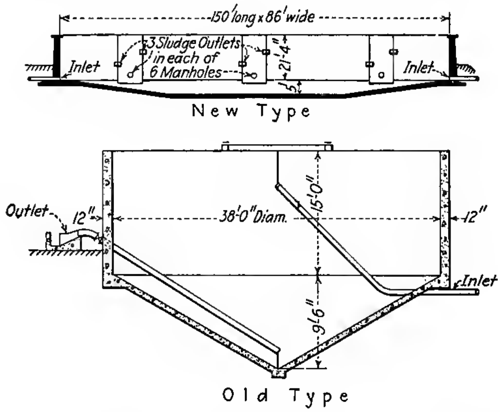


FIG. 104.—Two types of sludge-digestion tanks at Baltimore, Md.

type of roof; type of mechanical equipment to be provided, if any; heating of tanks; gas collection and utilization and possibly waste-gas disposal; and sludge piping.

Capacity of Digestion Tanks.—The theoretical considerations governing the digestion of sewage sludge are given in Chap. XIV and their application in the estimation of required sludge-storage capacity in Imhoff tanks is exemplified in Chap. XVI. Calculations for determining the required capacity of separate sludge-digestion tanks without provision for artificial heating may be similar to those for Imhoff tanks. On the other hand, the temperature of the sludge in separate digestion tanks does not follow that of the sewage. If the tanks are built above the ground, the temperature of the sludge more nearly approaches that of the atmosphere and if they are built below the ground, it approaches that of the ground air or the ground water, depending upon the elevation of the water table. Since these controlling temperatures are frequently considerably lower in winter than those of the sewage, the space required in unheated digestion tanks is commonly larger than the sludge space in

Imhoff tanks. To reduce it, artificial heating is resorted to. The temperature of heated tanks is usually maintained at 70° to 90°F. The digestion period provided at these temperatures is commonly 2 months, when allowance is made for drawing the digested sludge throughout the year.

In the northern part of this country it is almost impossible to dewater sludge on open sand beds during the colder months of the year. In the case of covered beds, the drying rate is greatly retarded during this period. Mechanical dewatering of sludge and its disposal on land may be carried on practically independently of weather or climatic conditions. Unless means are available for the treatment and disposal of the digested sludge as fast as produced, provision is generally made for storing such sludge. This may be done either in lagoons or in relatively inexpensive storage tanks, as at Peoria, Ill. (1). The storage of any considerable quantity of digested solids in heated, separate sludge-digestion tanks does not appear to be economical.

In Imhoff tanks, part of the digestion compartment is occupied by a neutral zone in the vicinity of the slots and part by scum space. In separate digestion tanks, similarly, a capacity allowance 25 to 30 per cent in excess of the theoretical requirements is often made for circulation and distribution of the sludge, for the formation of a clear-liquor zone, and as a factor of safety in the operation of the tanks.

Where appropriate, allowances for the digestion of trickling-filter humus or excess activated sludge are made as in the case of Imhoff tanks. Activated sludge is digested either alone or in conjunction with primary sludge. Rudolfs (2) concluded from tests at Milwaukee "that activated sludge digests more rapidly than fresh sewage solids, and that no greater digestion capacity is required, in spite of the fact that more solids must be treated from the activated-sludge process (without preliminary sedimentation) than from settling processes." Until more information is available, it appears reasonable to allow as great a capacity for activated sludge as for primary sludge, while taking into account the higher percentage of moisture in the activated sludge, both as added to the tank and as digested.

The practicability of the digestion of fine screenings in conjunction with sewage sludges has been discussed in Chap. X. Where this practice is followed, the capacity of the tank is usually increased in accordance with the quantity of screenings to be digested. Similarly, where provision is made for the digestion of sludge from chemical-precipitation plants, it is reasonable to allow for a larger quantity of sewage solids than in the case of plain sedimentation, as well as for a suitable quantity of chemical sludge due to the precipitants added to the sewage.

The capacity requirements for heated sludge-digestion tanks are influenced by or dependent upon a number of factors, among them being the temperature to be maintained, the weight of total and volatile solids to be added, the character of sludge,—whether primary, humus, activated, chemical precipitation or a mixture of such sludges—the process employed,—whether of a single stage or of two or more stages—the storage requirements for digested sludge, if any, and the degree of digestion required, as determined by subsequent methods of sludge treatment and disposal. With a given set of the above conditions, other factors of importance in the capacity requirements are: the concentration of sludge, as added to the tanks, during digestion, and possibly during storage; the reduction of solids by digestion and the rate at which it progresses; and the requirements for clear-liquor zone to insure a relatively clear overflow. With so many variables, it is not surprising that capacity allowances at various plants vary widely. It is not logical to place the allowance upon a per capita basis, without due regard to variations in factors influencing the requirements.

In a report on sewage disposal for the Hackensack Valley, Fuller recommended the equivalent of 16.8 cu. ft. of digestion-tank capacity per pound of suspended solids to be digested daily. An additional capacity of 11.8 cu. ft. per pound of suspended solids to be digested daily was proposed in sludge-storage tanks to be operated in conjunction with covered drying beds. In a report by Gascoigne on the Easterly plant at Cleveland, the total allowance for digestion and storage in conjunction with covered drying beds was equivalent to 12.5 cu. ft. per pound of suspended solids to be digested daily, based on removal from the sewage of 90 per cent of the suspended solids by the complete treatment process.

The loading on the Peoria, Ill., digestion tanks from November, 1931, to March, 1932, inclusive, operated as single-stage digestion tanks, was 1 lb. daily for 16.2 cu. ft. of tank capacity. The equivalent capacity of the Springfield, Ill., digestion tanks, as operated during 1930, 1931 and 1932, was 19.3, 19.5 and 13.5 cu. ft., respectively, per pound of solids added daily. During tests made at Grand Rapids, Mich., from June to October, 1931, the loading on the digestion tanks was equivalent to one lb. of solids daily for 18 cu. ft. of tank capacity. During tests at Elyria, Ohio, from April to September, inclusive, 1931, the digestion tanks had a capacity of 23.5 cu. ft. per pound of total solids added daily. During 1930 and 1931, the digestion-tank capacity at the North Toronto plant in Toronto, Ont., was equivalent to 12.1 and 11.0 cu. ft., respectively per pound of total solids added daily. The sludge drawn during these years, although otherwise suitable and inoffensive, did not have rapid drying qualities.

Based on experience in the operation of sludge-digestion tanks at San Antonio, Tex., Stanley (3) has estimated a required capacity of 40 cu. ft. per pound of volatile matter added daily, equivalent to 29.5 cu.

ft. per pound of total solids added daily. The digestion period is assumed to be 60 days and no allowance is made for the storage of digested sludge. Stanley states that he has not been able at any time to obtain a digested sludge, free from objectionable features for drying on open beds, in less than 60 days' digestion period.

Based on a tank capacity of 15 cu. ft. per pound of suspended solids removed daily by the sewage-treatment process, a sewage flow of 100 gal. per capita daily, total suspended solids of 300 p.p.m. in the sewage and 50 per cent removal of suspended solids by plain sedimentation, the required capacity of digestion tanks on a per capita basis would be found as follows:

$$\frac{100 \times 8.33 \times 300 \times 0.50 \times 15}{1{,}000{,}000} = 1.88 \text{ cu. ft. per capita}$$

On the basis of continuous operation and daily withdrawal of digested sludge, the Committee on Sludge Digestion of the Sanitary Engineering Division, American Society of Civil Engineers, has proposed the following formula (4):

$$C = \frac{AD - \dfrac{PVAD}{2}}{62.5(1 - W)} = \frac{0.008(2 - PV)AD}{1 - W}$$

The symbols used are as follows:

C = theoretical capacity of sludge-digestion tank, cu. ft.
A = dry solids added, lb. daily
D = minimum period of digestion, days
P = volatile matter digested, per cent
W = mean water content of sludge in tank, per cent
V = volatile matter in sludge, per cent.

On the assumptions of a sewage flow of 100 gal. per capita daily, 300 p.p.m. suspended solids in the sewage, 50 per cent removal of suspended solids by plain sedimentation, A = 0.125 lb. daily, D = 60 days, P = 75 per cent, W = 95 per cent and V = 75 per cent, the required capacity of digestion tank would be

$$\frac{0.008(2 - 0.75 \times 0.75)0.125 \times 60}{1 - 0.95} = 2.07 \text{ cu. ft. per capita.}$$

Storage requirements for digested sludge depend upon the methods of dewatering and disposal used in conjunction with the digestion process. When digested sludge is dewatered by mechanical filters, no storage capacity may be required. In combination with covered drying beds, storage for the equivalent of 2.5 months' production of digested sludge may be required in the northern states. In such a case the storage capacity may be estimated along the following lines. On the assumptions that reduction of solids by digestion = 40 per cent; average moisture content of sludge in storage = 90 per cent; specific gravity of sludge in storage = 1.03, and maximum storage period = 2.5 months; the required storage capacity would be

$$\frac{100 \times 8.33 \times 300 \times 0.50 \times 0.60 \times 2.5 \times 30}{1{,}000{,}000 \times 62.4 \times 1.03 \times 0.10} = 0.88 \text{ cu. ft. per capita.}$$

In conjunction with open drying beds in the northern states, a storage capacity equivalent to 5 months' production of digested sludge may be

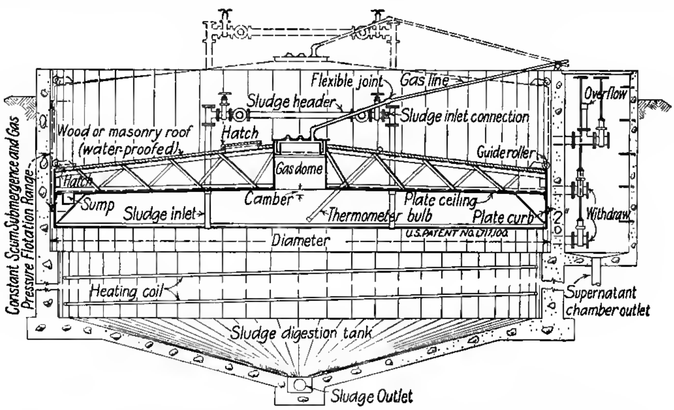


FIG. 105.—Downes floating cover in separate sludge-digestion tank.

required. On the assumptions previously made, the sludge-storage capacity required with open beds would be

$$\frac{5 \times 0.88}{2.5} = 1.76 \text{ cu. ft. per capita}$$

The capacities of a number of American tanks are given in Table 74.

Roofs.—Separate sludge-digestion tanks are usually covered, even when they are not heated and when the gases of decomposition are not collected. If exposed to the air, the floating scum becomes partly dried and has been known to build up a thick, dense mass which is the source of objectionable odors and flies. Roofs may be either stationary, as shown in Figs. 103*d* and 103*c*, in which a water seal prevents the escape of gas, or floating, as shown in Fig. 103*b*. The gas collectors at Birmingham, England, are constructed of reinforced concrete in the form of inverted pontoons that float on the digestion tanks.

The floating cover shown in Fig. 105 was suggested by Downes at Plainfield, N. J., and developed by the Pacific Flush-Tank Co. Covers of this type have been installed at Elgin and Springfield, Ill., Annandale,

N. J., Mamaroneck and Middletown, N. Y., and Cleveland and Elyria, Ohio. At the Easterly plant in Cleveland, floating covers are installed in 12 hexagonal tanks, in which the distance between parallel sides is 95 ft.

The fixed roof of the standard type of Dorr digester (Fig. 103*c*) is slightly arched to facilitate the collection of gases near the center of the tank. Part of the roof load is carried by the truss which supports the scum breaker and sludge plows. A departure from this standard form is made at the North Toronto plant, where the original tanks, which are 40 ft. in diameter, have fixed, domed roofs with a 4-ft. rise above the springing line and none of the roof load is carried by the truss. Several dome-roof tanks of Hardinge design, which contain sludge-removal and scum-stirring mechanisms, have been installed, including 3 tanks 75 ft. in diameter at Wichita, Kan., 2 tanks of 45-ft. diameter at Pottstown, Pa., and tanks 70 ft. in diameter at Findlay, Ohio, 55 ft. in diameter at North Chicago, Ill., and 30 ft. in diameter at McPherson, Kan.

Where relatively thin concrete roof slabs are used, supplementary means may be required to make the roof gastight under the required working pressure, normally not more than 18 in. of water. In the design of tanks with concrete roofs, it is reported that Prüss plans to cover them with sheet lead (5).

Mechanical Equipment.—In order to promote efficient operation, the Dorr sludge-removal equipment has been adapted for use in separate sludge-digestion tanks. It includes a scum breaker attached to the shaft as shown in Fig. 103*c*. The plows attached to the lower arms are held about 3 in. above the floor and cause the thick sludge to move toward the center of the tank. For tight-roofed tanks the upper arms are located just under the roof and act as scum breakers. The mechanism is operated at a speed of 0.04 r.p.m.

For open digestion tanks the scum-breaker arms rotate about 4 in. above the surface of the liquid. These arms are built in the form of channels from which, in one type, are suspended heavy chains that are submerged in the liquid. The function of these chains is to assist in breaking up scum. The raw sludge may enter the digester at the center and be distributed over the entire surface by the revolving channel arms.

Equipment similar to the Dorr has been developed by the Hardinge Co. for both flat- and dome-roof tanks.

Heating of Tanks.—In order to reduce the sludge-storage capacity of digestion tanks, which, as shown in Chap. XIV, is so largely dependent upon the prevailing temperatures of digestion, the tanks are usually heated. The gases of decomposition are commonly collected and utilized for this purpose. Heated tanks are roofed and, in some instances, insulated to reduce heat losses. Earth cover is commonly employed to protect the tanks from direct exposure to low air temperatures. It is advantageous either to place the tanks above ground-water level or to take special measures to insure against excessive heat losses if they penetrate into ground water. Sufficient heat is commonly supplied to the tanks to raise the temperature of the incoming sludge to 70 to 90°F. and to compensate for heat losses from the tanks.

The heat required for raising the temperature of the sludge added to the digestion tanks varies directly with the volume of sludge and rise in temperature required. The volume variations of sludge have been described in Chap. XII. The temperature of the sludge added may vary with the season and normally corresponds to that of the sewage. Data relative to mean annual and monthly temperatures of sewage are given in Tables 26 and 27. Based on a production of 2910 gal. of primary sludge per million gallons of sewage, as derived in the hypothetical case considered in Chap. XII, at a temperature of 45°F., representing the minimum month, and on a temperature of 85°F. within the digestion tanks, the heat required to raise the temperature of the sludge added would be

$$2910 \times 8.33 \times 1.03 \times 40 = 1{,}000{,}000 \text{ B.t.u. per mil. gal. of sewage.}$$

For tanks serving 10,000 persons, with a per capita sewage flow of 100 gal. daily, this would be equivalent to 1,000,000 B.t.u. daily. In the above calculation, the assumption is made that 1 B.t.u. is required to raise 1 lb. of sludge 1°F., as in the case of water.

The heat losses from digestion tanks under conditions comparable to those at the North Toronto plant, given in Table 73, may be equivalent to 0.6°F. daily. A similar loss of heat is reported from the digestion tank at Antigo, Wis. (6). Based on a population of 10,000 and a digestion-tank capacity of 1.88 cu. ft. per capita, as derived in the hypothetical study, the maximum heat requirements to compensate for losses from the digestion tank would be

$$10{,}000 \times 1.88 \times 62.4 \times 1.03 \times 0.6 = 726{,}000 \text{ B.t.u. daily.}$$

The total maximum heating requirements for single-stage sludge-digestion tanks receiving primary sludge, based on the above assumptions, would be 1,726,000 B.t.u. daily.

The most common method of heating digestion tanks is by circulating hot water through coils placed in the tanks. Other methods are heating the incoming sludge to a degree which will compensate for the heat losses, adding hot water to the tanks, or drawing sludge from the tanks, heating it and returning it to the tanks.

At the initial North Toronto plant the primary sludge and excess activated sludge were heated in a heat-exchanger tank prior to discharge into the digestion tanks, which had a total capacity of about 100,000 cu.

ft. of sludge. The heat-exchanger tank was of concrete, about 12 ft. long, 6 ft. wide and 5 ft. deep. Steam coils were placed in this tank and the sludge was heated by direct contact with the coils. A row of diffuser plates was set along one side of the tank at the bottom. This permitted the use of diffused air for inducing rapid circulation of the sludge and facilitating heat transfer. The tank was covered with sectional steel plating, which might be raised with the assistance of counterweights. Ventilation was provided by means of a fan and ducts.

The effectiveness of the North Toronto heat-exchanger tank is shown in Table 73.

The rate of heat transfer from the steam coils under the conditions at North Toronto was about 40 B.t.u. an hour per square foot of radiation per degree difference in temperature between steam and sludge. The

TABLE 73.—OPERATION OF HEAT-EXCHANGER TANK AT NORTH TORONTO SEWAGE-TREATMENT PLANT, TORONTO, ONT.

Month, 1931	Average temperature of air, °F.	Cu. ft. of sludge passed through heat-exchanger tank	Per cent solids in sludge	Average sludge temperatures, °F.					Apparent loss of heat from digestion tanks, °F. daily
				Heat exchanger		Sludge-digestion tanks			
				Inlet	Outlet	Beginning of month	End of month	Average during month	
Jan.	27.4	55,610	3.70	48.7	113.1	81.5	80.0	80.5	0.35
Feb.	28.9	52,483	4.40	46.0	113.9	80.0	79.2	79.5	0.67
Mar.	35.1	78,837	5.24	44.7	108.5	79.2	82.5	81.0	0.59
Apr.	44.0	93,696	4.38	47.8	105.9	82.5	85.5	84.3	0.57
May	54.4	155,363	3.89	52.5	97.3	85.5	88.5	88.9	0.31
June	63.9	135,561	4.18	56.6	89.1	88.5	87.5	87.8	0.09

coils and heat-exchanger tank were cleaned daily with a hose and brush. When the sewage-treatment plant was enlarged in 1933, provision was made for heating the digestion tanks by the circulation of hot water through coils placed in the tanks.

It is obviously easier to remove incrustations from the outside of pipes than from the inside. Since crusting is likely to occur, especially if high temperatures are used, it is best to pass the heating medium through the pipes and to immerse the pipes in the sludge to be heated. The heating of activated sludge in small pipes by means of exhaust steam in a jacket about the pipes proved unsatisfactory at Milwaukee, because the sludge clogged the tubes so rapidly as to make it impracticable.

The area of heating coils required is governed by several factors, including the rate of heat transfer from the coils, the difference in temperature between hot circulating water and sludge in the digestion tanks, and the heat units required to heat the sludge added and to compensate for heat losses. The rate of heat transfer may vary with the density and viscosity of the sludge and the efficiency of the installation, as regards promoting convection currents. Walraven (7) states that at Springfield, Ill.,

> using galvanized, wrought iron heating coils, a coefficient of heat transfer was found as low as 10.2 B.t.u. per hour per 1°F. difference in temperature per sq. foot of heating surface, with the coils surrounded by sludge which averaged 12 per cent solids. With other coils surrounded by thin sludge, approaching supernatant liquor in density, the coefficient was found to be 39.

Keefer (8) reports the overall efficiency of heat transfer in piping coils during tests at Baltimore, Md., as 10.7 B.t.u. hourly per square foot per degree Fahrenheit difference in temperature. The heat transfer through copper-pipe coils in the digestion tank at Halle, Germany, as reported by Heilmann, was 10.2 B.t.u. hourly per square foot per degree Fahrenheit temperature difference (9).

Queer (10) has suggested that the heating coils be distributed in the region between 25 and 60 per cent of the height of sludge in the tank. If the coils are located in the region above 60 per cent of the sludge height and within the supernatant liquor zone, they may be ineffective in supplying heat to aid digestion. Furthermore, the loss of heat from this section of the tank may be greater, because of the lower viscosity of the supernatant liquor. Heating in the region below 25 per cent of the sludge height may be ineffective because of the relative inertness of the material. For a temperature of 80°F. within the tanks, Queer recommends, as a result of tests, that the heating coils be designed on the basis of a unit rate of heat transfer per square foot hourly per degree Fahrenheit difference in temperature, amounting to 12 in the region of 25 to 35 per cent of the height, 18 in the region of 35 to 50 per cent and 24 in the region of 50 to 60 per cent. On account of the high viscosity and poor convectional heat transfer in sludge, he recommends that the heating coils be spaced 15 to 24 in. upon centers both horizontally and vertically.

The limiting factor in temperature of hot circulating water is the crusting of sludge on the coils. Walraven has reported to one of the authors that serious crusting occurred on the coils at Springfield at 180°F. Downes (11) has found a crust about ⅜ inch thick on the hot end of the coil, tapering off to zero about two thirds of the distance around the coil, after three and one half years of service at Plainfield,

N. J., where hot water is used at temperatures ranging from 100 to 140°F. He concludes that, if the inlet temperatures are kept below 110°F., there will be little tendency to form crust and that above 120°F. the tendency to incrustation is much more pronounced. Rudolfs (12) has reported that "experiments made at the New Jersey Sewage Experiment Station have shown that water heated to 140°F. caused no caking" (12). Besselievre states that no caking results at Antigo, Wis., with the initial temperature of hot water entering the heating coils at 120°F. (12).

The drop in temperature through the coils depends on the rate of flow, the rate of heat transfer, the area of pipe coil and the difference in temperature between circulating water and sludge.

In a preceding hypothetical study, it was estimated that the maximum heating requirements for single-stage digestion of primary sludge at 85°F. would be 1,726,000 B.t.u. daily for a sewage flow of 1 m.g.d., from which 150 p.p.m. of suspended solids settle out. Based on a rate of heat transfer of 10 B.t.u. hourly per square foot per degree Fahrenheit difference in temperature and an average temperature of hot water in the coils of 100°F., the surface area of coils required under the above conditions would be $\frac{1{,}726{,}000}{15 \times 10 \times 24} = 479$ sq. ft.

Sludge Piping.—The piping system of a digestion tank includes facilities for adding and drawing sludge and for discharging comparatively clear liquor. Provision for circulating sludge during digestion and for drawing foam or scum, as well as for heating and conditioning purposes, may also be desirable.

Under the influence of sedimentation, compacting and digestion, the sludge commonly collects at the bottom of the tank, whence it may be withdrawn. Scum may rise to the top and an intermediate zone of relatively clear liquor may be formed. The scum may at times accumulate to considerable depths and, instead of collecting at the top of the tank, may collect in an intermediate section above the sludge level with relatively clear liquor above and below. It may be advisable, therefore, to provide either connections at various levels or a flexible drawoff pipe, so that the overflow liquor displaced when a charge of sludge is received, may be taken from the most suitable point. Such facilities will also permit the withdrawal of excessive accumulations of scum for drying on sand beds.

Fischer (13) reports 12 in. of scum on the digestion tanks at Hartford, Wis., 33 in. at Kiel, Wis., and 10 ft. at Ridgewood, N. J. These are maximum figures but are indicative of what may take place. With similar conditions in mind, Downes (14) states: "An overflow at the top might carry with it some suspended matter. Additional 'take-offs' at different levels would facilitate operation." Along the same lines

Gould (12) reports: "By careful selection of the point of withdrawal it is possible to obtain liquid with but little suspended matter, which may be returned to the plant influent without causing much trouble."

Since sludge is usually pumped to the tanks, by suitable connections it is possible to provide both for recirculating the sludge, for purposes of heating or seeding, and for pumping sludge from one tank to another.

Piping systems which carry sludge are generally designed in such a way as to facilitate cleaning in the event of clogging.

Disposal of Digestion-tank Overflow Liquor.—As sludge is ordinarily drawn from plain-sedimentation tanks and added to separate sludge-digestion tanks, it contains a large proportion of water. Since the sludge settles in the digestion tank during storage, relatively clear liquor is formed and provision must be made for its withdrawal and disposal. Some difficulties have been experienced in withdrawing a relatively clear liquor and in the disposal of the liquor as drawn.

When the plant at Ridgewood, N. J., was placed in operation, the fine screenings and settled sludge from plain-sedimentation tanks were pumped to the digestion tank, the overflow from the latter being returned to the sedimentation unit. The digestion tank was seeded with a small quantity of ripe sludge, but a scum formed to a depth of 10 ft. As the overflow pipe extended only 4 ft. below the normal liquid level, a mixture of septic screenings and raw sludge, containing 13 per cent solids, was being discharged into the settling tank instead of clear liquor. Part of this septic scum settled in the sedimentation tank and part was discharged with the effluent on to the trickling filters, where clogging of the nozzles and filter medium resulted. To correct conditions, the overflow was diverted to a sand bed and an adjustable outlet was made, so that material could be drawn from the tank at different levels. The tank was cleaned and reseeded and the pH was controlled by adding lime, so that foaming and excessive depths of scum were eliminated.

For the Sugar Creek plant at Charlotte, N. C., McConnell (15) reported as follows:

> Considerable difficulty has been encountered with the bulking of sludge in the aerating and final settling tanks. This is attributed to the overflow from the sludge digestion tanks being passed through the aeration units. Though the digestion tanks are maintained in a satisfactory stage, the liquors displaced from them during pumping periods are at best a very unsatisfactory addition to the aeration tanks.

McConnell informed the authors that the procedure in September, 1930, was to discharge the excess activated sludge into the preliminary settling tank and to pump as thick a mixed sludge as possible into the digestion tanks, in order to minimize the quantity of overflow liquor. The digested sludge is not allowed to accumulate, but is drawn off, in order to enable a clear-liquor zone to form and to improve the quality of the overflow liquor. Also, an extra quantity of activated sludge is maintained in the aeration tanks, to take care of the oxygen demand of the digestion-tank overflow. These measures have reduced considerably the difficulties formerly experienced with the tank overflow and excellent results have been obtained at the plant.

Piping connections at various depths and cross connections between tanks have been provided at the North Toronto plant in Toronto, Ont., so that the operator is enabled to draw overflow liquor from relatively undisturbed sections of the digestion tanks. The overflow liquor discharged when sludge is being added to the tanks under normal operating conditions has averaged about 0.4 per cent solids. Under abnormal conditions, such as obtain when the tanks are filled with sludge solids, the overflow liquor has contained more than 1 per cent solids. At this plant the discharge of overflow liquor into the incoming sewage has apparently had no detrimental effect upon the final effluent.

The most common method of disposal of overflow liquor is by discharge into the influent of primary-sedimentation tanks.

According to Gould (12),

> a separate sludge well, in connection with digestion tanks, is often of distinct advantage. Surplus liquid can be drained to it from the tanks. The solids can be settled out or floated to the surface by the use of alum, the clear liquid pumped to the main influent of the plant, and the solids returned to the digestion tanks.

Besselievre (12) reports that

> several State Departments of Health have issued rules requiring the treatment of this displaced liquid [from separate sludge-digestion tanks] separately. This usually entails only the construction of a small sand filter.

Goudey states that the overflow liquor from separate sludge-digestion tanks at Salem, Ohio, because of its high B.O.D., upset the activated-sludge plant when returned to the plant influent (16). This was overcome by installing a separate settling tank for the excess activated sludge, concentrating it and pumping thickened activated sludge to the digestion tank. The overflow from the digestion tank was always clear thereafter, showing that the water content of the excess sludge pumped to the digestion tank had a great bearing on the character of the overflow liquor.

Gas Collection.—The collection and burning of the gas produced in digestion tanks present problems of design and operation which require careful consideration. Scum, gas-lifted solids or foam may clog gas-collecting domes and gas piping and provision may be required for its removal or control. In this connection Emerson (17) states:

Equipment for preventing scum from rising into the gas bells themselves is still in the development stage. Screens of a type found satisfactory in one installation may become clogged in a very few days in another. Elevation of gas domes or bells above the surface has been of little value in some installations where foaming has caused froth and scum to rise several feet above normal level.

At Peoria, as reported by Kraus (18),

Due to splashing and the consequent entrainment of sludge in the gas, it was found necessary to raise the vertical gas withdrawal pipes on the collecting domes from 2½ ft. to 8 ft. above the sludge level. This was accomplished by raising the 4-in. gas withdrawal line and housing it in an 8-in. sheet-metal pipe closed at the top. This procedure entirely obviated the difficulty.

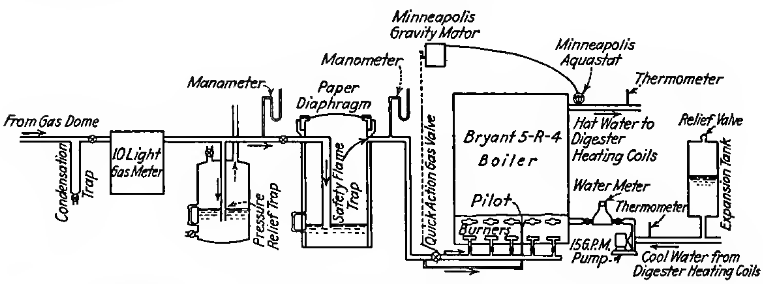


FIG. 106.—Diagram of gas piping and water-heating apparatus at sewage-treatment plant, Antigo, Wis.

Where sludge digestion proceeds under favorable temperature and reaction and the scum is suppressed by roofs or other means, little trouble is ordinarily experienced in gas collection. The breaking up of the scum by stirrers, which liberate entrained gases and cause the scum to settle, may also be helpful in minimizing gas-collection difficulties.

Tanks are generally designed so that the withdrawal of sludge or supernatant liquor does not permit the entry of air beneath the gas collectors, thereby forming an explosive mixture. A mixture of gas and air is highly explosive when the gas comprises 5 to 12 per cent by volume of the mixture. This is not so acute a problem where floating roofs are provided, if the tank is operated so that the roof always exerts a positive pressure and is not allowed to rest in its lowest position. In tanks with fixed roofs, the withdrawal of sludge causes a drop in the water level of the tank, unless the period of withdrawal is timed to correspond with the period of sludge addition. This is not always easy of accomplishment. At the North Toronto plant float-actuated valves on the city water supply permit a flow of compensating water into the tanks, whenever the water level in the tanks tends to drop because of the withdrawal of digested sludge at rates in excess of the addition of fresh sludge.

Donahue (12) suggests the possibility of having a gas container in series with the gas line, so that, as sludge is withdrawn from the digestion tank, gas will be discharged into the tank to replace the sludge removed. This would maintain constant gas pressures. At Lancaster, Pa., each of the two sludge digesters, having an inside diameter of 50 ft., is covered with a concrete slab roof, in the center of which is located a gas holder, 20 ft. in diameter and 17 ft. 4 in. in height, with a capacity of 5500 cu. ft.

In Fig. 106 is shown a diagram of the gas piping and water-heating apparatus at the sewage-treatment plant in Antigo, Wis., where sludge gas is burned under a boiler, in order to heat water which is passed through heating coils in the sludge-digestion tanks.

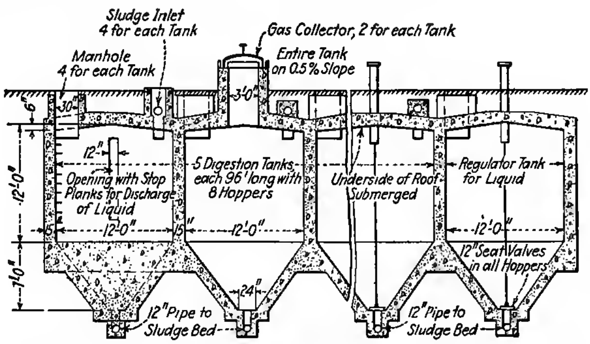


FIG. 107.—Separate sludge-digestion tanks at Boonton, N. J.

A further discussion of gas collection, storage, utilization and protective devices is given in Chap. XIV.

Statistics of Separate Sludge-digestion Tanks.—The design characteristics of several American installations of separate sludge-digestion tanks are given in Table 74 and a cross section of the tanks at Boonton, N. J., is shown in Fig. 107.

Separate Sludge Digestion in England.—The best known examples of this method of sludge treatment in England are at Bath and Birmingham. Plain sedimentation is provided for the sewage from 1,000,000 people at the Birmingham plant and the settled solids are discharged into seven digestion tanks, some of which are provided with raft-supported gas-collection domes, of precast reinforced concrete. During July, 1929, 95 per cent of the power used for pumping sewage and for other purposes about the works was produced from the gas collected. It is expected that when additional tanks are equipped for gas collection, all the gas needed at the works will be obtained from this source.

At Bath two primary digestion tanks, with a total capacity of 675,-000 cu. ft., and nine secondary digestion tanks, with a total capacity of

TABLE 74.—COMPARISON OF SEPARATE SLUDGE-DIGESTION TANKS

	Antigo, Wis.	Aurora, Ill.	Boonton, N. J.	Charlotte, N. C., Sugar Creek plant	Elyria, Ohio	Peoria, Ill.	Salem, Ohio	Springfield, Ill.	Topeka, Kan.	Toronto, Ont., No. Toronto plant
Population of design	10,000	75,000	20,000	50,000	36,000	140,000	15,000	60,000	54,000	100,000
Date of beginning operation	1926	1929	1924[1]	1928	1930	1930	1929	1929	1929	1929, 1933
Average sewage flow, m.g.d	0.9	6.5	....	5.0	3.6	22.0	....	....	5.4	9.0
Separate or combined sewers	Separate	Combined	Separate	Separate	Combined	Combined	Combined	....	Combined	Combined
Kind of sludge digested	Primary	Primary	Primary	Primary and excess activated	Primary and excess activated	Primary and excess activated	Primary and excess activated	Primary and excess activated	Primary	Primary and excess activated
Type of tank	Dorr	Dorr	Hopper-bottom	Hopper-bottom	....	....	Dorr	Hopper-bottom	Dorr	....
Type of roof	Fixed	Fixed	Fixed	Fixed	Floating	Fixed	....	Floating	....	Fixed
Provision for gas collection	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Number of units built	1	3	5	3	2	4	1	....	2	10
Length and width, ft	50, diam.	50 × 50	96 × 12	75 × 73	50, diam.	85, diam.	50, diam.	40, diam.	65, diam.	40–50, diam.
Side water depth, ft	17	16	11.5	10	20	29	20	19.5[3]	23	20
Maximum water depth, ft	17.5	....	19	19.5	....	....	....	31.5	....	22.9
Slope on floor, vertical on horizontal	1 on 48	....	1.4 on 1	1 on 2	....	....	....	1 on 1.6	....	1 on 6.8
Total sludge-storage capacity, cu. ft	33,700	150,000	80,000	210,000	80,000	660,000[2]	39,500	120,000	150,000	400,000
Per capita sludge-storage capacity, cu. ft	3.4	2.0	4.0	4.2	2.2	4.7	2.6	2.0	2.8	4.0
Method of heating	Hot-water coils	Hot-water coils	None installed	None	Hot-water coils	....	Hot-water coils	....	Hot-water coils	Hot-water coils

[1] Date of completion of construction.
[2] Additional sludge storage of 850,000 cu. ft. is provided in unheated tanks.
[3] Assuming 2-ft. freeboard.

615,000 cu. ft., are provided. Combined primary and humus sludges are digested in the tanks.

Separate Sludge Digestion in Germany.—Separate sludge digestion together with collection and utilization of the gas produced is practiced on an extensive scale in Germany. A summary of present practice from the observations of Keefer (5) during 1930 follows:

At Essen-Rellinghausen, Imhoff has constructed a circular digestion tank which is provided with a submerged cover. The gases, which are collected, are burned in a boiler in order to heat water, which flows into the sludge tank at a temperature of about 180°F. Even though the quantity of water added is constantly less than 1 gal. a minute, it is possible to maintain a temperature of about 70°F. in the tank.

At Gelsenkirchen, in the Essen district, Prüss has recently constructed six digestion tanks, all of which are provided with concrete covers for gas collection. The gas, which is burned in a boiler, heats water to a temperature of about 180°F. This water passes to an insulated heat exchanger, which consists of a metal cylinder, containing a series of tubes or pipes. As the raw sludge is pumped into the digestion tanks it passes through the heat exchanger and its temperature is raised.

The digestion tanks at the Gelsenkirchen plant, as well as at Frohnhausen and several other plants in the Essen district, have been equipped with vertical pumping units for stirring and mixing the sludge. The impeller of each pump is set in a vertical pipe which extends nearly to the tank bottom. When the pump is started, sludge is drawn into the lower end of the pipe, and after passing through the impeller is discharged on the surface of the sludge. This operation provides a thorough mixing of the digesting materials. Prüss' observations have been that sludge digests twice as rapidly under such conditions as in a tank where pumping is not resorted to and that the rate of gas production is also doubled.

At the Essen-Nord plant the sludge is circulated for 60 min. twice a day, each pump having a capacity of 150 g.p.m. Prüss reports that by reversing the pump the floating scum is removed from the top and made harmless by mixing it with ripe sludge (19).

Not all German authorities agree, however, that such vertical mixing is a wise procedure. According to Sierp (20):

Intensive mixing of the total contents of a tank prevents the necessary stratification of sludge according to its age and degree of digestion. It could easily happen, if the contents are mixed, that fresh, undigested sludge would be withdrawn from the bottom of the tank, while the digested sludge would remain in the tank, probably in the upper part. The latter difficulty would certainly not occur with horizontal stirring, as in the Dorr digester. The constant stirring, however, would unfortunately disturb the outer layer of insulating sludge.

In order to utilize the heat of the sewage, much as in Imhoff-tank sludge-digestion compartments, Prüss has built a tank, shown in

Fig. 108, in which plain-sedimentation compartments are located adjacent to, and on either side of, a sludge-digestion chamber (19). There are no direct connections between chambers. The sludge is removed daily from the sedimentation compartments by a portable air-lift pump and discharged into the digestion compartment. Vertical screw pumps are provided for circulating the sludge in the digestion compartment.

The Kremer-Kusch tank (Fig. 103*d*), which is trapezoidal in section and plan and is divided into two compartments, is used for two-stage sludge digestion. Following destruction of the more readily decomposed materials in the first compartment, the more resistant materials are finally digested in the second chamber. The first compartment is

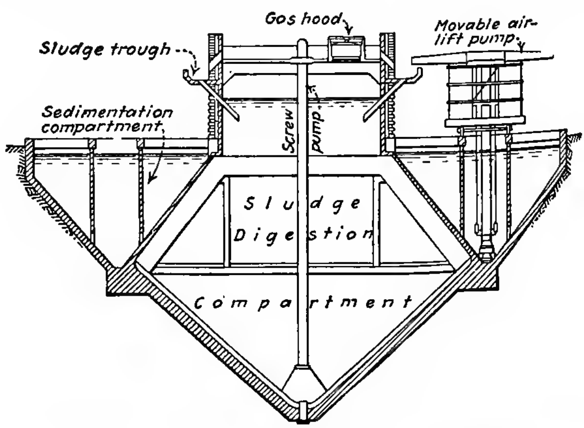


FIG. 108.—Sedimentation tanks with adjacent sludge-digestion tank at Essen-Frohnhausen, Germany.

shallow with large surface area and the second is deep with small surface area. This arrangement is believed by the designers to favor digestion. It should be noted, however, that the fresh solids are not seeded with digested material in this type of tank. The Kremer-Kusch digestion tank is generally used in connection with the Kremer sedimentation tank, a vertical flow tank with a deep sludge cylinder, designed to facilitate the withdrawal of fresh solids.

Imhoff (21) has reported that at the end of 1933 there were 70 towns in Germany with separate sludge-digestion tanks and that the separate sludge-digestion process is being adopted rather than the Imhoff process, since Imhoff digestion compartments cannot be heated effectively.

Cost of Construction, Operation and Maintenance of Separate Sludge-digestion Tanks.—For comparative purposes the costs of separate digestion tanks may best be compared on the basis of unit capacity. Plain hopper-bottom tanks without roofs commonly cost 30 to 50 cents per cubic foot of water capacity. Including roofs, the cost may be 40 to 60 cents a cubic foot and including both roofs and mechanical stirrers, \$0.75 to \$1.00 a cubic foot. The cost of heating equipment and housing will add to the above costs. Based on 2 cu. ft. per capita, a common allowance, digestion-tank costs vary from \$0.60 to \$2.00 per capita. At Peoria, heated digestion tanks of 660,000-cu. ft. capacity, costing about \$200,000, and unheated storage tanks of 850,000-cu. ft. capacity, costing about \$107,000, are provided. Based on 140,000 persons, which was the population of design, the cost per capita is about \$2.20 for sludge-digestion and sludge-storage capacity. Provision is made at this plant for a great load of industrial wastes.

The cost of operation of the Baltimore digestion tanks was about 50 cents per mil. gal. of sewage flow and 1.6 cents annually per capita during 1927 and 1928. This is a large plant and operation costs of smaller plants may be many times as great as that at Baltimore. The operation of the stirrers in the four tanks at North Toronto, which are 40 ft. in diameter, requires about 100 kw.-hr. a day. The unit power cost at this plant is low, but on the basis of an average figure of 1.5 cents a kilowatt hour the power costs for the operation of the stirrers would be about \$550 a year, \$0.30 per mil. gal. of sewage treated and 1 cent annually per capita served. The factors entering into the cost of operation of sludge-digestion tanks include adjustments of sludge flow, labor and supplies for heating, power for and maintenance of mechanical equipment, and maintenance of embankments, superstructures, piping and valves.

Bibliography

1. Greeley, S. A.: *Eng. News-Rec.*, 1930; **105**, 249.
2. Rudolfs, Willem, and H. M. Heisig: *Sewage Works Jour.*, 1929; **1**, 146.
3. Stanley, W. S.: *Sewage Works Jour.*, 1933; **5**, 309.
4. Rept. of Comm. on Sludge Digestion, Sanitary Engineering Division, Am. Soc. C. E. (1931).
5. Keefer, C. E.: *Eng. News-Rec.*, 1930; **105**, 218.
6. Fischer, A. J., and F. K. Quimby: *Public Works*, 1928; **59**, 134.
7. Walraven, J. B.: *Eng. News-Rec.*, 1934; **112**, 8.
8. Keefer, C. E.: *Water Works and Sewerage*, 1933; **80**, 81.
9. Heilmann: *Sewage Works Jour.*, 1933; **5**, 179.
10. Queer, E. R.: *Sewage Works Jour.*, 1933; **5**, 937.
11. Downes, J. R.: *Sewage Works Jour.*, 1932; 4, 72.
12. Gould, R. H.: *Trans.*, A. S. C. E., 1930; **94**, 704.
13. Fischer, A. J.: *Public Works*, 1928; **59**, 382.
14. *Proc.*, A. S. C. E., 1929; **93**, 1065.
15. McConnell, E. G.: *Public Works*, 1928; **59**, 438.

16. FROEHDE, F. H.: *Cal. Sewage Works Jour.*, 1929; **2**, 82.
17. EMERSON, C. A., JR.: *Sewage Works Jour.*, 1929; **1**, 231.
18. KRAUS, L. S.: *Sewage Works Jour.*, 1933; **5**, 623.
19. KOZMA, ALBERT: *Eng. News-Rec.*, 1928; **100**, 362.
20. SIERP, F.: *Sewage Works Jour.*, 1930; **2**, 191.
21. IMHOFF, KARL: *Surveyor*, 1933; **83**, 161.

CHAPTER XVIII

INTERMITTENT SAND FILTERS

The oxidation of the organic matter in sewage by biochemical agencies may be accomplished by intermittent application to beds of sand or other fine-grained material. The action of the filter may be separated into two functions, physical removal of suspended matter, largely by straining, and biochemical removal of colloidal and dissolved organic substances, or their transformation into stable material in the effluent. The mechanical straining action is easily understood, but the oxidation action is more complicated. A gelatinous film is formed, covering each grain of sand. This film is the home of bacteria which feed upon and break down the complex organic matter adsorbed from the sewage and transform it into stable substances. The manner in which changes in sewage are accomplished by filtration is discussed further in Chap. XXI.

Preliminary Treatment.—Treatment of sewage before applying it to filters is not so essential to efficient operation of intermittent sand filters as it is to that of contact beds and trickling filters. Removal of the coarser suspended matter from the sewage in fact results in deeper penetration of the remaining solids into the filter, necessitating the removal of greater depths of sand when the beds are cleaned. The advantages of using settled sewage, however, generally outweigh the disadvantages.

At Marlborough, Mass., the sewage is settled in plain-sedimentation tanks prior to intermittent sand filtration. At filter plants in Framingham and Milford, Mass., presedimentation is provided in Imhoff tanks. Detention periods of 1 to 2 hr. are commonly employed.

Where a high degree of purification is desired, intermittent sand filters are sometimes used for the final treatment of oxidized sewage from contact beds, as at Boonton, N. J., from trickling filters, as at Brockton, Mass., or from the activated-sludge process, as at Tenafly, N. J.

Selection of Filtering Material and Filter Sites.—Where suitable sand is available in place, a filter bed can be constructed by stripping off loam and subsoil and grading the surface to receive sewage. The extent of removal of soil is usually determined by the cost of the work. Large stones and roots of trees, together with the fine earth about them, are generally cleared away.

The limit for excavation may be determined in several ways: by the color of the sand; by loss of weight of the sand on ignition, due to the volatilization of the organic matter; by taking a small portion of the sand in a glass of water, shaking thoroughly, and permitting the sand to subside, the quantity of organic matter and fine sand found upon the top of the sand, when the material has settled, furnishing a ready guide as to the relative content of objectionable matter; and by mechanical analyses of representative samples of the sand.

The filtering material selected for sand filter beds affects in one way or another the rate of filtration, the quality of the effluent, the frequency of clogging and the cost of construction and operation. Where natural sand deposits are not available at the site of the filters, the cost of the material is of great moment.

Clean quartz is best suited for use in intermittent sand filters. If the sand is too coarse, it permits too rapid filtration, insufficient contact and deep penetration of fine solids. If, on the other hand, it is too fine, it limits the water load too greatly and decreases aeration by too long retention of the sewage and capillary saturation of the sand. The aim in selecting the filtering material is to obtain a bed of uniform permeability, so that differences in hydraulic resistance will not cause overloading at places where the sewage passes through the sand most freely. Loam and silt have a tendency to hold water by capillarity and to reduce aeration, unless the filter is operated at a low rate. Clay, cementitious sand and other relatively impervious materials are useless for intermittent filters.

Where no suitable sand is available in place, the site for the filters is often selected with regard to economical haul of the available suitable material from a borrow pit. When material is excavated from a borrow pit, screening and mixing of the sand may be advisable, to obtain uniform permeability in the bed and a material of desirable size and uniformity. Other factors governing the location of the filtration area are isolation, topography, disposal of effluent, length of outfall and pumping requirements. Where either raw or settled sewage is applied to sand beds, it is generally desirable to have the plant at a distance from any substantial settlement.

Mechanical Analysis of Sand.[1]—The present standard methods of mechanical analysis of sand were developed by Hazen in 1890 at the Lawrence experiment station and were described in the report of the Massachusetts State Board of Health for the year 1892 (1). The accumulation of forty years of practical experience has resulted in no change in basic method.

In brief, the mechanical analysis affords information as to the size distribution of the sand grains in a sample, through separating the sand by means of sieves into portions having grains of different average sizes.

[1] For a discussion of this subject see "Water Works Practice," Am. Water Works Assoc., 1925.

From the weights of the several portions the relative quantities of grains of different average sizes are computed.

Two related measures are commonly employed to describe the distribution of particle sizes of filter sand, namely the "effective size" and the "uniformity coefficient." They are determined by passing a known weight of sand through a nest of sieves of wire cloth, with progressively smaller mesh openings, and weighing the quantity of sand passing each sieve. The "effective size" is that size, commonly expressed in millimeters, than which 10 per cent of the sample by weight is smaller; the "uniformity coefficient" is the ratio of the size, than which 60 per cent of the sample is smaller, to the effective size. Hazen fixed upon the effective size as best representing the relative position of different sands with respect to capillarity and frictional resistance to the passage of water. The uniformity coefficient defines the size range within which 50 per cent of the particles lie and reflects to a certain extent the porosity of the sand. Generally speaking, the more uniform a material the greater is its pore space.

Desirable Sand Size.—It is desirable for the effective size of sand used in intermittent filter beds to lie between 0.20 and 0.35 mm., although good work may be done by materials outside these limits. Materials of 0.10 to 0.20 mm. will give admirable effluents but cannot pass sufficient quantities of sewage and tend to clog quickly. The coarser materials may lead to difficulties in satisfactory distribution of the sewage over the bed and to a low degree of purification. Sands with effective sizes even lower than 0.10 mm. are being successfully used in Massachusetts and some good filters contain sand of an effective size as low as 0.03 mm. The deleterious effect of even a small proportion of extremely fine sand in filter beds has led some engineers to the opinion that it may be economical, where beds are constructed by hauling sand into place, to remove the finest material by washing. In fact, it has been specified in some cases that not over 1 per cent of the sand grains shall be less than 0.13 mm. or thereabouts in diameter.

The nearer the uniformity coefficient is to unity the more desirable will be the sand. Nevertheless good work has been done by beds containing sand with a high uniformity coefficient. The coefficient of the majority of beds graded *in situ* in New England is between 3 and 15.

Hydraulics of Filtration.—The frictional resistance of sand to water has been studied in America by a number of investigators. A useful formula for the determination of this resistance was developed by Hazen (1). On the basis of effective size of the sand the capillary rise of water was represented by Hazen in the following expression:

$$H = \frac{1.5}{d^2} \qquad (a)$$

and the frictional resistance offered by the sand was represented by the expression,

$$v = cd^2\frac{h}{l}\left(\frac{T + 10}{60}\right) \tag{b}$$

where H = height in millimeters to which water will be lifted by capillarity in sufficient quantity to prevent circulation of air

d = effective size, millimeters

v = velocity in meters daily of a solid column of water of the same cross section as that of the sand; also approximately equals the rate of filtration in million gallon per acre daily.[1]

h = head of water causing motion

l = thickness of sand; h/l = loss of head, feet per foot of sand

T = temperature, degrees Fahrenheit

c = a coefficient varying with the compactness of the sand and, in the case of intermittent sand filters, with the displacement of the air in the filter. In water filtration, values of 500 to 1200 generally are observed.

Owing to the frequent misapplication of this formula and failure to observe the conditions under which the Massachusetts experiments were made, particular attention is called to the following statement made by Hazen in discussing a paper by Koenig (2):

> These results have been quoted many times, but in doing this their form has been sometimes changed, new assumptions have been introduced, and limitations originally made have been omitted, so that in using them at the present time, the only safe way is to refer to the original publication.

Depth of Bed.—Although little added improvement is obtained by making sand filters deep, it has been found advantageous practically to have 3 or 4 ft. of sand above the underdrains, in order to prevent the sewage from breaking through and reaching the collectors in an inadequately oxidized state. The greater depth has a steadying effect upon the bed efficiency, requires a less elaborate system of underdrains and during the life of the bed permits removing more sand in the course of cleaning operations. To ensure unsaturated sand near the surface, filters are usually made deep enough also to offset the capillary rise of water. Fine sand with an effective size of 0.1 mm., for example, will raise water by capillarity 6 in. more or less. On the other hand, if beds are made too deep, ventilation is impaired. If anaerobic conditions are established in the bottom layers, the effluent is deteriorated in quality, iron may be taken into solution and growths of iron bacteria, *Crenothrix polyspora*, then may clog the drains.

[1] With a porosity of 40 per cent, the actual average velocity of flow through the pore spaces will be 2½ times that given by Equation (*b*).

Clark (3) has stated that, other things being equal, filters of greater depth give effluents of a higher degree of purification than those of less depth. In the case of coarse sands, with an effective size of 0.25 mm. or more, 4 or 5 ft. in depth of bed was found desirable. A filter 10 ft. deep gave a somewhat better effluent, but not markedly so. Shallow filters, with 2 ft. of coarse sand, gave fair results when operated at low rates.

Where intermittent filters are provided to follow contact beds or other oxidation processes, they are sometimes given depths of only 2 to 3 ft. Underdrains are generally spaced more closely in shallow beds than in deep ones.

The depth of sand above the underdrains at a number of American plants is given in Table 76.

Filter Loading.—The most common methods of expressing filter loadings are the volume of sewage treated per unit of filter area and the population served per unit area.

Under average conditions the rates of treatment shown in the following schedule are employed in intermittent sand filtration:

	Gal. per acre daily	Persons per acre
Little or no preliminary treatment....	20,000– 75,000	400– 1,000
Settled sewage....................	50,000–125,000	500– 1,500
Effluent from contact beds, trickling filters or activated-sludge process...	100,000–800,000	1,000–10,000

The filter loading at a number of installations is given in Table 76 and the loading on filters in Massachusetts is shown in Table 75.

The rate at which sewage is applied to the filters in Massachusetts has been fairly well standardized by experience. In 1911, Barbour (4) stated his practice as follows:

> The size of dose may be changed but that usually applied is equivalent to a little more than 1 in. in depth on the sand surface, and experience has proved that a rate of discharge equal to 1 cu. ft. per second for each 5,000 sq. ft. of area will effect, on the ordinary sand bed, good distribution.

The authors' practice is to apply the sewage at the same rate, 1 cu. ft. per second per 5,000 sq. ft. of area, but they aim to cover the bed to an average depth of 3 in. This is because the average bed has an uneven surface and a 3-in. depth of sewage seems necessary to make every square foot serviceable. A dose covering a 1-acre bed 1 in. deep is equivalent to approximately 27,000 gal.

The loading of sand filters is based upon the surface area of the bed with no reference to the volume of filtering material, because the fineness of the filtering medium and the formation of a surface film of sludge concentrate the purifying process in large measure at the surface.

Dosing Cycle.—It is essential in all processes of biological filtration that an abundant supply of oxygen be admitted to all portions of the filter. For this reason the dosing and resting of the filter are of vital importance. It has been found desirable to apply sewage to sand filters in intermittent doses, in order to ensure thorough aeration of the bed and good distribution of the sewage upon it.

The doses may be regulated by a dosing tank or similar device or, in the case of large plants, by manual control of gates on the distributing system. Descriptions of a number of common dosing devices are given in Chap. XXIII.

The appropriate size and frequency of the dose depend largely upon the effective size of the sand, the condition of the bed and the character of the sewage applied, particularly with reference to preliminary treatment. One to three doses a day may be successfully applied, but in practice it has ordinarily been found advantageous to apply not more than one, the quantity being proportioned so as to give the filter ample time to recuperate through the admission of air after dosage. In some cases it has been found better to dose the filter with twice the quantity of sewage on alternate days, the filter thus having a much longer time to become aerated, but also being in service longer. In some places the dosing has been more infrequent than this, ranging from 1 dose in 3 days to 1 dose at long and irregular intervals.

The frequency of dosing also depends upon the capacity of the bed and the size of the individual dose. If a heavy dose, such as one 3 in. deep, is applied to a bed having a nominal capacity of 30,000 gal. per acre, the bed should theoretically be dosed only once in three days. In practice, however, the frequency is governed largely by the quantity of sewage produced at the time.

Size, Shape and Grouping of Beds.—The size, shape and grouping of sewage filters are commonly dictated by considerations of topography; arrangements for distributing the sewage over the beds and collecting the effluent in underdrains; cleaning, resting or repair of beds; economy of large units versus small units; and storage of sewage where intermittency of operation requires.

In the larger intermittent sand-filtration plants, beds having areas from ¾ to 1 acre generally have proved most desirable. In smaller plants the size may be much less, 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.

Except in extremely small plants, beds are rectangular in shape when the topography permits. The cost of embankments, which are usually

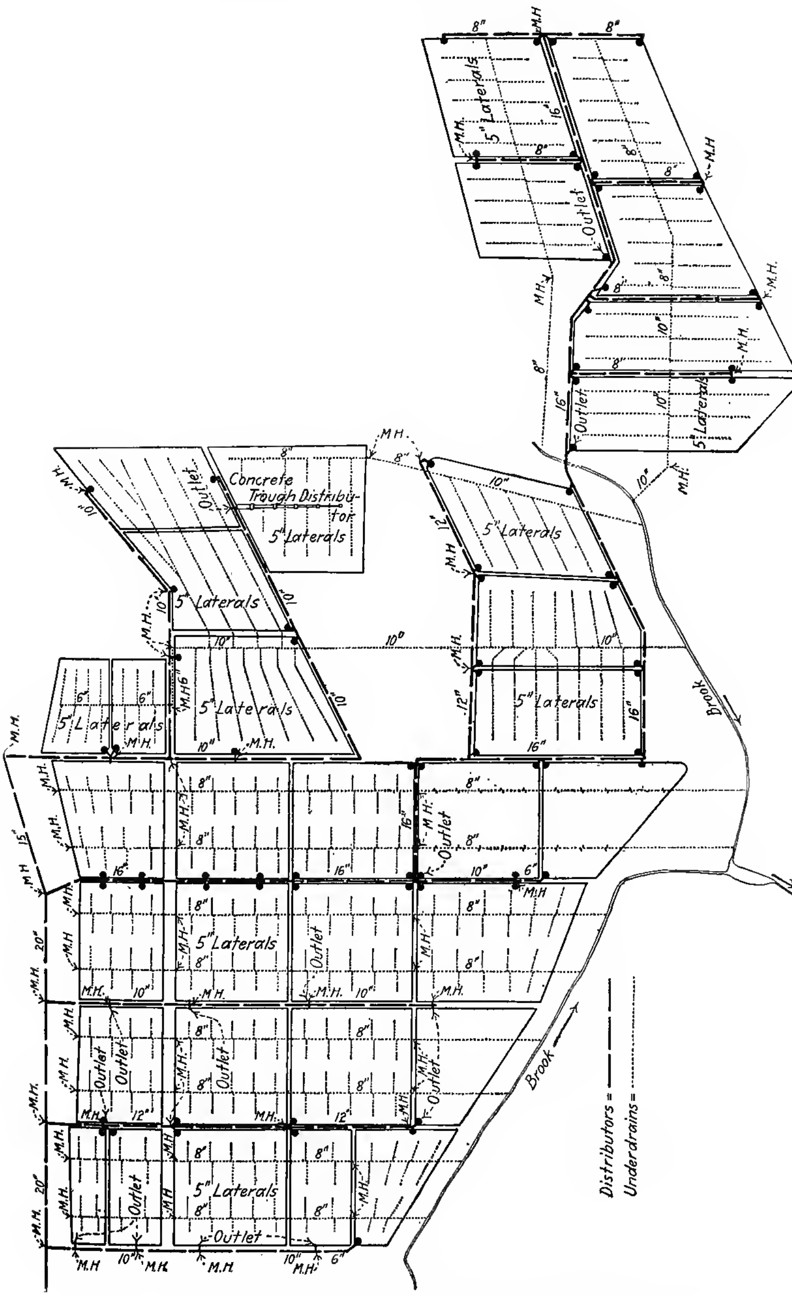


FIG. 109.—Arrangement of intermittent sand filters, Marlborough, Mass.

made of the loam 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 means for distribution of sewage over the beds are usually of importance in determining the shape. If the beds are square, it is practicable to flood them satisfactorily from the corners. On the other hand, the distribution of sewage over long beds is not so uniform as it is over square beds, unless troughs are used. Most operators dislike these structures, because they interfere with cleaning operations.

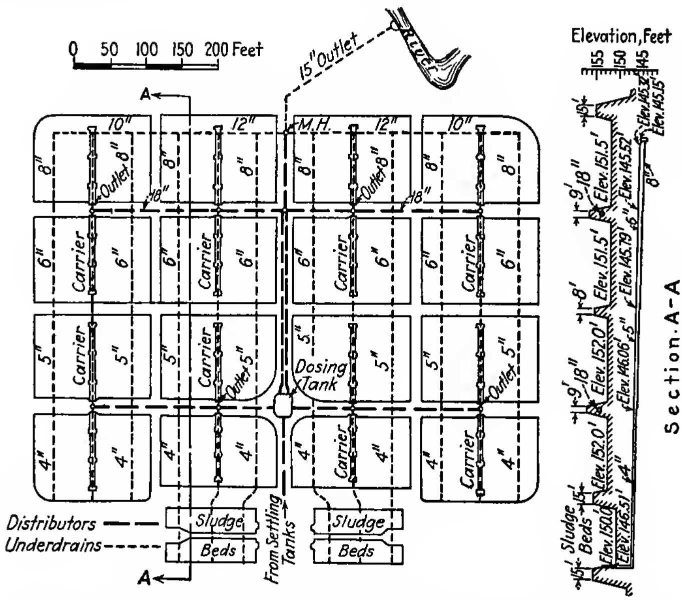


FIG. 110.—Arrangement of sand filters, North Attleborough, Mass.

As a general thing, several arrangements of beds, distributing conduits, drains and roads are practicable and preliminary studies are needed to determine which is best. The cost of the whole installation rather than that of one or two beds is the deciding factor, since main drains or main carriers may prove unexpectedly expensive, if judged by an examination of the needs of only one or two beds.

The number of units in several installations of sand filters is given in Table 76. The arrangement of filters at Framingham, Mass., is shown in Fig. 9, at Marlborough, Mass., in Fig. 109 and at North Attleborough, Mass., in Fig. 110.

Banks and Roads.—For convenience of access and for the removal of surface deposits roadways are provided between successive rows of filters. For the roadway embankments a top width of about 8 ft. has been found to be sufficient. The height of the embankment above the bed is 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. usually being sufficient cover. The side slopes of the embankments are commonly made steep, as a matter of economy. Ordinarily a slope of 1:1 can be maintained but 1 vertical on 1½ horizontal is preferable. The subsoil and loam stripped from the surface of the beds are used for the embankments, which are grassed over to reduce the cost of maintenance.

A sloping driveway is generally provided, leading into each filter bed. It is often 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 are made low and narrow, in order to economize area. A height of 18 in. with a top width of 2 ft. is generally found sufficient.

Distribution of Sewage.—The control of the distribution-pipe system depends upon the size of the filter plant and topographic conditions. In a large plant the sewage may be applied to the beds in groups, the pipe system being controlled by master valves at certain central points, each point of discharge of the sewage upon a bed being further regulated by a gate upon the lateral. In small filter plants the distribution can be most advantageously controlled by operating the gates upon individual beds. As previously stated, control may be effected either automatically or by hand.

The distribution mains are usually built of vitrified salt-glazed pipe, laid with cement, sulfur, or other type of tight joint, to line and grade, like pipe sewers. The distribution-pipe system is often designed on a liberal basis, in order to permit of rapid application of doses to individual beds. Under some conditions it has proved economical to operate the distribution system under a substantial pressure, necessitating cast-iron distributors.

The surface of filter beds is generally made substantially level. There is little advantage in sloping the surface under ordinary conditions, for if the discharge of sewage on to the bed is rapid, satisfactory distribution is obtained with a level bed. The treatment of the surface of the bed for the winter is described hereafter, under winter maintenance.

The following methods of distribution have been successfully used:

1. Single graduated troughs running nearly the full length of the beds, as shown in Figs. 110 and 111.

2. Radiating or arterial troughs, used particularly for irregular-shaped beds, or where more even distribution is to be secured, as with filters dosed at a high rate.

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 one or more of its four corners, as shown in Figs. 109 and 112.

5. An outlet, located at the midpoint of one or both of the long sides of the bed, which is designed to distribute sewage radially through a number of small holes in a semicircular concrete curb. 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 is usually 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.

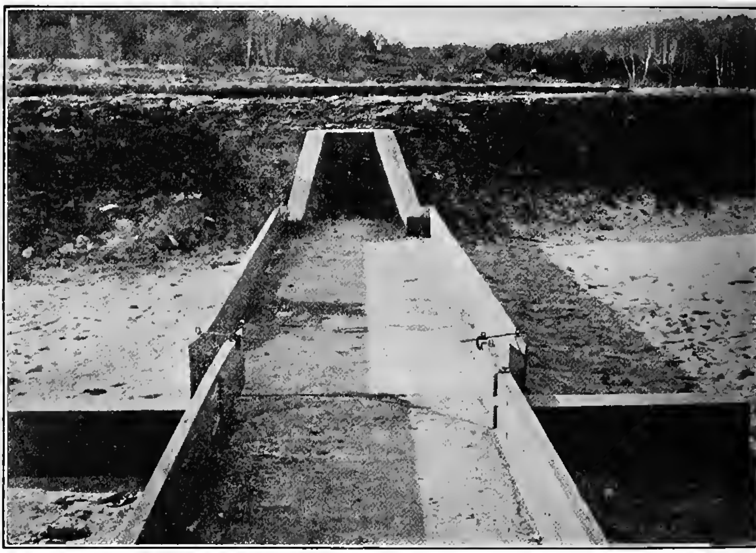


Fig. 111.—Distribution trough of sand filter at Marlborough, Mass.

The quantity of sewage distributed along the line of the trough is fixed by moving the hinged wickets on the sides of the trough.

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, as shown in Fig. 112.

Underdrainage.—Underdrains are generally laid true to line and grade. The bottom of the bed, when constructed artificially, either is level or slopes 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. Commonly the drains have a free outlet, but submerged outlets may operate satisfactorily and sometimes trapped outlets are advantageous.

Vitrified salt-glazed sewer pipe appears to be most satisfactory for underdrains. It is durable and can be cleaned easily. Cement pipe has not proved durable in some cases, apparently being attacked by acids formed in the beds. Blind drains are not desirable, on account of the difficulty of ridding them of deposit and organic growths and because they do not afford means for the rapid escape of the effluent.

In laying the underdrain pipe, the spigot end of the pipe is usually

Fig. 112.—Outlet at sand-filter bed, Marlborough, Mass.

The sewage is under a head of 20 ft. at the end of the pipe and the block of concrete was placed as shown to break the rush of the sewage. The concrete and stone pavement prevents wash of the sand while the sheet of sewage is spreading out and losing its high initial velocity.

separated from the shoulder in the bell by a distance of about 3/8 in., to permit the ready flow of water through the joint into the pipe. The authors have found it advantageous to break the upper part of the bell off the pipe, leaving the lower portion to assist in maintaining the alignment of the pipe, as shown by Fig. 113. The underdrain is then surrounded with screened gravel or broken stone of different grades, to prevent the sand of the bed from entering and silting up the pipe. Two or three grades of gravel are used, obtained by sifting it on at least two screens, the first having about 1-in. mesh and the second about 1/4-in. mesh. This gives three different grades of material. It is desirable to discard stones larger than 2 1/2 in. in diameter or thereabouts.

In placing this gravel the drain pipe is surrounded by a layer of the coarsest material, 3 in. in thickness, which is covered by a 3-in. layer of the next grade, and unless the sand of the bed is very coarse, a third layer of the finest material is required. The sand of the bed may then be placed upon the underdrain without fear of its passing through. In order that the sand may not wash laterally into the underdrains, the layers of gravel surround the pipe instead of merely covering it.

For the purpose of placing the gravel with certainty and rapidity, the authors have found the device illustrated in Fig. 113 satisfactory. Its cost is insignificant.

Underdrains are usually laid at depths of 3 to 4 ft. at the upper end, on flat slopes with 6 in. in 100 ft. as a minimum. The spacing of the underdrains is determined by the effective size and depth of the material

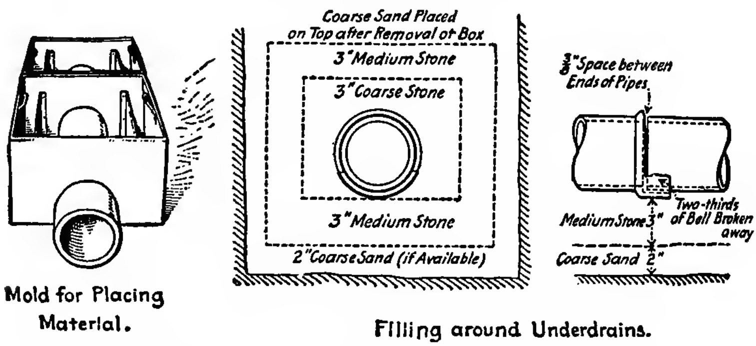


FIG. 113.—Method of constructing underdrains of sand filters.

and the shape of the bed. Ordinarily an interval of 40 ft. has been found satisfactory, though with sand having an effective size of 0.08 to 0.12 mm. it has been found advantageous to decrease this interval to about 30 ft.

The size and slope of the underdrains are determined by the quantity of liquid to be handled and the area of the beds contributing to them. A diameter of 4 in. may be considered the minimum; in some cases, particularly where deep deposits of sand and gravel are found, no underdrains are necessary. Some data on underdrains are given in Table 76.

Opinion is somewhat divided as to the desirability of terminating underdrains in risers coming to the surface of the ground for purposes of aeration and inspection. Where this is done, it is desirable to protect the pipe against breakage and admission of sewage. Such risers were so troublesome at Worcester that they were removed.

The underdrainage pipe is 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 stream or open channel may be advantageously provided with a headwall of masonry. If the lines of the underdrains themselves are long, manholes or lampholes may be desirable at convenient intervals. Provision is commonly made for sampling the effluent from each bed in small plants and from each group of beds in larger ones.

Clogging and Resting of Beds.—On beginning the use of an intermittent filter bed the surface is smooth and level. For the first few days of use uniform distribution of sewage over the entire bed is not obtained, for so much sewage penetrates the sand near the distributor outlets that the dose is exhausted before it has an opportunity to cover the bed. Gradually the surface becomes 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, as discussed in detail by Eddy and Fales (5). If it contains a large quantity of coarse material, such substances settle and accumulate on the surface of the sand, the coarsest in the immediate vicinity of the distributor outlet and the finest at the points farthest from such outlet. In other words, the material is more or less gradually distributed according to the size of the particles. With the lapse of time a surface mat of fibrous material forms, 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 fine substances into the body of the bed. On the other hand, it must be removed at frequent intervals, if the bed is to receive its proportionate 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. Therefore it is necessary, in order to remove the clogging material, to scrape off a substantial portion of the surface sand.

The finely divided silt washed from street surfaces in times of storm, if applied to sand filters, is likely to cause serious surface clogging. Filters which are in good condition may be clogged by the storm water from a single brief shower to such an extent 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 beds, enabling them to receive sewage much more freely than they did just prior to freezing. Beds which are clogged to a considerable degree may be made so porous by freezing that their capacity is temporarily increased greatly, provided the frost does not go deeper than an inch or two.

In spite of surface cleaning the upper layers of sand store up organic matter, which is gradually oxidized by bacterial action, but a certain quantity of humus remains and eventually requires removal.

When a 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 may be avoided by careful attention to dosing and care of surface, but if it exists, the condition may be improved by throwing the bed out of commission and allowing it to dry out, so as to overcome the effect of capillarity and become charged with air. Bacterial activity continues 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 extremely 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.

An example of the effect of this resting period was afforded by the filter beds at Hudson, Mass., after they had been overdosed with sewage containing a large admixture of wool-scouring wastes. Several acres of beds became substantially useless before a plant could be built for treating the wastes separately. The clogged beds were allowed to lie fallow during the winter, and early in the following summer, when they were again put into commission, they were found to have substantially recovered their filtering capacity.

In a few of the larger plants in Massachusetts, the underdrains have become clogged by the passage of sand into the gravel surrounding them. This clogging has been so serious in filters at Marlborough and Natick as to require the relaying of the underdrains. Drains have also become clogged in some instances by organic growths.

Cleaning and Winter Maintenance of Beds.—Where raw sewage is applied to sand beds, the heavy mats of sewage solids crack and curl up on drying and may be peeled off in this condition or scraped off while moist. The mats contain much organic matter and may have a small value as fertilizer.

Clogged beds are sometimes harrowed or plowed, in order to relieve clogging. Unless they are carefully cleaned before this is done, the solids which have been deposited upon the surface by the sewage will become mixed with the sand and thus decrease the capacity of the bed. This is a serious objection to such a method of relieving a clogged condition.

When the solids penetrate deeply into the sand, as happens when settled sewage is applied to the beds, the clogged surface sand requires removal from time to time. This material seems to have little practical use and generally is disposed of by filling low-lying land near the plant. Washing dirty sand for replacing in the filters is open to the objection that the finer sand particles are likely to be lost with the foreign material; therefore, the replaced sand is coarser than the original sand and permits deeper penetration of fine solids and their accumulation as a sealing layer at the surface of the undisturbed sand. Addition of new sand to make up for the sand removed during cleaning has been resorted to at Pittsfield and Framingham, Mass.

Filters are not generally so efficient in winter as in summer. As in the case of all biologically activated processes, warm weather is more favorable than cold. In addition there are certain physical obstacles to be overcome during cold weather. More care is required in dosing filters during the winter than at other seasons. Large doses of sewage are required 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 so large as 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 flow. 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.

When sewage is applied to furrowed filters, the suspended matter tends to settle to 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. 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 is so thick and the quantity of sand clinging to the mat so great that cleaning is impracticable.

Efficiency of Intermittent Sand Filters.—When sand filters are doing their best work, the effluent from the underdrains is clear, substantially free from suspended matter and practically odorless. It may remain stable indefinitely, as measured by the methylene blue test.

The efficiency of various sand filters in Massachusetts is shown in Table 75 (6). These results are for the most part yearly averages of monthly samples. It will be seen that there is considerable difference in the quality of the sewage applied as well as in the effluents. It is probable that most of the analyses of applied sewage represent the stronger day sewage. Filtration does not affect the quantity of chlorides, so that the chloride 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

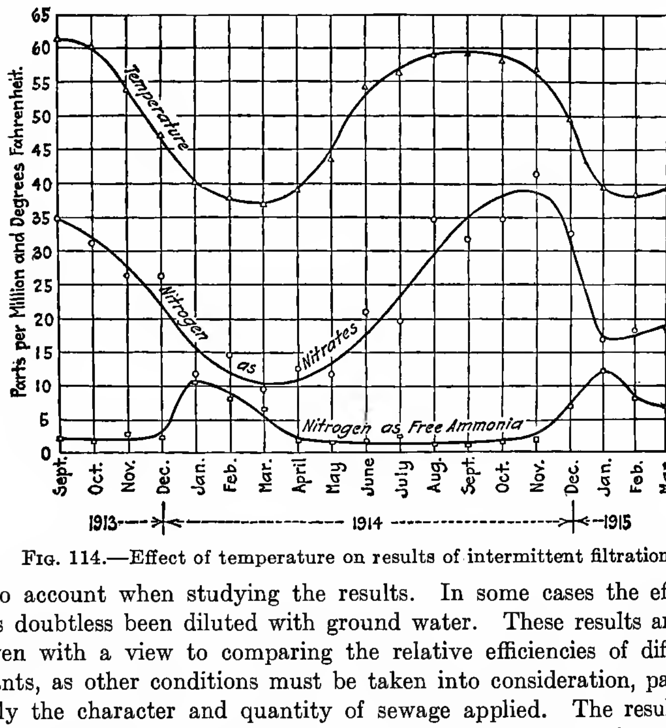


Fig. 114.—Effect of temperature on results of intermittent filtration.

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 comparing the relative efficiencies of different plants, as other conditions must be taken into consideration, particularly the character and quantity of sewage applied. The results do show in a general way, however, the purification accomplished in practice.

The efficiency of sewage filters in a northern climate is not so great during the winter months as during the remainder of the year. 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 results during the cold weather, are not favorable to a high degree of purification. The important influence of the seasons upon the quantities of nitrates and free ammonia in the effluent from a sand-filter plant is shown in Fig. 114, based upon averages of 229 analyses of the effluent from 33 filter beds.

TABLE 75.—EFFICIENCY OF SAND FILTERS IN MASSACHUSETTS
Results of Analyses in P.P.M.

City or town	Free ammonia			Albuminoid ammonia			Chlorides		Nitrogen in effluent as		Rate of operation with even distribution, gal. per acre daily
	Applied sewage	Effluent	Per cent removal	Applied sewage	Effluent	Per cent removal	Applied sewage	Effluent	Nitrates	Nitrites	
Attleborough	26.1	7.2	72	4.1	0.74	82	34.7	38.0	15.06	0.17	68,000
Brockton	38.4	11.9	69	4.5	0.60	87	58.3	73.9	19.03	0.06	
Clinton	28.5	15.8	45	6.6	0.59	91	48.6	41.7	0.62	0.01	51,000
Concord	20.3	0.1	99	5.8	0.17	97	26.1	25.3	16.42	0.04	70,000
Easthampton	41.2	13.3	68	6.0	0.70	88	46.0	35.4	14.13	0.16	
Framingham:											
Settled sewage	48.7	18.2	63	6.0	0.70	88	61.4	55.7	9.44	0.13	41,000
Raw sewage	36.1	23.5	35	13.4	0.90	93	64.8	55.4	5.14	0.10	
Franklin	25.4	11.2	56	2.5	0.90	64	35.2	60.3	3.91	0.13	72,000
Gardner:											
Gardner area	63.0	44.8	29	12.0	2.36	80	61.5	65.0	18.92	0.17	
Templeton area	54.2	40.1	26	5.4	1.35	75	50.3	55.8	3.21	0.07	
Hopedale	48.5	10.0	79	4.1	0.59	86	40.4	36.5	31.85	0.02	44,000
Hudson	51.5	19.1	63	6.0	1.27	79	49.5	48.4	9.28	0.18	63,000
Leicester	27.8	6.1	78	5.2	0.35	93	29.9	26.6	2.96	0.12	
Marion	18.0	9.8	46	3.5	0.37	89	33.7	36.9	4.38	0.08	147,000
Marlborough	44.7	14.9	67	6.2	0.68	89	62.7	51.8	11.21	0.14	56,000
Milford	45.0	21.3	53	4.1	0.81	80	58.4	53.2	7.62	0.09	46,000
Natick	30.5	27.7	9	5.0	0.86	83	55.5	68.9	1.11	0.06	55,000
North Attleborough	21.7	5.5	75	2.8	0.31	89	27.9	25.9	1.34	0.04	93,000
Northbridge	28.1	5.7	80	3.1	0.37	88	25.9	26.5	8.90	0.09	59,000
Norwood	34.8	12.1	65	5.9	0.56	91	114.7	89.3	3.81	0.11	61,000
Pittsfield	27.2	20.5	25	5.2	1.93	63	51.4	43.7	2.93	0.09	97,000
Southbridge	46.6	36.7	21	6.5	0.64	90	40.0	45.8	2.90	0.09	70,000
Spencer	37.4	12.2	67	46.3	0.47	99	38.9	38.9	0.26	0.02	
Stockbridge	27.6	2.1	92	2.9	0.53	82	20.6	17.2	4.99	0.10	
Westborough	42.5	13.2	69	8.1	0.68	92	118.8	90.2	1.36	0.03	44,000
Winchendon	28.8	7.7	73	2.9	0.18	94	44.5	24.0	3.67	0.02	

TABLE 76.—COMPARISON OF INTERMITTENT SAND FILTERS

	Boonton, N. J.	Bordentown, N. J.	Concord, Mass.	Framingham, Mass.	Gloversville, N. Y.	Marlborough, Mass.	Morristown, N. J.	Pittsfield, Mass.	Tenafly, N. J.
Date of construction	1925	1911	1899–1928	1890–1924	1912	1891, 1910–11	1910	1901, 1915	1927
Tributary population	...	...	2,400	14,000	22,080	15,080	12,507	...	5,000
Population designed for	20,000	4,300	3,050	...	...	14,000	15,000	45,000	7,500
Volume of sewage, m.g.d.									
Maximum rate of flow	...	...	0.69	...	...	3.25			
Average rate of flow	2.0[3]	0 2	0.42	1.17	0.15[1]	1.18	0.65	4.5[4]	0.75[4]
Minimum rate of flow	...	...	0.27	...	...	0.33			
Presedimentation	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	
Other preliminary treatment	Contact beds	Contact beds	None	None	Trickling filters and humus tanks	None	Contact beds	None	Activated sludge
Number of beds	8	4	7	33	2[2]	33	4	52	5
Average depth, ft	3–3.5	2.5	...	4–6	3	4.5–6	...	4	3
Area, acres	6.7	0.5	6.3	29.1	1.9	20.9	1.5	41.14	1.5
Effective size of sand, mm	...	...	0.06–0.50	...	0.25–0.35	0.07–0.20	...	0.15	
Dosing apparatus	...	...	Pump	Pump and 12-in. siphon	...	18-in. siphon	...	None	Dosing chamber
Method of distribution	...	...	2-4 inlets per bed	Wooden troughs and plain inlets	Concrete and wooden troughs	1–4 inlets per bed	Troughs	Inlets	
Underdrains:									
Size, in	...	...	None	4–6	6	5	...	3	
Spacing, ft	...	...	...	...	11.5–20	20–50	...	35	
Average loading, gal. per acre daily	300,000[3]	400,000	70,000	40,000	80,000	60,000	433,000	110,000	500,000[4]
Population per acre:									
Tributary	...	...	381	480	11,620	721	8,340	...	3,333
Design	3,000	8,600	485	...	...	670	10,000	1,100	5,000

[1] Average quantity of secondary tank effluent filtered, 1923.
[2] Three filter beds were provided in design, but one is used for treating sludge-bed effluent.
[3] On basis of 100 gal. per capita daily.
[4] Design basis.

Odors from Intermittent Sand Filters.—Opinions differ widely as to the odor coming from sewage filter beds. 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 beds, which receive raw or settled sewage, upward of one quarter of a mile away from any substantial settlement and the larger the area in use the more desirable is it that the beds lie at a considerable distance.

Statistics of Intermittent Sand Filters.—Statistics of a number of American installations of sand filters are given in Table 76.

Cost of Construction, Operation and Maintenance of Intermittent Sand Filters.—The cost of construction of intermittent sand filters depends upon whether or not suitable filtering material is found in place and, if sand must be obtained from a borrow pit, upon the length of haul and the degree of preparation required, such as screening and washing. Additional factors are the extent of preparation of site, including stripping of top soil and excavation and fill for leveling, and the underdrainage system required. The cost, exclusive of engineering and land, commonly varies from $5000 to $15,000 an acre. The cost of 8 acres of beds constructed in 1924 at Framingham, Mass., including a dosing tank, was $70,800, or about $8850 an acre. These filters are composed of sand found in place or transferred from high points to fill in low points. Since the construction of these beds it has been deemed necessary to excavate a portion of the filtering material and replace it with more suitable sand, at a cost of $4000 an acre. The cost of constructing 2.8 acres of beds at Concord, Mass., in 1929, was $10,800, or $5500 an acre. The sand was found in situ and underdrains were not required.

The operation of intermittent sand filter beds usually involves removing accumulations of sewage solids, renewal of sand, renewal and repair of underdrains, regulation of flow to the beds and upkeep of embankments and grounds. Operating costs vary from $150 to $500 annually per acre of filter, from $10 to $40 per mil. gal. of sewage treated and from $0.30 to $1.50 yearly per capita served. Common values are $300 an acre annually, $15 per mil. gal. and $0.50 a year per capita.

Bibliography

1. Ann. Rept., Mass. State Board of Health, 1892.
2. Koenig, A. C.: *Trans.*, A. S. C. E., 1911; **73,** 175.
3. Ann. Rept., Mass. State Board of Health, 1908.
4. Barbour, F. A.: *Jour.*, Assoc. Eng. Socs., 1911; **47,** 59.
5. Eddy, H. P., and A. L. Fales: *Jour.*, Assoc. of Eng. Socs., 1906; **37,** 67.
6. Ann. Rept., Mass. Dept. of Pub. Health, 1929.

CHAPTER XIX

CONTACT BEDS

Another method of oxidizing the organic matter in sewage is treatment of the sewage in contact beds. A *contact bed*, or *contact filter*, is an artificial bed of coarse material, such as broken stone or clinkers, in a watertight basin provided with a controlled inlet and outlet. It is operated in cycles of filling with sewage, standing full in contact, emptying and resting empty, in order to remove some of the suspended matter from the sewage as well as oxidizing organic matter by biochemical agencies. Most of the plants in this country which employ contact beds as the only means of oxidation treatment have tributary populations of less than 5000. At several larger plants where contact beds were installed, these beds have been replaced by other forms of treatment, in which it is possible to secure a higher rate of operation and degree of purification.

Characteristics of Contact Beds.—The broken stone, clinker or similar inert material with which contact beds are filled is sometimes called

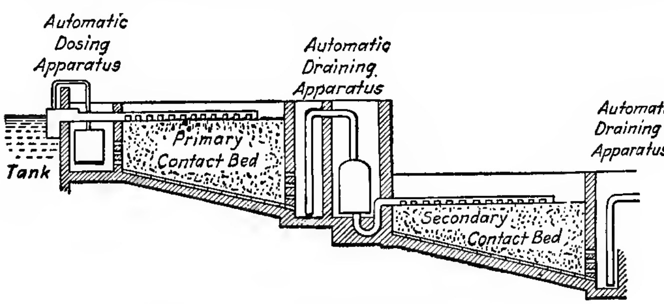


FIG. 115.—Arrangement of double contact beds.

"ballast," this being the usual term in England. Sewage is commonly applied to the bed from above, the outlet from the bed being kept closed. The sewage fills the voids in the contact medium and is permitted to remain in contact with it for a short time. The bed is then drained and allowed to rest, the voids in turn being filled with air drawn in by the receding sewage. The cycle is then repeated. As shown in Fig. 115, contact beds may be built in series, the effluent from the first or *primary* bed passing to a *secondary* bed. This is called *double contact.* Triple contact has also been used. In double contact, the secondary beds can usually treat a somewhat greater quantity of sewage per acre than the primary beds, with satisfactory results. Triple contact generally requires so much head for operation that it becomes uneconomical.

The medium in new contact beds contains voids equal to 40 to 50 per cent of its volume. In the course of a few years, however, the voids generally become filled with sewage solids, which are humus-like in character, and the contact material must then be removed and either cleaned and replaced or renewed. If relatively coarse ballast is employed, the beds may "unload," much as trickling filters do, and final settling is then required.

Contact beds generally provide less opportunity for the dissemination of odors than do intermittent sand filters or trickling filters, as the sewage is not sprayed upon the beds and need not be exposed to the open air. In general, if the sewage reaches the beds in a relatively fresh condition and if the filters are well operated, the odors should not be intense or especially offensive. In contrast with many trickling filters, contact beds are practically free from flies.

Preliminary and Final Treatment.—In the operation of contact beds both raw and settled sewages have been applied. To avoid rapid clogging, however, preliminary treatment in sedimentation tanks is usually provided.

The effluent from fine-grained contact beds may be clear and stable, so that no final treatment is required. The effluents from coarse-grained contact beds, however, usually contain considerable quantities of suspended solids, as the beds unload from time to time, although not to the same extent as trickling filters. Final sedimentation is therefore generally required when the average size of material exceeds 1 in. Where a high degree of purification is required, the effluent of contact beds may be passed on to intermittent sand filters, as at Morristown and Boonton, N. J. High rates of sand-bed dosing are employed in such cases.

Selection of Filtering Material.—Many different materials have been used as media in contact beds, including crushed stone, pebbles, blast-furnace slag, cinders, coke, coal, broken brick and waste from potteries. A detailed discussion of filter media is given in Chap. XXII, in connection with trickling filters. Crushed stone, clinkers and coke are most commonly used. Coke is so light that it tends to float when the bed is filled, causing more or less disintegration with each such movement.

The Royal Commission on Sewage Disposal (1) has indicated British experience with the size of ballast and corresponding permissible loadings for contact beds to be as follows:

Type of sewage	Suspended solids, p.p.m.	Size of ballast
Crude	400	3 in., upward
Septic tank effluent	80–100	$\frac{3}{8}$–$\frac{5}{8}$ in.
Chemical precipitation effluent	10–30	$\frac{1}{4}$ in.

Another point affecting the choice 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 the media 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, serious clogging of the beds can be avoided.

It is questionable, however, whether the average results obtained with such coarse beds are as good as those secured with the finer beds, unless a much greater quantity of contact material is used, so as to provide adequate area of gelatinous surface to effect the desired changes in the applied liquid. Furthermore, in the case of such coarse beds, the underdrainage system may require more expensive construction than is necessary with beds of finer material. The effluent may contain such a large quantity of suspended organic matter at times that settling tanks are needed for its removal, as is the case with the effluents from trickling filters. Some contact beds are covered to a depth of 6 in. with white pebbles or other clean material, which is not dosed with sewage and presents a pleasing appearance. Where double contact is employed, the primary bed is often constructed of coarse material, 3 or 4 in. in diameter, and the secondary bed of fine material.

Data relative to the size and type of materials used at various American plants are given in Table 18.

Depth of Bed.—The method of drainage, the maintenance of beds in good condition and the restriction of area occupied are the chief factors to be considered, besides available head, in choosing the depth of contact filters. Beds are commonly made 4 to 6 ft. deep.

The sewage which stands in the underdrainage system while the bed is full is not oxidized so well 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. Ordinarily the depth of underdrainage system is at least 6 in. It seems undesirable to have this least efficient part of the bed occupy more than one fifth of the whole depth and for this reason the Royal Commission suggested $2\frac{1}{2}$ ft. as the minimum allowable depth of bed.

Deep beds place considerable weight on the material in the lower part, which may lead to its disintegration, if friable material is used. Furthermore, when washing becomes necessary, it is much more difficult to remove and handle the material from deep beds than from shallow ones and the danger of breaking it is increased. For this reason the Royal Commission suggested that 6 ft. was probably the maximum limit of depth. Experiments at Lawrence, Mass., indicated that the depth had little effect on the biological activity of contact beds (2).

The depths of various contact beds are given in Table 18.

Dosing Cycle and Filter Loading.—Dosing schedules for contact beds vary according to the design and the rate of sewage flow, the time of resting depending upon the number of fillings per day. The following schedule illustrates a dosing cycle with two fillings per day:

	Hours
Time of filling	2.0
Time of contact	2.0
Time of emptying	2.0
Time of resting	6.0
Time of cycle	12.0

The cycle actually employed, however, is determined by experience in operating under the conditions encountered at the particular plant. If computations are based upon the water capacity of the bed after it has been in operation for a considerable period of time, when the void space may have been reduced to about 20 per cent, the volume of sewage which can be treated by a bed 4 ft. deep and ½ acre in area will be $4 \times \frac{1}{2} \times 43{,}560 \times 7.48 \times 0.20 = 130{,}000$ gal. per cycle, or 520,000 gal. an acre daily if the beds are filled twice daily. If allowance is made for a rest of 1 or 2 days in 5 or 6 weeks, the capacity is reduced to about 500,000 gal. an acre daily.

It is difficult to determine the most suitable 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 contact medium, the extent to which the medium 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 is 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 at least the quality of the effluent produced 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.

Generally it is considered that the most important biological action upon the organic matter takes place while the bed is standing empty and that this period should be made as long as possible, the necessary time for filling, standing full, and draining being taken into account. British experience has indicated that 4 hr. of rest are 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.

As filling and draining 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 material in the bed or the film adhering to it.

If the period of standing full is too long, appreciable anaerobic decomposition may take place. In some cases Clark and Gage (2) found 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 at Columbus, Ohio, that 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 hours' contact generally gave the best results in the practical operation of contact beds but added that no rule of general applicability could be laid down.

The method of operating contact beds is such that they can be loaded in direct proportion to their depth. Their activity does not change with depth so long as they are in good working condition.

Under average conditions the following rates of treatment on contact beds are commonly employed:

Gallons per acre-foot daily	75,000–200,000
Persons per acre-foot	750–2000

	4-ft. bed, single contact	4-ft. beds, double contact
Gal. per acre daily	400,000–800,000	300,000–500,000
Gal. per cu. yd. daily	62–124	47–78
Persons per acre	4,000–8,000	3,000–5,000
Persons per cu. yd.	0.62–1.24	0.47–0.78

The loadings on contact beds at a number of American plants are given in Table 18.

Advisability of Automatic Control.—The dosing cycles of contact beds are controlled readily either by hand or by automatic dosing apparatus, which may be supplied for flooding the beds and emptying them after a definite period of contact has elapsed. Owing to the facts that both quantity and quality of sewage vary greatly from hour to hour and from day to day and that capacity and efficiency of contact beds change from time to time, in a small plant with three to five beds, automatically controlled, it is possible that the same bed may receive day after day the strong sewage of the day time, while the other beds, receiving weaker sewage, are called upon to do less work. In most cases, however, the fluctuations in flow from day to day cause more or less change in the hourly cycle and, under such conditions, the use of automatic apparatus for dosing tends to promote better work on the part of the beds and less expense for caretakers than the operation of the control 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 better results can usually be obtained by the intelligent operation of gates by manual labor than by automatic dosing apparatus.

Various types of automatic dosing apparatus are described in Chap. XXIII.

Size, Shape and Grouping of Beds.—The size of the units is more important with contact beds than with most other methods of treatment, since it governs the time required for filling and draining the beds. At Manchester, England, with a sewage flow of about 36,000,000 gal. daily, the individual beds were made ½ acre in area.

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 four beds at Washington, N. J., are each 43 ft. square. At Bordentown, N. J., there are eight beds, each 80 by 50 ft. The eight beds at Boonton, N. J., are each 105.5 ft. square. At Auburn, N. Y., four beds are provided, each 144 by 132 ft. The beds at Bellefontaine, Ohio, consist of a concrete tank, 200 by 240 ft. in plan, divided into four compartments by diagonal division walls. At Morristown, N. J., each of the five contact beds has an area of ¼ acre and all combined form a pentagon. Each bed is a triangle, the equal sides being about 148 ft. long.

Distribution of Sewage.—The chief aim in the design of the distribution system for contact beds is to fill the beds without disturbing the deposits in the filters and the bacterial films on the contact surfaces. Small beds are sometimes filled from one or more openings in the sides or corners; this is known as lateral filling. Larger beds are more commonly provided with troughs running over the surface or with pipes laid about a foot beneath the surface. The distributing system may radiate from one corner or may consist of a main carrier from which laterals branch off like the arteries of a leaf. Troughs are generally provided with side outlets and pipes are usually laid with open joints. Subsurface distribution avoids unsightliness of the bed surface and sewage odors. Filling from the bottom through the underdrains has been employed but is open to the objection that the sewage remaining in the underdrains is not subjected to contact treatment. At Manchester, England, open-bottom troughs partly filled with fine material, through which the sewage is screened, have been employed. In some American plants clogging has been reduced by replacing 6 to 12 in. of ballast at one corner of the bed with fine cinders and constructing a low bank of cinders around this area. Sewage is applied at this corner and is filtered through the cinders before filling the bed. The clogged cinders are raked off from time to time.

Sewage is delivered to the beds at Bellefontaine, Ohio, through the underdrainage system. Settled sewage is applied to the top of each bed at Morristown, N. J., through distributing troughs laid on the surface of the bed. The distribution system at Auburn, N. Y., consists of 12-in. tile main feeders, laid diagonally across the bed, and 8-in. laterals extending parallel to the side walls. The pipes are laid with open joints just beneath the surface.

Underdrainage.—In general, concrete has been employed in this country for the walls and bottoms of contact beds. Ordinarily such beds have pipe underdrains laid in large stones, making an underdrainage system at least 6 in. deep. Better drainage is afforded, however, by a floor system similar to those now used in trickling filters. At Mansfield, Ohio, the beds are underdrained by open-joint tile, laid in depressions about 6 in. deep and covered to a depth of 4 to 9 in. with cinders ½ to 1½ in. in size. At Bellefontaine, Ohio, lines of 6- and 8-in. tile, spaced 10 ft. on centers, form the drainage system. Between the tiles and extending up to the top of them, broken stone 2 to 3 in. in size affords a rough, lateral collector system. At Morristown, N. J., the floor of each bed is covered with parallel rows of 6-in. split-tile pipe, laid close together on edge and discharging from either side into concrete drainage channels.

Maturing of Beds.—As is the case with other types of filters, contact beds are less effective in producing a good effluent when they are first placed in operation than after they have matured. The reason for this is that purification depends upon the changes that take place at the jelly-sewage interface and upon the activities of bacteria and other organisms living in the zoogloeal film which covers the filtering medium. Time elapses before favorable conditions are established and the bed has become "broken in," "mature" or "ripe." The period required varies with the season of the year from a few weeks in warm weather to several months during the cooler seasons.

Clogging of Beds.—Contact beds are more or less subject to clogging. This reduces their capacity and impairs the quality of the effluent. If the contact medium is coarse and some unloading takes place, the beds may not clog for years. Clogging may occur, however, at any depth and may require complete removal of the medium for cleaning.

Clogging of contact beds may proceed until the voids are completely filled, 4 or 5 years being a common length of service, or it may stop after the bed has matured. Contact beds which have become clogged after several years of operation are cleaned by removing the medium, washing it by hand or in machines, often similar to concrete mixers, regrading it and replacing the clean material in the beds. Temporarily overloaded beds recuperate during resting periods of varying lengths, depending upon the circumstances.

Treatment of Different Sewages.—Clark and Gage (2) have found that any advantage from the treatment of sewage in septic tanks before application to contact beds is largely mechanical, *i.e.*, the clogging by suspended matter is largely reduced and a considerable load is removed from the bed in that these suspended matters do not have to be taken care of by biological processes. They have found it impossible to obtain a satisfactory effluent when strong septic sewage is applied, but when the sewage is first thoroughly aerated, satisfactory nitrification follows. At Boonton, N. J., provision is made for utilizing a 30-ft. available head between settling tanks and contact beds to secure aeration of the settled sewage, by means of aerator manholes.

Efficiency of Contact Beds.—The degree of purification effected by contact filters varies greatly with the many factors that influence their operation. In general, it may be said that 85 to 90 per cent of the suspended matter, 60 to 80 per cent of the organic matter and 50 to 75 per cent of the bacteria are removed. Where a high percentage removal of suspended matter is desired with coarse-grained beds, the effluent is generally subjected to final sedimentation. The effluent from fine-grained contact beds is fairly clear and stable.

The results of analyses of samples collected at Alliance and Canton, Ohio, by the United States Public Health Service during a study of American sewage-treatment plants are shown in Table 77 (3). The samples collected at Alliance covered a period of 14 days, while those at Canton covered 10 days.

Statistics of Contact Beds.—Statistics of contact beds at a number of American municipalities are given in Table 78. The plant at Boonton, N. J., is the only really modern plant, the others being twenty years or more old. Plans were prepared for an activated-sludge plant at Boonton, but the location of such a plant on this site was opposed by interested parties and, to avoid an injunction, the contact-bed, sand-filter plant was adopted as being acceptable to the opponents of the activated-sludge process.

Cost of Construction, Operation and Maintenance of Contact Beds. Contact beds are of similar construction to trickling filters, except that they are not usually so deep. The cost per acre-foot is somewhat greater for contact beds than for trickling filters, as the cost of the floor

TABLE 77.—AVERAGE ANALYSES OF INFLUENTS AND EFFLUENTS OF CONTACT BEDS

Results Are Stated in P.P.M., Except as Noted

City	Suspended matter			Settleable solids, 2 hr., cc. per liter		Oxygen consumed, 30 min. in boiling water			Alkalinity as $CaCO_3$ by methyl orange			5-day biochemical oxygen demand			Nitrate nitrogen
	Influent	Effluent	Per cent removal	Influent	Effluent	Influent	Effluent	Per cent removal	Influent	Effluent	Per cent removal	Influent	Effluent	Per cent removal	Effluent
Alliance[2]	69	17	75	Tr.[1]	Tr.	38	26	32	192	167	13	68	20	71	2.8
Canton[3]	93	41	56	0.3	Tr.	38	18	53	343	354	−3	90	22	76	0.3

[1] Tr. indicates "trace," less than 0.1 cc.

[2] This plant was replaced in 1929 by Imhoff tanks, trickling filters and humus tanks.

[3] This plant was replaced in 1926 by Imhoff tanks, trickling filters and humus tanks.

TABLE 78.—COMPARISON OF CONTACT BEDS

	Auburn, N. Y.	Bellefontaine, Ohio	Boonton, N. J.	Bordentown, N. J.	Mansfield, Ohio	Morristown, N. J.
Date of construction	1909	1912	1924	1911	1902	1910
Population designed for	8,000	14,000	20,000	4,300	10,000	15,000
Average sewage flow provided for, m.g.d.	....	1.0	....	0.2	1.0	1.0
Total area of beds, acres	1.675	1.1	2	0.73	1.25	1.25
Area of primary beds, acres	1.675	1.1	2	0.365	1.25	1.25
Area of secondary beds, acres	None	None	None	0.365	None	None
Character of filtering material	Stone	Stone	Trap rock	Trap rock and gravel	Cinders	Stone
Size of material, in.	½–2½	½–1	....	....	¼–¾	½–2
Average depth of material, ft.	4	4.5	6	4	4.75	6.33
Gallons daily per acre-ft., basis of design	....	200,000	....	70,000	170,000	127,000
Persons per acre-ft., basis of design	1,200	2,830	1,670	1,470	1,700	1,900
Number of beds	4	4	8	8	5	5
Dosing apparatus	Siphons	Ansonia[1]	Siphons	Siphons	Concentric cylinders	
Preliminary treatment of sewage	Septic tanks	Settling tanks	Settling tanks	Septic tanks	Septic tanks and aeration	Settling tanks
Further treatment of sewage	None	None	Sand filters	Sand filters	None	Sand filters

[1] The Ansonia dosing apparatus consists of a set of flap valves, actuated by a float, as explained in Chap. XXIII.

system is not divided into so many feet of depth. The load per unit of area and depth is less for contact beds than for trickling filters, making correspondingly greater the unit cost per mil. gal. daily of sewage treated and per capita served.

Elements entering into the operating cost of contact beds include periodical cleaning or renewal of the contact medium and control of the sewage flow, including regulation of the dosing cycle.

Bibliography

1. Fifth Rept., Royal Comm. on Sewage Disposal, 1908.
2. Ann. Rept., Mass. State Board of Health, 1908.
3. *Pub. Health Bull.* 132, U. S. Pub. Health Service, 1923.

CHAPTER XX

CONTACT AERATORS

A relatively new device for the treatment of sewage by oxidation is the *contact aerator.* This consists of a crate holding broken stone, coke, brushwood or some other medium, which is placed in a single- or two-story sedimentation tank. Compressed air is usually introduced at the bottom of the aerator, causing a circulation of the sewage through it.

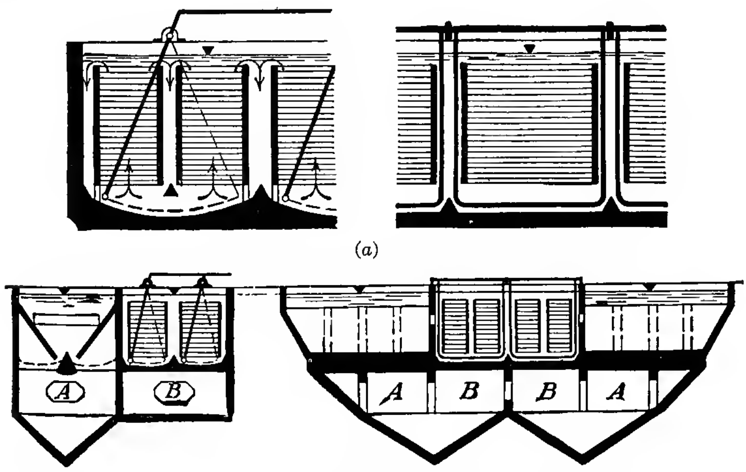


Fig. 116.—Sections of contact aerators. *a*, Built into single-story tank; *b*, built into two-story tank.

Characteristics of Contact Aerators.—Figure 116 illustrates the method of building contact aerators in both single- and two-story tanks. First suggested by Buswell, these devices occupy a position, with respect to physical conditions, more or less intermediate between sewage filters, especially trickling filters, and activated-sludge tanks. Imhoff (1) classifies them as contact beds which operate while continuously submerged in sewage, aerobic conditions generally being maintained by blowing air through the contact material. The air, when used, performs the further function of maintaining circulation in the units. As developed in Germany, complete treatment of the sewage is secured by arranging the units in two or three stages, the first stage alone being employed when only partial treatment is to be secured and further oxidation is to take place in activated-sludge tanks or trickling filters. The three stages may be compared with the three zones of self-purification observed in polluted streams—polysaprobic, mesosaprobic and oligosaprobic. This comparison is based upon the nature of the organisms developing upon the contact material and the changes brought about in the sewage treated. Differentiation from other treatment methods is particularly marked in the first stage, in which sewage fungi predominate and the high-molecular, organic dissolved and colloidal substances are converted into simpler organic substances but not into mineral matter. These intermediate products of decomposition are thrown out of solution by the life processes of the organisms and to a large extent are changed into living cell matter. It is therefore desirable to remove growths more or less continuously, together with such suspended matter as settles on the contact surfaces. Since aerators seem best fitted for the removal of dissolved and colloidal substances from sewage, they are commonly made to follow presedimentation tanks. The growths and solids flushed from them are caught in final-settling tanks. As shown in Fig. 116, presedimentation and final clarification may be secured by building the units into the central flowing-through compartment of two-story tanks. On the other hand, separate sedimentation tanks may be provided. Where more than one stage is employed, settling units are interposed between the successive aerators.

Circulation of the sewage through the contact material is important and is obtained by confining the contact material in boxes, open at top and bottom, which do not occupy the entire width of the tank. Air, blown into the boxes from below, drives the sewage through them and sets up a return current in the space between the boxes and the sides of the tanks. The sewage circulates not once but many times.

The first stage of treatment is commonly accomplished in ½ to 1 hr., while complete treatment requires about 4 hr.

Elements of Design.—Some of the design characteristics of German contact aerators, as given by Mahr and Sierp (2), are shown in Table 79.

German experience with one-stage units, as given by Mahr and Sierp (2) and by Imhoff (1), may be summarized as follows:

Presedimentation period	¾ to 1 hr.
Dimensions of contact units:	
Width of boxes	Less than 5 ft.
Effective depth of boxes	2 to 10 ft.
Length of boxes	About 10 ft.
Capacity of all boxes in terms of detention period	10 to 80 min.
Depth of top of box below sewage level	2 to 3 in.
Width of space between boxes and between walls and boxes	0.5 to 1 ft.

Air supply:

Per gallon of sewage	0.13 to 0.4 cu. ft.
Per square foot of horizontal box area per minute	0.1 to 0.8 cu. ft.
Final-sedimentation period	½ to 1 hr.
Required increase in sludge-digestion capacity	10 per cent

TABLE 79.—DESIGN AND OPERATING CHARACTERISTICS OF CERTAIN GERMAN CONTACT AERATORS

	Hagen	Velbert	Blanken-stein	Menden
Rate of sewage flow, gal. per day[1]	683,000	1,140,000	160,000	1,140,000
Character of sewage	Domestic	Domestic	Septic, strong	Strong, domestic
Contact units:				
Surface area, sq. ft.	264	700	240	515
Effective depth, ft.	2.5	4.8	4.9	8.5
Total volume, cu. ft.	650	3,400	1,180	4,400
Contact period, min.	10	32	80	40
Air supply:				
Cu. ft. per gal.	0.44	0.19	0.28	0.33
Cu. ft. per sq. ft. per min.	0.8	0.2	0.13	0.5
Method of distribution	Swinging pipe	Swinging pipe	Pipe grid	Swinging pipe
Average operating results, p.p.m.:				
Oxygen consumed, influent	244	269	302	341
effluent	184	163	194	141
per cent reduction	25	39	36	59
B.O.D., influent	134	219	362	240
effluent	97	139	228	74
per cent reduction	28	37	37	69
Ammonia N, influent	33	44	53	45
effluent	29	33	49	28
Organic N, influent	21	28	17	25
effluent	11	23	11	7

[1] Day flow.

Contact Material.—Experiments by Buswell (3) have indicated that veneer strips woven into mats, which are fastened into a frame with any desired spacing, prove more practical than laths, brushwood, strings of cotton or hemp, copper gauze or galvanized iron gauze. It has been found that irregular material like coke becomes clogged. Strips of veneer, 1 in. wide, $\frac{1}{16}$ in. thick and 72 in. long, were used in experiments carried on by the Illinois State Water Survey and were found satisfactory (4). The construction of the mats is shown in Fig. 117. Lath was found to be a satisfactory medium but was more bulky than the veneer mats. The latter are lighter and can be handled more easily. Furthermore, they become water-soaked more quickly and do not have to be weighted down. Laths are laid in a horizontal position but are tilted, edge upward, so as to reduce the surface area upon which the sludge may settle. Veneer mats are placed in a vertical position, ¾ to 1½ in. apart.

Aeration.—Buswell's experiments have shown the need of maintaining aerobic conditions in contact aerators. The importance of the circulation of the sewage many times through the racks has been stressed.

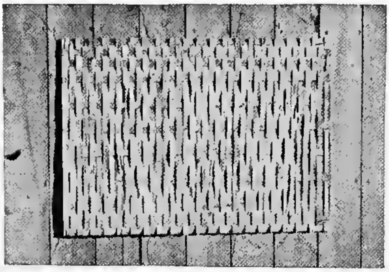


FIG. 117.—Photograph of veneer mat for contact aerator.

Air distribution by means of a perforated pipe, swinging from side to side like a pendulum, is held to be most satisfactory. A stationary pipe grid may be substituted, especially when industrial wastes, such as wool-scouring wastes, require large volumes of air. The pipes are generally 2 in. in diameter, with perforations on the bottom side ⅛ to ¼ in. in diameter and spaced 6 in. apart, giving a combined area 20 to 50 per cent greater than the cross section of the pipe. The pipes swing

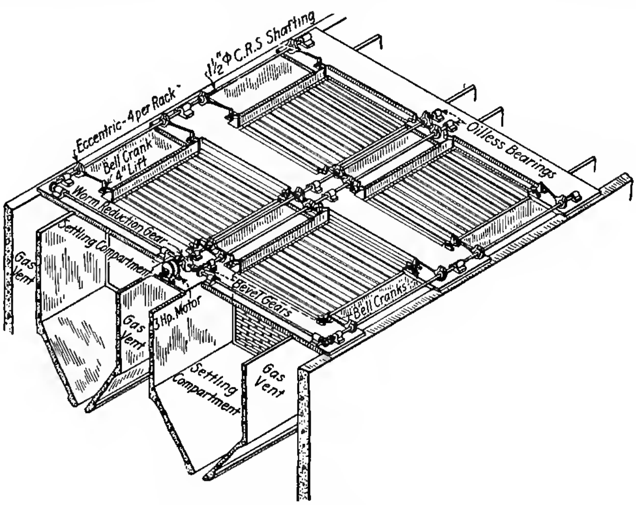


FIG. 118.—General arrangement of mechanism for cleaning contact aerators by agitation.

back and forth two to six times a minute. The swinging pipes are attached to the air mains by short sections of rubber hose, a valve being provided to control the air flow. The tank bottom is curved to fit the arc of the swinging pipe, a clearance of 2 in. being allowed. Sludge deposits are flushed out by the air currents.

Box widths are limited to ensure continuous upward movement of the sewage. Depths are made as great as possible to economize air, the supply per square foot being perhaps more important than the supply per gallon, since circulation is an all-important factor.

Cleaning Contact Medium.—In order to make the contact aerators effective, it is necessary to remove the accumulated matter at frequent intervals. This may be done either by flushing out the aerators with increased quantities of air intermittently or by agitation. The former scheme may be carried out by shutting off the air from all but one unit at a time, perhaps once a day.

For agitation, Buswell (3) suggests a scheme to raise and lower the racks a distance of about 4 in. vertically. The mechanism, illustrated in Fig. 118, consists of a motor operating through a worm reduction gear, revolving a shaft with rod connections to bell cranks, so connected as to raise and lower the racks.

Operating Results.—Experiments by Shive (4) in 1924 to 1926 indicated that submerged surfaces, when used in a ratio of surface to volume of 20 or more to 1, would remove 30 per cent of the nonsettleable or colloidal organic matter from domestic sewage, provided that the accumulated matter was cleaned from the surfaces at frequent intervals and that aerobic conditions were maintained. They also showed that bioprecipitation occurred even though the oxygen supply was low.

The results of subsequent experiments, as summarized by Buswell and Pearson (3), are given in Table 80, showing 41 per cent decrease in the biochemical oxygen demand. If the decrease in B.O.D. due to plain sedimentation is considered as 30 per cent, the submerged contact

TABLE 80.—DATA ON OPERATION OF LATH CONTACT AERATOR BY ILLINOIS STATE WATER SURVEY

Volume of tank	2,040 gal.
Temperature	12–15°C.
Sewage flow, g.p.m.:	
Median	13.6
Maximum	16
Minimum	9.4
Detention period	2 hr.

Composite Samples, Jan. 14 to Apr. 19, 1927

	Influent, p.p.m.	Effluent, p.p.m.	Per cent removal
Settling solids	101.4	49.2	51.4
Oxygen consumed	53.6	39.6	26.4
B.O.D.	112	65.6	41.2

aerators may be said to have increased the efficiency of the installation by 30 per cent.

Further experiments have confirmed previous observations that the purification obtained by this process is 30 to 50 per cent greater than can be obtained by plain sedimentation for a corresponding detention period.

Tests of contact aerators, operated with presedimentation and final clarification, as employed in Germany for partial treatment of sewage, are reported by Mahr and Sierp (2). Table 79 gives some of the German findings. The data indicate reductions in organic matter similar to those obtained in the United States by sedimentation alone. Overloading, while producing a poorer effluent, it is stated, does not otherwise affect the working of the process. This is claimed as a distinct advantage over other biological treatment methods. Contact aerators, furthermore, are said to be relatively tolerant to toxic substances and have been employed successfully in treating phenolic and wool-scouring wastes.

Examples of Contact Aerators: *Jacksonville, Tex.*—Submerged contact aerators have been in continuous operation at Jacksonville, Tex., since Sept. 7, 1927 (5). The sewage passes first through a grit chamber, coarse bar racks and a preliminary-sedimentation tank having a detention period of 1 hr. The effluent from the preliminary-sedimentation tank enters a brushwood contact aerator, illustrated in Fig. 119. After aeration for 1 hr., a short sedimentation period is provided, the sludge being discharged into a separate sludge-digestion tank, which also receives the primary-sludge solids. The effluent from the secondary settling tank passes over a step aerator into a dosing tank, where it is chlorinated and then distributed upon a coarse-grained trickling filter. The filter effluent is settled in an Imhoff tank and the tank effluent passes down an aerator of 12 steps into a small stream.

The contact aerator consists of a reinforced-concrete tank with filtros plates in precast containers laid side by side on the floor. The plate area is one third that of the water surface of the tank. Brushwood is supported on a monel-metal hammock over the air-diffuser plates. Compressed air is introduced through the plates and distributed evenly throughout the brushwood medium. Air is compressed by a positive-pressure blower and introduced under a head of 4½ lb. per square inch. The quantity of air delivered is equal to 0.25 cu. ft. per gallon of sewage and requires 3½ hp. The brushwood is laid in layers, some being parallel to the flow of sewage and some perpendicular to it. The plant was designed for a flow of 400,000 gal. a day.

Experience during a year's operation led to these conclusions by Thackwell (5):

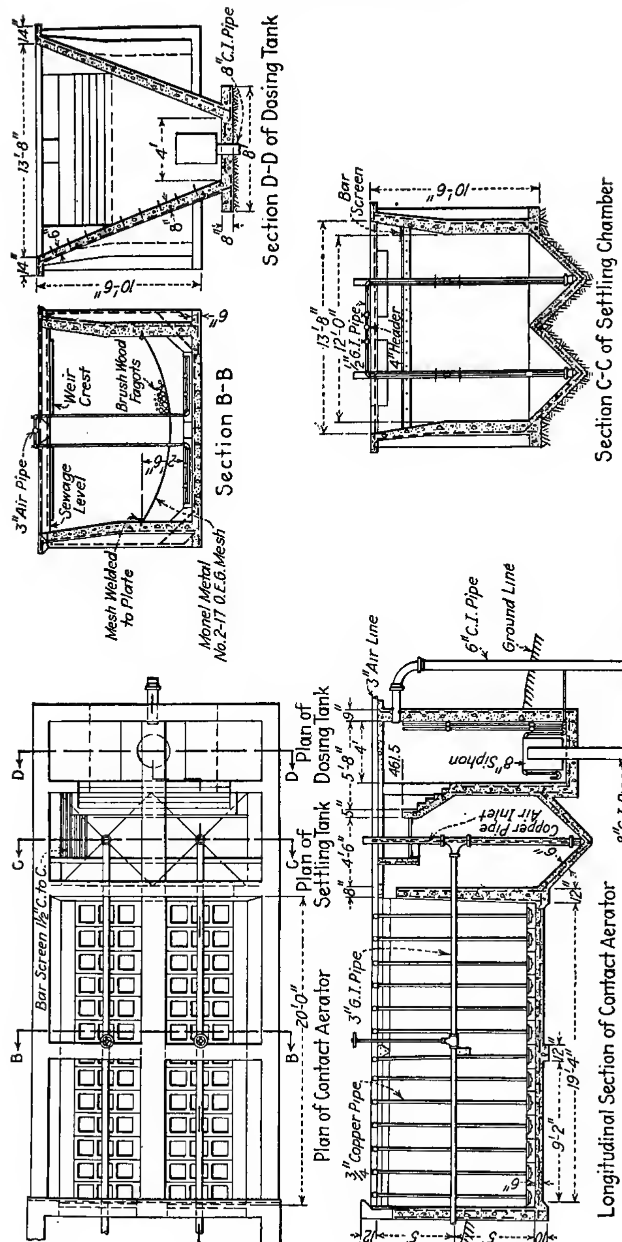


FIG. 119.—Contact aerator, secondary settling tank and dosing tank at Jacksonville, Tex.

The value of the aerator as a treating unit is about equal to that of a small contact bed and it has about one fourth the effectiveness of a trickling filter under normal rates.

There is a rise in pH from 7.1 to 7.5 and the dissolved oxygen is increased from zero to 0.5 p.p.m.

From my experience I should say that the value of contact aeration would only be one where partial treatment is desirable. As a forerunner of trickling filters, I should prefer a coarse-grained roughing filter, provided sufficient head were available. Straight activated sludge of twenty-minute aeration period would be just as satisfactory for a preliminary treatment.

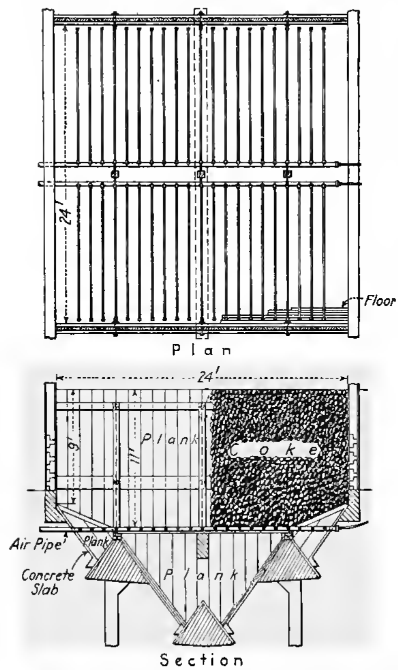


FIG. 120.—Contact aerator in Imhoff tank at Brighton treatment plant, Rochester, N. Y.

Brighton Plant, Rochester, N. Y. A contact aerator was constructed in one of the Imhoff tanks at the Brighton treatment plant, Rochester, N. Y., in the hope that its operation would enable the trickling filters to operate at increased rates and thereby would postpone additions to the filters. The aerator was placed in operation in August, 1928, and regular operating records were kept and tests were run to determine its efficiency (6).

The contact aerator occupied the central portion of the sedimentation compartment of an Imhoff tank. It was 24 ft. square in plan, occupying the full width of the sedimentation compartment, and was symmetrical about the transverse axis of the tank. It had a maximum depth of 11 ft. and a minimum depth of 9 ft., its base resting upon the two ridges of the partition which separates the sedimentation compartment from the digestion chamber, as shown in Fig. 120.

The aerator was formed by constructing two wooden partitions across the sedimentation compartment, 24 ft. apart, and building between these partitions a floor consisting of 2 by 4 pine joists set on edge with ¾-in. slots between. The air-supply piping was laid on top of this flooring and the compartment thus formed was filled with selected coke of 1-in. size. Since the coke showed a tendency to float, probably on account of the disturbance created by the rising air, wire mesh covered with about 2 in. of broken stone was placed over the surface of the coke to hold it in place.

To prevent the passage of sewage through the open space between the bottom of the contact aerator and the partition separating the sedimentation compartment from the sludge-digestion chamber, a vertical bulkhead was constructed across the center of this portion of the sedimentation compartment. Thus the sewage, in order to pass through the sedimentation tank, first had to pass into the body of the contact aerator.

Following the suggestion of Imhoff, who visited the plant in May, 1929, vertical baffles were placed across the sedimentation compartment about 18 in. from each end of the contact aerator. They extended nearly 5 ft. below the flow line of the tank. The tops of the baffles were slightly above the sewage surface.

The air-distribution system was divided into two sections, each supplying one half the aerator. Two 3-in. galvanized-iron headers extended across the bottom of the aerator near the center, each supplying one half the unit. Air-distributing laterals consisted of ½-in. copper pipe, spaced at intervals of 1 ft. along each header. Along the underside of each of the copper distributors there were pairs of ⅛-in. holes at intervals of 1 ft. Air was supplied by either one or two air compressors, each rated at about 180 cu. ft. of air a minute under a pressure of 6 lb. per square inch and operated by a Pelton wheel, driven by the flow of sewage which reaches the plant under a head of about 70 ft.

During the early months of operation the contact aerator effected a marked reduction in suspended solids in excess of that by the Imhoff tank alone, but during later months this advantage entirely disappeared. The effect of the contact aerator in reducing the biochemical oxygen demand was noticeable at times, but over the entire period to June, 1929, considered as a whole, it produced no distinct improvement in this respect. It seems probable that the unsatisfactory results were attributable in part to the fact that accumulations of fungi and organic solids were not being flushed out of the coke effectively. Remaining therein these solids decayed and in so doing interfered with the normal growth of the oxidizing fungi. Furthermore, the circulation through the medium was inadequate previous to installing the baffles outside the aerator, as suggested by Imhoff.

Changes were made in construction and method of operation of the contact aerator, so that a marked improvement in efficiency took place in June and continued through July, 1929. More adequate flushing of the medium was secured by passing the entire volume of air through each side of the unit once a day and some cleaning was effected by the temporary use of chlorine.

Tests made during July, 1929, indicated that the aerator was capable of removing oxygen demand equivalent to approximately 30 per cent of that residual in the Imhoff-tank effluent. It was evident that the coke in the aerator was not so suitable a medium as laths or veneer mats.

Cost of Construction, Operation and Maintenance of Contact Aerators. Estimates prepared for Buswell and Pearson (3) indicate that the cost of construction per mil. gal. of sewage flow provided for would be $1460 for the racks, $400 for blower equipment and $1000 for rack-shaking equipment, a total cost of $2860 per mil. gal. On the basis of 0.2 cu. ft. of air per gallon of sewage at a pressure of 4 lb. per square inch, 60 per cent overall efficiency and power at 1.5 cents a kilowatt-hour, the cost of power for air compression would be about $1.10 per mil. gal. of sewage treated.

Bibliography

1. Imhoff, Karl: "Arithmetic of Sewage Treatment Works," 1929.
2. Mahr and Sierp: *Technisches Gemeindeblatt*, 1928; **31**, nos. 10 and 11.
3. Buswell, A. M., and E. L. Pearson: *Sewage Works Jour.*, 1929; **1**, 187.
4. *Bull.* No. 25, Illinois State Water Survey, 1928.
5. Thackwell, H. L.: *Eng. News-Rec.*, 1929; **102**, 35.
6. *Public Works*, 1928; **59**, 369.

CHAPTER XXI

THEORY AND OPERATION OF TRICKLING FILTERS

In addition to the types of filters already discussed in detail, trickling filters are employed to convert the organic matter in sewage into a more stable form or into mineral matter through the agency of living organisms in the presence of free oxygen. A *trickling filter* is an artificial bed of coarse material, much the same as that used in contact beds,

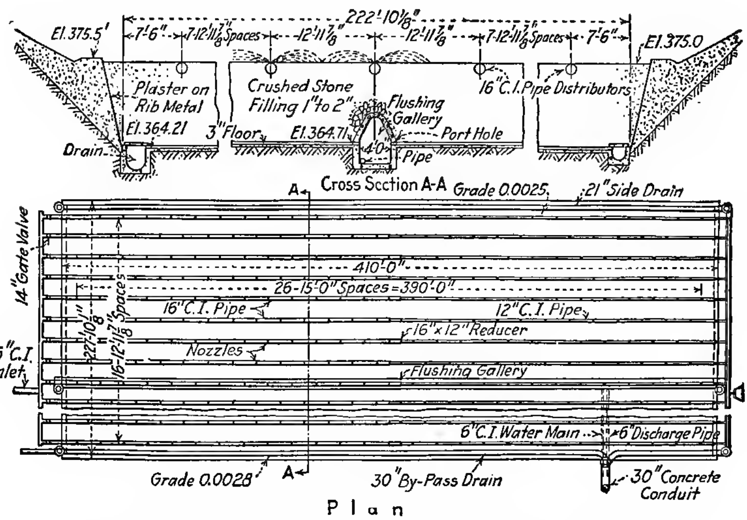


FIG. 121.—Arrangement of trickling filters, Fitchburg, Mass.

over which sewage is distributed intermittently and through which it trickles to underdrains. The filtering material commonly consists of broken stone or slag, 6 to 10 ft. in depth.

Characteristics of Trickling Filters.—In trickling filters the filtering medium is sometimes placed in watertight tanks and sometimes not. Figure 121 shows a plan and section of one of the trickling filters at Fitchburg, Mass., built from the plans of Hartwell and the authors. Operating at a much higher rate than contact beds, trickling filters require a more elaborate system of sewage distribution and underdrainage. Sewage is applied to the surface by means of nozzles and other sprinkling devices of varying design, which approach uniform dosing of the area. The pores of the bed are not filled with sewage and ventilation is commonly such that oxygen may be absorbed continuously by the jelly-like covering on the medium and by the sewage, rather than intermittently as is the case with the older types of sewage filters. Dosage is far more frequent than with intermittent sand filters and contact beds.

In further contrast to intermittent sand filters and most contact beds, which must be cleaned from time to time, well-designed and carefully operated trickling filters are self-cleaning. They unload the accumulated, more or less stabilized solids and hence the filter effluent requires secondary or final treatment in sedimentation tanks, if these solids are not to be discharged with the final effluent.

How Changes in Sewage Are Accomplished by Filtration.—The manner in which not only suspended solids but also colloidal and dissolved materials are removed from sewage by filtration is complex. The mechanical straining out of the larger suspended solids needs no explanation and the factor of sedimentation which is operative in filters has been discussed in Chap. XII.

Neither straining nor sedimentation, however, accounts for the removal of colloidal and dissolved organic matter which takes place during filtration. This action is closely associated with the development upon the filtering medium, during the ripening period of the filter, of a slimy, gelatinous growth of bacteria, called zoogloea, and other organisms such as fungi, protozoa and certain higher forms of life. Furthermore, it is associated with the *contact* between the sewage and the slimy film.

At the beginning of the present century two opposing theories were advanced in explanation of the phenomena of colloid removal and oxidation of dissolved matter by filtration. The first is known as Dunbar's theory, the second as the Hampton doctrine of Travis. Generally speaking, the former stressed the importance of biological activity, while the latter gave greatest weight to physical action. There was some truth in each theory.

In the light of modern developments in the chemistry of colloids Buswell (1) explains the way in which changes are brought about in sewage by filtration on the following basis.

1. In the surface film of liquids the concentration of dissolved matter tends to change in such a way as to decrease surface tension. If, therefore, a substance dissolving in a liquid increases the surface tension of the solution, the film concentration of the substance tends to become less; if it decreases the surface tension its film concentration tends to become greater. (Most salts and all strong bases increase the surface tension of water; but ammonia and nitric and hydrochloric acids decrease it.) Substances in the colloidal state are believed to act in a similar way and there are found in

sewage colloidal soaps and proteins that tend to concentrate in a surface film. These changes in film concentration are not restricted to the air-liquid interface, but seem to apply also to the interface between the bacterial slime and the liquid. The extent of the interface or contact surface is, therefore, important.

2. At the jelly-sewage interface the following occurs:

a. The substances concentrating at the interface are adsorbed to the contact surfaces and thus removed from the sewage being filtered;

b. The adsorbed substances are attacked by the enzymes and living organisms present in the slime;

c. As rapidly as these substances are removed by digestion or direct absorption into the living cells, others come to the interface and further removal is thus effected;

d. In the presence of air the products of decomposition of organic matter are chiefly carbon dioxide, nitrates and a humus-like residue. The gas escapes from the filter; nitrates, being salts, increase the surface tension of water and therefore pass from the interface into the flowing sewage; the humus-like residue must either be removed, as in intermittent sand filters and contact beds, or sloughs off from time to time, as in trickling filters. It is an important fact that the humus from trickling filters settles readily, unlike the colloidal matter from which it largely originated.

3. Experience has shown that the oxygen present in the sewage applied to sewage filters is insufficient to maintain their oxidation reactions. The filters must, therefore, be permitted to rest between doses, in order that the biological jelly may dissolve or adsorb a sufficient quantity of oxygen from the air that penetrates into the interstices of the filter while it is standing idle.

To summarize, it may be said that the changes accomplished by filtration are due to surface phenomena involving changes in the film concentration of the sewage at the jelly-sewage interface and adsorption of the substances concentrating at the interface, the activity of living organisms and their enzymes found in the gelatinous films covering the contact surfaces, and the maintenance of favorable conditions by an adequate supply of oxygen. All these changes are spoken of as *contact* phenomena.

Preliminary Treatment.—Some form of preliminary treatment is commonly employed with trickling filters, to prevent the clogging of distribution nozzles and of the filter medium. How far treatment should be carried depends upon the economy of plant construction and operation. Fine screens have been advocated in some instances, but it is doubtful if the solids passing screens but removed by sedimentation tanks can be handled as economically by filters as by preliminary tank treatment. Plain-sedimentation tanks, septic tanks, and Imhoff tanks have all been used to remove settleable solids.

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 the dissemination of odors. There is also a general belief that sewage can be oxidized by bacterial agencies more advantageously when fresh than when septic.

Partial oxidation of clarified sewage prior to filtration is discussed in Chap. XXIV.

Final Treatment.—The effluent from trickling filters usually contains considerable quantities of suspended matter. On this subject Wagenhals, Theriault and Hommon (2) comment as follows:

> Very little importance can be attached to the amount of suspended matter in trickling filter effluents or the reduction in this constituent effected by them unless results are available for long periods. The amount of suspended matter in the effluent and its ratio to that in the influent will normally vary within wide limits, depending upon the state of the filters as regards unloading. The function of the filter, as regards suspended matter, is not to remove it permanently but to hold it till it has been worked over and oxidized. The character of the solids in the effluent is almost always entirely different from that of the solids in the influent, and it is this change of character that is important.

Notwithstanding losses due to digestion, as much suspended matter may be expected collectively over a protracted period of time in the effluent as is present in the influent; indeed, colloidal matter is often precipitated within the filter, so that the effluent frequently averages higher in suspended matter than the influent. This difference may be augmented by bits of organic growths, worms and the like washed from the filter.

In appearance, the suspended matter in 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 gm. of each sediment was mixed with 4000 cc. of river water saturated with oxygen. The results are given as follows (3):

	Dissolved-Oxygen Saturation at End of 5 Days, Per Cent
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 raw 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 particles of organic growths.

In a majority of trickling-filter installations it has been deemed necessary to remove suspended matter from the effluent by means of tanks, strainers or filters. Much of the suspended matter lends itself readily to sedimentation, but there is considerable fine matter which cannot be removed effectively by this method. For complete removal, strainers or filters are used. In the latter case it is common practice to pass the trickling-filter effluent through sedimentation tanks first. Intermittent sand filters not only remove the suspended matter but afford further oxidation and bacterial purification. They may be operated at comparatively high rates, as at Brockton, Mass., and formerly at Gloversville, N. Y. The permissible loading of sand filters following trickling filters is touched upon in Chap. XVIII.

The design and operation of final-sedimentation tanks, or humus tanks, are taken up in Chap. XXII.

Maturing of Beds.—As in the case of other types of filters, trickling filters are less effective in producing a good effluent when they are first placed in operation than after they have matured. The reasons for this are given in connection with contact beds in Chap. XIX. The period of maturing varies with the season of the year from a few weeks in warm weather to several months during the cooler seasons.

Unloading of Filters.—The periodic storage and unloading of solids by trickling filters are among their most important attributes. Since the beds harbor huge numbers of living organisms, it is not surprising that they respond quickly to changes in temperature and other conditions. In summer, in the northeastern states, the quantity of solids in the effluent is about the same as in the applied sewage. Oxidation is then more active and nitrification takes place if doses are not so great as to prevent 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 also is 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 filters emit 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 filters, which thus recover 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.

Great numbers 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 perform some function in the general transformation of organic matter into more stable matter, which goes on inside the filters.

Results of studies of film accumulation in the trickling filter at Plainfield, N. J., during the year ended June 16, 1926, are reported by Rudolfs and Peterson (4). Chemical analyses indicate that nitrification is best shortly after the filter medium has been cleaned by the sloughing off of the accumulated film and that it decreases gradually with the increase in thickness of the film. Consequently, such a large weight of film as accumulates prior to the slough is undesirable.

Clogging of Filters.—Trickling filters are subject to clogging by the breaking down of the filtering material to fine particles which compact, by the retention within the voids of excessive quantities of sewage substances, and by the formation of a mat of algae and fungi in the top layers. The selection and testing of material so as to secure a medium not subject to disintegration and compacting are treated in Chap. XXII. Clogging due to an excess of sewage substances in the voids may be caused by the size of the filtering material, the relative roughness of the filtering material, or a greater burden of suspended matter than the filter can treat successfully. The size and relative roughness of material are considerations also covered in the discussion of the selection of filter media.

Adequate allowances for sedimentation of the sewage prior to filtration as a rule permit filters to operate without clogging. Industrial wastes, particularly iron wastes, may cause serious clogging of trickling filters. At Fostoria, Ohio, precipitated iron so seriously clogged a trickling filter, 35,000 sq. ft. in area and 7 ft. in depth, which had treated a total of only 392 mil. gal. of settled sewage, as to require the removal and cleaning of the filter medium at an expense of some $15,000. The raw sewage at Fostoria during 1929 contained on an average about 80 p.p.m. of iron, the settled sewage 54 p.p.m. and the humus tank effluent 17 p.p.m.

Under normal conditions of installation and operation, trickling filters remain self-cleaning and clogging of the main body of the bed is an exceptional occurrence.

Cleaning of Filters.—There are several methods of combating surface clogging of trickling filters and the resulting ponding of sewage:

1. Resting and drying the bed without permitting destruction of the contact film to take place.
2. Hosing the surface of the bed with a stream from a monitor or fire hose.
3. Picking over, loosening, forking or harrowing the surface layer.
4. Chlorination of the applied sewage for the destruction of surface growths, or use of copper sulfate, caustic soda or chlorine compounds for this purpose.

5. Cultivation of the water springtail, *Achorutes viaticus*, a member of the family *Poduridae;* this is a wingless insect which feeds on the organic slime accumulating on the surface.

Clogged underdrains can sometimes be freed by flushing the drainage system by means of hose streams. Chlorination of the applied sewage may be effective in causing an unloading of the humus-like materials collecting on the filter medium.

During the spring of 1928 trouble was experienced with the trickling filters at Lackawanna, N. Y., due to a growth of slime and an accumulation of solids on the beds (5). Excessive ponding occurred on the filters and deposition inside the pipes and spray heads caused friction losses so that the sprays became feeble, with resultant poor distribution of sewage on the beds. Upon examination it was found that the slime consisted of gelatinous vegetable growths, which apparently caught and held grease and particles of scum from the Imhoff tanks, as well as dust and cinders.

For about five days an average dosage of 32 p.p.m. of chlorine was applied to the septic Imhoff-tank effluent, insuring a residual of 0.5 to 5 p.p.m. in the applied sewage. The application of chlorine resulted in disintegration of the slime, which sloughed off and was washed out of the filter; the header pipes and spray nozzles were cleaned and the sprays became vigorous and full again. The cost of chlorination per acre was $110.29, with chlorine at 6 cents a pound.

At Schenectady, N. Y., a heavy, black surface film formed on the trickling filters during the winter of 1926–1927 (6). No pooling was caused. A 48-hr. period of chlorination at a rate of about 25 p.p.m. cleared all the filter units. Although chlorination was not repeated during the entire summer, the film did not develop unduly.

Chlorination to cure pooling of trickling filters proved effective at Elgin, Ill. (7). Pooling was caused by a slime of gelatinous, plantlike nature that sealed the surface between the stones and filtered and held the scum and colloidal matter of the Imhoff-tank effluent. Chlorine was applied between the hours of 10 P.M. and 8 A.M. After some experimentation the chlorine was applied at a rate of 11.2 p.p.m., with a residual in the applied sewage of 4 to 7 p.p.m. A total of 4500 lb. of chlorine was used for a filter 1.5 acres in area. With chlorine at 7.9 cents a lb., the cost was $237 an acre.

After a month's chlorination the chlorine had caused disintegration of the surface growth and cleared the filter so as to do away with ponding. The chlorine loosened and killed off the growth that had accumulated in the distribution system and after chlorination the sprays were even and vigorous.

To economize on chlorine, Wing and Williams make the following suggestions for the chlorination of trickling filters for cleaning purposes (5):

1. Do the chlorinating at night only, for then the sewage has its lowest biochemical oxygen demand[1] and is least in volume.
2. Add sufficient chlorine to ensure residual chlorine values of 3 to 5 p.p.m. in the sewage at the nozzles.
3. If possible, reduce the flow of sewage to the filter being chlorinated by diverting all except sufficient flow to insure against failure of the siphon to function properly. This will insure maximum benefits with the minimum quantity of chlorine consumed in the process.
4. Repeat each night until the beds are cleaned.
5. Following the initial treatment, succeeding treatments should not be nearly as costly and should be infrequent.

Nozzle Clogging.—Spraying nozzles are subject to clogging by materials which pass through preliminary treatment units and by fungi and other growths or slimes from the walls of dosing tanks and from inside distribution pipes. At Fitchburg, Mass., Allen (8) reports:

> The most troublesome cause of nozzle clogging is the great number of matches found, while balls of grease, bits of growth, small pieces of wood, fish and even chewing gum have been found in the nozzles.

During 14 years of operation of the Fitchburg filters 20.3 nozzles were cleaned each day, on an average, or 5.28 per cent of the average number in use. At Worcester, Mass., during 1928, the average daily number of nozzles cleaned was 10.2 per acre, or 140 for the entire area. This is equivalent to 4.7 per cent of the total number of nozzles. At Schenectady, N. Y., in 1930, the average number of nozzles cleaned daily on an area of 3 acres was 116, equivalent to 7.7 per cent of the total number of nozzles, or 14 nozzles per million gallon of sewage treated.

The better and more complete the preliminary treatment, the less clogging of nozzles occurs. The screening of tank effluent prior to discharge on to the filters has proved advantageous at some plants. The occasional flushing out of the distribution system and cleaning of the dosing tanks are also helpful. The cleaning of the distribution system may be facilitated by the application of chlorine.

Odors from Filters.—The intensity of the odors produced by trickling filters varies with the concentration, composition and condition of the sewage and with the air temperature, humidity and wind movement. In general, if the sewage reaches the plant in a relatively fresh condition and if the filters are well operated, the odors should not be intense or especially offensive. Sewage odors are particularly noticeable on relatively calm, damp days.

It is generally desirable to keep trickling filters ¼ to ½ mile distant from residences. If septic sewage is to be treated, even greater isolation is desirable. Small filters are sometimes covered in order to

[1] There is a marked correlation between B.O.D. and chlorine demand.

reduce the travel of odors. Screening of the plant by stands of evergreen trees has been employed with indifferent success, while chlorination of the sewage prior to filtration has been markedly successful, but excessive destruction of the filter life by chlorine must be guarded against. The covering of trickling filters for the control of odors is described in Chap. XXII. The hygienic significance of sewage odors does not appear to be appreciable.

Investigations of the control of odors at the Imhoff-tank, trickling-filter plant in Macon, Mo., during 1928 are described by Johnson and Bosch (9). They find:

> As a control procedure to prevent the production of objectionable odors from the sprinkling filters, a maximum of 1.5 p.p.m. of volatile hydrogen sulfide in the Imhoff tank effluent should not be exceeded. Odor control by chlorination is effective and is the most economical known remedy of preventing odors at the Macon sewage-treatment plant. Good operation will reduce considerably the amount and cost of chlorine required.

At Macon a dosage of 3.94 to 10.9 p.p.m. of chlorine was required to reduce the intensity of the odors to an unobjectionable point.

Results of odor control by chlorination at Schenectady are summarized by Cohn (10) as follows:

> At Schenectady, original investigations carried out in 1925 indicated that the application of chlorine to tank effluent in the siphonic chambers at rates as low as 4 p.p.m. resulted in a marked change in the odor conditions of the sewage applied to the treated filter units. At rates of 6 to 8 p.p.m. a clean chlorosubstitution odor replaced the usual sewage odor from the beds. No effect was noted on the stone film. At rates above 10 p.p.m. distinct chlorine residuals were noted in sewage collected at the nozzles and the chlorine bleached out the black growth on the stone and on the nozzle spindles and domes. Within a few days, the film sloughed off the stone and passed into the beds, producing the phenomenon of unloading. The beds looked like new units. No pooling was noted and the beds gave off no odors even when allowed to dry out under the sun's rays. In a regular program of filter odor control, heavy chlorination to remove surface film could be practiced at very infrequent intervals, and applications of low chlorine dosages for deodorization of the applied liquid could be used only during seasons of the year or periods of the day when conditions of tank effluent or atmosphere indicated that odor control was necessary.

Filter Flies.—A small, gray moth fly, *Psychoda alternata*, is sometimes troublesome about trickling filters, although it does not occur on sand filters or contact beds. These flies do not bite but may get into the eyes, nostrils and ears. Being so small that some pass through ordinary window screening, they can be excluded only with difficulty from dwellings or other buildings in the immediate vicinity of the plant. The radius of their flight is generally limited to a few hundred feet, but they may be carried by the wind to greater distances, perhaps ½ to ¾ mile. The eggs of the *Psychoda* are laid in the filter and the larvae feed upon the filter slime. Fair (11) states that the life cycle varies from 7 days at a temperature of 85°F. to 22 days at 60°F.

Two methods of checking filter flies which have been used are flooding the beds for about a day once a week, during the periods of greatest prevalence, and chlorination. The latter is carried out either by applying chlorine to the sewage in a heavy dose or by spraying with ortho-dichlorobenzene. Cultivation of the water springtail, *Achorutes viaticus*, also is said to be effective in reducing fly prevalence. This may be due to destruction by the springtail of the organic slime upon which the larvae of the filter fly feed.

Cohn (6) states that at Schenectady during 1927

. *Psychoda* inhabited the beds in small numbers throughout the winter and increased in number early in the spring. Chlorination temporarily controlled their development during the early spring months. The beds were flooded for 24 hr. on eight occasions from May to October and the flies were thus kept under satisfactory control. . . . The secret of effective fly control seems to be the application of control measures at very frequent intervals, but flooding more often than twice a month reduces filter efficiency. The Schenectady effluent suffered marked reductions in stability that lasted for 7 days after bed submergence.

Hommon (12) reports flooding to be quite effective in the control of flies. Referring to experience with the flies at Canton, Ohio, he states: "The sloughing (following flooding) caused the removal of the majority of the fly larvae and practically all of the flies were drowned."

Relative to chemical methods of *Psychoda* control, Fair (11) makes the following statement:

Experience with chemical control agents seems to have established the value of kerosene and ortho-dichlorobenzene, as well as mixtures of these substances or of kerosene and pyrethrum, for the destruction of adult flies. Substantial destruction of fly larvae appears to have been accomplished by chlorination. The use of creosote is favorably reported, as is high acidity of the applied sewage.

The effect of housing trickling-filter beds in glass-overs upon the control of flies is described in Chap. XXII.

Efficiency of Trickling Filters.—The degree of purification effected by trickling filters is influenced by many factors of design and operation. In general, the reduction in B.O.D. is 60 to 85 per cent and in bacteria 70 to 85 per cent.

Trickling filters are capable of converting putrescible settled sewage into highly stable effluent with low B.O.D. During the course of a

TABLE 81.—AVERAGE ANALYSES OF INFLUENTS AND EFFLUENTS OF TRICKLING FILTERS, AS DETERMINED BY U. S. PUBLIC HEALTH SERVICE IN 1920

Results Are Stated in P.P.M., Except as Noted

City	Suspended matter			Settleable solids, 2 hr., cc. per liter		Oxygen consumed, 30 min. in boiling water			Alkalinity as $CaCO_3$ by methyl orange			5-day biochemical oxygen demand			Nitrate nitrogen in effluent
	Influent	Effluent	Per cent removal	Influent	Effluent	Influent	Effluent	Per cent removal	Influent	Effluent	Per cent removal	Influent	Effluent	Per cent removal	
Atlanta, Ga., Intrenchment plant	67	42	34	0.6	Tr.[1]	21	10.0	52	82	48.0	41	23	4.2	82	3.0
Peachtree plant	68	38	44	0.2	Tr.	18	7.0	61	69	35.0	49	20	3.9	81	2.0
Baltimore, Md.	107	46	57	0.6	Tr.	36	14.0	61	144	63.0	56	125	14.0	89	3.3
Columbus, Ohio	79	81	−2	Tr.	Tr.	46	28.0	39	219	192.0	12	134	49.0	70	0.6
Fitchburg, Mass.	63	52	17	Tr.	Tr.	37	19.0	57	99	7.9	92	95	20.0	79	6.1
Lexington, Ky.	67	40	40	Tr.	Tr.	22	11.0	50	194	152.0	22	87	16.0[2]	70	
Reading, Pa.	85	48	44	Tr.	Tr.	31	16.0	48	177	105.0	41	96	20.0	79	4.4
Rochester, N. Y., Brighton plant	38	26	32	Tr.	Tr.	15	9.7	35	189	131.0	31	36	7.0	80	2.8
Average	72	47	34	....	....	28	14.0	50	147	92.0	39	77	17.0	78	

[1] Tr. indicates "trace," less than 0.1 cc.
[2] Effluent of secondary sedimentation tanks.

TABLE 82.—ANNUAL AVERAGE OPERATING RESULTS OF TRICKLING FILTERS
Results Are Stated in P.P.M., Except as Noted

	Brockton, Mass. 1928		Fitchburg, Mass., 1928				Schenectady, N. Y. 1928		Worcester, Mass. 1928	
			Dec. to May		June to Nov.					
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
Nitrogen										
Ammonia	27.4	15.6	11.12	3.34	11.97	2.00	12.7	5.1	14.6	9.6
Albuminoid:										
Total	6.67	5.44	2.63	1.50	2.54	1.40	3.43	2.49		
Dissolved	5.60	3.96	1.72	0.98	1.70	0.83	2.03	0.96		
Suspended	1.07	1.48	0.91	0.52	0.84	0.57	1.40	1.53		
Nitrite	0.25	0.45	0.084	0.219	0.028	0.111	....	0.484	....	0.173
Nitrate	0.15	10.15	0.962	6.236	0.437	9.287	....	4.742	....	5.71
Oxygen consumed:										
Total	95.4	40.7	88.7	58.8	80.2	50.5				
Dissolved	70.5	26.3	53.9	31.3	45.4	25.4				
Suspended	24.9	14.4	34.8	27.4	34.8	25.1				
Solids:										
Total	494	418	287	259	320	300	418	414	569	548
Volatile	237	161	123	113	147	141	....	....	173	145
Dissolved	404	358	239	227	276	266	369	352	465	448
Volatile	169	116	102	97	122	123	....	....	112	107
Suspended	90	60	48	32	44	34	49	62	104	100
Volatile	68	45	21	16	25	18	....	....	61	38
Settling, 1 hr., cc. per liter	....	....	....	....	....	....	....	....	0.77	2.82
Chlorine	77.8	80.2	42.2	41.8	46.0	46.0	38.6	37.2	87.0	87.6
Biochemical oxygen demand	....	....	....	....	....	....	107	35	98.8	20.4
Dissolved oxygen, per cent saturation	6.8	53.8	....	....	....	....	13.8	47.3		
Hydrogen ion concentration, pH	....	....	....	....	....	....	7.3	7.4		

normal year's operation the suspended solids in the effluent are about equal in quantity to those in the influent, provided that unloading is complete and there is no clogging. However, the suspended solids in the influent are fine and do not settle readily, while those in the effluent are coarse and flocculent and can be removed in large measure by sedimentation. The volume of suspended matter in the effluent is particularly great during the spring unloading period. At this time masses of sludge worms differing greatly in variety are mixed with the solids.

The results of studies of filter efficiency conducted by the U. S. Public Health Service in 1920 are shown in Table 81 (2). Operating data of typical American plants, taken from annual reports of operation, are given in Table 82. The efficiency of certain Ohio trickling-filter installations, as measured by their removal of B.O.D., is shown in Table 83, compiled from data furnished by Hatch (13).

TABLE 83.—LOADING AND EFFICIENCY OF OHIO TRICKLING FILTERS, AS MEASURED BY B.O.D. APPLIED TO FILTERS AND REMOVED BY THEM, MAY TO OCTOBER, 1930

Location	5-day B.O.D. at 20°C., p.p.m.		B.O.D. applied daily to filter		B.O.D. removed daily by filter		Efficiency of filter, per cent
	Settled sewage	Filter effluent	Lb. per acre-ft.	Lb. per capita	Lb. per acre-ft.	Lb. per capita	
Akron	91.7	17.4	122.6	0.0858	99.3	0.0695	81.0
Alliance	146	35.5	188.7	0.1187	142.8	0.0900	75.7
Canton[1]	100.5	25.2	171.1	0.0808	128.2	0.0605	74.9
Cleveland, Southerly plant	110.8	31.6	126.8	0.0952	90.7	0.0680	71.5
Columbus	153.8	64	757.5	0.1262	441.8	0.0736	58.4
Delaware	269	31.4	187.3	0.0916	165.5	0.0809	88.3
Fostoria	124.8	29.7	248.8	0.1176	189.5	0.0897	76.2
Marion	189.2	31.7	157.8	0.0921	131.5	0.0767	83.2

[1] May to October, 1929.

Relative Merits of Trickling Filters and Contact Beds.—Trickling filters have several advantages in comparison with contact beds. They permit much higher rates of operation and consequently require less area and involve less expense for construction than contact beds. Trickling filters usually give much less trouble from clogging and on the average produce a more satisfactory effluent than do contact beds.

The latter, on the other hand, require less head to operate, are practically free from flies, which sometimes infest trickling filters, and present less opportunity for the dissemination of odors. In the case of contact beds, it is not necessary to expose the sewage to view. For municipalities where isolated sites are difficult or expensive to obtain, or for residential, institutional or industrial plants, contact beds may be advantageous.

There have been few recent installations of contact beds by municipalities, the total number of such installations having decreased rather than increased during recent years, as contact beds in the older treatment works are superseded by the construction of trickling filters or activated-sludge plants.

Bibliography

1. Buswell, A. M.: "Chemistry of Water and Sewage Treatment," 1928.
2. *Pub. Health Bull.* 132, U. S. Pub. Health Service, 1923.
3. Ann. Rept., Mass. State Board of Health, 1908.
4. Rept. of Sewage Substation, N. J. Agricultural Experiment Sta., 1926.
5. Wing, F. K., and R. C. Williams: *Eng. News-Rec.*, 1929; **102**, 621.
6. *Public Works*, 1928; **59**, 223.
7. Smith, E. E.: *Sewage Works Jour.*, 1930; **2**, 387.
8. Ann. Rept., Dept. of Pub. Works, Fitchburg, Mass., 1918.
9. Johnson, W. S., and Herbert Bosch: *Public Works*, 1929; **60**, 400.
10. Cohn, M. M.: *Sewage Works Jour.*, 1929; **1**, 568.
11. Fair, G. M.: *Sewage Works Jour.*, 1934; **6**, 966.
12. Hommon, C. C.: "Rept. of Investigation of Crushed Blast Furnace Slag as a Medium in Sewage Trickling Filters," 1930.
13. Hatch, B. F.: Ann. Rept., Ohio Conference on Sewage Treatment, 1931.

CHAPTER XXII

DESIGN OF TRICKLING FILTERS

Testing and Selection of Filtering Materials.—Many different materials have been used as filtering media in trickling filters, including the following: crushed stone, pebbles, blast-furnace slag, cinders, coke, coal, broken brick, waste from potteries, wooden laths, brushwood and corncobs.

In the United States crushed stone seems to be preferred to other materials. Crushed stone does not disintegrate so rapidly as do some other materials under the influence of sun and air, freezing and thawing, and wetting and drying. It is not broken down by the weight of overlying material and is not subject to organic decay. Being heavier than water, it does not rise and settle, as may coke, when the filters are flooded for fly control. Trap rock, granitic rock and limestone are commonly employed. Of these, trap rock appears to be a particularly satisfactory material. While preferences are well established in favor of hard rock, it may be more economical in some cases to use cheap materials like cinders, even though they must be renewed occasionally, than to build of more permanent but considerably more costly material.

A committee of the Sanitary Engineering Division of the American Society of Civil Engineers is working on the subject of filter media. The following condensed discussion is taken from a progress report of the committee issued in 1929 (1):

The essential considerations in determining the suitability of trickling-filter media may be classified as disintegration, cementation and consolidation, and ability to unload.

Since the size of filtering medium largely affects the capacity and efficiency of trickling filters, the selected size should be substantially maintained throughout the life of the filter. The exposed surface of filters is subjected to natural weathering through the atmospheric agents of rain, frost, changes of temperature and wind and also to frequent alternate wettings and dryings by the intermittent discharge of sewage on to the beds. Sewage may contain appreciable quantities of dissolved carbon dioxide. When the atmospheric temperature is below freezing and the period between sprayings is ½ hr. or more, the wetted surface freezes between doses. Thus the material in a trickling filter is subjected to all the effects of natural weathering in greatly concentrated form, particularly as regards the mechanical effect of hydration and frost action. Furthermore, any chemical changes, such as oxidation of iron or slacking of free lime, are certain to be noticeable in much less time than under natural conditions. The splitting of material into sizable pieces along seams is not of such serious consequence as the crumbling away of the medium. This latter form of disintegration seems to be confined mostly to the softer varieties of limestones and to the slags. As the particles crumble away to granular bits, consolidation and clogging become more and more likely.

In some instances among the older trickling-filter installations, especially where clinkers and other relatively friable materials had been used, it was found that these materials were reduced to fines, which were cemented together, compacted or consolidated so completely as to render the filter inoperative.

Little is known as to the effect of roughness of material on the unloading ability of the filter and the resultant effect on the overall filter efficiency.

The following tentative conclusions have been reached by the committee:

a. Material for filter media should be sound, hard pieces, with all three dimensions as nearly the same as possible, clean and free from dust, screenings, and other fine material. It should be of as nearly uniform size as possible and the material should not disintegrate under service conditions either by breaking into smaller pieces or by crumbling into fine material.

b. In selecting the most suitable material for a given installation from the available sources of material, careful investigation of the several sources of material should be made, including geological inspections of quarries, a study of the facilities for obtaining, screening, and cleaning the desired size of material, and a laboratory analysis of samples selected from the several sources, in order to determine the comparative merits of the several materials as to the probability of the breaking-down of the materials under conditions of filter service. For the laboratory analysis samples should be selected from the various ledges in the several quarries or from the several possible storage piles.

c. The investigation of the Committee to date indicates that the sodium sulfate soundness test[1] is the most informative test for predetermining the disintegrating qualities of the proposed material. Other physical tests, including the determination of specific gravity, water absorption and porosity, give useful information. Chemical analyses are of value in determining the effect of impurities, such as earthy materials, iron, sulfur and, in some cases, lime and magnesium. Microstructural analyses made by qualified examiners may also be useful.

d. Great care should be taken in obtaining the filter material from a selected source to insure a uniform grade and size, free from dirt and fine matter. [In its progress report for 1934, the Committee states that a tolerance of not more than 5 per cent of material less than the smallest specified size has been suggested.] There is evidence to indicate that

[1] A description of this test is contained in the 1929 progress report of the committee (1).

inadequate screening for size and insufficient cleaning of filter material for many of the existing trickling-filter installations have been of influence in the deterioration of the filter beds, perhaps more so than the breaking down or disintegration of the individual pieces of filter material. It appears that proper sizing and cleaning of filter material cannot be obtained without special arrangements for screening, rescreening and perhaps washing of the material, either at the source or at the filter plant immediately before placing the material in the filter bed.

From tests on the various filter media collected from 12 trickling-filter plants the following general conclusions were indicated: Limestones with low specific gravities, say less than 2.50, must be regarded with suspicion, since, when this occurs, there is likely to be an undesirable quantity of earthy material or sandstone and the low-specific-gravity limestones show a decided tendency toward surface abrasion. Finely crystalline, uniform, low-silica limestones, having a specific gravity of 2.67 or more and an absorption of less than 1.10 per cent by weight may be expected to resist disintegration. On the other hand, if the specific gravity is less than 2.67 or the absorption is greater than 1.10 per cent, there is no certainty that the material will break down.

The tests indicated a relatively low abrasion loss for the dense limestones, trap, granite and hard quartzite. However, it would appear that some materials with a low abrasion loss would prove only partly satisfactory, particularly from a disintegration standpoint, whereas others might be regarded as satisfactory where the abrasion loss is high. This seems to be especially true of slags.

In general, 20 cycles of the sodium sulfate treatment are sufficient to establish whether or not a filtering material will break down in actual service.

The preceding study was confined to crushed stone and slag. Of the other materials listed, coke and cinders are undesirable, coal is usually impracticable, while brush,[1] broken brick and waste from potteries are seldom available in the quantities required and little is known of their suitability. Laths have been used only in small installations.

Hommon (2), after an inspection of 48 filter plants, of which 37 employ slag as a medium, and as a result of laboratory tests, concludes that crushed blast-furnace slag is a desirable material for use in trickling filters and that the rough texture of the slag does not prevent the periodic unloading of the filters. He finds no evidence to indicate that, when used in the sizes now commonly specified, it will cement or consolidate.

In trickling filters where the surface layers are particularly exposed to such weathering as occurs in the northern United States, a less resist-

[1] The use of brushwood at Toronto, Ont., which was abandoned a few years ago, was described by Phelps in the *Can. Eng.*, February, 1917, and was developed as a result of experiments with a small lath filter by Nasmith.

ant but locally cheaper medium, such as limestone, may be employed for the lower layers, while materials such as trap rock may be used for the upper ones. Such construction has been used at Lexington, Ky., at the Southerly plant in Cleveland, Ohio, and elsewhere. At Delaware, Ohio, Gascoigne (3) has provided 7 ft. of broken limestone, 1¼ to 2¾ in. in size, overlaid with 1 ft. of crushed granite.

Stanley (4) states that

> . . . an incomplete survey of trickling filters in the United States made in 1925 indicated that 83 per cent of the filter media for the plants covered was crushed rock of one type or another, 2 per cent gravel, 11 per cent slag and most of the remaining 4 per cent cinders or coke.

Size of Filtering Material.—The greater the surface area of the filtering material the greater the opportunity for contact between the sewage and the bacterial jelly covering the filter medium. The extent of surface varies directly as the roughness and inversely as the size of the ballast. Surface contour, however, is rapidly suppressed by the growths which cover the medium. Roughness is helpful in holding the gelatinous film but tends to retard the unloading of trickling filters and may induce clogging. Extremely smooth materials such as water-worn pebbles, on the other hand, afford insufficient hold for the zoogloeal growths. Size affects the opportunity for oxidation also by controlling, together with depth, the time of contact.

The time of contact or passage through filters of different depths and sizes of medium, which was observed at the Lawrence experiment

TABLE 84.—TIME OF PASSAGE OF SEWAGE THROUGH EXPERIMENTAL TRICKLING FILTERS, LAWRENCE, MASS.

Material	Size of material, in.	Depth		Ave. daily rate[1]		Time of passage of sewage, hr.	Nitrates, p.p.m.
				Gal. per acre	Gal. per cu. yd.		
		Ft.	In.				
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.

station, is given in Table 84, but this time relates only to the conditions of those particular experiments (5). Dunbar (6) found that if a quantity of sewage equal to the water-retaining capacity[1] of a filter 3 ft. deep is applied to such a bed, 20 to 30 min. elapse before that sewage is discharged from the bottom.

Numerous experiments with trickling filters of different sizes of coal, gravel and clinker from refuse destructors were carried out by Clifford (7), who concluded 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 quantity of water taking part in the general water movement through the bed, the quantity of water in motion being generally represented by the interstitial water.

American practice shows a tendency to use stone ranging from 1 to 3 in. in size for trickling filters. It is common practice to use as small material as is consistent with abundant air supply, avoidance of surface clogging 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, it may be desirable to rescreen the stone before placing it in the filter.

According to Winans (8), for an extension to the filters at Fort Worth, Tex., dust was blown out of the rock at the crusher plant by using air under pressure from a blower. The material was then delivered to the filter in cars, which were run right out over the bed and unloaded. At Wichita Falls, Tex., as outlined by Helland (8), the stone "was placed by shovel from tracks running over the stone. Immediately after being placed, the fire hose was turned on it and it was washed that way. Since that time, as far as I know, there has been very little fines coming out of the plant."

The character of the applied sewage affects the selection of the size of filtering medium. The more thoroughly the suspended matter is removed by preliminary treatment, the finer may be the filtering medium which it is safe to choose. Generally sewage is fairly well settled before it is applied to trickling filters, in order to reduce to a minimum the danger of clogging and to avoid placing upon the filter a burden of work which can be performed by less expensive structures like tanks.

The stone in the bottom 5 ft. of filters at Aurora, Ill., ranges from 1½ to 2 in. in size and that in the top foot from 1 to 1½ in. The

[1] The water-retaining capacity of a soil is defined as the volume of water which remains in the soil after drying the soil, filling it with water and allowing the excess water to drain away.

filters at Madison, Wis., are filled with crushed dolomite limestone, 10 ft. deep. At the bottom is a 12-in. layer of 3- to 5-in. stone, then 7 ft. of 1½- to 3-in. stone and on top 2 ft. of ½- to 1-in. stone.

The sizes of filter medium selected for various trickling filters in this country are given in Table 92.

Choice of Depth of Bed.—Among the factors to be considered in the choice of trickling-filter depth are the effect of bed depth on efficiency, unit loading, surface clogging, ventilation and distribution of sewage; the head available; the economy of construction; the economy of operation; and local topography.

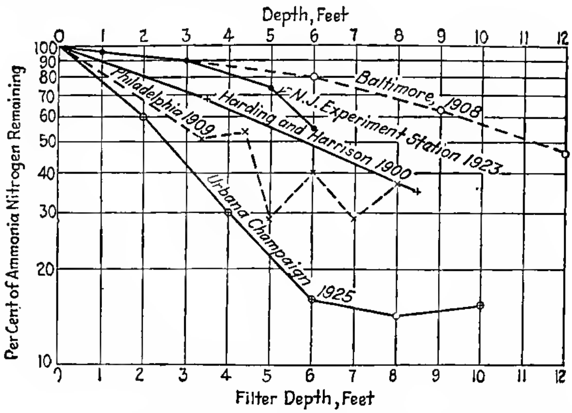


FIG. 122.—Effect of trickling-filter depth on ammonia reduction.

Opinions differ as to the effect of depth upon permissible filter loading. There is shown in Fig. 122 the ammonia removal at different depths recorded by various observers for trickling filters receiving the same quantity of sewage per square foot of surface area. It appears from this plotting that the percentage ammonia reduction per foot of depth is practically constant; the quantity of ammonia removed per foot of depth naturally decreases, however, as the quantity remaining diminishes. The lower depths of the filter are therefore required to do less work than the upper ones. Studies of nitrification effected yield similar information. Filtration is an oxidation process and the effectiveness of the lower filter strata depends to some degree upon the efficiency of ventilation.

Offsetting to a considerable extent these possible disadvantages of greater depth is the fact that the time of contact seems to increase geometrically with depth and the quantity of sewage which can be treated per unit area of bed increases at a greater rate than the increase in depth. Thus analysis of Clark's observations at Lawrence shows that, for a given dosage of sewage per acre, the time of contact is practically doubled for every increase in depth of 2 ft. and that a filter 10 ft. deep has the same period of contact when receiving 2.5 mil. gal. an acre daily as a 4-ft. bed dosed at one fifth this rate.

Tests on trickling filters have been conducted for many years at the Lawrence experiment station. Results of operation during 1928 of eight filters constructed of similar materials, ranging from 4 to 10 ft. in depth and operated so as to secure a relatively uniform degree of purification, are given in Table 85 (9).

TABLE 85.—OPERATING DATA ON EXPERIMENTAL TRICKLING FILTERS OF VARIOUS DEPTHS, WITH RELATIVELY UNIFORM DEGREE OF PURIFICATION

Filter No.	Depth of bed, ft.	Quantity of sewage applied, gal. per acre daily	Gal. per acre-ft. daily	Ammonia		Kjeldahl nitrogen, p.p.m.	Nitrate nitrogen, p.p.m.	Oxygen consumed, p.p.m.	Bacteria per cc.
				Free, p.p.m.	Albuminoid, p.p.m.				
452	4	787,000	196,000	20.5	4.1	7.5	12.6	26.7	500,000
453	6	984,000	164,000	26.1	4.1	7.1	12.7	25.9	400,000
454	8	1,521,000	190,000	14.5	3.7	6.8	18.3	21.3	200,000
455	10	3,432,000	343,200	19.8	3.5	6.8	14.4	22.9	140,000
473	6	489,000	81,500	22.3	3.7	6.3	7.5	23.3	600,000
474	8	1,465,000	183,000	23.5	3.5	6.1	4.3	22.1	310,000
475	10	3,432,000	343,200	16.6	3.7	7.1	13.4	24.2	420,000

Filters Nos. 452, 453, 454, and 455 were put in operation in May, 1914, and were constructed of stone that would pass a 1½-in. screen and be retained by a ¾-in. screen; filters Nos. 473, 474, and 475, constructed of a coarser broken stone, all of which would pass a 3-in. screen and be retained by a 1½-in. screen, were placed in operation in April, 1915. Analyses of the settled sewage treated show an average of 37 p.p.m. of free ammonia, 6 p.p.m. of total albuminoid ammonia and 35.8 p.p.m. of oxygen consumed.

With reference to the tests Clark (9) states

A study of this table will make clear the fact, as has been stated frequently in these reports, that the deeper filters will receive sewage at a much greater rate than would be expected by a comparison of their depths. For instance, Filter No. 455, 10 ft. in depth, was operated during the year at a rate somewhat more than four times as great as that of Filter No. 452 constructed of the same material and 4 ft. in depth and with slightly better purification results not only in the removal of organic matter but of bacteria.

Filter No. 475, 10 ft. deep, was operated at a rate about seven times as great as that of Filter No. 473, 6 ft. deep, and effected better purification.

Unfortunately most, if not all, of the other tests which have been used as a basis of judgment upon this subject were not performed in such a way as to secure such a uniform degree of purification and the value of the tests in indicating volumetric efficiency is subject to doubt.

During the year 1920, filters 4, 6, 8 and 10 ft. deep were operated at Lawrence at about the same rate per foot of depth, or approximately 170,000 gal. per acre-foot daily. The results of operation are given in Table 86 (10).

Table 86.—Average Results of Operation of Experimental Trickling Filters of Various Depths, Dosed at a Uniform Rate per Foot of Depth

Filter No.	Depth of bed, ft.	Quantity of sewage applied, gal. per acre daily	Ammonia		Kjeldahl nitrogen, p.p.m.	Nitrate nitrogen, p.p.m.	Oxygen consumed, p.p.m.	Bacteria per cc.	Stable samples, per cent
			Free, p.p.m.	Albuminoid, p.p.m.					
452	4	686,000	37.1	7.3	13.3	12.6	40.3	1,323,000	85
453	6	967,000	32.1	5.8	10.4	21.0	32.4	637,000	90
454	8	1,357,000	30.0	5.8	9.9	18.3	34.3	532,000	95
455	10	1,751,000	27.7	5.2	10.0	20.5	33.8	553,000	100

These results indicate that the degree of purification obtained, when the filters were loaded in proportion to their depth at a rate of about 170,000 gal. per acre-foot daily, increased somewhat with the depth, particularly as shown by the relative stability of the effluents.

Studies made by Buswell and Strickhouser (11) on samples of filter effluent taken at intervals of 2 ft. in depth from a trickling filter at Urbana-Champaign, Ill., led them to conclude as follows:

> The data secured indicate that the quality of the final effluent from the 10-ft. filter is no better than that from the 6-ft. level of the filter. The additional depth over 6 ft. does not cause a more satisfactory effluent to be produced at materially higher rates than could be expected from a 6-ft. filter. Moreover, clogging and ponding become limiting factors in the employment of high rates of filtration.
>
> The results point to the generalization that there is an optimum depth of filter beyond which it is neither necessary nor advisable to go in the installation of a plant. In the filter under investigation at Urbana the maximum amount of purification was brought about at a depth of 6 ft., and the additional depth did not appear to be advantageous, but it is undoubtedly true that this same depth might not be the optimum with different sewages and under other conditions.

The item of ventilation requires consideration with respect to the depth of filters, for if the oxygen in the air and in the sewage passing through the filter is used up before the bottom is reached, the lower part of the filter will be ineffective. The oxygen requirements are greater in treating unusually strong sewages or industrial wastes having a marked avidity for oxygen than when ordinary domestic sewage is treated. If the capacity of a filter is based upon a certain per capita volume, such as 2000 persons an acre-foot, with abnormally dilute sewage a deep filter requires the application of a relatively large volume of sewage per unit of area. This consideration may lead to the adoption of a filter of moderate depth.

Organic growths have appeared at times on some filters, more or less clogging the surface layer. This condition is governed largely by the character and size of the surface filtering medium and the efficiency of preliminary clarification. For deep beds, a relatively greater quantity of sewage may be applied per unit of superficial area, as well as a correspondingly greater load of sewage solids, than for shallow beds, so that there is opportunity for greater surface clogging and consequent obstruction of the circulation of air. However, the 10-ft. filters at Fitchburg, Mass., have been operated at rates exceeding 3.5 mil. gal. per acre daily without impairment of efficiency.

Uneven distribution may result in clogging overdosed areas relatively quickly 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, or flooded for control of *Psychoda* or for cleaning of the medium, a larger proportion of the filter may be out of use in the case of deep beds.

Since the head lost in flow through the filter is equal to the depth of the bed, the head available is an item to be considered. The saving of 2 to 4 ft. in depth of filter may in some instances make pumping unnecessary or, if pumping is required, it may effect a saving in pumping charges.

The cost of trickling filters per unit of volume decreases with the increase in depth, owing principally to the reduced cost of floor, underdrainage and distribution systems with deeper beds. Estimates made in 1914 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., \$3.96 with 8-ft. and \$3.60 with 10-ft. depth. If the quantity of sewage which can be purified satisfactorily by trickling filters is proportional to the depth, the deeper filters will be the more economical ones. If the quantity of sewage which can be purified satisfactorily is greater than proportional to the depth, as indicated by Clark's experiments, the relative economy of the deeper filters will be still further increased. If, however, as concluded by Buswell, there is comparatively little purification below the 6-ft. level, there is no economy and little justification in building filters deeper than 6 ft. If the capacity of a filter is proportional to its depth, local topography and limitations of available area may influence the choice of depth in favor of the deeper filter.

Trickling filters are commonly 5 to 10 ft. deep, 6 to 8 ft. being considered a satisfactory mean. Depths of several installations are shown in Table 92.

Trickling-filter Loading.—Opinions differ as to the permissible loading of trickling filters, despite the long years of successful operation of a large number of installations. There is no really satisfactory way of expressing the loading. The rate at which sewage may be filtered to yield an acceptable effluent apparently depends, among other things, upon the quantity and nature of the organic matter in the applied sewage, the size and character of the filtering material and the depth of the filter. It is not possible to combine into a single unit these factors which themselves are constituted of a number of variables. The most common methods of expressing filter loading are, therefore, limited to a statement of either the population served per acre or per acre-foot or the quantity of sewage treated per acre or per acre-foot. Loadings are sometimes expressed in terms of cubic yards of filtering material. Where abnormal quantities or strengths of industrial wastes are included in the flow treated, the industrial wastes may be converted to terms of equivalent population, in order to derive comparative filter loadings. Correct interpretation of filter loadings, however expressed, requires familiarity with the factors listed above.

Tests were run on the trickling filters at Fitchburg to determine their effective capacity. Allen (12) reports the results as follows:

> The operation of 59 per cent of the total area of the trickling filter from Nov. 30, 1918, to May 26, 1920, has shown that the filter is capable of treating the sewage of more than 30,000 persons per acre per day. . . The efficiency of that portion of the filter in use prior to May 27 was well maintained as shown by the chemical analysis and all samples of final effluent were perfectly stable for at least 14 days.

During the four months ending June 30, 1919, the average flow treated per acre was more than 3.5 mil. gal. daily without impairment of efficiency.

The capacity of trickling filters is somewhat influenced by the strength of sewage treated. A method of determining filter loading in terms of the organic constituents of the applied sewage is explained in Chap. XVIII.

The requirements of the British Ministry of Health regarding the dry-weather flow per acre-foot of trickling filter, composed of material which will not pass a 1-in. screen, are given in Table 87. Such filters

TABLE 87.—STIPULATION OF BRITISH MINISTRY OF HEALTH AS TO ALLOWABLE LOADING OF TRICKLING FILTERS TREATING DRY-WEATHER FLOW
U.S. gal. per acre-foot daily

Preliminary treatment	Character of sewage		
	Strong	Average	Weak
Grit chamber	29,000	48,500	77,500
Plain-sedimentation or septic tank	87,000	136,000	194,000
Chemical precipitation	126,000	194,000	290,000

TABLE 88.—LOADINGS PROVIDED FOR IN DESIGN OF TRICKLING FILTERS

Location	Area, acres	Average depth, ft.	Rate of filtration		Population of design	
			Mil. gal. per acre daily	Gal. per acre-ft. daily	Per acre	Per acre-ft.
Akron, Ohio	13.74	10.0	2.362	236,200	18,930	1893
Aurora, Ill	3.6	6.0	1.8	300,000	20,800	3933
Baltimore, Md	30	8.5	2.5	293,000	20,000	2350
Bloomington, Ill	2.5	8	1.44[1]	169,000[1]	21,600	2700
Canton, Ohio	6.8	7.0	.....	....	20,000	2857
Cleveland, Ohio[2]	6.08	10.0	.....	....	40,000	4000
Columbus, Ohio	10.0	5.33	3.41	640,000	20,000	3750
Decatur, Ill	3.0	5.7	2.66	467,000	20,000	3500
Delaware, Ohio	0.55	8.0	2.64	330,000	26,400	3300
Durham, N. C	1.0	7	.....	....	10,000	1429
Elgin, Ill	1.5	8	2.18[1]	263,000[1]	24,500	2950
Fitchburg, Mass	2.07	10.0	2.0	200,000	20,000	2000
Fort Worth, Tex.[3]	3.0	9	2.5	278,000	41,700	4640
Fort Worth, Tex.[4]	4.0	9	4.0	444,000	56,250	6250
Fostoria, Ohio	0.85	7	2.35	335,000	17,500	2500
Gloversville, N. Y	3.07	5.0	1.0	200,000	6,500	1300
Lexington, Ky	2.0	6.0	1.75	292,000	14,000	2330
Lincoln, Neb	2 6	6	1.73	288,000	38,400	6400
Madison, Wis.[5]	2.22	10.0	2.25	225,000	18,000	1800
Marion, Ohio	1.4	10.0	0.85	85,000	28,500	2850
Maynard, Mass	0.24	8.0	2 0	250,000	20,800	2600
Pontiac, Mich	1.74	6.0	2.32	370,000	29,800	4950
San Bernardino, Cal	2.5	7	.....	....	24,000	3429
Schenectady, N. Y	3.07	5.0	2.0	400,000	21,670	4334
Springfield, Mo	2.5	6	.....	....	19,000	3167
Urbana-Champaign, Ill	1 6	10	0.85[1]	85,000[1]	28,100	2810
Worcester, Mass	5.12	10.0	3.38	338,000	25,000	2500

[1] Loading during 1927.
[2] Southerly plant.
[3] Initial plant, 1924.
[4] With additions, 1929; 3½ hr. preliminary aeration with activated sludge.
[5] Nine Springs plant.

may be used for storm flows up to three times the dry-weather flow. In this connection, it should be remembered that weak British sewage is much stronger than American sewage.

The rate of dosing trickling filters can be greatly increased if preliminary partial treatment by aeration is provided. Partial aeration is being utilized at Birmingham, England, Decatur, Ill., and Lemoore, Cal. This method of treatment, discussed in Chap. XXIV, may be especially advantageous where extension of existing trickling filters is difficult, uneconomical or unsuitable.

At Decatur the strength of the sewage is greatly increased by the discharge of starch-works wastes into it. Studies at a testing station showed that, within reasonable limits, trickling filters with an average depth of 6 ft. could be operated at a relatively constant loading of 5-day biochemical oxygen demand amounting to 7500 lb. an acre daily (13).

During warm weather filter loadings might be appreciably increased, but generally the volume of sewage at such times is below the average for the year. The increased efficiency of the filter is more likely to be evidenced by the production of a better effluent, which is a favorable condition, because at higher temperatures the oxygen demand of streams increases and their flow at such times is usually relatively low.

The loadings provided for in the design of various filters are given in Table 88. It may be observed that there is a wide range in the loadings tabulated. The average rate of filtration provided for is about 2 m.g.d. per acre and 300,000 gal. daily per acre-foot. The population provided for averages about 23,000 persons per acre and 3200 persons per acre-foot.

The loadings of certain Ohio trickling filters, expressed in terms of B.O.D., have been given in Table 83.

Dosing Cycle.—Intermittency of operation is secured in trickling filters by the movement of rotating or traveling dosing mechanisms over the bed, by the intermittent discharge of sewage onto splash plates or, more generally, through nozzles, the head on the nozzles being varied during the dosing period, or, in the case of small units, by the movement of a tipping-tray mechanism, described in Chap. XXX. In general, the cycle is completed in 5 to 15 min., with approximately equal periods of rest and sewage application. Resting periods of too great length foster drying or freezing of the filter and excessive loading at times of dosing, entailing less satisfactory dispersion of the sewage through the bed. There seems to be a general tendency to reduce the period of rest to a minimum, when nozzles are employed which operate under varying heads, because this method of distribution ensures sufficient intermittency for different parts of the bed.

The average dosing and resting times for several trickling filters in Illinois are given in Table 89 (14).

TABLE 89.—DOSING CYCLES OF TRICKLING FILTERS IN ILLINOIS

Location	Area of filters, acres	Depth of filters, ft.	Number of units	Dosing time, min.	Resting time, min.
Bloomington-Normal	2.5	8.5	4	8.1	29.7
Chicago Heights	1.1	6.0	4	9.2	3.4
Decatur	3.0	6.0	2	3.4	6.8
Elgin	1.53	8.3	2	4.2	22.8
Johnston City	0.34	6.5	2	3.2	13.0
Urbana-Champaign	1.6	10.0	2	2.4	12.7

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 is governed by the extent of the clogging and the results obtained by actual operating tests.

The dosing of trickling filters is usually made automatic. Discussion of automatic dosing apparatus is deferred to Chap. XXIII, but dosing tanks, together with the hydraulics of nozzles and dosing tanks, are considered later in the present chapter.

Size, Shape and Number of Filters.—The area of filters may be controlled by limitations of depth, or depth may be controlled by limitations of area available. When adequate area is available at suitable elevation and the depth has been selected in conjunction with unit loading, the total area of the filter beds becomes a question of arithmetic. When the filters are dosed by fixed nozzles, the distribution system is designed to provide as many dosing units as desired.

If, for purposes of controlling the development of filter flies or cleaning the filters, as suggested by Hommon, it is desired to flood the filters from time to time, it may be advantageous to divide the filter into a number of units, so that one unit at a time may be flooded without unduly loading the units remaining in operation. This arrangement involves the construction of division walls and separate drainage systems for each unit.

At Columbus, Ohio, the 10 acres of trickling filters[1] are divided into four units, in the form of equilateral triangles with a common apex, as

[1] The trickling-filter installation at Columbus is now being replaced by a larger activated-sludge plant.

shown in Fig. 123. At Canton, Ohio, the trickling filter is composed of four units of 1.7 acres each, making a total of 6.8 acres. At some plants filter units are much larger, the unit area being 13.74 acres at Akron, Ohio, 6.08 acres at Cleveland, Ohio, and 5.12 acres at Worcester, Mass.

Filters dosed by rotating distributors are commonly circular. The size of such filters is limited by the economical and practical size of the

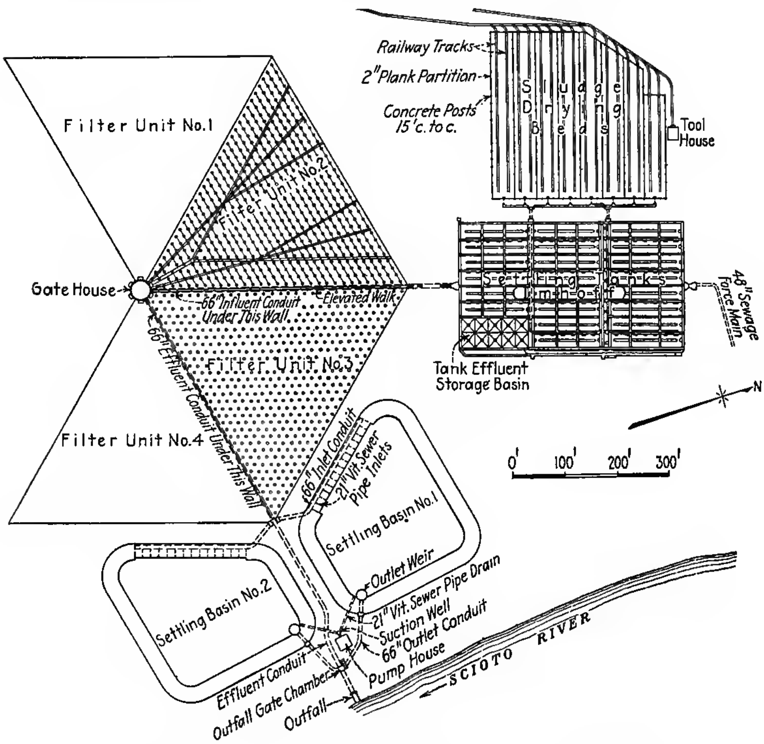


FIG. 123.—Arrangement of trickling filters[1] at Columbus, Ohio.

distributing device and is influenced by the fact that it is less expensive to build a few large beds than many small ones. Stationary nozzles may be employed in connection with rotating distributors, filling in the spaces between circles of influence. For filters using mechanical distributors more than one distribution unit is commonly 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 such units for each plant. There is a decided advantage

[1] The trickling-filter installation at Columbus is now being replaced by a larger activated-sludge plant.

in having small distribution units, for they permit the periodic resting of portions of the filter.

Distribution of Sewage.—The most desirable means of distributing sewage over the beds produces as nearly uniform loading of the filter as is obtainable, subject to considerations of economy. The higher the rate of dosing, the more important becomes uniformity of distribution. It follows that the distribution systems developed for trickling filters are more elaborate than those employed in the older filtration methods.

Several different means have been devised for securing moderately uniform distribution of sewage over trickling filters. Some of these are outlined in the following schedule:

1. Moving distributors
   a. Revolving
   b. Traveling the length of the bed
   c. Tipping trays
2. Stationary distributors
   a. Splash plates
   b. Spraying nozzles
   c. Filtering layer of fine material

Objections raised to the use of moving distributors for large installations are troubles with interruptions due to wear of the moving parts, deposition of solids in troughs feeding traveling distributors, freezing troubles in cold climates and clogging of small holes or openings discharging sewage over the bed.

Revolving and traveling distributors have been used quite extensively in Great Britain and its dominions and to some extent in the United States. They are considered in detail below.

Tipping trays are used only for small installations, notably in connection with lath filters, where they require triangular boards to assist in obtaining uniformity of distribution.

Splash plates commonly receive streams of sewage from openings in the bottoms of troughs or pipes. Each plate consists of a concave metal disk, placed above the bed surface, upon which the sewage falls and from which it rebounds upward and outward in the form of spray. The use of splash plates is open to the objection of clogging of the supply openings and freezing of the exposed distribution system. They have been used to only a limited extent.

Dunbar at Hamburg secured an even distribution of sewage by using small-sized ballast at the surface of the filter. Such beds are dosed by means of a central trough and sewage percolates into the filtering material. Dosing is continued until the surface of the bed clogs; the bed is then thrown out of service for rest and surface cleaning. This type of filter has found only limited application.

Spraying nozzles are employed almost exclusively for dosing trickling filters in the United States. They are considered in detail on page 510.

Revolving Distributors.—Revolving spray distributors for dosing circular filters commonly consist of four horizontal arms revolving about a central post. The distributors are made in sizes ranging from 15 to 225 ft. in diameter.

The largest installation in the United States is at the plant designed by Hubbell, Hartgering and Roth for Pontiac, Mich., where eight revolving distributors, 110 ft. in diameter, are installed. The population provided for in design was 60,000. The distributors, illustrated in Fig. 124, are driven by the available hydrostatic head of 2 ft. 8 in. between the water surface in the Imhoff tanks and the surface of the

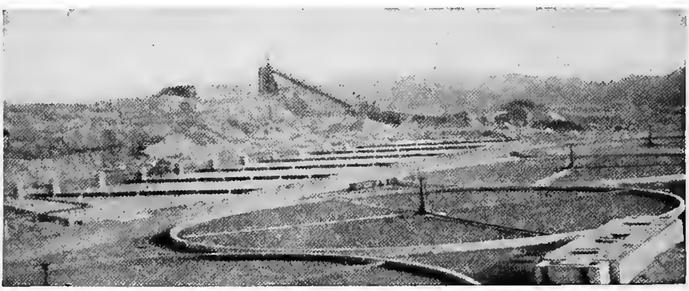


Fig. 124.—Revolving distribution on trickling filters at Pontiac, Mich.

filters. The manufacturers claim that the machines can be operated on 18 in. of head while discharging at a rate of 3 mil. gal. per acre daily. The revolving arms, four to each bed, are suspended from a central bearing. Each arm has a series of 3/8-in. openings spaced at varying intervals, from 12 in. near the center to 3 in. at the periphery of the filter. The normal peripheral speed of rotation is about an average walking gait. Operation of the Pontiac distributors is described by Shephard (15) as follows:

It seems to be the general opinion that filters of this type cannot be operated in climates similar to those of the northern United States. This has not been the case at Pontiac. These filters have operated through seven or eight Michigan winters, where zero temperatures are not at all unusual, and have given no trouble. A ring of ice will sometimes form around the outside wall. The operator readily overcomes this difficulty by opening shear gates at the ends of the arms for a few minutes. The increased flow melts the ice ring sufficiently to prevent trouble. This is necessary only under exceptional circumstances. In fact the net area of filter under ice is a materially less percentage with this type than with the fixed-nozzle type operating at Plymouth, about thirty miles away. In the eight years of operation of this plant the only repairs to the filter mechanism were the replacement of some bearings. Probably the entire repair bill did not exceed $250, including labor. In operation these distributors are relatively easy to keep clean. When the holes in the arms become stopped up a hose stream is found very effective in cleaning them. The entire eight units can be cleaned in about two hours.

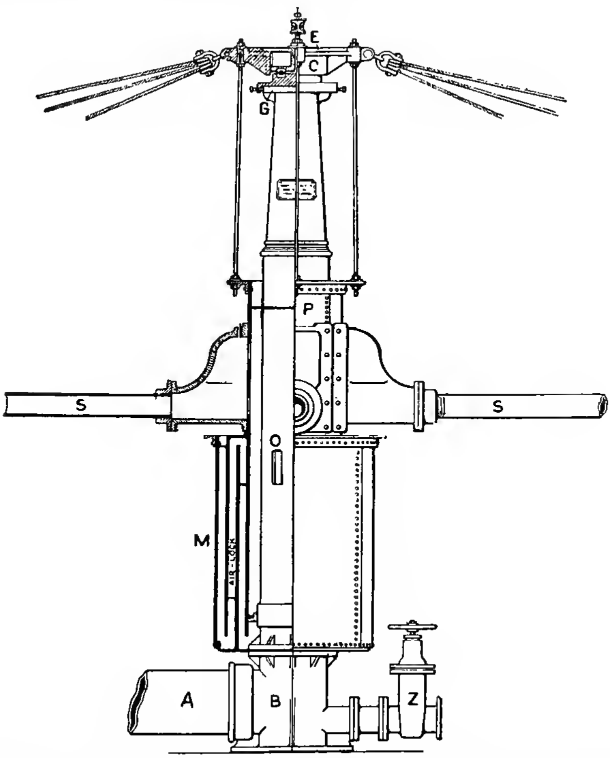


FIG. 125.—Standard for ball-bearing-supported distributor. (*Adams.*)

A. Supply pipe.
B. Duck-foot bend.
C. Adjustable head carrying ball bearings.
E. Lubricator.
G. Flange carrying bead.
M. Entrapped air.
O. Slots.
P. Revolving body.
S. Spray arms.
Z. Wash-out valve.

The weight of a revolving distributor may be carried either by ball bearings at the top of the central vertical standard, as in Fig. 125, or by a float, as in Fig. 126.

The cost of distributors 50 ft. in diameter is about $1500 per unit, while 100-ft. units cost about $2300 and 120-ft. units about $2600.

Normally filter beds are made circular when dosed by revolving distributors. Fixed spray nozzles are used at Darwen, England, for areas in a rectangular bed not covered by revolving distributors. Such nozzles operate at a low fixed head.

Traveling Distributors.—Although traveling distributors have been used to some extent in England, British possessions and Germany, they have seldom been installed in the United States. They usually take the form 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 type of machine. The liquid is commonly siphoned out of a channel running either along one side of the bed or

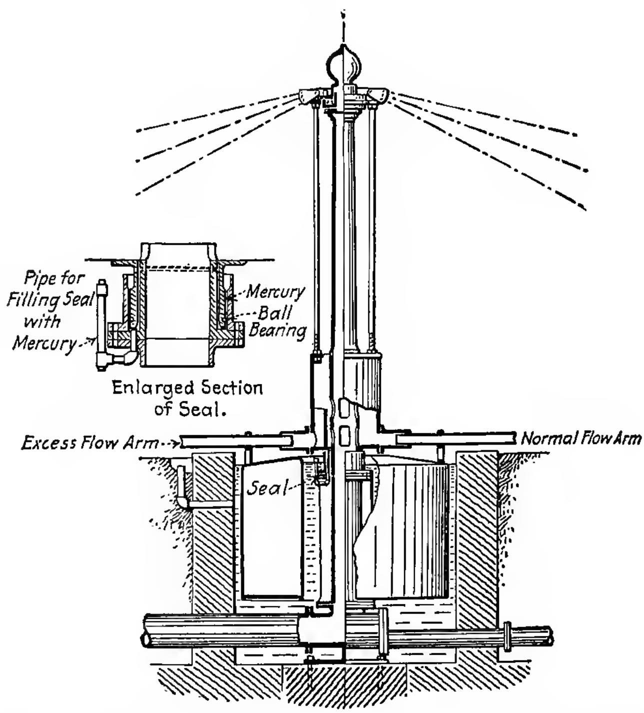


FIG. 126.—Standard for float-supported distributor. (*Candy-Whittaker.*)

between pairs of beds and passes through a feed tube running the whole length of the distributor. This tube has apertures through which the liquor drops into the buckets of a long water wheel. The feeding arrangements are reversed at either end of the bed by a buffer attached to the masonry and, as soon as reversal occurs, the apparatus begins its return trip. Another form of traveling distributor provides for continuous travel down half of the filter unit, around a semicircular end and back on the other half.

Traveling distributors, provided for trickling filters at Springfield, Mo., were placed in operation in 1913. The plant has a design capacity of

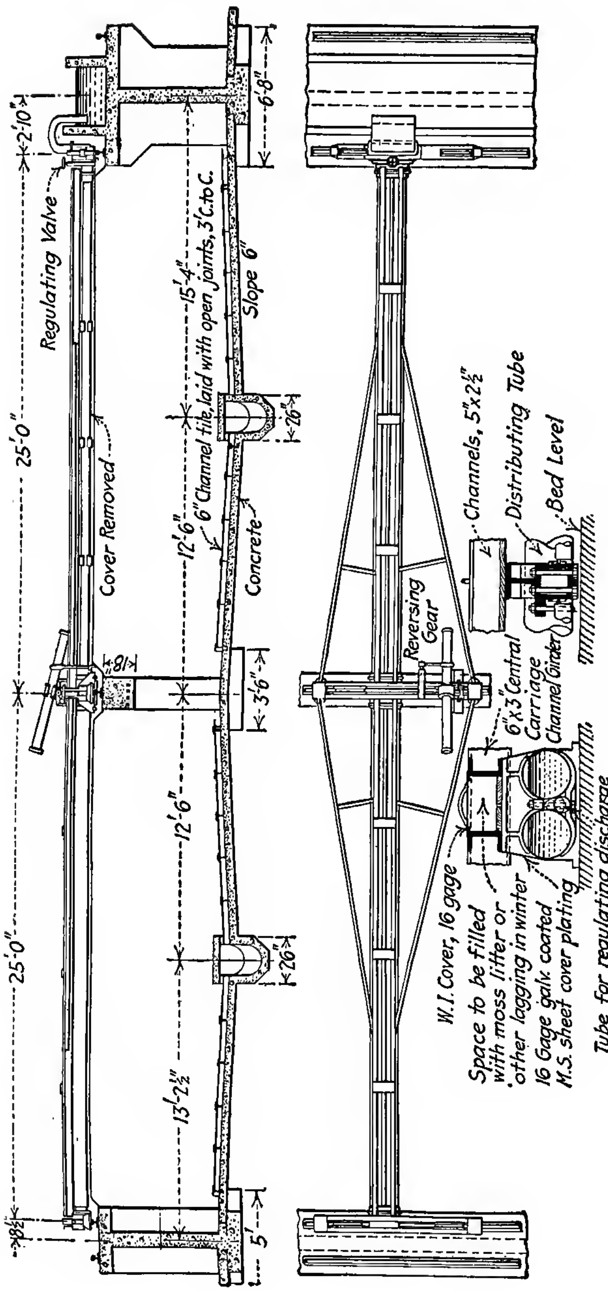
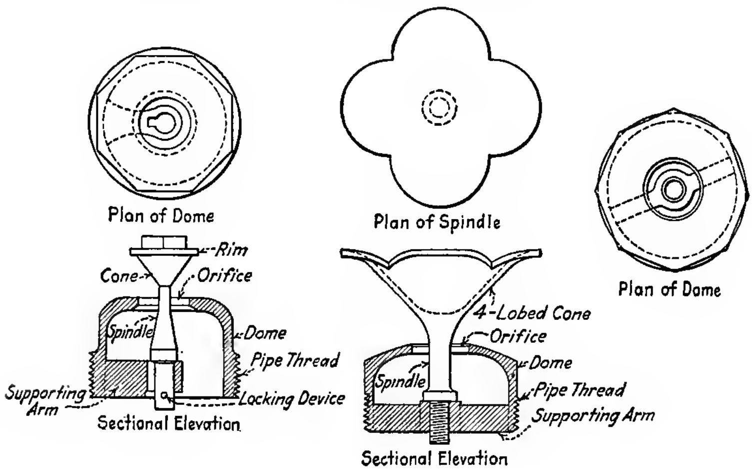


Fig. 127.—Traveling distributor at Springfield, Mo.

3,500,000 gal. daily. The filter beds comprise six units, each 53 ft. 9 in. wide and 200 ft. long. At designed capacity the rate of dosing is 2.35 mil. gal. per acre daily. The distributors are designed to distribute the sewage upon the beds with a loss of head not to exceed 12 in., when the liquid is applied at the maximum rate of 3.5 mil. gal. per acre daily. Six distributors are provided, each 50 ft. long and spanning one filter unit. Each distributor is supported on 3 rails spaced 25 ft. center to center. The length of travel is 200 ft. As first installed the distributors were driven by an endless wire cable with a speed of 38 ft. a minute. Because of the expense of maintenance of the driving equip-



(a) (b)

FIG. 128.—*a*, Worcester type of nozzle; *b*, Taylor square-spray nozzle.

ment and the difficulty of securing spare parts, the distributors were later equipped with direct motor drive. One of the distributors is illustrated in Fig. 127 (16).

Although the winters at Springfield are severe, Potter stated in 1916 that it never had been found necessary, because of freezing weather, even temporarily to abandon the use of mechanical distribution (17).

During the World War considerable difficulty was experienced with the distributing machinery, as it was almost impossible to get replacement parts from England, and as a result the plant became more or less inefficient.

Spraying Nozzles.—Spray nozzles for trickling filters are made in several patterns, some throwing a spray which covers an approximately square area and others dosing a circular area. A typical circular-spray nozzle is shown in Fig. 128*a*, 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 fine spray and spread it over a wide area. The spindle has a locking device permitting its ready removal for cleaning the orifice. A typical square-spray nozzle, designed by Taylor, is shown in Fig. 128*b*. It has an orifice 1 in. in diameter, through which a spindle passes, carrying a four-lobed spreading cone, which is intended 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

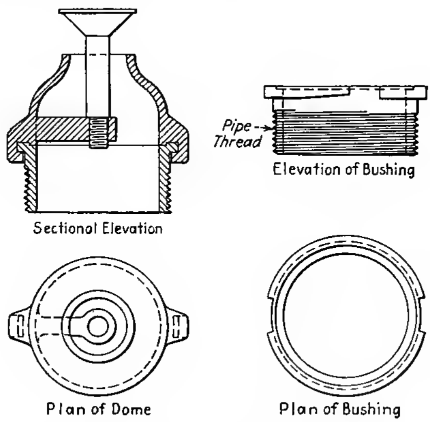


FIG. 129.—Akron type of nozzle.

depends upon rapidly varying the head of the applied liquid. Taylor nozzles also are made to spray hexagonal and circular areas.

A nozzle used at Akron, Ohio, which was designed by Allton and Backherms and patented by the latter, is shown in Fig. 129. It consists of a threaded bushing that is screwed into the distribution pipe, and a dome attached to this bushing by lugs, which bear against the underside of a notched collar on the bushing. A threaded spindle is screwed into an arm on the inside of the dome. The spindle-and-dome assembly is removable for cleaning by giving it only part of a turn before lifting out.

A type of nozzle recently developed by the Pacific Flush-Tank Co. provides for ease in removal for cleaning the orifice section and cone. As shown in Fig. 130*A*, by means of a special pair of pliers, a spring which engages three lugs on the base is slightly compressed, thus permitting the removal of the orifice section. A further advantage claimed for the nozzle is that, by removing the orifice section, inverting it and replacing it in the base, the cone will serve as a plug and stop off the nozzle. The size of the nozzle base is 1½ in., making possible its installation in smaller distribution piping than the 2-in. size heretofore commonly employed. The nozzle is manufactured in ¾-in. and ⅞-in. sizes.

Two other types of circular-spray nozzles furnished by the Pacific Flush-Tank Co. are shown in Fig. 130.

A flotation type of nozzle, shown in Fig. 131, has been developed by Nelson (18) for the distribution of sewage on trickling filters, from the

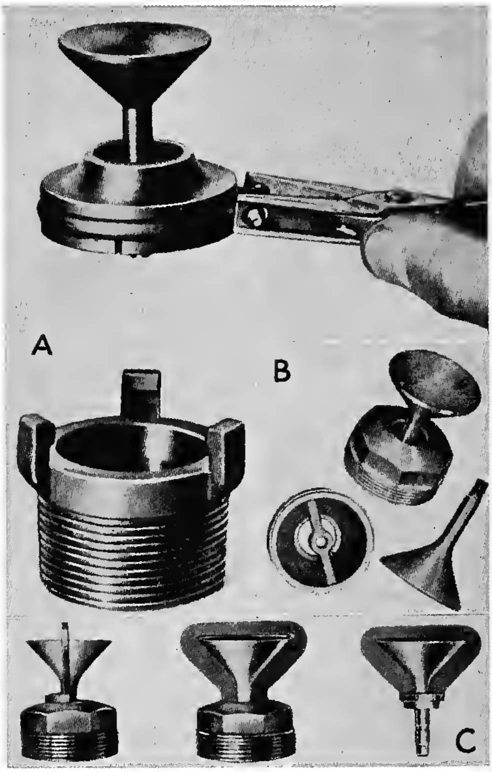


Fig. 130.—Circular-spray nozzles made by Pacific Flush-Tank Co. *A*, Type "D" circular-spray nozzle; *B*, type "A" circular-spray nozzle; *C*, type "C" half-spray nozzle.

Sacramento nozzle used in the aeration of water. An advantage claimed for this type of nozzle is that clogging is practically eliminated and, if it should occur, it is necessary only to lift the cone in order to remove the obstruction.

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 substantial item. If the orifice is so large that the sewage is delivered to the bed in a sheet, the streaming effect may prevent efficient oxidation. 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 depends 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. The hydraulics of spray nozzles will receive further attention in the following section and methods of automatic dosage control are outlined in Chap. XXIII.

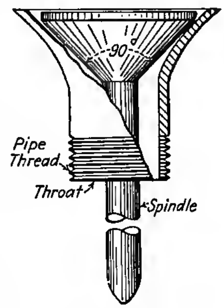


FIG. 131.—Flotation-type nozzle.

Mains for distributing sewage to fixed nozzles may be of concrete or vitrified pipe encased in concrete, but the smaller sizes are preferably of cast-iron pipe with lead or composition joints. The pipes are made large enough to avoid 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 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 then would have to be supported or temporarily removed. Facilities for flushing and draining the distribution system are generally provided.

In certain cases fractures have occurred in lateral distributing pipes owing to excessive pressures. If these pipes are laid at the bottom of the filter, repairs upon them entail excavation of filtering material, which adds greatly to the cost of the repairs. Placing the laterals at or near the surface makes them more accessible, but large pipes 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 in winter 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.

Nozzles are generally provided with 2-in. pipe-thread connections, which cannot be tapped into cast-iron pipes smaller than 6-in. Screwed pipe and fittings are therefore generally employed below the 6-in. size. If the lateral distributors are placed beneath the surface, riser pipes are necessary. The most common form of riser is cast-iron pipe jointed with bitumen or some other flexible joint-material. Screw pipe has also been employed. The riser is designed large enough to minimize friction, due allowance being made for a considerable deposit or growth on the inside of the pipe. Risers are commonly 3 in. in diameter. Long risers are more likely to become bent or broken during construction than are shorter ones. The elevation of the top of the riser above the filter does ordinarily not exceed 6 in., although certain types of nozzles afford better distribution when the nozzle is placed 12 in. or more above the filter surface. In cold climates the exposed portion of the riser is

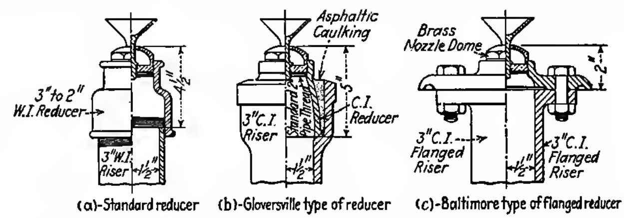


FIG. 132.—Nozzle connections to riser pipes. (*Pacific Flush-Tank Co.*)

likely to cause trouble from freezing during the resting periods of the dosing cycle. Sloping the bed to equalize the heads on the nozzles, while feasible, results in a slow discharge or "dribble" of sewage from the nozzles at the lower elevations. This is not desirable.

Connections between nozzles and riser pipes are made in three common ways: When 3-in. wrought-iron pipe is used for risers, a standard 3 by 2-in. reducer coupling may be used to make the connection, as shown in Fig. 132*a*. When the risers are of cast-iron bell-and-spigot pipe, a special reducer, 3 by 2 in. in size, may be calked into the bell end of the riser, as shown in Fig. 132*b*. For connections to flanged pipe, a special 3- by 2-in. reducing flange may be used, as in Fig. 132*c*.

Experience indicates that there is less likelihood of freezing in the larger pipes at the bed surface than there is in the small riser pipes of the subsurface system. The surface system is coming into wide use, as it eliminates the riser pipes, the fittings necessary for connecting them and also the concrete piers which are required for the subsurface system.

Hydraulics of Nozzles for Trickling Filters.—The hydraulic characteristics of various types of nozzles have been studied by the manufacturers, such as the Pacific Flush-Tank Co., by the engineering experiment station of Purdue University, by the authors and by others. Figure 133 shows the distribution curves obtained in the Purdue experiments for a Worcester-type nozzle (19). Generally speaking, the discharge through spray nozzles can be represented by the standard orifice

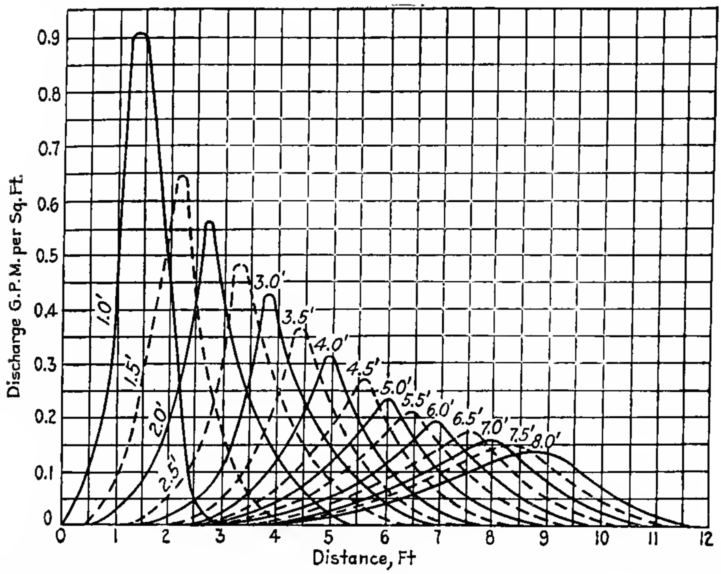


FIG. 133.—Distribution curves for 0.813-in. Worcester circular nozzle under different constant heads.

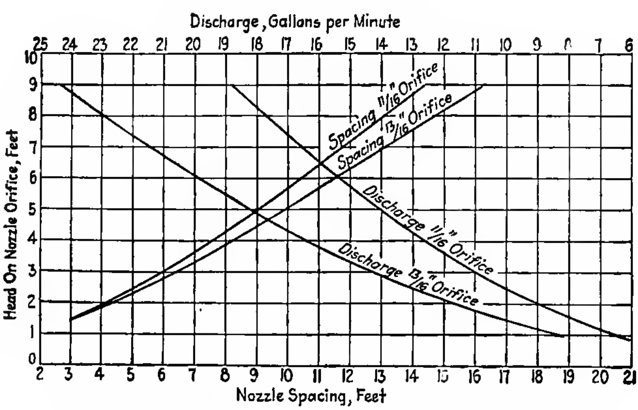


FIG. 134.—Discharge and spacing of Worcester-type nozzles.

formula, $Q = C_n a_n \sqrt{2gh_n}$. The coefficients, C_n, obtained at Purdue for the Worcester nozzle range from 0.66 at heads of 8 ft. to 0.69 at heads of 1 ft. The discharge of Worcester-type nozzles is shown in Fig. 134.

Circular-spray nozzles which are not spaced to give overlapping sprays cover only 78.5 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 undesirable. 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. 135. When fixed nozzles are used and the head remains constant, the quantity of liquid

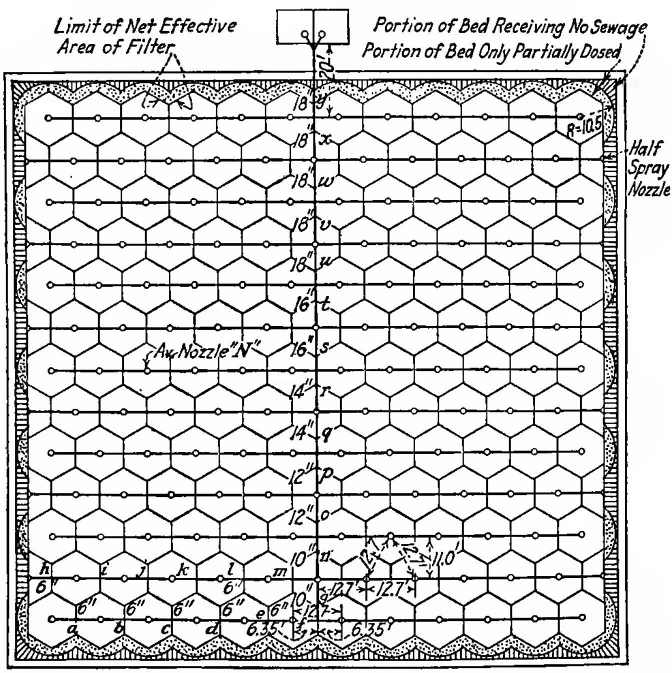


FIG. 135.—Arrangement of nozzles and distribution system for a trickling filter.

falling upon a unit area at the periphery of the wetted area is less than that at some lesser distance; and still nearer the nozzle the rate of fall is again less than the maximum. Uniform distribution over the surface therefore necessitates overlapping of the sprays at the time of maximum head, as well as a variation in head upon the nozzles.

While square-spray nozzles apparently will overcome the necessity for overlapping sprays, wind action and nozzle clogging affect all sprays in such a manner as in a measure to offset refinements of design of nozzles. Circular-spray nozzles seem to be preferred by operators generally.

Nozzle spacing has been worked out for the most common types of nozzles from studies such as those illustrated in Fig. 133 and from tests of experimental and existing installations. If allowance is made for overlapping of sprays and placing of nozzles at the apices of equilateral triangles, the net effective area of each nozzle is hexagonal in shape, as shown in Fig. 135. Along the edges of rectangular beds there remain areas, stippled in Fig. 135, which are dosed only in part and other areas, cross-hatched in Fig. 135, that receive no sewage whatever. Use of "half-spray" nozzles along the edges has been resorted to. These are generally made by covering one half of the nozzle opening and attaching flanges which direct the spray inward.

On the assumption that the capacity of a trickling filter is approximately proportional to its depth, a smaller volume of filtering material

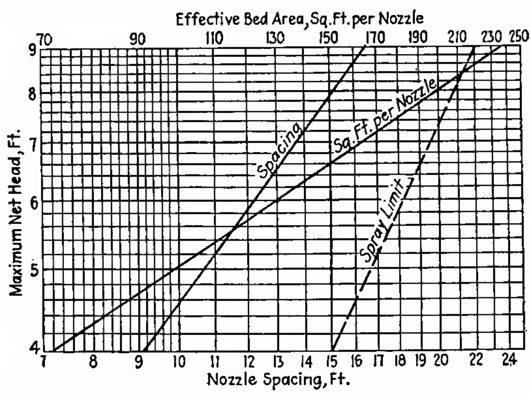


FIG. 136.—Optimum spacing and effective area of 7/8-in. Pacific Flush-Tank nozzles.

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.

Complete utilization of the marginal strip is not practicable if wind-blown spray is to be kept on the filter. Marginal allowances of 18 to 24 in. are frequently made. Selection of nozzle spacing is illustrated by an example in a following section.

The spray limit, optimum spacing and effective area per nozzle for Pacific Flush-Tank Co. nozzles at various maximum heads are shown in Figs. 136 and 137 (20). The discharge in gallons per minute at various heads for these nozzles is shown in Fig. 138. The distribution curves of Pacific Flush-Tank nozzles with 3/8-in. spindles under different constant heads are shown in Figs. 139 and 140.

The discharge and distribution curves for flotation-type nozzles with ⅜-in. spindles are shown in Figs. 141 and 142 (18). It is claimed that

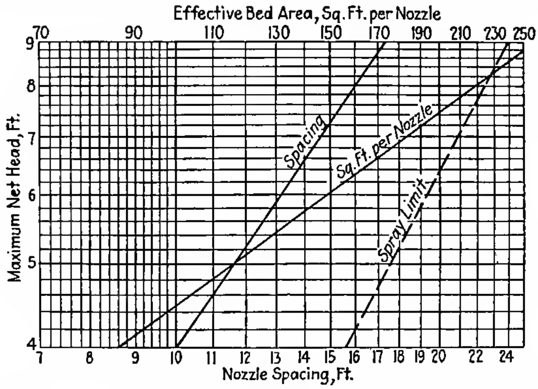


FIG. 137.—Optimum spacing and effective area of 1-in. Pacific Flush-Tank nozzles.

this type of nozzle secures better distribution, as the discharge falls off to a lower rate at the low heads than in the case of nozzles with fixed cones.

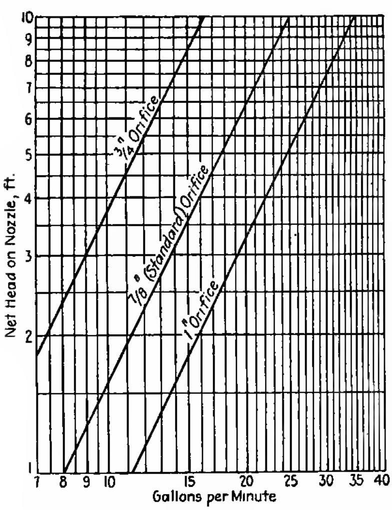


FIG. 138.—Discharge of Pacific Flush-Tank nozzles.

Dosing Devices.—Control of the application of sewage to trickling filters is usually made automatic. There are three classes of dosing apparatus in general use: air-controlled siphonic apparatus, mechanically controlled siphonic apparatus and mechanical devices. These devices are considered in detail in Chap. XXIII.

In order to obtain the variation in head on the nozzles of trickling filters, which is necessary to distribute the sewage as uniformly as possible over the bed, use is made of dosing tanks or head-varying valves. Discussion of head-varying valves is deferred to Chap. XXIII.

Dosing Tanks.—The usual form of dosing tank is hopper-shaped or is tapered by the use of 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 throw a larger proportion of the spray on to the more extensive outer areas. The operating head commonly ranges from a maximum between 5 and 10 ft. down to a minimum of 1 or 2 ft. Automatic air-lock siphons are used in conjunction with these tanks.

There are three common types of dosing-tank installations:

1. Single dosing tanks into which sewage flows continuously, the siphon capacity being such that the rate of outflow when discharging always exceeds that of inflow, even under the minimum operating heads and with

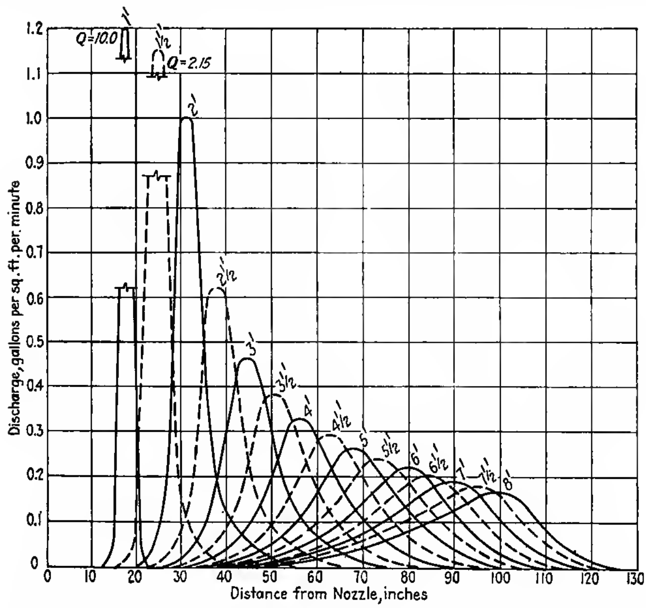


FIG. 139.—Distribution curves for 7/8-in. Pacific Flush-Tank nozzle.

maximum inflow. If suitably proportioned these tanks will alternately fill and discharge.

2. Twin dosing tanks with air-lock feeds discharging to a common nozzle field; one tank fills, while the other discharges and stands empty during a brief rest period. Deep bells on the siphons or flap valves shut off intercommunication between the tanks. The maximum rate of inflow is such as to fill the tanks in a longer time than is required for discharge.

3. Twin dosing tanks with air-lock feeds discharging on to separate nozzle fields, each half of the field having its separate dosing cycle. Since only half the number of nozzles is supplied at one time, the dosing-tank size is cut in two.

Until the time of construction of the Fitchburg plant, dosing tanks for trickling filters were commonly of the single-dosing type. There is usually a wide fluctuation in the rate of sewage flow and this variation in flow produces an unequal distribution from the nozzles, if this type of dosing tank is used. Inflow into the dosing tank during time of discharge tends to overdose the filter, especially at low heads, and this may cause pooling on the filters adjacent to the nozzles.

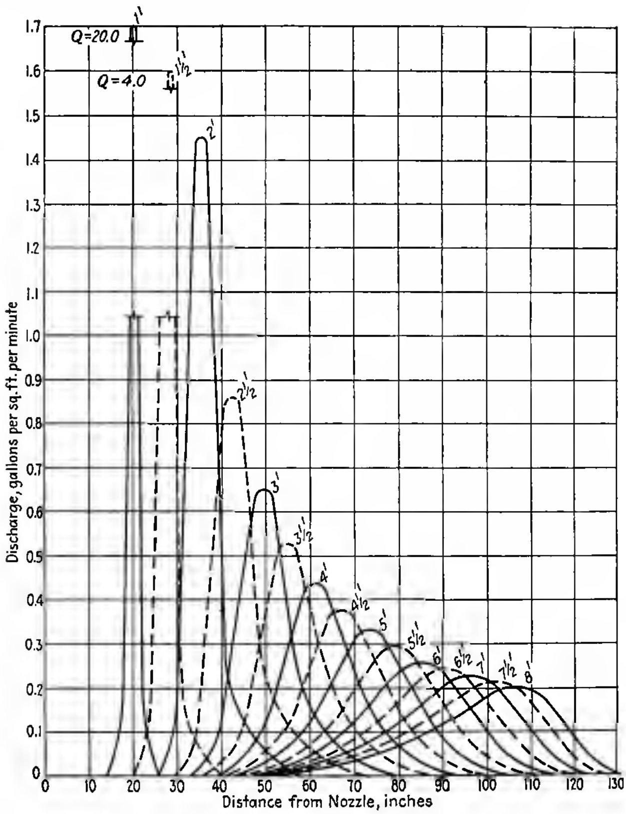


FIG. 140.—Distribution curves for 1-in. Pacific Flush-Tank nozzle.

Twin dosing tanks assure the same size of dose and the same distribution of the sewage on the filter at each dose, regardless of the rate of sewage flow, until a maximum storm-water rate is reached, when one of the tanks goes into continuous operation. By the use of two dosing tanks which operate alternately, the point at which continuous operation begins is kept higher. Such an installation at Fitchburg is illustrated in Fig. 143.

The dosing tanks are built in pairs and at the inlet end of each tank there is located an automatic air lock, which shuts off the inflow when the tank to which it is connected is filled. At the same time that the air lock for one tank shuts off, the air lock connected to the other tank is released, thereby permitting the inflow to enter the empty tank. By

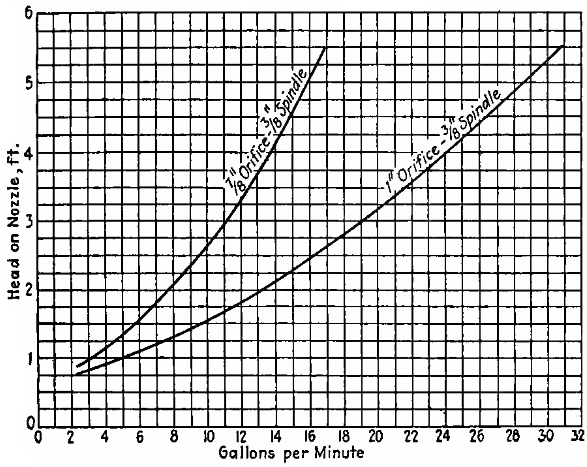


FIG. 141.—Discharge of flotation-type nozzles.

this method, a regular, definite quantity of sewage is applied to the filters at each dose and no sewage enters the dosing tank during the time of discharge to disturb the desired distribution.

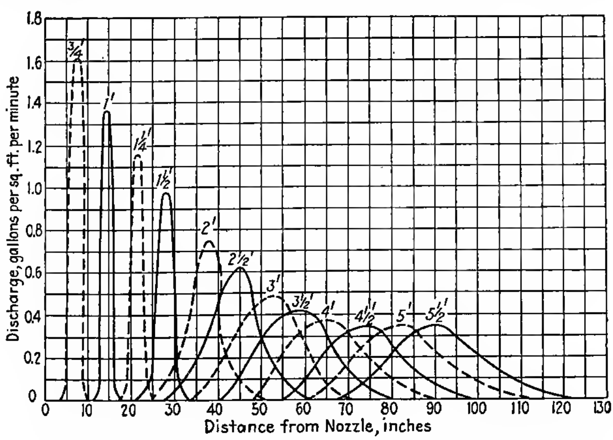


FIG. 142.—Distribution curves for 1-in. flotation-type nozzle.

The design of a dosing tank and nozzle field is exemplified on page 525. Suitable design takes the following hydraulic factors into account as relating to flow and losses of head:

1. Friction in air-lock feeds and over inlet weirs
2. Siphon losses
3. Dosing-tank losses
4. Velocity heads and friction losses in distributing pipes
5. Entrance losses, losses in passing openings and losses at change in section and direction
6. Losses in gates

Losses 2 to 6 commonly equal 15 to 35 per cent of the maximum available head on the nozzles, with 25 per cent as a fair average. These losses are about equally divided between the siphon, dosing tank and

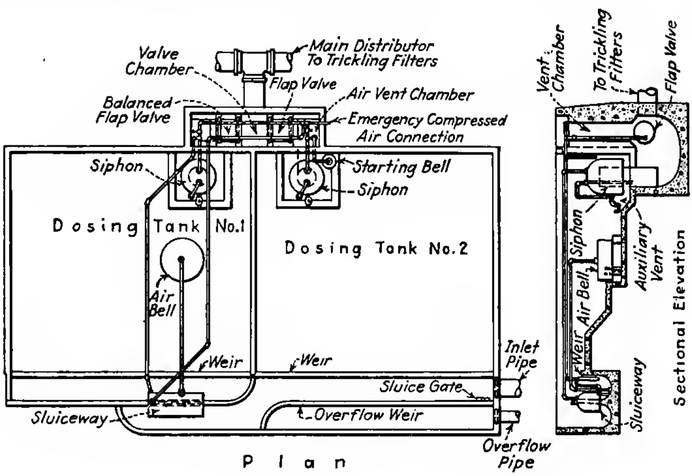


FIG. 143.—Dosing tanks and siphonic apparatus, Fitchburg, Mass.

distribution system. Losses in the latter can be greatly reduced by using rounded orifices, bell-mouth openings and gradual changes in section.

The Pacific Flush-Tank Co. has studied the operation of trickling-filter dosing systems using the company's dosing apparatus and has derived formulas for the computation of the special losses involved (20); these the authors have reduced to general terms as given below. The formulas were derived from observations at five trickling-filter plants using siphons from 12 to 30 in. in diameter, operating under a "total head" of 7 to 10 ft. and using dosing tanks with maximum area of water surface from 200 to 800 sq. ft. The formulas naturally apply only to the company's apparatus. The expression for siphon loss (h_s) holds, furthermore, only for siphons of the trapless type and the formulas for dosing-tank loss (h_d) and time of discharge (t) should be used only when the maximum velocity in the siphon is between 2 and 4 ft. per second. It will be noted that the expressions are empirical, with the exception of that for siphon loss. So far as the authors know, these are the best data to be had on the subject.

$$h_f = 0.129\left(\frac{Q_i}{L_f}\right)^{1.64} \tag{1}$$

$$h_w = 0.470\left(\frac{Q_i}{L_w}\right)^{0.565} \tag{2}$$

$$h_s = 0.0433\left(\frac{Q}{a_s}\right)^{2} \tag{3}$$

$$h_d = \frac{364a_s^2}{AQ_{\text{max.}}} \tag{4}$$

$$t_a = 454\left(\frac{a_s}{Q_{\text{max.}}}\right)^{2} \tag{5}$$

Nomenclature:

h_f = loss of head in inlet feed, for 6-in. feed, ft.
h_w = loss of head over inlet weir, ft.
h_s = loss of head in dosing siphons, ft.
h_d = loss of head at starting, or dosing-tank loss, ft.
t_a = time elapsed from time siphon starts to time of maximum discharge, or time during which dosing-tank loss occurs, sec.
Q_i = rate of inflow to tank, c.f.s.
Q = rate of discharge, c.f.s.
$Q_{\text{max.}}$ = maximum rate of discharge, c.f.s.
a_s = cross-sectional area of siphon, sq. ft.
A = area of dosing tank at maximum water level, sq. ft.
L_f = length of inlet feed, ft.
L_w = length of inlet weir, ft.

Generally good distribution is secured if the water level in the dosing tank falls at a uniform rate, or nearly so. The surface area of the tank, though determined to some extent by considerations of nozzle distribution, is influenced even more by the dosing-tank loss. Since this loss is volumetric and must be limited to a certain value, it generally controls the choice of the tank area. This maximum area must be carried down a distance equal to the dosing-tank loss. From here on the area must decrease in proportion to the rate of discharge. A parabolic curve results and is generally approximated by continuing the vertical sides of the tank for a short distance and then sloping the sides in a straight line to the low-water line.

Clogging of nozzles is reduced and operation of the filters facilitated if the distribution laterals are occasionally flushed out. Thus accumulated sludge solids and organic growths are removed. Otherwise, these solids are dislodged from time to time and increase the frequency of nozzle clogging. At Worcester, Mass., flushing is done by removing the end nozzles and allowing the siphons to discharge. The 6-in. valves which empty the distribution system into the main drains are opened regularly for a short period of time as an aid to maintaining the system in as clean a condition as possible.

At Fitchburg, the ends of the distribution laterals are valved and joined to a 6-in. cast-iron pipe collector discharging on to a small sand bed. The laterals are flushed three to four times a year on the average. An appreciable decrease in nozzle clogging is effected.

At Akron, the ends of the laterals are valved and joined to an 8-in. cast-iron pipe collector, discharging to the humus-sludge line. During flushing, the discharge is pumped to the Imhoff tanks.

The practice of discharging the flushings on to the trickling filters at the ends of the laterals has resulted in some places in the clogging of the stone in the vicinity to an undesirable extent.

Nozzle Fields.—There are two types of nozzle fields in use, the common nozzle field and the separate nozzle field. In the common field, a filter served by twin dosing tanks is built as a single unit and the two siphons are connected to a common distribution main serving the entire nozzle field. The main advantage of this type of field is the ability to handle inflow rates two or three times the normal and still maintain automatic operation.

Where a great variation of flow is not to be expected, there are some advantages in the separate field. In this type of plant each dosing tank with its siphon serves an area independent of all other dosing equipment. In a plant where twin dosing tanks are to be installed, the separate-field installation requires dosing tanks just half as large as those required by a common field, as well as correspondingly smaller dosing equipment.

Outline of Hydraulic Computations for a Trickling Filter.—The following outline is suggestive of one of a number of approaches that may be taken in the solution of the hydraulic problems connected with the design of the dosing devices and distribution system of a nozzle field. A hypothetical problem is set, rather than an actual case, because it can be presented more simply and will illustrate the issues more readily. Repeated trials of different layouts and studies of individual parts of the system, involved in the economical design of filters in practice, are not given, in order to conserve space.

Assumed Basic Data.

Average volume of sewage to be treated	1.0 m.g.d.
Maximum rate of flow	3.0 m.g.d.
Minimum rate of flow	0.5 m.g.d.
Loading of filter	0.333 m.g.d. per acre-foot
Depth of filter	6 ft.
Difference in elevation between maximum discharge line of dosing tank and nozzle openings	8.9 ft.

Computations for Nozzle Field.—Let it be assumed that the nozzle field is to be constructed with cast-iron pipes laid near the surface of the bed, and that circular-spray Worcester-type nozzles are to be employed.

Selection of Nozzles.

1. The economical range in the maximum net head on the nozzles of a trickling filter is commonly found to extend from 65 to 85 per cent of the total head. For a head of 8.9 ft., therefore, it will generally lie between 5.8 and 7.6 ft. *Try* 6.7 *ft.*

2. The minimum net head or net terminal head is ordinarily kept between 1.0 and 2.5 ft. *Try* 1.5 *ft.*

3. Circular-spray Worcester-type nozzles will have the following characteristics when operating under heads of 6.7 and 1.5 ft.:

Nozzle characteristics	Size of nozzle, in.		Remarks
	$1\frac{1}{16}$	$1\frac{3}{16}$	
1. Nozzle spacing, ft.	11.4	12.7	From Fig. 134
2. Spacing of laterals, ft.	9.9	11.0	0.866 × nozzle spacing
3. Number of nozzles required	193	156	$\dfrac{1.0 \times 43{,}560}{0.333 \times 6 \times (1) \times (2)}$
4. Maximum rate of discharge per nozzle, g.p.m.	16.2	21.0	From Fig. 134, or using
5. Minimum rate of discharge per nozzle, g.p.m.	7.7	10.0	P = 0.66 to 0.68 in $Q = Ca\sqrt{2gh}$
6. Maximum rate of dosing nozzle field, g.p.m.	3,130	3,280	(3) × (4)
c.f.s.	6.96	7.30	
7. Minimum rate of dosing nozzle field, g.p.m.	1,490	1,560	(3) × (5)
c.f.s.	3.31	3.48	
8. Average of maximum and minimum dosing rates, c.f.s.	5.14	5.39	½[(6) + (7)]
9. Maximum rate of sewage flow, c.f.s.	4.64	4.64	3.0 × 1.547

Try using a $1\frac{3}{16}$*-in. nozzle.* The average of the maximum and minimum dosing rates of this nozzle is well in excess of the maximum rate of sewage flow. Its spray limit is 10.5 ft., from Fig. 133.

Selection of Distribution System.

4. *Try a distribution system layout consisting of* 13 *laterals, each with* $^{156}/_{13}$ = 12 *nozzles,* as shown in Fig. 135. A number of systems should be studied and the most economical one selected.

5. Computations for the head lost in the pipe system shown in Fig. 135 at the maximum rate of dosing are illustrated in the schedule shown in the table on page 526.

6. The pipe loss, h_p, to the average nozzle N in Fig. 135 is the sum of the losses in lines c to f and s to y, together with the entrance T loss, the Y loss, and the loss in passing openings. It is found to be 0.77 ft. This is

Pipe	Length, ft.	Diameter, in.	Sewage flow, c.f.s.	Velocity, ft. per second	Friction losses			
					Pipe losses[1]		Passing openings,[2] ft.	Bends,[3] ft.
					Ft. per 1000	Ft.		
a	12.7	6	0.047	0.24	0.095	0.001		
b	12.7	6	0.094	0.48	0.34	0.004		
c	12.7	6	0.140	0.71	0.71	0.009		
d	12.7	6	0.187	0.95	1.22	0.015		
e	12.7	6	0.234	1.19	1.85	0.024	0.001	
f	6.35	6	0.281	1.43	2.60	0.017	0.001	T = 0.040 NOTE.—This is the loss to each lateral.
g	11.0	10	0.562	1.03	0.78	0.009		
n	11.0	10	1.12	2.06	2.82	0.031	0.004	
o	11.0	12	1.68	2.14	2.45	0.027	0.004	
p	11.0	12	2.25	2.87	4.20	0.046	0.008	
q	11.0	14	2.81	2.62	2.98	0.033	0.006	
r	11.0	14	3.37	3.15	4.18	0.047	0.009	
s	11.0	16	3.93	2.82	2.91	0.032	0.007	
t	11.0	16	4.49	3.22	3.72	0.041	0.010	
u	11.0	18	5.05	2.86	2.61	0.029	0.008	
v	11.0	18	5.62	3.18	3.18	0.035	0.009	
w	11.0	18	6.18	3.50	3.79	0.042	0.012	
x	11.0	18	6.74	3.81	4.42	0.049	0.013	
y	20.0	18	7.30	4.13	5.08	0.102	0.016	Y = 0.265

[1] By Williams and Hazen formula, using $C = 100$.

[2] 0.03 × velocity head for each opening or branch. Flinn, Weston and Bogert; "Water Works Handbook," 1st ed., p. 583.

[3] *Jour.*, N. E. W. W. A., 1913; **27**, 520. T loss = 1.25 velocity heads; Y loss = 1 velocity head.

about ⅓ of the total loss assumed to be available (8.9 − 6.7 = 2.2 = 3 × 0.73) and represents a common value.

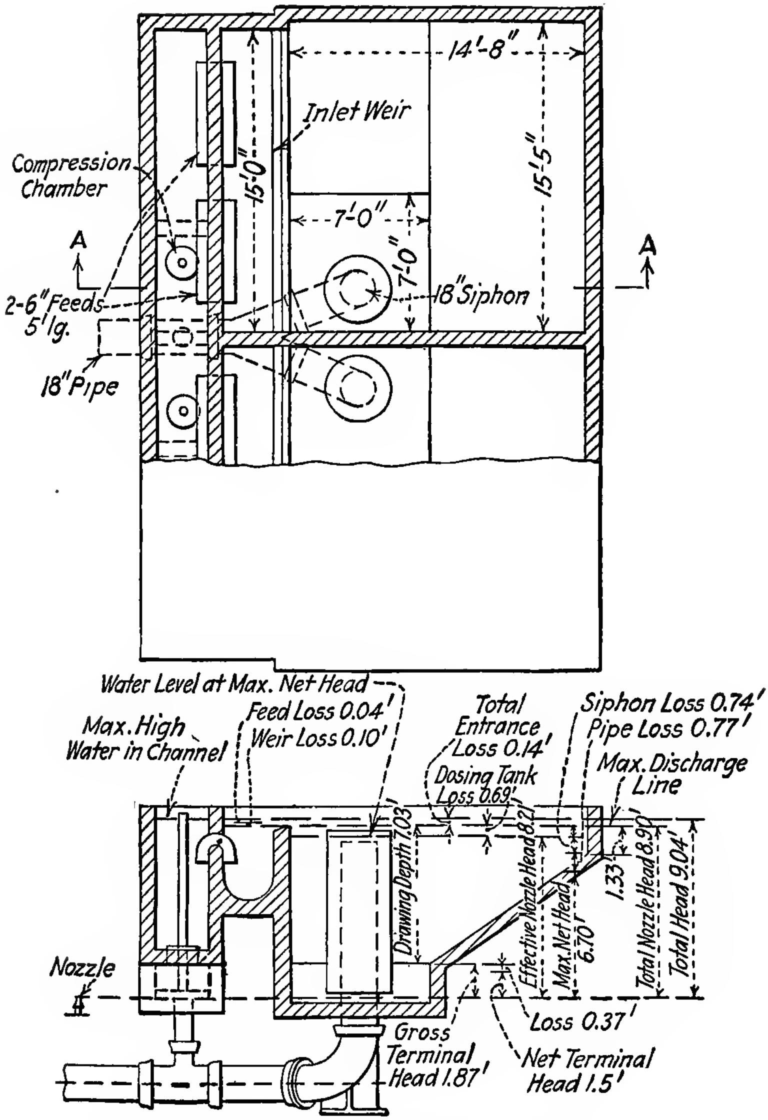
Nozzles other than N should also be studied.

Computations for Dosing Tank.—Investigate the use of twin dosing tanks supplying a common nozzle field and equipped with air-lock feeds and deep-bell, trapless siphons, illustrated in Fig. 144. Try a dosing cycle of 4.5 min. for the maximum rate of flow of 3.0 m.g.d. = 4.64 c.f.s. This is a common cycle time.

Size and Shape of Dosing Tank.

7. For a dosing cycle of 4.5 min. the effective volume of dosing tank is 4.64 × 4.5 × 60 = 1,250 cu. ft.

8. By following common practice and selecting a siphon of such size that the maximum siphon loss, h_s, is about equal to the pipe loss, h_p, about 1/12 of the total head or ⅓ of the total loss assumed to be available, it is found, by using a coefficient of discharge of $C_s = 0.6$, that the nearest size is 18 in. The siphon loss is, therefore,



Section A-A

FIG. 144.—Twin dosing tanks with air-lock feeds and deep-bell, trapless siphons.

$$h_s = \left(\frac{Q}{C_s a_s \sqrt{2g}}\right)^2 = \left(\frac{7.30}{0.6 \times 1.767 \times 8.02}\right)^2 = 0.74 \text{ ft.}$$

where Q is the maximum rate of dosing the nozzle field.

9. The effective nozzle head for maximum discharge is, therefore, 6.7 + 0.77 + 0.74 = 8.21 ft.

10. On the assumption that the heads for other rates of discharge vary approximately as the squares of the rates, the minimum discharge head or gross terminal head will be $8.21 \times \left(\frac{3.48}{7.30}\right)^2 = 1.87$ ft. In actual design this value would be checked by computations similar to those given for maximum discharge.

11. The bottom width of the tank must be such as to accommodate the 18-in. siphon and to result in an economically proportioned tank. The siphon needs 6.67 ft., according to Pacific Flush-Tank Catalog 30. *Try 7 ft.*

12. If the tank areas are proportioned to the rates of discharge, the top width becomes about $7 \times \frac{7.30}{3.48} = 14.67$ ft., because the tank length L is commonly held constant.

13. The dosing-tank loss, h_d, may now be expressed in terms of the top area or length by applying the formula suggested by the Pacific Flush-Tank Co.:

$$h_d = 364\frac{a_s^2}{AQ} = \frac{364 \times (1.767)^2}{A \times 7.30} = \frac{156}{A} = \frac{156}{14.67L}$$

14. To allow (*a*) for the dosing-tank loss, h_d, which is usually of the same magnitude as the siphon loss, h_s, and the pipe loss, h_p, and (*b*) for the use of a uniformly sloping tank wall rather than a parabolic one, vertical tank walls are required at the top, of a depth greater than the dosing-tank loss. *By inspection, try 1.33 ft.*

15. If the tank capacity computed from the assumed dimensions is equated to the tank volume obtained from the cycle time, the tank length L = 15.42 ft. from

$$1{,}250 = \left(8.21 - 1.87 + \frac{156}{14.67L}\right) \times 14.67L - \left[(8.21 - 1.87 - 1.33) + \frac{156}{14.67L}\right] \times \frac{(14.67 - 7.0)}{2}L$$

16. The dosing-tank loss for the tank dimensions chosen is

$$h_d = \frac{156}{14.67 \times 15.42} = 0.69 \text{ ft.}$$

which is approximately equal to h_s and h_p. The total computed loss is, therefore, $h_d + h_s + h_p = 2.20$ and the total difference in elevation between the discharge line of the dosing tank and the nozzle openings is 6.7 + 2.20 = 8.90 ft., which equals, in this case, the value of 8.90 ft. assumed in the basic data.

Dosing Cycle.

17. A dosing cycle of 4.5 min., under conditions of maximum sewage flow, was assumed. This is made up of the dosing time and the rest period. Although good results can be secured with continuous operation at times of maximum flow, it is well to ensure satisfactory tank and nozzle operation by providing for a minimum rest period of say 0.5 min. To estimate the probable rest period, the dosing time may be assumed to consist of (*a*) the siphon-starting time, t_a, during which the tank level drops a distance equal to the dosing-tank loss, (*b*) the time, t_b, required to empty the remaining vertical portion of the tank and (*c*) the time, t_c, required to draw down the sloping portion of the tank.

a. Time t_a may be found from the equation proposed by the Pacific Flush-Tank Co.:

$$t_a = 454\left(\frac{a_s}{Q}\right)^2 = 454\left(\frac{1.767}{7.30}\right)^2 = 26.5 \text{ sec.}$$

b. Time t_b is given by the common formula of theoretical hydraulics for discharge under a falling head:

$$t_b = -\frac{A}{C_n a_n \sqrt{2g}}\int_{h_1}^{h_2}\frac{dh}{\sqrt{h}} = \frac{2A}{C_n a_n \sqrt{2g}}(h_1^{1/2} - h_2^{1/2})$$

where C_n is the coefficient of discharge of the nozzle field and a_n is the total area of the nozzle openings. The area of a $1\frac{3}{16}$-in. Worcester nozzle with $\frac{3}{16}$-in. spindle is 0.00340 sq. ft. The combined area $a_n = 156 \times 0.0034 = 0.530$ sq. ft. C_n may be computed approximately from the loss of head, 8.21 ft., excluding the dosing-tank loss, found for the average nozzle N for the maximum rate of discharge, 7.30 c.f.s.

$$C_n = \frac{Q}{a_n\sqrt{2gh}} = \frac{7.30}{0.530 \times 8.02\sqrt{8.21}} = 0.60$$

$$t_b = \frac{2 \times 14.67 \times 15.42}{0.60 \times 0.530 \times 8.02}[(8.21)^{1/2} - (8.90 - 1.33)^{1/2}] = 21.2 \text{ sec.}$$

c. To find time t_c the tank area must be expressed in terms of the head, since the area decreases with the head. The area A at any head h is

$$A = \left[7.0 + \frac{(h - 1.87)(14.67 - 7.0)}{(8.90 - 1.33 - 1.87)}\right] \times 15.42$$

$$A = 20.8(3.33 + h)$$

$$t_c = \frac{-20.8}{0.60 \times 0.530 \times 8.02}\int_{h_1}^{h_2}\frac{(3.33 + h)}{\sqrt{h}}dh$$

$$t_c = 8.15[6.66(h_1^{1/2} - h_2^{1/2}) + \tfrac{2}{3}(h_1^{3/2} - h_2^{3/2})]$$

Since $h_1 = 8.90 - 1.33 = 7.57$ and $h_2 = 1.87$,

$$t_c = 174 \text{ sec.}$$

The dosing period is, therefore, $t_a + t_b + t_c = 3.7$ min. and the resting period, $4.5 - 3.7 = 0.8$ min., which satisfies the requirements. Were this not so, it would be necessary to select different nozzles or change some of the assumptions.

Computations for Inlet Structures.—It is assumed that the inlet structures consist of air-lock feeds and weirs as shown in Fig. 144.

Air-lock Feed.

18. *Try two 6-in. feeds, 5 ft. wide.*

The feed loss h_f for a maximum inflow rate of 3 m.g.d. = 4.64 c.f.s. becomes:

$$h_f = 0.129\left(\frac{Q_i}{L_f}\right)^{1.64} = 0.129 \times (0.464)^{1.64} = 0.04\,\text{ft.}$$

Inlet Weir.

19. *Try a weir 15 ft. long.*

From the Francis formula, the weir loss, h_w, with a coefficient of discharge of 3.0, is found to be:

$$h_w = \left(\frac{Q}{CL_w}\right)^{2/3} = \left(\frac{4.64}{3.0 \times 15}\right)^{2/3} = 0.20\,\text{ft.}$$

The Pacific Flush-Tank formula, $h_w = 0.470\ (Q_i/L_w)^{0.565}$, yields about the same result.

20. The combined loss of the inlet structures is $h_f + h_w = 0.24$ ft. for maximum flow. The sewage level in the inlet channel must, therefore, rise 0.24 ft. above the maximum level of the dosing tanks at times of highest sewage flow. If this value is unsatisfactory the inlet loss may be cut down by changing the dimensions of the feeds or by changing the length of the weir.

Underdrainage.—Underdrains, as well as distribution systems, are necessarily more elaborate as the volume of sewage treated per unit area becomes greater. Therefore, for trickling filters, suitable design of underdrainage facilities is of considerable importance.

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. These objections are met if such filters are built upon floors of concrete or other masonry.

One of the earliest drainage systems consisted of a concrete floor upon which is a layer of stones 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 may 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 carried away more readily with the effluent or flushed through the drains and that the gases of decomposition may have a ready means of escape, false floor systems are generally adopted.

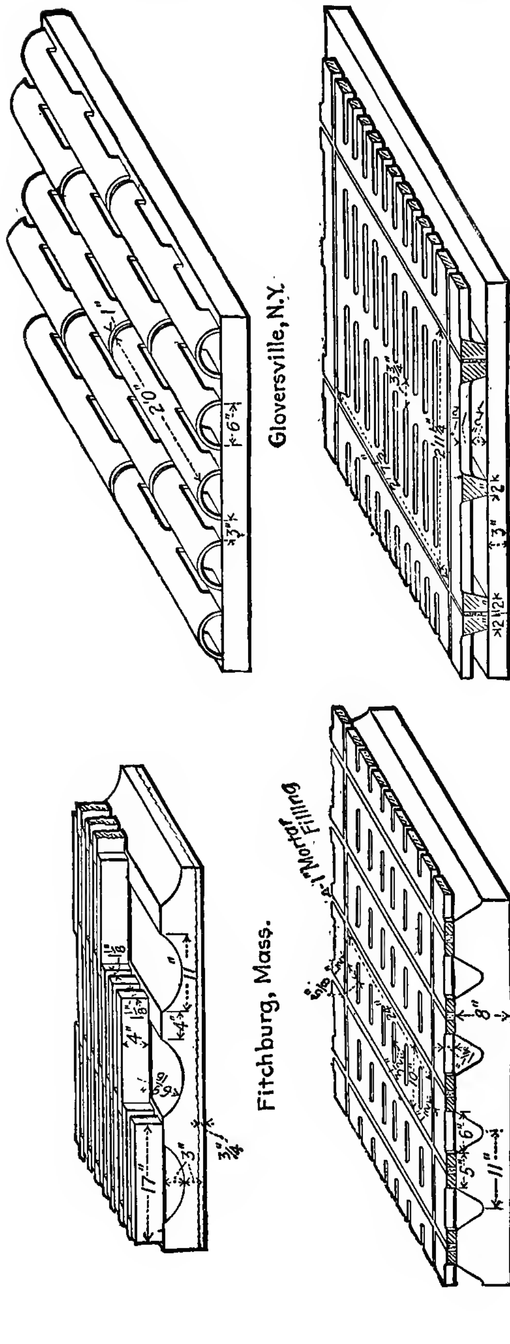


FIG. 145.—Types of floors for trickling filters.

The floor system used at Gloversville and some other places, shown in Fig. 145, is the least expensive type under some conditions. The concrete slopes toward the main drains and the lateral drains of inverted half-round, slotted tile practically cover the entire floor. Floor slopes of at least 1 per cent are usually provided. The tiles 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. Instead of slotted pipes of this kind, half-round bell-and-spigot pipes laid with open joints have been employed. The opening should be made as large as possible without sacrificing the overlap of the bell. A 1-in. opening is desirable.

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 likely to disintegrate, the valleys between the pipes may gradually become 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.

Figure 145 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 large part of the floor, however, so that the self-cleansing qualities and facilities for flushing are not ideal.

The Baltimore floor system, Fig. 145, 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 is of uniform depth.

A type of floor designed by the authors for use at Fitchburg, Mass., and subsequently installed at other plants, is shown in Fig. 145. As the lateral drains in the concrete are covered with narrow concrete beams, a large percentage of drainage area is afforded, as well as a concentrated flow of effluent. Cobble stones, which are abundant at the site, were placed by hand over the openings before placing the crushed stone on

FIG. 146.—Constructing trickling-filter floor, Fitchburg, Mass.

the bed. The method of constructing the floor of the Fitchburg filters is illustrated in Fig. 146.

A type of underdrainage system manufactured by the Metropolitan Paving Brick Co. and used at Akron and Cleveland, Ohio, as well as at

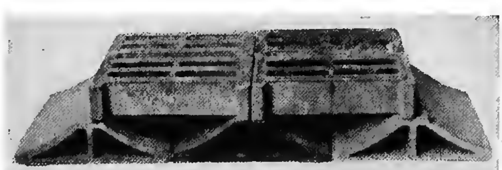


FIG. 147.—Underdrainage system made by Metropolitan Paving Brick Co.

other plants, is shown in Fig. 147. The concrete floor is cast in a flat surface sloping toward the main drains. The underdrainage system consists of vitrified-clay pier and grill blocks. The pier blocks form a channeled floor on which the grill blocks are laid. The channels formed are 3 in. deep and 12 in. wide at the top and are spaced 13½ in. center to center. The openings in the grill blocks are 1⅛ in. wide and the space between openings is 1⅛ in. The depth of the underdrainage system is 7 in. Figure 148 shows such a system being installed in trickling filters at Akron, Ohio.

It is evident that the depth of the underdrainage system 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 are generally as large as can be provided at reasonable cost. The false floor is necessarily made amply strong for the load coming upon it.

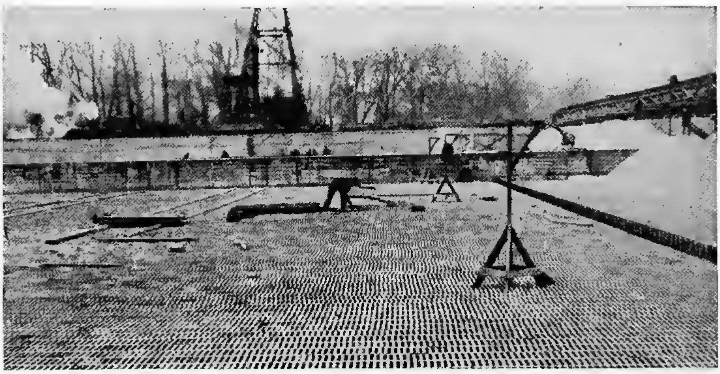


Fig. 148.—Underdrainage system of trickling filters at Akron, Ohio.

The possibility that the drains may become clogged with solids unloaded by the filter or with 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. It is desirable that the lateral drains be of sufficient size to carry away the effluent promptly, to make flushing possible and to afford space for the circulation of air. Where tiles or channels are carried through the outer walls, it is impossible to flood or submerge the filter for the purpose of destroying the larvae of the filter fly. This is also true where walls are not watertight or where they are omitted, the ballast taking its own angle of inclination.

The main drains may be rectangular or semicircular channels covered with slabs, or they may be circular. It is desirable that the grades of all parts of the drainage system ensure freedom from backing up of effluent in the ducts and channels.

Loss of Head.—The loss of head through trickling filters varies with the depth of filters, area of filters, type of distribution system and characteristics of nozzles, if used, and of dosing equipment. Revolving, traveling and other mechanical distributors require considerably less head than spraying nozzles. The principal loss of head is commonly due to the depth of the filter, including the underdrainage and floor system. This loss varies from 6 to 11 ft. Next in importance is the maximum net head on the nozzles, which varies from 4 to 8 ft. Other losses in the dosing equipment, distribution system, collection system and connecting conduits may assume considerable proportions.

The loss of head in a number of trickling filters is given in Table 90.

TABLE 90.—LOSS OF HEAD IN TRICKLING FILTERS

Location	Elevation, ft.			Total head for filter, ft.	Depth of filter, ft.	Head excluding depth of filter, ft.
	Flow line of prelim.-sedimentation tanks	Surface of trickling filters	Flow line of final-sedimentation tanks			
Akron, Ohio	753.6	740.52	729.0±	24.6	10.0	14.6
Allentown, Pa.	29.5	18.2	6.2	23.3	10.0	13.3
Bloomington-Normal, Ill.	43.2	33.1	22.0	21.3	8.5	12.8
Cleveland, Ohio, Southerly plant	32.6[1]	23.1	10.4	22.2	10.0	12.2
Columbus, Ohio	28.24	20.15	11.00	17.24	5.33	11.91
Decatur, Ill.	602.6	595.0	585.8	16.8	5.7	11.1
Elgin, Ill.	742.1	732.5	722.1	20.0	8.3	11.7
Fitchburg, Mass.	385.4	375.0	360.0	25.4	10.0	15.4
Gloversville, N. Y.	67.0	61.5	55.0	12.0	5.0	7.0
Schenectady, N. Y.	13.75	9.0	3.67	10.08	5.0	5.08
Washington, Pa.	995.0	991.0	980.5[2]	14.5	6.83	7.67

[1] High water in dosing tank.
[2] Invert of main underdrain.

Ventilation.—Inasmuch as the effective 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 solids on or in the bed or to insufficient drainage capacity, causes a decrease in the efficiency of the filter.

With deep filters special precautions are required in the selection of materials of suitable size and permanent character and in the design and construction of the underdrainage system so that sufficient air will circulate in the lower part of the filter.

The value of ventilation through cowls or through side walls of open construction has not been demonstrated and such ventilation, no doubt, causes lower temperatures and reduces bacterial activity in cold weather. The natural downward currents of air have ordinarily maintained aerobic action in filters, except that sometimes slight troubles have been experienced with fine filter material. Clogging of the underdrains, which tends to interrupt free circulation of air, is detrimental but usually can be overcome by flushing.

Walls.—Some form of masonry is best for filter walls. These need not be made strong enough to act as retaining walls to support either the natural banks or the filtering material, although if such support is not provided for, extra care may be required during construction and during removal of the filtering material for washing, if this becomes necessary. At Fitchburg ribbed metal lath, plastered on both sides,

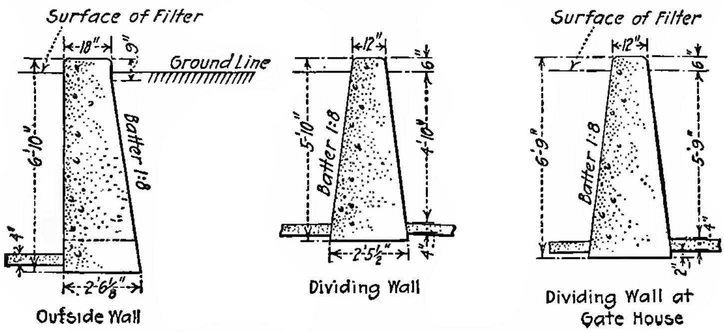


FIG. 149.—Walls of trickling filter at Columbus, Ohio.

was used as a curtain wall between the natural soil and the filtering material. In this case, the wall is inclined from the bottom outward. At Durham, N. C., the trickling filters were placed so that existing sand filters could be used as the bottom of the new trickling filters, the old embankments being built up and paved. The paving is a 4-in. reinforced-concrete slab, cast in place, with side slopes at an angle of 45 deg.

Types of wall used at Columbus are shown in Fig. 149. The wall of the trickling filters at Gloversville, N. Y. (Fig. 150), is buttressed and reinforced to support the roof system.

If the ground around the filter is higher than the surface of the bed, care is required to prevent the washing of soil into the filtering material by sewage spray or by storm runoff. This may be accomplished by extending the walls above the ground elevation or by providing drains to carry away the surface wash.

If trickling filters are constructed above the surface of the ground, the filtering material may be allowed to assume its natural angle of repose. This method of construction, however, involves the use of considerable filtering material which is ineffective in the treatment of sewage and the

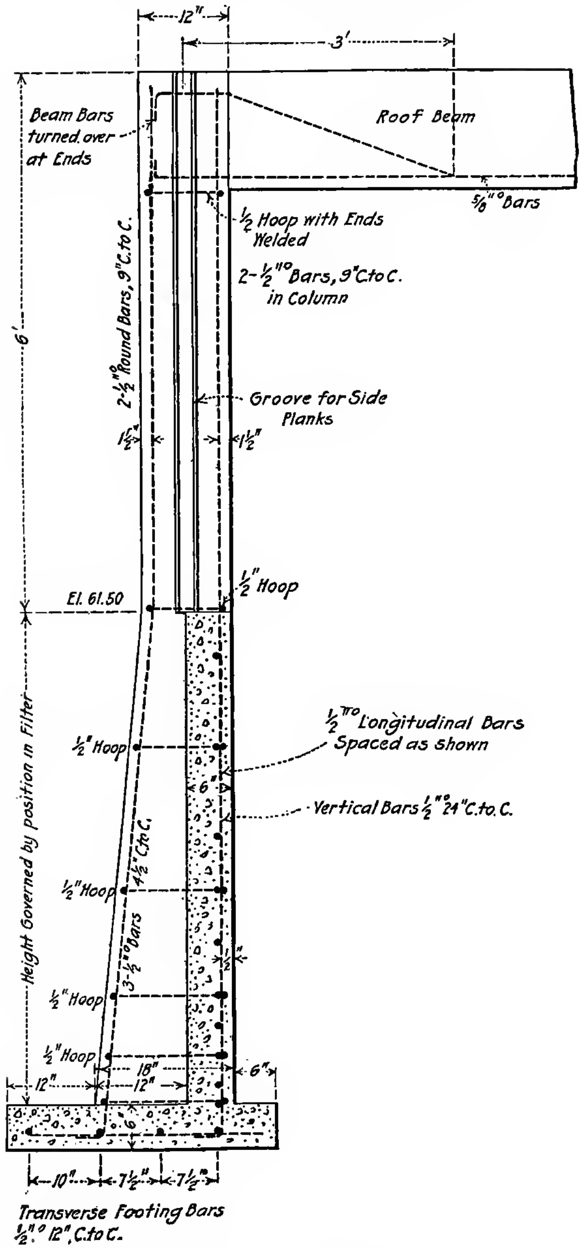


FIG. 150.—Wall and roof beam of trickling filter at Gloversville, N. Y.

construction of a larger floor system than is required to collect the effluent. For these reasons such construction is not economical in most cases where crushed stone is used for filtering, but where an inexpensive material is available, it may be the cheapest construction.

If the filtering material is not sloped, some form of wall is necessary. Such walls have been constructed of tile, brick and concrete. The brick wall, commonly seen in England, is calculated to facilitate ventilation within the bed. To accomplish the same purpose tile have sometimes been built into solid masonry walls.

Covers for Small Trickling Filters.—Small trickling filters, which are adjacent to residences or are likely to cause annoyance for other reasons, have been covered at Clear Lake, Ia., Haddon Heights and Totowa,

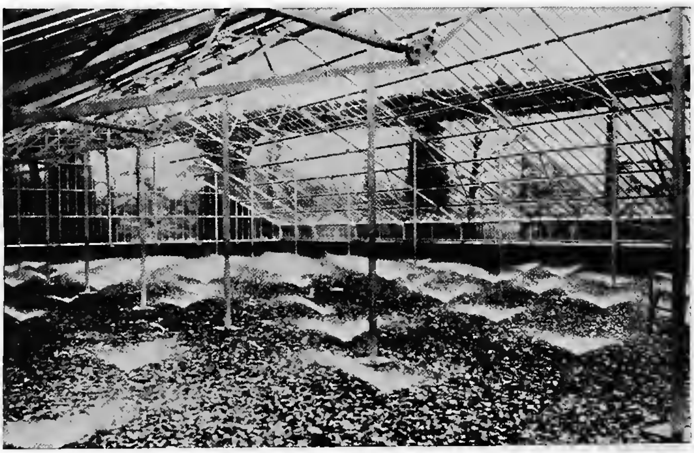


Fig. 151.—Glass-over for trickling filter at Totowa, N. J.

N. J., Mt. Vernon, N. Y., and elsewhere. In some places, such as Gloversville, N. Y., covers have been built to protect filters from the low temperatures of winter.

The cover at Haddon Heights is a glass house extending over both trickling filters and sludge-drying beds, measuring 173 by 80 ft., the filters being 136 by 80 ft. in plan and the sludge-drying beds 37 by 80 ft. The glass-covered filter was provided for the control of odors and filter flies. Operation during the first year has been described by Shissler (21). No pronounced disagreeable odor was observed either within or without the house. Swarms of *Psychoda* flies developed inside the house, but few were observed outside. By closing the ventilators high temperatures within the house were secured, which were said to have a marked effect on destroying fly life.

The air from within the trickling-filter glass-over at Totowa, illustrated in Fig. 151, will be collected and discharged through a blower into a wooden flume, into which a small quantity of chlorine gas will be introduced. According to Capen (22), "a period of contact will complete the chemical combination of the chlorine with the organic substances responsible for odors before they are discharged into the atmosphere."

Final-sedimentation Tanks.—Sedimentation tanks which follow trickling filters commonly are given a detention period of ½ to 2 hours. Any one of the common types of sedimentation tanks may be employed and plants have been built with horizontal-flow, flat-bottom tanks as at Akron, Ohio; with horizontal-flow hopper-bottom tanks as at Worcester, Mass.; with vertical-flow tanks of the Dortmund type as at Fitchburg, Mass., and with tanks equipped with sludge-removal mechanisms, such as the Link-Belt at Durham, N. C., and the Dorr at the Southerly plant in Cleveland, Ohio. The principles of design do not differ materially from those applicable to preliminary-sedimentation tanks.

The quantity of sludge deposited in trickling-filter humus tanks varies from 300 to 800 gal. per mil. gal. of filter effluent, averaging about 500 gal. The water content averages about 92.5 per cent, varying from 90 to 95 per cent. Sludge is generally withdrawn daily from mechanically equipped tanks and weekly from tanks not so equipped. In some plants longer intervals elapse between cleanings and sludge is removed only when septicization or excessive accumulation of solids interferes with clarification. During unloading periods more frequent cleaning is essential.

The design characteristics of final-sedimentation tanks of various types are given in Table 91.

The sludge from humus tanks may be air-dried on sludge beds, as at Worcester, discharged into Imhoff tanks where the latter are provided for preliminary treatment, as at Fitchburg, or pumped to separate sludge-digestion tanks, as at Durham.

With frequent removal of sludge from humus tanks, the sludge does not contain gases of decomposition, as does Imhoff sludge. Hence there is no necessity for exercising the care in pumping humus sludge which is essential in dealing with Imhoff sludge, in order to prevent liberation of the gases. If humus sludge is discharged into Imhoff tanks for digestion, it is desirable to prevent breaking up the floc during pumping, but this is not such an important matter as in the case of activated sludge. However, it is desirable that the pumps which handle humus sludge be simple in construction and capable of passing solids of reasonable size. Centrifugal, diaphragm and plunger pumps are commonly used. The humus-sludge pump at Akron is a vertical centrifugal pump, with a capacity of 250 g.p.m. while pumping against a 53-ft. head.

Statistics of Trickling Filters.—Data for a number of trickling-filter installations are presented in Table 92.

TABLE 91.—DESIGN CHARACTERISTICS OF FINAL-SEDIMENTATION TANKS FOLLOWING TRICKLING FILTERS

Location	Akron, Ohio	Bloomington, Ill.	Decatur, Ill.	Delaware, Ohio	Durham, N. C.	Elgin, Ill.	Fitchburg, Mass.	Gloversville, N. Y.	Madison, Wis., Nine Springs plant	San Bernardino, Cal.	Worcester, Mass.
Sewage flow designed for, m.g.d.	33.0	5.6[1]	10.2[1]	1.45	2.0	5.14[1]	4.0	3.0	5.0	5.0	28.0
Type of tank	Plain flat-bottom	Dorr	Dorr	Dorr	Link-Belt	Dorr	Dortmund	Dortmund	Dorr	Dortmund-Hardinge	Plain hopper-bottom
Number of units	12	1	1	1	2	2	4	2	2	1	4
Inside dimensions, ft.	65 × 50	70 × 70	....	30 × 30	13 × 58	....	30 (diam.)	36 (diam.)	50 × 50	40 (diam.)	60 × 120
Maximum water depth, ft.	7.5	....	....	7	10	11.2	22.5	33.2	9.2	25	15
Total water capacity, cu. ft. per capita	1.22	1.15[1]	0.80[1]	0.41	1.5	0.46[1]	1.17	1.38	0.9	0.84	1.23
Average detention period, hr.	1.72	1.94[1]	1.15[1]	0.75	....	0.70[1]	2.21	2.12	1	....	1.4
Bottom slope	4 per cent	1 on 12	1 on 12	1 on 12	1 per cent	1 on 12	1 on 1½	1 on 1	1 on 12	....	23.3 per cent
Sludge removed by	Flushing, gravity and pumping	Pumping	Pumping	Pumping	Pumping	Pumping	Gravity and pumping	Gravity	Pumping	Pumping	Pumping
Type of sludge pump	Centrifugal	....	....	....	....	....	Centrifugal	....	....	....	Centrifugal
Sludge disposal	Discharged into Imhoff tanks or lagooned	Discharged into Imhoff tanks	Discharged into Imhoff tanks	Discharged into Imhoff tanks	Digested in separate tank	Discharged into Imhoff tanks	Discharged into Imhoff tanks	Dried on sand beds	Discharged into Imhoff tanks	Discharged into Imhoff tanks or to drying area	Dried on sand beds

[1] Operating data for 1927.

TABLE 92.—COMPARISON OF TRICKLING FILTERS

Location	Johnson City, Ill.	Rochester, N. Y., Brighton plant	Delaware, Ohio	Fostoria, Ohio	Gloversville, N. Y.	Elgin, Ill.	Fitchburg, Mass.	Madison, Wis., Nine Springs plant	Marion, Ohio	Champaign-Urbana, Ill.
Date of construction	1923	1915	1927	1928	1911	1926	1914	1927	1923	1924
Population designed for	3,000	10,000	14,500	15,000	20,000	37,500	40,000	40,000	40,000	45,000
Capacity designed for, m.g.d.	....	1	1.45	2.0	3.0	....	4.0	5.0	4	6.75
Area of filters, acres	0.34	1	0.55	0.85	3.1	1.53	2.1	2	1.4	1.6
Number of units	2	2	2	1	3	2	1	2	4	2
Character of filtering material	....	Limestone	Limestone and granite	Limestone	Limestone	....	Granite and trap rock	Dolomite limestone	Limestone	Limestone
Size of filtering material, in.	....	1	....	1½–2½	1–2½	....	1–2	½–5	1¼–2¾	....
Average depth of material, ft.	6.5	6	8	7	5	8.3	10–10.5	10	10	10
Method of distribution	Taylor circular-spray nozzles	Columbus nozzles	Nozzles	Taylor circular-spray nozzles	Taylor square-spray nozzles	Taylor circular-spray nozzles	Worcester circular-spray nozzles	Taylor nozzles	Circular-spray nozzles	Taylor circular-spray nozzles
Size of orifice of nozzle, in.	⅞	....	1	⅞	¾	1	1³⁄₁₆	....	1³⁄₁₆	1
Height of orifice above surface, in.	....	....	Flush	Flush	8	Flush	Flush	....	3	....
Distance center to center of nozzles, ft.	12	14.3	15	12.25	12	14	15	14	15	14
Number of nozzles per acre	378	226	....	300	303	247	230	286	223	247
Head on nozzles, ft.	5.0	6.0	8.0	6.0	6.3	8.3	8.5	8.5	8.2	8.0
Character of underdrainage system.	6-in. split tile	6-in. tile	Metro-block	Metro-block	6-in. slotted channel pipe on 3-in. concrete floor	Metro-block	Grooved concrete floor and blocks	Grooved concrete floor and blocks	Grooved concrete floor and blocks	Ridge-and-furrow
Type of nozzle field	....	....	Common	Common	....	Separate	Common	Common	Common	Separate
Dosing tanks:										
Type	Twin	Twin	Twin	Twin	Single	Twin	Twin	Twin	Twin	Twin
Number	2	2	2	2	1	2	2	2	2	2
Capacity per unit, gal.	2,150	....	19,400	8,300	16,700	13,237	40,000	26,000	38,500	6,038
Size of siphon, in.	8	20	20	20	30	24	30	24	30	20
Type of siphon	....	....	Deep-bell	Deep-bell	Deep-seal	....	Flap-valve	Deep-bell	Flap-valve	Trapless
Dose, gal. per acre	....	....	....	....	8,100	....	20,000	....	19,250	....
Dosing time, min.	8.2	....	....	2.5	3.5	4.2	5	....	5	2.4
Resting time, min.	12.4	....	....	3.5	....	22.8	....	....	....	12.7

TABLE 92.—COMPARISON OF TRICKLING FILTERS.—(*Continued*)

Location	Springfield, Mo., new Southwest plant	Pontiac, Mich.	Bloomington-Normal, Ill.	Decatur, Ill.	San Bernardino, Cal.	Canton, Ohio	Worcester, Mass.	Akron, Ohio	Baltimore, Md.
Date of construction	1928	1920	1927	1922	1928	1926	1925	1927	1911
Population designed for	47,500	52,000	54,000	60,000	60,000	135,000	242,000	260,000	600,000
Capacity designed for, m.g.d.	....	4	....	8.0	....	....	28	33	75–90
Area of filters, acres	2.5	1.74	2.5	3.0	2.5	6.8	14.0	13.7	30
Number of units	....	8	4	2	2	4	2	1	10
Character of filtering material	....	Stone	Stone	Stone	....	Slag	Trap rock and granite	Hard limestone	Limestone and trap rock
Size of filtering material, in.	....	....	....	....	1¾–2½	1½–2¾	1¼–3	1–2	1–2½
Average depth of material, ft.	6	6	8.5	5.7	7	7.5	10	10	8.5
Method of distribution	Taylor nozzles	Revolving distributors, ⅜-in. holes	Taylor circular-spray nozzles	Taylor circular-spray nozzles	Taylor nozzles	Taylor circular-spray nozzles	Worcester nozzles	Circular-spray nozzles	Merritt and Taylor nozzles
Size of orifice of nozzle, in.	....	....	⅞	⅞	⅞	⅞	13/16	0.906	Approx. ¾
Height of orifice above surface, in.	Flush	....	....	....	Flush	....	Flush	2	8
Distance center to center of nozzles, ft.	....	....	14	13.25	11.25	15	15	11	15–21
Number of nozzles per acre	....	....	238	300	....	....	217	300	194
Head on nozzles, ft.	....	1.2	8.75	5.6	....	....	7–1	5.93–1.39	8–1
Character of underdrainage system	Tile	....	Vit. clay blocks on ridge-and-furrow	Shale bricks on ridge-and-furrow	Plymouth underdrain pipes	....	Grooved concrete floor and blocks	Metro-block	Tiles on grooved floor
Type of nozzle field	Separate	....	Common	Separate	Separate	Common	Common	Common	....
Dosing tanks:									
Type	Twin	....	Twin	Single	Twin	Twin	Twin	Twin	Butterfly valve
Number	2	....	4	2	2	4	8	14	
Capacity per unit, gal.	25,500	....	18,500	15,606	13,750	18,600	36,930	23,000	
Size of siphon, in.	30	....	30	30 and 20	20	30	36	36	
Type of siphon	Trapless	....	Deep-bell	Trapless	Trapless	Flap-valve	Deep-bell	Deep-bell	
Dose, gal. per acre	....	....	....	....	....	....	21,000	11,700	
Dosing time, min.	....	....	8.1	3.4	....	....	3	2	
Resting time, min.	....	....	29.7	6.8					

Cost of Construction, Operation and Maintenance of Trickling Filters.—The unit cost of trickling filters may be expressed in dollars per acre, per acre-foot, per million gallon daily of sewage flow or per capita provided for in design. The number of acre-feet in a filter is the product of the area in acres and the depth of stone in feet. Costs per acre-foot, based on prices prevailing from 1925 to 1929, vary from $10,000 for large plants to $15,000 for small plants. The cost per million gallon daily varies from $30,000 to $50,000 and the cost per capita from $3 to $6.25. Important items affecting the cost are: depth and character of excavation, pumping requirements, depth of filter, character of underdrainage system, type of distribution system, provision of division walls, design of outside walls and cost of filter stone. The last is the largest single item of cost in a trickling filter. Filter stone may range in cost from $2 to $3.25 a cubic yard before placing in the filter, with an additional cost of $0.15 to $1.00 for placing. Rescreening at the treatment plant to remove dust and chips costs 10 to 15 cents a cubic yard. Filter floors vary in cost from $5 to $7 a square yard.

The cost of operating trickling filters depends upon the requirements for pumping, cleaning nozzles, relieving surface clogging by chlorine application or turning over the top stone, cleaning clogged beds by removing, screening, washing and replacing the media and maintenance of dosing tanks. Few data are available showing actual costs of operation of trickling filters alone. Laborers engaged in cleaning nozzles are employed on other parts of the plant as well and, unless some sort of time-clock or time-card system is used, the time allocated to the trickling filters alone is an estimate at best. From such data as are available it appears that operating costs range from $0.70 to $4.30 per million gallons treated and from 2.2 to 6.3 cents annually per capita.

The cost of excavating, cleaning and replacing stone in the trickling filters at Fostoria, Ohio, during 1929 was $1.25 a ton.

Bibliography

1. *Proc.*, A. S. C. E., 1929; **55,** 1799.
2. Hommon, C. C.: "Rept. of Investigation of Crushed Blast Furnace Slag as a Medium in Sewage Trickling Filters," 1930.
3. Havens, W. L.: *American City*, 1928; **38,** no. 3, p. 78.
4. Stanley, W. E.: *Sewage Works Jour.*, 1930; **2,** 486.
5. Ann. Rept., Mass. State Board of Health, 1908.
6. Dunbar: "Principles of Sewage Treatment," 1908.
7. Clifford, William: *Proc.*, Inst. C. E., 1908; **172,** part 2, p. 283.
8. Mahlie, W. S.: *Proc.*, A. S. C. E., 1929; **55,** 1405.
9. Ann. Rept., Mass. Dept. of Pub. Health, 1928.
10. Ann. Rept., Mass. Dept. of Pub. Health, 1920.
11. *Bull.* No. 26, Illinois State Water Survey, 1928.

12. Ann. Rept., Comm. of Pub. Works, Fitchburg, Mass., 1920.
13. Greeley, S. A., and W. D. Hatfield: *Trans.*, A. S. C. E., 1930; **94,** 544.
14. Ann. Rept., Ill. Dept. of Pub. Health, 1926–27.
15. Shephard, W. F.: *Sewage Works Jour.*, 1929; **1,** 486.
16. *Eng. News*, 1913; **70,** 1112.
17. Potter, Alexander: *Eng. News*, 1916; **75,** 1028.
18. Nelson, F. G.: *Sewage Works Jour.*, 1928; **1,** 64.
19. *Bull.* No. 20, Eng. Experiment Sta., Purdue Univ., 1925.
20. Catalog 30, Pacific Flush-Tank Co.
21. Shissler, Ed.: *Proc.*, N. J. Sewage Works Assoc., 1930, p. 19.
22. *Public Works*, 1928; **59,** 360.

CHAPTER XXIII

AUTOMATIC DOSING APPARATUS

The dosing cycles of sand filters and contact beds can be controlled readily by hand. The dosing of trickling filters, however, is usually made automatic. In considering the advisability of automatic control for sand filters and contact beds, important items to be remembered are 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 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 weaker sewage, are called upon to do less work. In most cases, however, the fluctuations in flow from day to day cause more or less change in the hourly cycle and, under such conditions, automatic apparatus for dosing probably insures better work on the part of the beds and less expense for caretakers than the operation of the control gates by manual labor. In large plants, on the other hand, 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.

Generally speaking, there are three classes of apparatus for automatic dosing of sewage filters, namely, air-controlled siphonic apparatus, mechanically controlled siphonic apparatus and mechanical devices. For use with intermittent sand filters and trickling filters these devices are installed only for the purpose of applying sewage to the beds. In contact beds they are also used for emptying the beds after a definite period of contact has elapsed.

Air-controlled Siphonic Apparatus.—Both trapless and deep-seal types of siphon have been used. There may be considerable advantage in the trapless type, particularly in the larger sizes. The coefficient of discharge for trapless siphons remains practically constant regardless of size, whereas the coefficient for the deep-seal type, which has three abrupt 90-deg. turns, decreases with the increase in the size of siphon. The larger the siphon of the deep-seal type, the greater is the loss of head. The largest coefficient for a deep-seal siphon in no case equals the smallest coefficient for a siphon of the trapless type, the nearest approach being in 6-in., 8-in. and 10-in. sizes, where the coefficient for the deep-seal is almost equal to the coefficient for the same sizes of trapless siphon. Consequently, for these three sizes the deep-seal type can be used without making compensation for the additional loss, which does not exceed 3 per cent of the total head.

Siphons for use with trickling filters are equipped with adjustable blowoff traps, so that the maximum discharge elevation in the dosing tank can be varied within certain limits, to make adjustments in the nozzle distribution in case such is necessary.

The features of the deep-seal type of siphon are shown in Fig. 152. Its principal parts 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

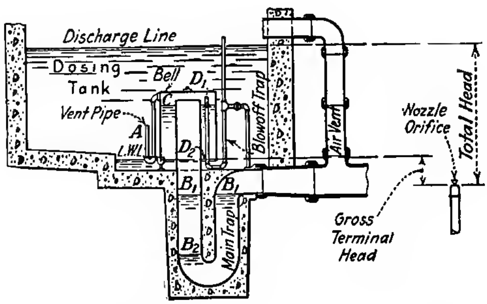


FIG. 152.—Sewage siphon with adjustable blowoff trap, deep-seal type. (*Miller.*)

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.

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

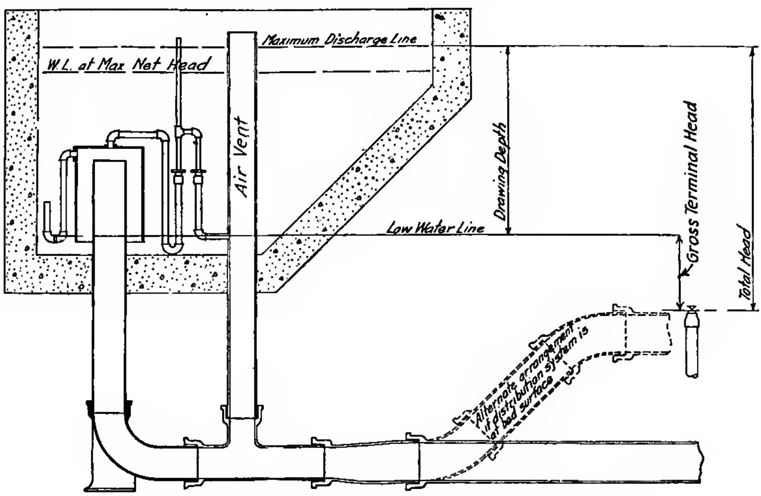


FIG. 153.—Trapless siphon. (*Pacific Flush-Tank Co.*)

bed and arranged to discharge in rotation automatically. Two siphons of the type illustrated by Fig. 152, 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.

Siphonic equipment of the trapless type is shown in Fig. 153 for two types of construction, one where the distribution system is at the bottom of the bed and the other where it is at the surface of the bed (1). In the latter case, there is a gradual slope from the bottom of the siphon up to the distribution piping at the bed surface.

Two types of controlling equipment are adapted to use with twin dosing tanks and common nozzle fields in trickling filters. One of these is the deep-bell type, shown in Fig. 144, in which there is a direct connection between the siphon and distribution system. This type is not adapted to plants where the maximum net head[1] is less than 4½ ft.

The other type of apparatus for use with common fields is the flap-valve type, shown in Fig. 154, which was developed and used prior to the deep-bell type (1). It will be noted that the siphons discharge into a common chamber and that the discharge end of each siphon has a balanced flap valve to prevent the discharge of one siphon from backing up in the idle siphon. Generally this type requires more head than the deep-bell type, as there are losses of head in the flap valve and discharge chamber.

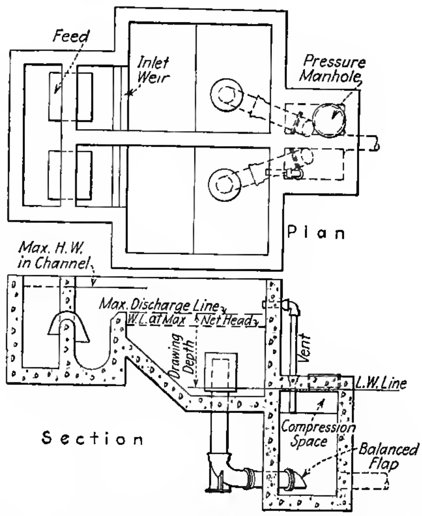


Fig. 154.—Flap-valve type of twin dosing-tank equipment for common nozzle field. (*Pacific Flush-Tank Co.*)

Siphonic equipment for separate nozzle fields is illustrated in Fig. 155 (1). The separate-field equipment has all the advantage of the controlled-inflow feature and it is also possible to make direct connection from the siphon to the distribution system.

The operation of the twin dosing-tank equipment is not complicated and it is positive in action. In Fig. 155 it will be noted that there is a vertical pipe rising from the main discharge line and connecting to the compression chamber. When the siphon starts, a part of the discharge is introduced through the vertical pipe into the compression chamber and as the latter is airtight, a pressure is developed, which is transmitted by means of air piping to the operating feed, creating an air lock and stopping the flow into the dosing tank. A small air bell is located in the compression chamber and the pressure from this bell is transmitted to a

[1] Differential head between nozzles and water level in dosing tank, when the siphon is in full operation and the maximum head on the nozzles is reached.

blowoff trap connected to the feeds. When the seal of this trap is forced, the air lock is released and the feed is started. The siphons are started in the usual manner by forcing the seal of a blowoff trap connected either to the main siphon or to an auxiliary starting bell suspended in the dosing tank.

The discharge of siphons can be represented by the common orifice formulas, $Q = C_s a_s \sqrt{2gh_s}$. Here h_s represents the head causing flow, a_s the area of the siphon and Q the rate of discharge. For free discharge this head is measured from the surface of the sewage in the tank to the

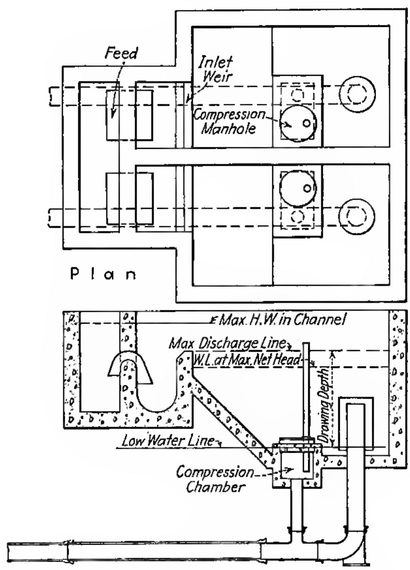


FIG. 155.—Twin dosing-tank equipment for separate nozzle fields. *(Pacific Flush-Tank Co.)*

center of the outlet from the main trap. When the discharge is not free, for example, when sewage is discharged into the distribution system of a trickling filter, h_s is the head lost in the siphon. The coefficient of discharge, C_s, is approximately 0.6. In emptying sewage from a tank, however, there is a delay in bringing the siphon into full operation and in overcoming the inertia of the sewage in the distribution system. During this time there is a fall of sewage level in the tank. This effect produces what is called the "dosing-tank loss" which, according to investigations of the Pacific Flush-Tank Co., is measured approximately by the relation

$$h_d = 364\frac{a_s^2}{AQ}$$

where h_d = dosing-tank loss, feet
Q = rate of discharge, cubic feet per second
A = area of dosing tank at maximum level, square feet
a_s = area of siphon, square feet.

To determine the characteristics of the dosing device, therefore, both these relations are taken into account, as illustrated in Chap. XXII. Where a single tank is used, allowance also is made for the volume of sewage flowing into the tank while the siphon is discharging.

Air-lock Feeds.—Figure 156 shows in outline the general features of an assembly of siphons and air-lock feeds for four contact beds. At the

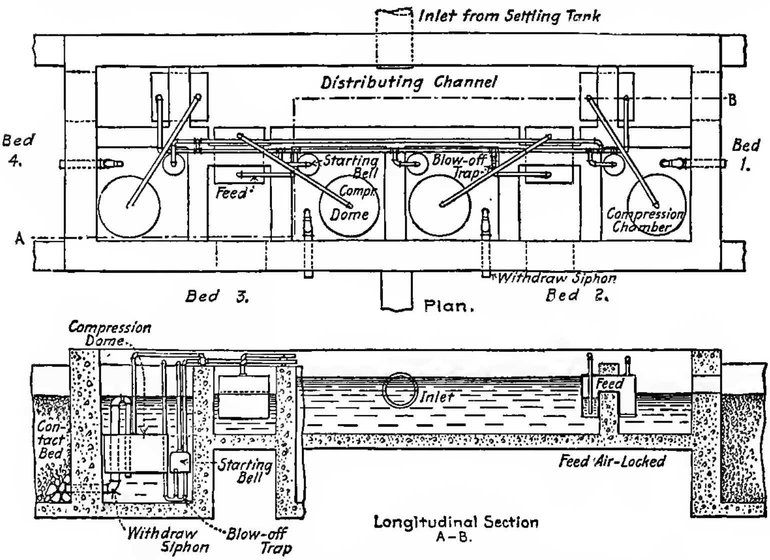


FIG. 156.—Air-lock feeds for contact beds. (*Miller-Adams.*)

beginning of a cycle of operation the wells in front of the air-lock feeds are filled with water, except the one that is to operate first. All the blowoff traps are filled with water.

Sewage entering the channel from the settling tanks flows through the feed 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 the first feed is air-locked, the seal in the second blowoff trap is forced, thus releasing the air confined in the second feed and allowing the sewage to discharge into the second contact bed. 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,

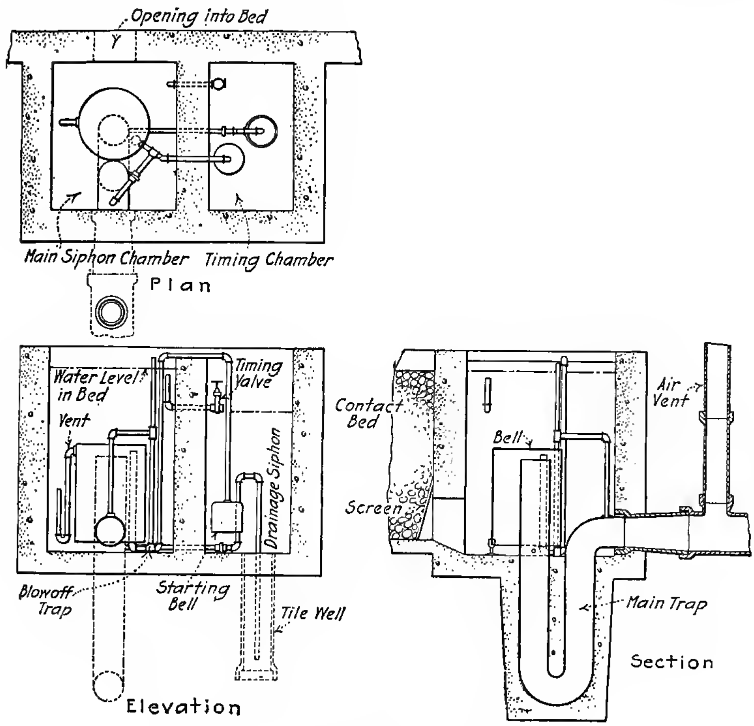


FIG. 157.—A timed siphon. (*Miller.*)

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.

Hydraulically, feeds may be considered as siphons and the discharge through them may be expressed by the orifice formula, $Q = C_f a_f \sqrt{2gh_f}$. On the assumption that coefficient $C_f = 0.7$, the head required to operate a 6-in. air-lock feed 1 ft. in length is given by the following relationship,[1] $h_f = 0.127Q^2$, where h_f is expressed in feet and Q in cubic ft. per second.

Timed Siphons.—To render the operation of contact beds completely automatic they require apparatus controlling the time the sewage stands in the beds. Timed siphons are frequently used for this purpose. The general details of the apparatus are shown in Fig. 157.

At the start, the main trap, blowoff trap and tile well in the timing chamber are filled with water. The size of the timing chamber and the size

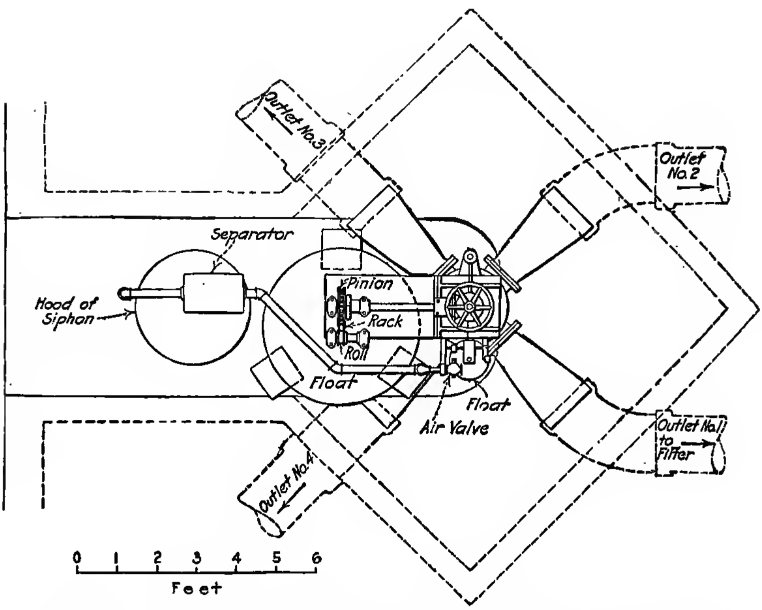


FIG. 158.—Plan of Barbour dosing apparatus at North Attleborough, Mass.

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

[1] The Pacific Flush-Tank Co. in its Catalog 30 gives losses of head for air-lock feeds which may be represented by the equation, $h_f = 0.129Q^{1.64}$.

bell is vented, the timing chamber is emptied and the apparatus is ready for the next filling of the bed.

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, in addition to the depth of the bed.

Mechanically Controlled Siphonic Apparatus.—Several kinds of mechanical devices are used for directing the discharge of siphons to one pipe after another. Figures 158 and 159 illustrate one used at a number of sewage-treatment works by Barbour.

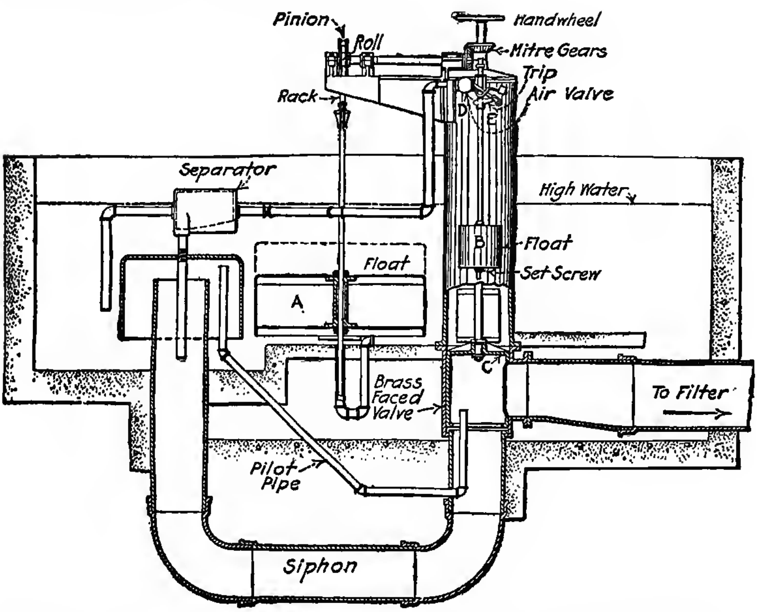


FIG. 159.—Sectional elevation of Barbour dosing apparatus at North Attleborough, Mass.

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 discharge ceases. When float A falls with the sewage level. the cylindrical gate valve is not turned back, but is held in

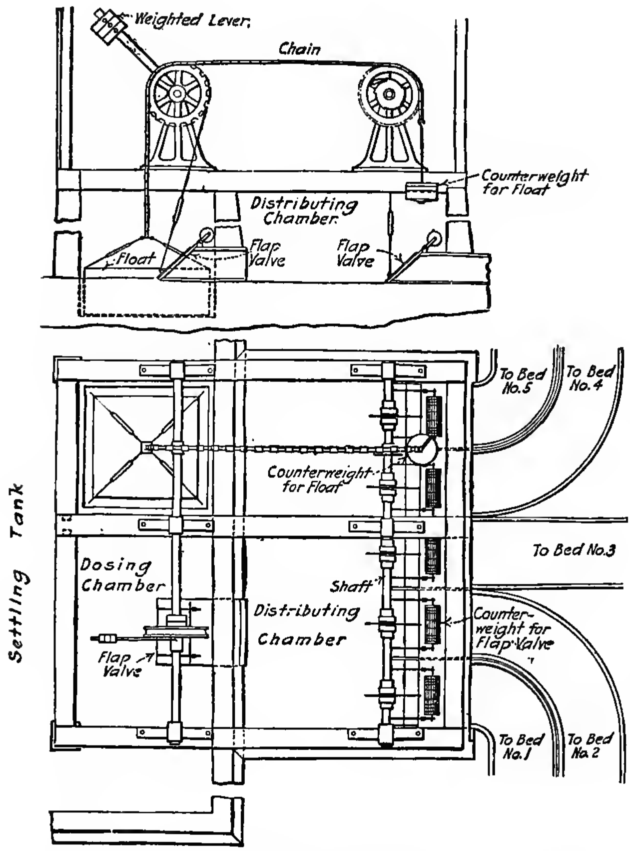


FIG. 160.—Dosing apparatus at Newton, N. J. (*Ansonia.*)

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. 160. 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 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 revòlves 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 a pawl and ratchet. The five flap valves leading to the five filter beds are 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 is necessary 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.

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.

One of the largest filter plants employing this means of regulating head is that at Baltimore, Md., where the head on the nozzles runs from almost zero to 9 or 10 ft. The installation is shown diagrammatically in Figs. 161 and 162. 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 beds. The opening to each of these castings is controlled by an electrically operated sluice gate. Each motor is connected to an autostarter 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 autostarter is raised sufficiently to trip the latter and open a sluice gate on the inlet line to section 3 of the trickling filters.

Inside 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, or 2 cycles, approximately every 7 min., thus alternately

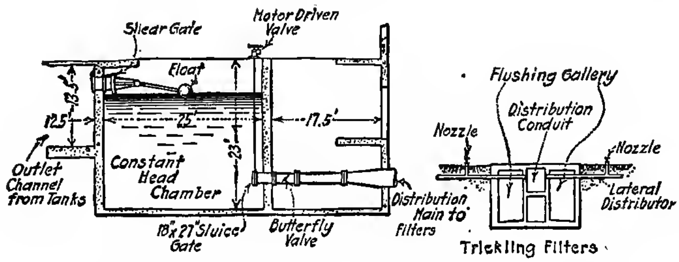


FIG. 161.—Sectional elevation of dosing apparatus, Baltimore.

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. from the nozzle, at which moment the flat plate in the casting has turned till it is in a horizontal position and the maximum volume of sewage is flowing.

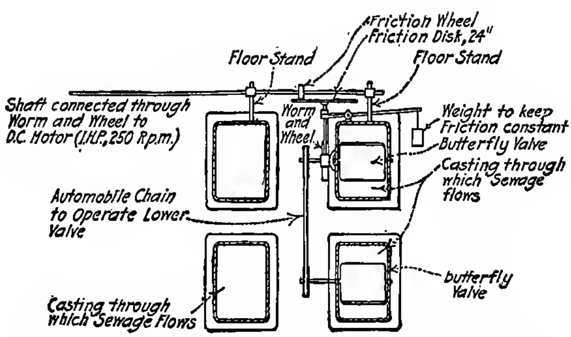


FIG. 162.—Operating device for butterfly valve, Baltimore.

Among other trickling filter installations which include butterfly valves in the dosing equipment are those at Brockton, Mass., and at the Pennypack Creek plant in Philadelphia, Pa.

Dosing Apparatus for Intermittent Sand Filters.—The types of apparatus commonly used for dosing intermittent sand filters include air-controlled siphonic apparatus, mechanically controlled siphonic apparatus and mechanical devices. These appliances have already been discussed.

It is desirable that siphons for dosing intermittent sand filters have sufficient discharging capacity at the minimum head to empty the dosing tank, even under conditions of maximum rates of inflow from the trunk sewer. If such maximum rates are higher than it is feasible to provide for, the balance above a predetermined rate may be by-passed or the siphon may be allowed to go into continuous operation. Operating continuously, the siphon will discharge at a maximum rate dependent upon the maximum head available. Table 93, prepared by the Pacific Flush-Tank Co., is a guide in the selection of a siphon to cover this contingency. Fig. 163 indicates the dimensions referred to as "minimum" and "maximum heads." The minimum head can usually be varied only a slight amount for one size of siphon, but the maximum head can be varied to suit the conditions of each case. Table 94, also prepared by the Pacific Flush-Tank Co., gives the rates of discharge for siphons discharging freely, that is, into an open trough or similar unrestricted channel.

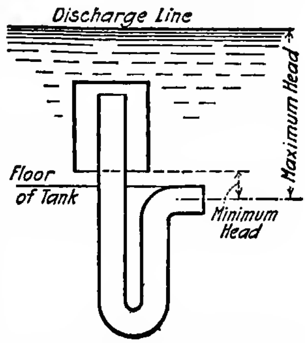


Fig. 163.—Relation of maximum and minimum heads on a siphon.

TABLE 93.—SIZES OF SIPHONS TO BE USED WITH VARIOUS RATES OF INFLOW

Max. inflow in gallons per 24 hr.	Diameter of siphon, in.	Minimum head, in.	Discharge at minimum head, gal. per min.
Up to 100,000	5	6	160
100,000–135,000	6	6½	230
135,000–280,000	8	7½	450
280,000–480,000	10	9½	800
480,000–720,000	12	10½	1,200
720,000–1,070,000	14	13	1,780
1,070,000–1,465,000	16	14	2,440
1,465,000–1,925,000	18	15	3,210
1,925,000–2,540,000	20	17	4,230
2,540,000–3,860,000	24	19	6,430
3,860,000–6,360,000	30	22	10,600

Table 94.—Rates of Discharge in Gallons per Minute for Siphons Discharging into an Open Trough

Head	Diameter of siphon, in.										
	5	6	8	10	12	14	16	18	20	24	30
1'3"	255	345	640	1000	1420	1950	2560	3210	3,960	5,720	8,950
1'6"	280	375	690	1060	1545	2120	2770	3485	4,320	6,210	9,720
1'9"	300	410	750	1180	1675	2300	3010	3780	4,680	6,740	10,530
2'0"	320	435	800	1260	1785	2450	3210	4030	5,000	7,200	11,250
2'3"	340	460	850	1340	1900	2605	3410	4290	5,320	7,650	11,970
2'6"	360	490	900	1420	2000	2760	3610	4530	5,580	8,060	12,650
2'9"	380	510	945	1490	2100	2890	3780	4750	5,900	8,450	13,230
3'0"	395	535	985	1550	2200	3010	3950	4960	6,120	8,850	13,830
3'3"	410	555	1020	1600	2275	3130	4100	5150	6,400	9,200	14,350
3'6"	425	575	1060	1675	2370	3250	4260	5350	6,620	9,550	14,930
3'9"	440	590	1100	1730	2455	3370	4410	5540	6,880	9,900	15,470
4'0"	455	620	1140	1790	2540	3470	4550	5720	7,060	10,210	15,960
4'3"	470	635	1170	1840	2610	3580	4690	5900	7,300	10,520	16,440
4'6"	485	650	1200	1880	2670	3690	4820	6070	7,510	10,800	16,900
4'9"	500	670	1235	1950	3760	3790	4960	6240	7,700	11,120	17,380
5'0"	510	690	1270	2000	2840	3890	5100	6410	7,920	11,430	17,870
5'3"	520	710	1300	2050	2900	3980	5220	6550	8,100	11,700	18,270
5'6"	535	725	1335	2100	2980	4080	5340	6720	8,320	11,980	18,740
5'9"	545	740	1365	2145	3045	4170	5460	6860	8,500	12,250	19,120
6'0"	555	755	1395	2190	3100	4270	5570	7000	8,650	12,500	19,530
6'3"	570	770	1420	2230	3165	4340	5680	7150	8,820	12,750	19,930
6'6"	580	780	1450	2280	3230	4430	5800	7300	9,000	13,000	20,340
6'9"	590	800	1475	2325	3290	4520	5910	7440	9,190	13,260	20,720
7'0"	600	815	1500	2365	3340	4600	6020	7570	9,360	13,500	21,130
7'3"	610	830	1525	2395	3400	4680	6130	7690	9,500	13,700	21,430
7'6'	620	845	1555	2440	3470	4760	6230	7830	9,680	13,970	21,850
7'9"	630	860	1580	2480	3520	4840	6330	7930	9,820	14,190	22,200
8'0"	640	875	1610	2525	3590	4920	6430	8070	10,000	14,450	22,580
8'3"	650	885	1630	2560	3640	5000	6530	8200	10,180	14,640	22,890
8'6"	660	900	1660	2600	3700	5080	6640	8310	10,350	14,900	23,290
8'9"	670	910	1680	2640	3740	5160	6730	8430	10,500	15,100	23,600
9'0"	680	920	1700	2675	3790	5240	6820	8550	10,630	15,300	23,930

Dosing Apparatus for Contact Beds.—For the automatic control of contact beds, two sets of apparatus are required, one for filling the beds with sewage and the other for emptying the beds after definite periods of contact. For filling the beds, alternating siphons in dosing tanks may be used if sufficient fall is available. In this case the apparatus is similar to that described for dosing sand filters. The dosing tank is generally given a capacity equal to the voids in the contact bed. Where there is only a limited head available, air-lock feed apparatus has been found satisfactory. Various mechanical devices have also been developed, working in conjunction with siphonic apparatus, to fill and empty contact beds. For discharging sewage from the beds after contact, timed siphons have been used extensively.

The automatic apparatus for dosing contact beds may be grouped as follows: air-lock siphonic apparatus, mechanically controlled siphons and miscellaneous mechanical devices. All three types of apparatus have been discussed above.

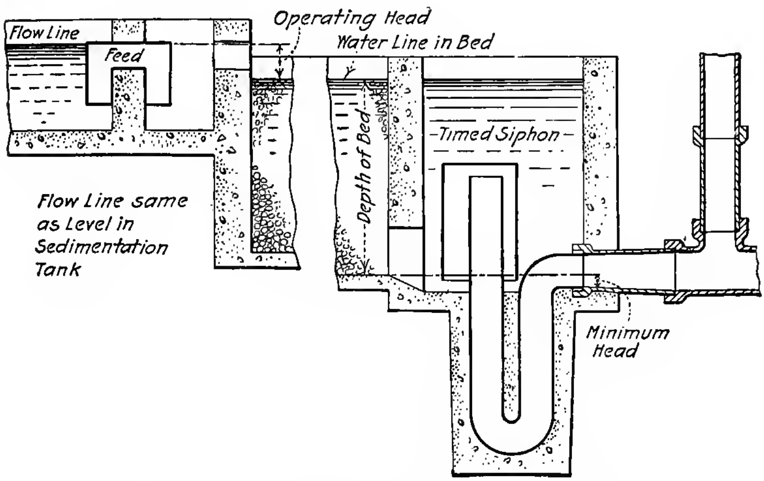


FIG. 164.—Loss of head with feeds and timed siphons.

The head required for contact-bed apparatus is often a vital feature and in this connection the figures given in Table 95, prepared by the Pacific Flush-Tank Co., are useful. Figure 164 indicates the dimensions referred to in the table.

Instead of building the filling apparatus at one end of the contact bed and the discharging apparatus at the other end, it is often more satisfactory to combine the two devices and locate all the control apparatus for the entire plant together in a single building.

Dosing Apparatus for Trickling Filters.—The design of adequate dosing apparatus for trickling filters is closely allied with the design of the distribution system. With the various forms of traveling distributors used extensively abroad no additional dosing apparatus is required, so long as the sewage can be delivered to the distributor by gravity. In the United States, however, pressure nozzles have been used commonly in preference to traveling distributors and the resulting

TABLE 95.—SIZES OF FEEDS AND TIMED SIPHONS FOR VARIOUS RATES OF INFLOW

Maximum inflow, gal. per 24 hr.	Size of Miller-Adams feeds required, in.	Operating head, in.	Depth of contact beds, ft.	Size of Miller timed siphons required, in.	Minimum head, in.
5,000 250,000	6 × 10	9	3.5–4.0	5– 6	2½
250,000 400,000	6 × 18	9	3.5–4.0	6– 8	2½
400,000 750,000	8 × 18	11	4.0–5.0	8–10	2½
750,000 1,000,000	10 × 24	13	4.0–5.0	10–12	2½
1,000,000 2,000,000	12 × 30	15	4.0–5.5	12–14	2½
2,000,000 4,000,000	15 × 36	18	4.0–6.0	14 up	2½

complications necessary to obtain uniform distribution of the sewage have brought out a number of devices for controlling the flow. Some of these are modifications of methods used in sand-filter or contact-bed installations, while others are entirely new.

The apparatus may be classified as follows: siphonic apparatus, head-varying valves and dosing tanks. This equipment has been described in this chapter and the hydraulic computations involved in the design of dosing equipment for trickling filters have been exemplified in Chap. XXII.

Bibliography

1. Catalog 30, Pacific Flush-Tank Co.

CHAPTER XXIV

THEORY AND OPERATION OF THE ACTIVATED-SLUDGE PROCESS

As was the case with filtration, two opposing theories of the manner in which activated-sludge treatment of sewage acts have been advanced. One stresses the physical factors; the other, the biological factors. In general, the changes accomplished are probably closely akin to those brought about by filtration.

Fundamental Principles of the Process.—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 sulfate of alumina or sulfate of iron and lime and the floc resembles the chemical coagulum, particularly the ferric hydrate from ferrous iron and lime treatment.

The floc is a sponge-like mass, or, as expressed by Stein, "an open-mesh network" which, in the process of formation, may envelop, entrap or entrain colloidal matter and bacteria (1). The sponge-like structure of the floc offers a large surface area for contact and appears to be able to adsorb colloidal matter, gases and coloring compounds. Buswell and Long (2) have estimated that the sludge surfaces present an area of 500 sq. ft. per cubic foot of tank volume. When the floc is driven about in the liquid, it has a sweeping action by which the colloidal substances may be said to be swept out of the sewage, or, as stated by Parker (3), the "process may be regarded as passing a filter through the water in place of passing the water through a filter."

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 finely divided matter and thus relieve the floc of its burden of such substances and restore its power 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 also will be consumed by the living organisms which thus cause regeneration of the floc. It is because of these properties that the sludge has come to be called "activated sludge," a term suggested by Ardern and Lockett (4).

Buswell and Long (2) have proposed the following statement of the biological theory of the activated-sludge process:

Activated-sludge flocs are composed of a synthetic gelatinous matrix similar to that of *Nostoc* or *Merismopedia*, in which filamentous and unicellular bacteria are imbedded and on which various protozoa and some metazoa crawl and feed. The purification is accomplished by ingestion and assimilation by organisms of the organic matter in the sewage and its resynthesis into the living matter of the flocs. This process changes organic matter from colloidal and dissolved states of dispersion to a state in which it will settle out.

Harris, Cockburn and Anderson (5) attribute mainly to enzymes the changes in character of sewage effected by the process.

Regarding the mechanism of the activated-sludge process we are confident that the biochemical changes brought about in sewage matter must be attributable to enzymic rather than to direct bacterial action. The rapidity with which it is possible to effect these changes, the disproportion between cause and effect that existed in some of our experiments, and the relationship between time and percentage of sludge required, fall into line with the experimental evidence of the action of proteolytic enzymes.

What part, if any, purely physical action plays in the activated-sludge process is, in the present state of our knowledge, very indefinite. Many observers ascribe the coagulation of sewage colloids to the physical process of adsorption, a process admittedly characterized by rapidity of effect. Our observations tend to negative this view, as we have invariably found that the separation of the colloidal matter in the Shieldhall sewage necessitates an appreciable period of contact.

One of the cardinal principles of the activated-sludge process is that there must be dissolved in the sewage an ample supply of 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 combus-

TABLE 96.—DEVELOPMENT OF NITROGEN CYCLE IN EXPERIMENTAL TANK AT MILWAUKEE, JUNE 25, 1915

	Hours of aeration	Air per gallon, cu. ft.	Bacteria removed, per cent	Nitrogen as		
				Free ammonia, p.p.m.	Nitrite, p.p.m.	Nitrate, p.p.m.
Raw sewage.....	0	0	..	12.6	0.30	0.54
Effluent.........	1	0.36	90	9.6	0.20	2.48
Effluent.........	2	0.71	96	5.1	0.47	5.20
Effluent.........	3	1.08	95	1.2	0.60	8.80
Effluent.........	4	1.42	..	0.0	0.10	11.60

tion being carried off in the form of dissolved or gaseous matter. This process, under favorable conditions, may extend to nitrification, by which considerable quantities of nitrates and nitrites are formed. Substantial nitrification, however, is not necessary to the maintenance of sludge activity.

Data regarding the development of the nitrogen cycle in an experimental tank at Milwaukee, Wis., are given in Table 96 (6).

The importance of the presence of activated sludge in the aerating tank is shown in Fig. 165, giving the results of parallel experiments made by Bartow and Mohlman (7) at the University of Illinois. In one case

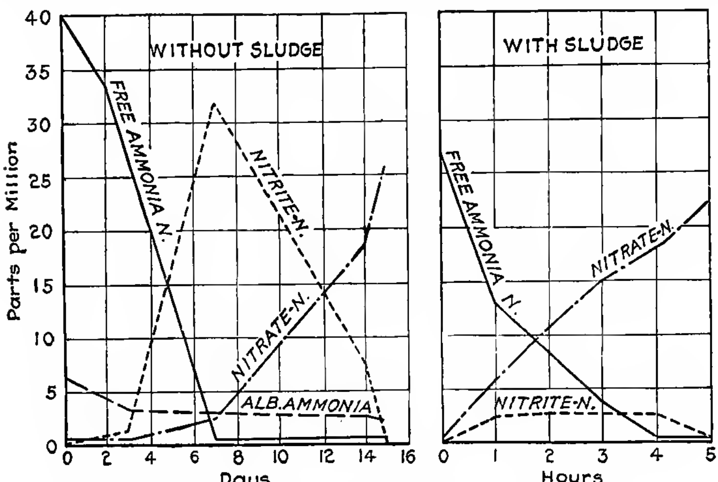


FIG. 165.—Effect of aeration of sewage with and without sludge.

sewage was aerated without sludge and in the other with sludge. In both cases nitrification followed the nitrogen cycle, but without sludge the completion of the cycle was a matter of days and the stages of change were distinct, while with activated sludge the cycle was completed in a few hours and the nitrite nitrogen was oxidized at once to nitrate nitrogen.

It is evident from the foregoing that the manner in which the activated-sludge process functions still is understood incompletely. It is probable that, under the common conditions of sewage treatment by this process, physical, physicochemical and biological action all play a part. Helpful in obtaining a mental picture of the workings of the process are the ideas incorporated by Parsons (8) in a paper in which he stresses differentiation between the three stages of activity which are evident in the course of the process, namely, clarification of the incoming sewage, reactivation of the sludge, or restoration of its powers of clarification, and nitrification of the sewage and sludge. That there are stages of activity was recognized by Ardern and Lockett (4) in their early work.

The clarification stage is confined principally to the first hour or two of aeration. It is chiefly physical in nature and produces the bulk of the improvement in the sewage effected by the process. There are two requisites, namely, an active sludge added to the sewage and circulation of this sludge through the sewage. Aeration, according to tests by Parsons and Wilson (9), is not essential during this stage. Buswell (10), too, has shown that stirring is more important in the clarification stage than oxygen and that activated sludge will function at oxygen levels so low that they can scarcely be measured. Grant, Hurwitz and Mohlman (11), on the other hand, have found that, given a chance to absorb oxygen, the floc will take up oxygen at a fairly uniform rate throughout the common aeration period. This observation does not imply, however, that oxygen is essential to the first stage.

Reactivation, or restoration of the powers of clarification, the second stage of the process, and nitrification, the third stage, may go on simultaneously or nearly so. The second stage is essential to the maintenance of the first, while the third stage, as previously stated, may be omitted. During these stages changes appear to be brought about chiefly by biological agencies and an ample supply of oxygen is one of the essential requirements. So far as improvement in the character of the sewage is concerned, the first stage is the most important. For the maintenance of sludge activity, however, the subsequent stage is cardinal, since upon this factor hinges the whole activated-sludge process. The second and third stages, depending as they probably do upon biological activity, are naturally more protracted. Hence the common periods of aeration reach 4 hr. or more, of which only the first and possibly the second hours are essential for clarification. It would be incorrect to surmise, however, that the fresh sewage solids entering an aeration unit are converted into activated sludge during a single continuous passage through the unit. If, for example, it is assumed that return sludge to the extent of 25 per cent of the sewage flow is kept in service, while the excess sludge which is withdrawn from circulation amounts to 1.35 per cent of the sewage treated, then the average period during which a sludge particle remains in circulation, neglecting losses by bacterial digestion, is $25/1.35 = 18.5$ times the detention period. The smaller the proportion of return sludge, the shorter is the period of circulation and vice versa; and there is conceivably an optimum period, the determination of which awaits further investigation.

The second and third stages may be accomplished in the aeration units, *i.e.*, with the sludge, whose clarifying or active powers are to be restored, in contact with the sewage, or separate tanks may be provided for the purpose of aerating the sludge settling from the treated sewage. The latter practice is known as reactivation or better "separate sludge re-aeration." Both practices are in use in America and abroad. Their relative values are discussed later in this chapter. The staging of the process also explains the use of short aeration periods, such as 1 hr., prior to treatment of the sewage on trickling filters. This is known as "bioflocculation," a practice which receives attention on page 571 of this chapter. Finally, a practice known as "stage treatment" is also based upon an appreciation of these principles.

Preliminary Treatment.—Grit and other heavy materials are generally removed from sewage which is to be treated by the activated-sludge process, because these substances are not readily maintained in suspension and tend to weigh down the sludge and cause it to settle. Their removal reduces accumulation and septic decomposition of sludge on the tank bottom and thus reduces obstruction to the passage of air from the diffuser plates. It is best, too, to keep bulky solids and floating matters out of aeration units, as they do not respond readily to treatment, tend to settle on the bottom or float in unsightly collections on the surface, and affect adversely the quality of the sludge settled from the tank effluent. Oil seems to interfere seriously with aeration in all types of activated-sludge units.

Before fine screening was applied to the raw sewage prior to treatment by the activated-sludge process at the Milwaukee experimental plant, quantities of sand, water-logged wood, waste and other coarse materials gathered on the plates. Copeland (12) states that "the deposits of sludge upon the Filtros plates forced the air to burst from the restricted open areas of the plates at high velocity. The air rose in geysers of large bubbles and much escaped without doing the sewage any good."

Ardern (13) has stated, referring to the experimental fill-and-draw tank at the Davyhulme works in Manchester, England:

The experience gained with this tank has also demonstrated:

1. The necessity for effective removal of excessive quantities of oily matter (trade discharges) from the sewage, which is liable to have an emulsifying effect and thus prevent satisfactory clarification.

2. The importance of adequate removal of heavy suspended matter such as grit, sand, etc., or otherwise difficulty may be experienced in the accumulation of the sludge, with the likelihood of the formation of deposits of de-aerated sludge which undergo secondary decomposition and thus produce conditions inimical to the complete success of the process.

3. The advantage of a filtered air supply in obviating trouble with diffusers.

4. In the case of certain industrial sewages the necessity for the provision of a re-aeration tank for economy in the maintenance of the process.

Aside from the removal of those matters which interfere with effective operation of the treatment process, it may be desirable under some conditions to remove also the readily settling solids from the sewage prior to aeration, in order to secure a reduction in air requirements or, during the period of aeration, a reduction in the bulk of the sludge to be disposed of and a reduction in the proportion of those solids which are of little fertilizing value. Preliminary-treatment works, furthermore, may be planned for the handling of sewage from combined systems so as to permit partial treatment of the entire flow received and complete treatment of the dry-weather flow only or a limited quantity in excess of it.

Sometimes the lower density of activated sludge produced from settled sewage may be disadvantageous, as it may be more difficult to dewater such dilute sludge. English experience seems to show that such trouble may be avoided, if aeration of settled sewage is carried to a point where the effluent is well nitrified, while with raw sewage a well clarified effluent is commonly sufficient. Where the readily settling solids are high in nitrogen content and the excess sludge is to be marketed as a fertilizer, it may be advantageous to dispense with presedimentation. As a general rule, however, it seems undesirable to produce more activated sludge than is required to carry on the treatment process.

The method of sludge treatment and disposal may influence the design of preliminary-treatment units. In the event that the sludge is to be digested, it has been found advantageous to provide preliminary-sedimentation tanks of sufficient capacity effectively to settle out the excess activated sludge with the primary sludge. A comparison of the relative volumes of sludge to be disposed of under various conditions may be afforded by the following calculations, based on a hypothetical sewage flow of 1 mil. gal. daily, with 300 p.p.m. of suspended solids in the sewage following racks, grit chambers, skimming tanks, or other preliminary devices. No allowances are made for the reduction of solids by oxidation in the activated-sludge process or for the specific gravity of the sludge, which in any event would be near unity.

CASE 1. *Fine screens and activated-sludge treatment.*

Suspended solids removed by fine screens:	20 p.p.m.
Suspended solids removed by activated-sludge process:	
$0.95 \times (300 - 20) =$	265 p.p.m.
Volume of excess activated sludge at 98.5 per cent moisture:	
$265 \times \frac{100}{1.5} =$	17,670 gal. daily.

CASE 2. *Preliminary sedimentation and activated-sludge treatment; primary sludge and activated sludge taken separately.*

Suspended solids removed by preliminary sedimentation:

$0.50 \times 300 =$ 150 p.p.m.

Volume of primary sludge at 94.5 per cent moisture:

$150 \times \frac{100}{5.5} =$ 2730 gal. daily.

Suspended solids removed by activated-sludge process:

$0.95 \times (300 - 150) =$ 135 p.p.m.

Volume of excess activated sludge at 98.5 per cent moisture:

$135 \times \frac{100}{1.5} =$ 9000 gal. daily.

Volume of combined sludge: 11,730 gal. daily.

CASE 3. *Preliminary sedimentation and activated-sludge treatment; the waste activated sludge discharged into the influent of the preliminary sedimentation tanks and settled out with the primary sludge.*

Suspended solids removed:

$0.95 \times 300 =$ 285 p.p.m.

Volume of combined sludge at 96 per cent moisture:

$285 \times \frac{100}{4} =$ 7125 gal. daily.[1]

[1] As shown in Chap. XII, with a specific gravity of 1.03, the volume of sludge produced under the assumed conditions would be 6,910 gal. daily.

Choice of preliminary-treatment methods depends upon local conditions. Racks, grit chambers, fine screens, skimming tanks and single- or two-story settling tanks all have their part in the economy of the process. Most of the newer American plants, including the large works at Chicago, Indianapolis, and New York and smaller ones at Pomona, Cal., Peoria and Springfield, Ill., Charlotte, N. C., Elyria, Ohio, and Toronto, Ont., provide for some sedimentation before aeration. The small plant at Mercedes, Tex., is equipped for pre-aeration of the sewage for 45 min. in advance of the addition of return sludge. Pre-aeration is apparently of value in "freshening" a septic sewage prior to activated-sludge treatment. Bell (14) has employed bleach for a similar purpose at Barnsley, England. The use of equalizing tanks to reduce peak flows and decrease the concentration of industrial wastes that are discharged periodically through the day is recommended for certain unusual conditions by some engineers. Submerged contact aerators have been employed in the Ruhr Valley, Germany, in advance of aeration units and a similar use of trickling filters is reported from the Emscher

District. The advantage claimed for these combinations of oxidation processes is the ability of contact aerators and trickling filters to care for sudden changes in the character of the sewage received at the plant, notably those due to toxic industrial wastes. Preliminary treatment by chemical precipitation has been successfully employed at Bolton, England, where the sewage receives strong industrial wastes (15).

In America there has been much discussion of sedimentation tanks in preparing sewage for treatment in aeration units. Goudey (16) has reported that experiments at Los Angeles have demonstrated that preliminary clarification in the activated-sludge process is helpful and that raw-sewage solids are not essential in the aeration tanks in order to maintain good activated sludge.

At Essen-Rellinghausen, Fries (17) has arrived at the conclusion that the efficiency of the activated-sludge process is enhanced by the removal of the solids from the raw sewage.

Activated Sludge, Ltd., has reported that pre-aeration of crude sewage by means of diffused air in the presence of surplus activated sludge, followed by settlement in preliminary-sedimentation tanks, has been of value in the operation of the activated-sludge plant at Oslo (18).

In the project report for the Wards Island activated-sludge plant at New York, Fuller (19) gave the following reasons for adopting sedimentation rather than fine screening as a preparatory process:

1. Settling tanks having 1 hr. detention period at average flow will reduce the volume and amount of sludge to be shipped to sea, as compared with a plant using fine screens, by several hundred tons per day. This is due to the fact that settling tanks will remove at least 30 per cent more of the total suspended solids than fine screens, and this sludge will have a moisture content of probably 95 per cent, whereas with screens this 30 per cent would have to be removed in the final tanks with a probable moisture content of 98 per cent.

2. Settling tanks are readily adaptable to grease removal, while fine screens require separate grease-skimming tanks.

3. The material removed in the preliminary tanks is mostly inert and of low nitrogen content, hence the remaining suspended solids are better for fertilizer manufacture.

4. With the heavier material removed by the settling tanks, less air and a shorter aeration period should produce the same degree of purification of the sewage.

5. Under local conditions screenings are somewhat more troublesome to handle and dispose of than settling tank sludge.

6. Fine screens require housing, and this, together with separate grease-skimming tanks, makes them more expensive than the uncovered settling tanks.

The kind of preliminary treatment most appropriate in any case depends upon a careful weighing of comparative advantages and disadvantages. So much depends upon local conditions that it is not possible to lay down any procedure which is generally applicable.

Final Treatment.—The effluent from aeration tanks requires sedimentation in order to remove the flocculated impurities and produce a clear effluent with low biochemical-oxygen demand, as well as for the purpose of capturing the activated sludge that must be utilized in the process. Vertical-flow tanks of the Dortmund type in England and horizontal-flow, mechanically cleaned tanks in this country have been employed most commonly as sedimentation units. The return sludge, depending upon the system of operation, may receive further aeration in separate re-aeration units.

Where the sewage is only partially treated by the activated-sludge process, the effluent after settling may be oxidized further on sewage filters. This matter will receive attention in a subsequent section of this chapter.

The effluent from activated-sludge plants is generally clear and the removal of bacteria is usually high. Disinfection of the plant effluent, therefore, is generally less essential than in connection with other treatment processes. When needed, however, it can usually be accomplished efficiently by using relatively small quantities of disinfectant.

A plant to study methods of sewage reclamation has been put in operation in Los Angeles (20). This plant employs sedimentation, activated sludge, coagulation, superchlorination, sedimentation, sand filtration and dechlorination with activated carbon. The sludge removed is digested in separate tanks. The effluent is said to be sparkling, clean, tasteless, odorless, colorless, free from suspended solids and turbidity and to have but a trace of B.O.D. and an oxygen-consumed value of 2.5 p.p.m. This effluent is discharged upon natural sand beds, through which it is expected finally to reach the infiltration galleries, 1½ miles distant, from which part of the domestic water supply is obtained. The cost of producing an effluent complying with the U. S. Treasury Department standards for drinking water is said to be less than the cost of water brought in from outside sources.

The sewage and laundry wastes from the resort hotels and camps at Grand Canyon, Ariz., are treated by sedimentation, activated sludge, rapid sand filters and chlorination (21). The plant effluent is used in boilers, for cooling water in oil engines, for irrigating lawns and for flushing toilets. Cost of treatment is $0.574 per 1,000 gal., including interest and depreciation, as compared with $3.09 for fresh water brought in tank cars with a haul of 100 to 120 miles.

A small sewage-treatment plant at Barrington, N. J., provides for screening, sedimentation, activated-sludge treatment, rapid sand filtration and chlorination prior to discharge into a small stream which contains little pollution (22). The rate of filtration is about 1½ gal. a minute per square foot.

Vacuum filters have been installed at Rockville Centre, N. Y., to treat the effluent from an activated-sludge plant (23). Paper pulp is added to the filter influent, the paper coagulating the suspended solids to form a mat on the filter. The purpose of the installation is to reduce the quantity of suspended solids reaching the sand filters, which form the final step in the sewage-treatment process.

Complete Treatment.—An interesting characteristic of the activated-sludge process is its adaptability, in the light of the staging of activities, to accomplishing several degrees of purification. If the removal of a substantial part of the suspended and colloidal solids and bacteria and the production of a moderate degree of stability are sufficient, aeration of the sewage—but not necessarily of the sludge—can be stopped short of material oxidation of organic matter and nitrification. These latter activities can be dispensed with, if circumstances permit, and a material saving in air and tank capacity effected. On the other hand, if complete stability and oxidation of a substantial quantity of organic matter are required, the process can be operated so as to accomplish this by protracted aeration or by combination with other oxidation devices, such as trickling filters.

Most of the existing American activated-sludge plants are designed for complete treatment of the sewage in the aeration units. The early plants, such as those at Houston, Tex., Milwaukee, Wis., and Pasadena, Cal., were provided with a minimum of preliminary treatment, usually fine screening. Later plants, as previously noted, include a certain degree of presedimentation.

In the past there has been a tendency in American plants to carry aeration to a point where there is appreciable nitrification of the effluent. In some cases this may be necessary in order to secure a sludge that settles or dewaters readily or attains a high nitrogen content. When the nitrates are reduced below 1 p.p.m., the sludge is reported as having a tendency to bulk. This phenomenon is discussed later in the chapter.

In other cases high nitrates in the effluent may become undesirable by inducing objectionable growths of algae and other aquatic organisms in the stream receiving the nitrified sewage. Furthermore, high nitrification may be uneconomical and there may be a considerable saving in power when nitrification is kept low.

During early tests at Milwaukee, Copeland (6) found that when the nitrates were 1.1 p.p.m. after 2½ hours' aeration the efficiency of the sedimentation tanks was not good. The settled liquors contained 15 to 20 p.p.m. of suspended matter and were noticeably turbid.

Partial Treatment.—The idea of partial treatment by aeration prior to filtration was inherent in the original experiments by Clark and Gage at the Lawrence experiment station. It was first put to use on a practical scale at Birmingham, England, where the process received the name "bioflocculation," and has been employed in this country at Decatur, Ill., and Lemoore, Cal., among other places. Aeration is generally carried through the first or clarification stage and the sludge is reactivated in separate re-aeration tanks. The effluent is generally passed on to trickling filters which can be operated at greatly increased rates, in the vicinity of 2 to 4 times the normal ones, as indicated below. Advantages claimed for this combination of processes are the utilization in many cases of existing filters, the restriction of plant area where expansion of trickling-filter works becomes necessary, the prevention of annoyance due to odors and flies, often associated with trickling filters, the reduction of serious disturbances in the treatment process caused by variation in the character and quantity of industrial wastes, and the more efficient utilization of available treatment processes. In explanation of the latter, Martin states that it is by no means certain that it is always wise to dispense with filters and that the combination of first-stage aeration and trickling filters may at times present the best arrangement (24). Whitehead has estimated the relative annual cost of a complete bio-aeration plant to be in excess of that of a combined bio-aeration and trickling-filter plant (25).

Experiments by Clark and Gage (26) at Lawrence, Mass., during 1913 showed that preliminary treatment by aeration made it possible greatly to increase the rate of dosing filter beds and still obtain satisfactory results. Aerated sewage to the extent of 350,000 gal. per acre daily on intermittent sand filters and 10,000,000 gal. per acre daily on trickling filters was successfully treated. The average results of operation for the year are given in Table 97.

The partial treatment of sewage by aeration prior to filtration is of particular importance where large areas of filters are already available. Martin (27) states:

> The Birmingham sewage, after settlement in tanks, can be deprived of about 60 per cent of its residual polluting matter by a short period of agitation with activated sludge. The effluent from this process can then be sprayed over the existing percolating filter, and purified at a rate at least double that at which settled sewage can be purified. The process has deprived the sewage of those constituents which give rise to smell and choking of the filter.

Extension of the filter area can thus be avoided. At Birmingham the Hartley-type aeration tanks remove about 60 per cent, or 80 p.p.m.,

TABLE 97.—RESULTS OF PRE-AERATION AND FILTER OPERATION AT LAWRENCE EXPERIMENT STATION, 1913

	Free ammonia, p.p.m.	Total albuminoid ammonia, p.p.m.	Nitrate nitrogen, p.p.m.	Oxygen consumed, p.p.m.
Trickling filter				
Raw sewage	38.3	6.0	....	40.7
Aerated and settled sewage	36.7	3.3	0.2	19.1
Trickling-filter effluent[1]	......	.....	17.2	13.3
Intermittent sand filter				
Raw sewage	38.3	6.0	....	40.7
Aerated and settled sewage	25.7	2.2	0.5	13.6
Sand-filter effluent[2]	0.515	0.344	28.3	3.1

[1] Average, 6.21 m.g.d. per acre.
[2] Average, 0.30 m.g.d. per acre.

of the fine suspended and colloidal matter from settled sewage in about 1 hr. and the effluent is treated on trickling filters at almost double the rate for unaerated sewage. Relative to the sewage flow, from 2 to

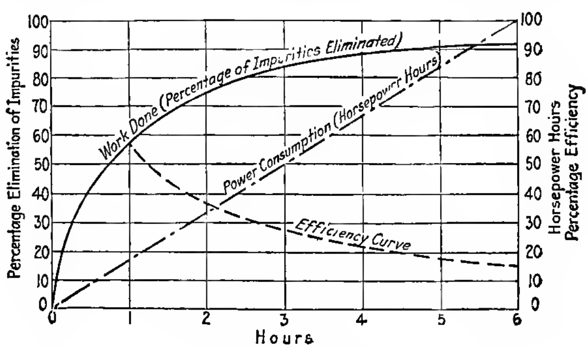


FIG. 166.—Relative efficiency of activated-sludge process with respect to power consumption.

5 per cent of return sludge, re-aerated for 18 hr. in Simplex units, is required in the aeration units to maintain the process. Watson (28) has determined the relative efficiency of the activated-sludge process with respect to power consumption under conditions applicable to Birmingham, the results being presented in Fig. 166. According to his

studies, 56 per cent of the impurities are removed during the first hour, while this value is raised only to 74, 83 and 88 per cent during the second, third and fourth hours, respectively. An example will illustrate the method of finding the efficiency curve. Calling the work done in the first hour 56 per cent, with a power consumption of 16 hp.-hr., and the work done during the first 2 hours 74 per cent, requiring 33 hp.-hr., the relative efficiency for 2 hours' aeration is $^{74}/_{33} \times 16 = 36$ per cent.

A new aeration unit to treat 12 million U. S. gal. daily has been constructed at Birmingham. The aeration tanks are of the diffused-air type, with a 60-deg. sloping bottom on both sides of a row of porous plates. Pre-aeration of the settled sewage before addition of the activated sludge is to be tried. The sewage with the activated sludge is to be aerated one hour. The sludge is to be re-aerated four to five hours (29).

According to O'Shaughnessy (30):

> Sewage contains sludge-forming material not in solution and other material which is in true solution. Of the sludge-forming material 78 per cent to 80 per cent is readily settled out in a plain sedimentation tank. The settled sewage then contains non-settling colloidal matter which may choke the trickling filter, but which may be flocculated very rapidly by the activated sludge. It also contains material which may be dealt with only slowly by the expensive activated sludge technique, but which is dealt with easily, rapidly and therefore economically on the bacteria bed.

Hatfield (31) has reported on the operation of the pre-aeration plant at Decatur, Ill., as follows:

> After two years of testing-station results (32, 33) the pre-aeration unit for the Decatur plant was designed and constructed. The testing-station data had indicated that with a 2.5-hr. aeration period from 30 to 40 per cent reduction in the 5-day oxygen demand of the settled sewage could be anticipated, and that the settled aerated effluent could be successfully applied to the sprinkling filters at three times the rate, in million gallons per acre per day, that could be used with the un-aerated sewage. . . .
>
> There are six aeration tanks of the Manchester or spiral-flow type which have a total displacement period of 2.5 hours at a sewage flow of 10 m.g.d. plus a sludge return of 1 m.g.d. . . . There are two 77.5 ft. Dorr clarifiers for settling the aerated sewage, which have a total displacement period of 2.6 hr. and a settling rate of 833 gal. per 24 hr. per square foot of surface. The sludge is returned to the influent conduit without provision for re-aeration and to the influent to the Imhoff tanks for sedimentation and digestion with the primary sludge. . . .
>
> The plant has never ceased to be a large experimental plant, due to the unusual sewage received and the idiosyncrasies of underaerated activated sludge. Many experimental runs or periods have been made, but in

general the operation may be divided into three groups which will be designated as (1) "normal aeration," (2) "re-aeration," and (3) "no sludge return."

Normal aeration consists in operating the aeration plant in the way it was designed to run, *i.e.*, using all six aeration tanks to aerate the sewage and return sludge and obtaining an average displacement period of about 2.5 hr. The volume of sludge return has varied from 0.5 to 2.0 m.g.d. or from 5 to 20 per cent of the sewage flow. The total quantity of air has varied from 0.35 to 2.62 cu. ft. per gallon of sewage aerated. In general, about 10 per cent of the air is used for aerating the conduits and 90 per cent in the aeration tanks proper. . . .

The averages for the normal-aeration periods are: sewage flow 9.83 m.g.d., sewage aeration period 2.54 hr., air applied 0.48 cu. ft. per gallon of

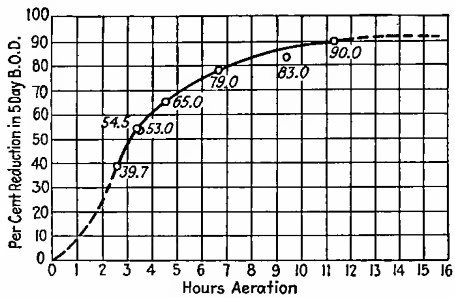


FIG. 167.—Reduction in 5-day biochemical-oxygen demand by pre-aeration at Decatur, Ill.

sewage, influent B.O.D. 163 p.p.m. and B.O.D. removal 32 per cent. The suspended matter both in volume and weight was consistently greater in the settled effluent than in the influent. . . .

Re-aeration of the sludge is accomplished by utilizing the first two aeration tanks as sludge re-aeration tanks, and the remaining four aeration tanks for aeration of the mixture of sewage and re-aerated sludge. This method of operation reduces the time of sewage aeration to 66 per cent of that which would have obtained with the same sewage flow under the normal-aeration method of operation, but allows for a sludge re-aeration period of from 4.5 to 22.0 hr., depending on the variation of sludge return from 2.0 to 0.41 m.g.d. . . .

The averages for the re-aeration periods were: sewage flow 9.95 m.g.d., sewage aeration period 1.75 hr., sludge re-aeration period 16.6 hr., air applied 0.29 cu. ft. per gallon of sewage and 2.77 cu. ft. per gallon of sludge or an overall total application of 0.44 cu. ft. per gallon of sewage treated, influent B.O.D. 142 p.p.m. and B.O.D. removal 30 per cent. . . .

No Sludge Return consists in operating the six aeration tanks similarly to the normal aeration plan except that no sludge from the Dorr clarifiers is pumped to the aeration tanks, but all the sludge is pumped back into the influent sewage just preceding the Imhoff tanks. There have been only a very few occasions when the Imhoff-tank effluent has shown any signs of containing this aeration-plant sludge. There may be some inoculation of the raw sewage with bacteria, protozoa and other materials which do not settle out in the Imhoff tanks, but this inoculation is not visible to the eye or under the low-power microscope. . . .

"The volume of air used has been about 0.4 cu. ft. per gallon of sewage. . . .

The average results . . . were: Sewage flow 10.2 m.g.d., sewage aeration period 2.48 hr., air applied 0.41 cu. ft. per gallon of sewage, influent B.O.D. 126 p.p.m. and B.O.D. removal 33 per cent.

As a result of the tests, Hatfield concludes that the process with re-aeration appears to work better than the other methods of operation tested, particularly from the standpoint of odor control. He makes the following statement:

Usually there is little or no offensive odor from the aeration tanks and sprinkling filters, but about twice a month a strong sewage (probably industrial wastes) that causes odors is received. The odor hazard from the aeration tanks and filters is apparently greatest with "no sludge return," less with "normal aeration" and least with "re-aeration." The larger amount of relatively good activated sludge in the system during "re-aeration" absorbs or oxidizes the odors and reduces the odor hazard, provided the sludge is carefully watched and not allowed to overload the oxygen-satisfying capacity of the plant. In the latter case the sludge odors are as bad as the other odors. With proper attention less odors are noticeable during periods of "re-aeration."

The effect of volume of air on the efficiency of the pre-aeration process under the "no sludge return" system was studied by Hatfield, with the results indicated in Table 98. In these tests the sewage flow was maintained at 3.48 m.g.d., the aeration period was 2.64 hr. and the B.O.D. in the tank influent was 101 p.p.m.

Table 98.—Effect of Volume of Air on B.O.D. Reduction at the Pre-aeration Plant in Decatur, Ill.

Air, cu. ft. per gal. of sewage	Reduction of B.O.D., per cent
0.172	17.5
0.288	28.8
0.417	37.2

Carbohydrate wastes from the corn-products industry greatly influence the operation of the Decatur plant.

The sewage-treatment plant at Lemoore, Cal., is designed for partial treatment by the Simplex system preceding trickling filters, operated at a rate of 4.5 m.g.d. per acre (34). The depth of filter medium is 6 ft.

Four hours of aeration are provided, with 40 per cent return sludge. Separate re-aeration of the sludge is possible. Plant operation is greatly affected by the presence of milk wastes with a 5-day B.O.D. of 600 to 700 p.p.m.

Stage Treatment.—Although, as previously mentioned, it was early recognized that the activated-sludge process, as a complete process, incorporates recognizable stages of activity, no attempt was made until recently to separate these different stages in plant design and operation. A number of different experiments are now under way and a measure of success has been reported.

In the light of Parsons and Wilson's analysis of the activated-sludge process, a stage-treatment plant should include a clarifying stage and

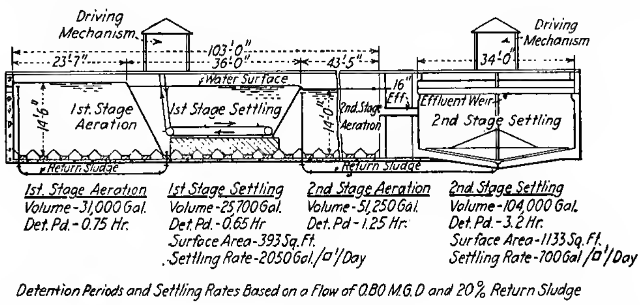


FIG. 168.—Two-stage activated-sludge process at Calumet treatment plant, Chicago.

a biological stage, or possibly two biological stages, one for reactivation, the other for nitrification (9). Furthermore, each of the stages should consist of an aeration tank followed by a sedimentation tank. There are two conceivable methods of introducing sludge into the different stages. In the first method, the sludge recovered by sedimentation in each stage is discharged into the influent of the stage producing it. In the second method, only well-activated sludge, *i.e.*, sludge settling in the final-sedimentation unit or separately re-aerated, is added as return sludge to the influent of each stage. Both methods have been employed experimentally. While the second method of sludge addition is essentially in accordance with the recommendation of Parsons and Wilson, the first does not appear to agree with their concept of the stage process.

At the Calumet treatment plant in Chicago a two-stage process has been under test since the fall of 1929 (35). A cross section of the experimental unit is shown in Fig. 168 and analytical and operating results are given in Tables 99 and 100.

As indicated in Fig. 168, sludge from the first-stage settling tank was added to the first-stage aeration tank and sludge from the second-stage

TABLE 99.—ANALYSES OF RAW SEWAGE AND EFFLUENTS FROM TWO-STAGE ACTIVATED-SLUDGE PROCESS AT CALUMET WORKS, CHICAGO

Parts per Million

Month, 1929–30	Raw sewage			Effluent, first stage				Effluent, second stage				
	Org. + NH_3 N	5-day B.O.D.	Sus-pended solids	NO_2 + NO_3 N	Dis-solved oxygen	5-day B.O.D.	Sus-pended solids	Org. + NH_3 N	NO_2 + NO_3 N	Dis-solved oxygen	5-day B.O.D.	Sus-pended solids
Oct.	11.8	79	107	1.1	5.5	38	81	6.9	1.4	6.6	11.6	25
Nov.	13.0	79	132	0.9	2.3	22	52	6.9	0.7	7.6	7.3	17
Dec.	13.2	89	119	0.6	1.7	23	45	7.5	0.6	7.6	8.4	13
Jan.	10.5	71	106	1.4	5.9	25	43	6.8	1.5	9.8	13.8	29
Feb.	10.0	78	122	1.2	4.5	32	70	6.2	1.4	8.5	13.2	28
Mar.	12.1	88	118	1.1	2.4	35.8	82	7.4	1.1	7.4	14.7	25
Apr.	11.3	76	101	1.2	3.6	35.8	69	6.4	1.4	5.9	14.8	29
May	12.6	94	150	0.5	0.9	35	68	6.9	0.6	4.7	9.1	18
June	13.2	93	139	0.4	0.6	39	92	7.0	0.7	4.6	7.0	14
July	12.7	84	135	0.4	0.9	37	57	3.7	3.0	4.5	7.6	13
Aug.	11.6	71	124	0.3	0.9	22	52	1.9	4.3	4.1	6.0	12

TABLE 100.—OPERATING RESULTS OF TWO-STAGE ACTIVATED-SLUDGE PROCESS AT CALUMET WORKS, CHICAGO

Month, 1929–30	Sewage treated, m.g.d.	Aeration period, hr.	Return sludge, per cent of sewage flow	Air consumption (free air)		Suspended solids in aeration tank, p.p.m.	Settling rate in Dorr tank, gal. per sq. ft. daily	Stage
				Cu. ft. per gal.	Cu. ft. per plate per min.			
Oct.	0.795	0.79	19.7	1.11	4.0	....	2019	1
		1.30	19.7			3418	700	2
Nov.	0.745	0.83	20.1	0.93	3.2	3860	1895	1
		1.38	20.0			3740	655	2
Dec.	0.750	0.83	20.2	0.77	2.6	3314	1903	1
		1.37	19.8			4414	660	2
Jan.	0.760	0.82	19.8	0.58	5.1	2557	1910	1
		1.35	19.6	0.34	1.9	1965	668	2
Feb.	0.788	0.80	18.9	0.47	4.2	2377	2005	1
		1.31	19.0	0.27	1.6	3400	695	2
Mar.	0.813	0.78	18.3	0.34	3.2	2526	2060	1
		1.28	18.3	0.22	1.4	2907	720	2
Apr.	0.800	0.79	18.7	0.31	2.9	1967	2037	1
		1.29	18.7	0.21	1.2	2745	705	2
May	0.805	0.78	18.6	0.30	2.8	3979	2050	1
		1.29	18.3	0.20	1.2	3055	710	2
June	0.800	0.78	18.6	0.29	2.7	3902	2045	1
		1.29	18.2	0.20	1.2	2978	708	2
July	0.800	0.78	18.6	0.30	2.8	3390	2040	1
		1.29	18.4	0.21	1.2	3056	709	2
Aug.	0.820	0.76	18.3	0.30	2.8	3530	2090	1
		1.27	17.9	0.20	1.2	2949	724	2

settling tank to the second-stage aeration tank. Mohlman and Wheeler (35) comment as follows upon their study of this form of the two-stage process:

The average results of analyses of raw sewage and effluents shown in Table 99 indicate that the final effluent has been quite satisfactory, with a comparatively low B.O.D. and a small amount of suspended solids. The highest B.O.D. and suspended solids in the final effluent occurred from January through April, the period of the year when results are usually poorest. [*Note:* During the summer lower values for B.O.D. and suspended solids were obtained for the final effluent, as shown in Table 99.] The results of analyses for both the effluents from the first and second stage show little nitrification, but notwithstanding this fact a low B.O.D. was obtained in the final effluent.

According to the operating results shown in Table 100 approximately 800,000 gal. of sewage has been treated per day, with an aeration period of approximately 0.8 hr. in the first stage and 1.3 hr. in the second stage, making a total aeration period of 2.1 hr. There was no reaeration of sludge. This total aeration period for production of a completely treated effluent is very much shorter than we have obtained previously, although approximately the same period was used during the pre-aeration experiments in 1928 and 1929. The air requirements shown in Table 100 range from 0.5 to 0.9 cu. ft. per gallon, excluding the first month. [*Note.* Approximately 0.5 cu. ft. per gallon was applied during the summer.]

Our experiments have not progressed far enough to warrant final conclusions with regard to the efficiency of the two-stage process as compared with the normal process of activated-sludge treatment. While we have obtained good results with a very short aeration period and comparatively low amount of air, we also obtained almost as good results last year in our pre-aeration experiments, in which a single stage of aeration was used plus reaeration of sludge. The two-stage process is very sensitive biologically and requires careful control, which we have not completely perfected as yet, although the sludge produced in the second stage from the Calumet sewage has generally been well flocculated, settleable sludge. It might be possible to retain sludge in the first stage, even with concentrated sewage, by returning sludge occasionally or even continuously from the final settling tank but there will probably be operating difficulty in preventing the sludge from passing through the primary settling tank. This difficulty has not been serious so far in the Calumet tests. The flexibility of the process commends it. The proper operating conditions will have to be found for various types of sewage, and consideration will have to be given in design for seasonal variation and peak flows, but our results indicate a possibility in this scheme for reduction of time required for aeration. Whether such reduction in size of aeration tanks will pay for the cost of additional settling tanks will have to be determined.

At Essen-Rellinghausen, Imhoff has tested a two-stage experimental plant in which the sludge added to both stages is secured from the second-stage settling tank. The sludge removed by first-stage sedimentation is wasted as devitalized excess sludge. According to Sierp (36), this arrangement reduces the overall detention period of sewage in the activated-sludge plant to about two-thirds of that required in single-stage treatment. Whether sufficient second-stage sludge can be obtained to serve both stages under American conditions of sewage flow remains to be determined. Separate re-aeration of first-stage sludge may be required.

Disposal of Excess Activated Sludge.—One of the major problems of the activated-sludge process is the disposal of the excess activated sludge. This problem looms large because of the great bulk of the sludge produced and its unstable character. Among the various methods of sludge treatment prior to disposal which have proved practical in American plants are dewatering on sand beds; digestion, either alone in separate sludge-digestion tanks, or with other sewage solids in Imhoff or separate sludge-digestion tanks, prior to dewatering on sand beds; lagooning; dewatering by filtration through cloth; and heat drying. These methods of treatment are discussed chiefly in Chaps. XXVII to XXIX, together with the ultimate disposal of the sludge. The digestion of excess activated sludge in Imhoff tanks is described in Chap. XVI and the use of separate sludge-digestion tanks is referred to in Chap. XVII.

As previously mentioned, the requirements of sludge disposal have a bearing upon the degree of preliminary treatment to be provided, as well as upon the requirements of the aeration process.

A method of concentrating excess activated sludge, developed at the Los Angeles experiment station by Goudey and Bennett (37), has been described as follows:

> The process entails withdrawal of excess activated sludge . . . in the form of mixed sewage and sludge from the main aeration tanks rather than from the final settling tanks as is usually practiced. This mixture is allowed to settle in a continuously operated clarifier designed on a basis of 600 gal. per square foot surface area per day. The inlet to the clarifier is at the side and influent enters at the bottom. The overflow liquor from this excess sludge clarifier is equally as good as the final effluent from the conventional final clarifiers and is mixed with the latter. A chlorine dose is applied to the top water of the clarifier to maintain a residual of 1 p.p.m. of chlorine in the water above the sludge zone. This dose amounts to about 25 lb. per million gallons of sewage treated. The residual chlorine carried by the overflow liquor is sufficient to disinfect the final effluent from the entire plant. . . . The thick concentrated sludge is pumped but once daily from the thickening tank. . . . The average water content (of the sludge) after nine months of operation is 94.80 per cent.

Success with this new method of sludge concentration is due primarily to maintaining the presence of the dissolved oxygen which the aerated mixture contains upon entering the thickening tank.

Establishment and Maintenance of Sludge Activity.—As with other biological sewage-treatment methods, some time elapses before effective operation of new aeration units is established. This is because of the necessity of building up the desired quality of activated sludge, the vehicle of the purifying agencies.

The production of activated sludge at new works, or at a plant which has been out of operation for some time, is accomplished without delay if good activated sludge can be obtained from a nearby treatment plant in sufficient quantity to start operations. Next best perhaps is the introduction of sludge from trickling-filter humus tanks, for this type of sludge is similar in character to activated sludge and has been found to aid in the accumulation of an adequate supply of activated sludge from sedimentation and 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.

When neither activated sludge nor trickling-filter humus is available, the aeration unit may be broken in either by operating it on a fill-and-draw basis, fresh sewage being added about four times a day, or by working the tanks at a greatly reduced rate and discharging into the influent all the sludge settling from the tank effluent. Ordinarily, normal operation becomes possible in 2 to 6 weeks.

When the North Toronto plant at Toronto, Ont., was placed in operation, all the sewage was passed through the plant and all the sludge settling in the final-sedimentation tanks was discharged into the mixing channel, whence it passed into the aeration tanks. An activated-sludge floc was developed and effective clarification secured in about 6 weeks.

The time required and difficulties to be encountered in starting up an activated-sludge plant are of particular importance in the case of a plant where the increase of stream flow and change in requirements make it unnecessary to provide complete treatment throughout the year. As reported by O'Brien (38), the aeration units at the plant of Lima, Ohio, were taken out of service from January to April, inclusive, 1933, with a resultant saving of about $3,500 in power charges. He states:

> No difficulties were encountered in building up a body of activated sludge on the two occasions of starting up the aeration units, a characteristic brown sludge was secured in less than a week and the effect of the treatment was noticeable in the effluent on the day after aeration was started.

Once a tank is broken in, the sludge normally retains its action. The quantity of sludge continues to increase until there is an accumulation of excess sludge which must be withdrawn from circulation. The maintenance of sludge activity is essential to the process. Some of the difficulties encountered in operation will be considered in other sections of this chapter.

Experiments with artificial sludges, such as iron, aluminum and manganese hydroxides and silica "gels," have shown that they are of little practical benefit. Although they may intensify oxidation, it is said to be difficult to maintain them "active." Ferrous iron and nitrites, which take up oxygen quickly, were found by Buswell to act as carriers of oxygen to the bacteria.

Ridenour and Henderson (39), after studying the relative advantages of high and low concentration of return sludge, report as follows:

> The rate and degree of purification through an activated-sludge plant may be decidedly impaired by concentration of the return sludge in the final settling tank sumps. This concentration allows the sludge to grow stale at a rapid rate. Even a moderate deviation of the sludge from its original condition by virtue of short concentrations greatly reduces the rate of purification through the aerators. The effect of this stale return sludge is most important on plants with comparatively short detention periods. . . .
>
> Some index for use in maintaining a good quality of return sludge is as vitally important to good activated-sludge plant operation as any of the other important indices used for plant control. At this plant the index used is the ratio between the suspended solids in the return sludge and the suspended solids in the aerator, which represents the degree of concentration of the return sludge in the final settling tanks. The lower the ratio or concentration the more efficiently the purification process will perform . . A ratio of 2.5 to 3 has proved satisfactory for normal operating requirements at this plant.

Ridenour has found it advantageous to maintain activated sludge in a fresh condition by carrying a large volume of return sludge and low concentration of solids, even though the aeration period was correspondingly reduced to a considerable extent.

Recent studies by workers of the U. S. Public Health Service have led to the identification of the adsorbent principle in activated sludge as a base-exchanging substance, chemically identical with the zeolites of water purification (40). These investigators assert that sterilized sludge can be regenerated by sodium chloride, in the same manner as the commercial zeolites. Under natural conditions, of course, regeneration of the sludge is called "reactivation," with bacteria as the active agents.

Sludge Re-aeration.—Three general arrangements of activated-sludge units have been employed. These are outlined by Pearse and Mohlman (41) as shown in Fig. 169. If the period of aeration, the quantity of return sludge and all other operating conditions are suitably regulated, the "direct return" method can be successfully employed and most of the recent American plants—Chicago, Ill., Indianapolis, Ind., New York, N. Y., North Toronto, Ont., and others—make use of it.

"Re-aeration of sludge return" was employed in some of the earlier American plants, such as those at Houston, Tex., and Pasadena, Cal. At the Des Plaines works of the Sanitary District of Chicago, parallel tests of "direct return" and "re-aeration of sludge return" seemed to show no economy in favor of the latter, either in air or in required tank capacity. In tests at Milwaukee, where sludge containing 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

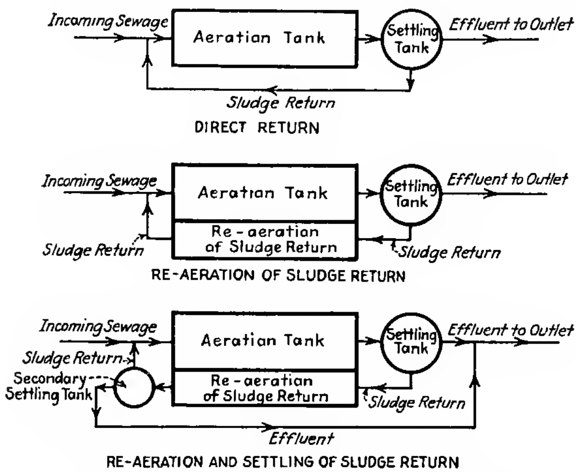


FIG. 169.—Schemes of flow in activated-sludge plants.

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 dissolved oxygen throughout the sludge.

Hatton (42) reported as follows:

> It was hoped that by reaerating the sludge the aerating capacity might be reduced due to a richer sludge, but our experiments indicated that this was not a fact, at least to the extent of warranting us in providing extra tankage and air for reaeration.

Concerning re-aeration at Pasadena, Orbison (43) states:

> The use of re-aeration tanks was discontinued for a while, but this has now been resumed, because it was found that the particles of floc which return into the raw sewage function much more effectively when reaerated, especially after being pumped from the clarifiers to the re-aeration tanks.

Re-aeration and settling of return sludge was employed experimentally at the Stockyards testing station in Chicago (41).

The following paragraphs are quoted from the report of the tests:

The average operating results are summarized [Table 101] during summer conditions in 1916, when the sludge was not re-aerated, and during 1917, when the sludge was re-aerated and resettled.

TABLE 101.—RESULTS OF TESTS OF RE-AERATION AND SETTLING OF RETURN SLUDGE AT STOCKYARDS TESTING STATION, CHICAGO

	Aeration period, hr.	Settling period, hr.	Air, cu. ft. per gallon	Ratio of return sludge to sewage	Sewage treated, gal. daily	Per cent of sludge[1]	Period compared
Aeration alone.......	9.0	1.3	3.5	0.60	61,000	33	July 2 to Oct. 22, 1916
Re-aeration and settling.	6.6	1.7	3.5	0.39	68,800	33	Mar. 27 to Nov. 14, 1917

[1] Percentage of sludge in volume in aeration tank, after settling 1 hr.

In general a comparable effluent was produced by each method of operation but the effluent from the primary settling tank in 1917 was of low grade. When mixed with the resettled effluent, however, the quality was improved sufficiently to approximate the other effluent.

The general summary indicates that a saving of approximately 27 per cent of aeration tank volume can be obtained with re-aeration and resettling of the sludge, but that about 30 per cent more settling tank capacity must be provided. As settling periods are much less than aeration periods, a saving of 20 per cent in total tank volume may be obtained by re-aeration.

It was concluded at Chicago that, although re-aeration effected a substantial saving in tank space, it increased the complexity of the process and necessitated more careful supervision of the plant. For these reasons it was doubtful whether re-aeration and resettling would be desirable in a practical working plant.

It may be advantageous, however, in special cases, such as the treatment of industrial wastes, or sewage containing large quantities of these wastes, and partial treatment by the activated-sludge process. English experience in general is rather favorable to separate re-aeration of the return sludge. At Birmingham, for instance, where it is desired simply to sweep out a sufficient portion of the suspended and colloidal matters so that the effluent may be applied at higher rates to trickling filters, it is considered essential to re-aerate the activated sludge, in order to maintain it in good condition.

As previously stated, Hatfield at Decatur, Ill., has found that re-aeration appears to work better than other methods of operating the pre-aeration plant which he has tested.

Properties of Activated Sludge.—From the standpoint of plant design and operation the most important properties of activated sludge are the ability of the return sludge to effect continuously the desired degree of purification of the sewage treated; the readiness with which the sludge in the effluent from the aeration units will settle as a relatively concentrated mass within the limits obtainable with this type of sludge floc; the facility with which the excess sludge will give up its water content after conditioning with or without other treatment, such as digestion; and, where sludge is to be employed as a fertilizer, the richness of the excess sludge in fertilizing constituents—nitrogen, phosphorus and potash. All these properties enter into the economics of the process and should receive consideration in planning and operating activated-sludge works.

Thoroughly-activated sludge floc generally has a golden-brown color and is relatively compact. Underaerated floc is usually light brown in color, fluffy and relatively light in weight. Overaerated floc may be of a muddy-brown color, probably due to the breaking-up and disintegration of the floc into relatively fine material which settles slowly and leaves a turbid supernatant liquid. A trained operator often relies upon the color of the sludge as an indicator of its quality.

Well-activated floc settles more readily than underaerated floc and forms a denser sludge, leaving the supernatant liquor clear. Underaerated floc is so light that it forms a thin and bulky sludge which is apt to be carried out of sedimentation tanks with the effluent.

As compared with sludge obtained in the treatment of sewage by other processes, activated sludge is relatively high in its water content. When well aerated and accumulated under favorable conditions, it generally contains about 2 per cent solids, or 98 per cent water. Particularly well-activated sludge may contain as much as 3 per cent solids, or slightly more, while underaerated sludge may contain only 0.5 per cent solids, or even less. This variation in density has an important bearing upon the design and operation of the plant, for a sludge containing 0.5 per cent solids has a volume four times as great as that containing 2 per cent solids.

On the whole it appears that a well-activated sludge is most amenable to dewatering and is richest in fertilizing constituents.

Microbiology of Activated Sludge.—The activities of living organisms in activated sludge and their relation to the theory of the process are discussed at the beginning of this chapter. As far as their significance in plant operation is concerned Ardern and Lockett (44) have concluded that the different species of protozoa found in activated sludge are indicative of the condition of the sludge; that in the treatment of sewage by the activated-sludge process, under suitable aeration conditions, protozoa have no adverse influence on the settling properties of the sludge; and that, from the standpoint of sewage treatment in general, protozoa play no important part in the purification effected by activated sludge, although it is possible that they facilitate the production of a more highly clarified effluent.

They classified the protozoan population of different sludges as follows:

Condition of sludge	Amoebae (*Sarcodina*)	Flagellates (*Mastigophora*)	Ciliates (*Infusoria*)
a. Bad	Preponderance	Preponderance	Very few
b. Unsatisfactory	Many	Many	Few
c. Satisfactory	Few	Few	Preponderance
d. Good, *i.e.*, nitrification well established	Rare	Very few	Preponderance

Sludges (*a*), (*b*) and (*c*) were more densely populated than sludge (*d*). It should be noted that the ciliates are largely bacteria eaters, while the other groups are scavengers and live chiefly on organic debris. Similar conclusions have been drawn by other workers (45).

On the other hand, Cramer has concluded, from the results of experiments at Milwaukee, that protozoa play an important part in the activated-sludge process (46). He explains that

. . . in the activated-sludge process the material to which both bacteria and protozoa cling, and which carries both down in the settling process, is mostly organic. It is decomposed by both aerobic and anaerobic bacterial life processes, and would remain in a colloidal, nonsettleable state if the protozoa did not resynthesize it, assembling it in particles (their own bodies) that are so large they will not disperse.

The sludge will not maintain its power to carry down with itself bacteria, protozoa, and more finely divided organic matter unless a vigorous and healthy protozoan life is maintained in it. A daily careful and intelligent microscopic examination of the sludge will soon be recognized as the most effective means of diagnosing operating conditions and difficulties.

Sludge Bulking.—Occasionally a phenomenon known as "sludge bulking" is observed in treatment plants. It is so called because the volume of the sludge swells and becomes unusually great. An unsatisfactory degree of purification generally results. There is some uncertainty as to the cause of sludge bulking. The phenomenon has been ascribed to septic conditions which may be due to factors such as underaeration, prolonged detention of sludge in settling tanks, accumulation of sludge on the bottom of aeration units or sludge-return conduits, return to the process of overflow liquor from separate sludge-digestion tanks or sludge-dewatering operations and sudden discharge of septic solids from the sewer system. Many workers, on the other hand, are of the opinion that sludge bulking is due chiefly to the presence in the sewage of certain industrial wastes, notably milk wastes, starch wastes and brewery wastes. All these wastes contain carbohydrates, which favor the growth of filamentous fungi, especially *Sphaerotilus*, sometimes identified as *Sphaerotilus natans*, the sewage fungus. Growths of this group of organisms have been associated with sludge troubles at Chicago (47) and Decatur, Ill. (45). Sludge bulking has not been reported in the treatment of sewage containing those industrial wastes, such as tannery wastes and iron wastes, which do not include carbohydrates.

Overaeration has also been blamed for sludge bulking. An increase in the protozoan population of the sludge is commonly noted when bulking occurs, but this is not believed to be a causative agency. Generally speaking, bulking seems to be more common with strong English sewages than with the more dilute American sewages. This is to be expected, no matter which explanation of sludge bulking be accepted.

At the present time, British engineers, who have had more experience with this phenomenon than American engineers, seem to be fairly well agreed that sludge bulking should not be ascribed to either underaeration or overaeration, but to the nature of the sewage undergoing treatment. Most of the British workers hold that a reduction in the proportion of return sludge to 15, 10 or even 5 per cent by volume will prevent bulking. Some believe, however, that increased aeration is also efficacious.

Some trouble has been experienced abroad with the destruction of activated sludge by the larvae of *Chironomus*, or blood worms, which seem to feed upon the sludge. At Essen-Rellinghausen these organisms have been effectively removed from the return sludge by passing it through fine screens.

Sludge Index.—The condition of activated sludge with regard to bulking may be indicated by the ratio of weight to volume, a relationship for which Donaldson has suggested the term "sludge index." Theriault (48) early suggested a method of determining the ratio of weight to volume of activated sludge, which he termed the "sludge ratio." This was determined by dividing the total suspended solids in the aeration tank, expressed in parts per million, by the settleable solids in a 30-min. period, expressed in cubic centimeters per liter. On this basis it was reported that sludge with a ratio of 20 or more was in good condition, whereas sludge with a ratio of 5 or less was in bad condition.

After reviewing the work of Theriault, Donaldson (49) has suggested that the sludge index be determined by dividing the weight of suspended solids, expressed in per cent, by the volume of sludge settled in 30 minutes, also expressed in per cent, the result being multiplied by 100. The sludge index, thus determined, is one tenth of the sludge ratio, as computed by Theriault.

Other methods of computing the sludge index have also been reported, in which the bulking sludge is expressed by a diminishing numerical index. This has seemed by some investigators to be inconsistent with the concept of bulking and they have inverted the equation, placing the results of the settling test in the numerator and thus giving an increasing numerical index for bulking.

Mohlman (50), after investigation of the various methods of determining the sludge index in use at the more important American activated-sludge plants, or research stations, has recommended the following uniform technique and method of computation, to the end that results may be comparable:

The sludge index is the volume in cubic centimeters occupied by one gram of sludge after settling 30 min. The sample is collected at the outlet of the aeration tank, settled 30 min. and the volume occupied by the sludge reported in per cent. The sample is thoroughly mixed, or the original sample taken, and the suspended solids determined and reported in per cent by weight. The sludge index is computed by dividing the result of the settling test, in per cent, by the result of the determination of suspended solids, also in per cent.

Plant	Sludge index		
	Average or normal	Minimum	Maximum
Chicago, Ill., Calumet plant	55	29	100
Chicago, Ill., North Side plant[1]	65	35	100
Cincinnati, Ohio, U. S. P. H. S.	...	40	910
Hagerstown, Md.	150	68	454
Indianapolis, Ind.[2]	185	74	312
Lima, Ohio[1]	71	59	115
Milwaukee, Wis.	75	44	100
New Jersey Exp. Sta.	...	35	500
Pasadena, Cal.	104	53	262
Peoria, Ill.[1]	100	..	250
Salinas, Cal.	...	83	167
San Antonio, Tex.[3]	168	72	305
Toronto, Ont., North Toronto plant	...	40	200

[1] 60 minutes' settling.
[2] 250-cc. cylinder.
[3] 500-cc. cylinder.

This inverse sludge index has been used by Palmer and Smith at the North Side and Calumet plants in Chicago, by Heisig at Milwaukee, by Kraus at Peoria, and by the authors during tests at Toronto. The index varies from 40 for rapidly settling sludge to 200 or more for slowly settling or bulking sludge. The Donaldson index can be converted to the recommended index by dividing into 100 and the Theriault by dividing into 1000. Mohlman has collected the essential data from a number of plants and stations and computed the sludge index on the recommended basis as shown in the table on page 588.

Bacterial Digestion of Sludge in Aeration Tanks.—Bacterial digestion of sludge is likely to be more marked at the moderate temperatures of summer than at the low ones of winter. Such action may be expected to reduce the quantity of sludge to be disposed of and may account, in a measure, for discrepancies in the observed quantities of sludge at different plants and at the same plant at different times. Indeed certain tests, which the authors have made, indicate that at times of favorable bacterial action the quantities of sludge and of solid matter contained in it have been reduced, presumably by such digestion, as compared with the quantity previously in the tanks, notwithstanding the accretions from additional sewage. These tests also indicate that it is possible to carry bacterial digestion so far that the sludge will become disintegrated and will lose its clarifying power. When this condition obtains, removal of a part of the overdigested sludge or reduction of the aeration period is followed by the rapid building up of a large quantity of fresh activated sludge, which results almost immediately in a well-clarified effluent.

Studies by Pearse and Mohlman showed a digestion of only 5 per cent during tests running from July 1, 1916, to Mar. 26, 1917 (41). From Mar. 27 to Nov. 14, 1917, on the other hand, 16 per cent of the sludge was lost by digestion. The estimated loss of sludge solids by digestion at the Milwaukee plant during a 6-month period ending in April, 1929, was 9 per cent. At Pasadena there was an apparent reduction of 6 per cent in sludge solids during the year ending June 30, 1929, and of 6.7 per cent during the year ending June 30, 1930.

Obtaining dependable estimates of the loss of suspended solids through digestion in the aeration process is attended by great difficulties, because of unavoidable inaccuracies in measurement and sampling; hence there probably will continue to be much uncertainty in the application of suitable allowances.

Nitrification and Nitrogen Balance.—The nitrogen contained in the sewage entering activated-sludge units may find its way into the effluent and into the sludge, or it may be lost as gaseous nitrogen. Fixation of atmospheric nitrogen, considered by some workers as being an important feature of the process, apparently does not play a significant part, if any.

It has been claimed that activated sludge normally contains more nitrogen than the fresh sewage solids from which it originates. This has been attributed by different workers to a number of different agencies, as follows:

1. The "burning out of carbon," while nitrogen remains in the sludge (7).
2. The initial absorption of ammonium salts by the sludge, with subsequent assimilation of these salts by ammonia-fixing organisms (51).
3. The taking up by living organisms of nitrates, nitrites and ammonia nitrogen and their resynthesis into protein (52).
4. The flocculation of colloids and secondly the growth of bacteria and higher organisms (53).

In general, therefore, there seems to be evidence that an interchange takes place between the nitrogen in the liquid and the nitrogen in the sludge, probably to the advantage of the latter.

Some nitrogen may be lost in gaseous form. In computing the nitrogen balance of the Packingtown plant at Chicago, Pearse and Mohlman (41) noted a recovery in the sludge of 19 and 22 per cent, respectively, of the total nitrogen in the stockyards sewage in the summer and winter while employing 3 to 7 cu. ft. of air per gallon of sewage. At the same time a loss of nitrogen to the atmosphere amounting to 41 per cent in the summer and 23 per cent in the winter was recorded. Reddie arrived at a loss at Bradford, England, of 8.6 per cent during January and February and of 18.6 per cent during April and May (54). Richards and Sawyer state that tests at Harpenden (51), England, showed that 15 per cent of the total nitrogen entered the sludge from strong sewage and 27 per cent from sewage of half the strength to which half the normal volume of air was supplied. Studies by Buswell and Neave show a recovery of 26.6 per cent of the influent nitrogen from Champaign, Ill., sewage and practically no loss of nitrogen to the atmosphere, when 0.7 to 1.4 cu. ft. of air per gal. of sewage were employed (52). Summarizing their study of the biochemistry of the activated-sludge process, these workers conclude as follows:

1. An effluent of reasonable stability can be obtained without using air sufficient to produce nitrification.
2. Denitrification[1] results in protein formation rather than loss of nitrogen.
3. There is apparently no loss of nitrogen when using a minimum volume of air for aeration.
4. There is no evidence of nitrogen fixation, even when treating with $FeSO_4$ to stimulate crenothrix-like organisms.

As previously stated, a limited degree of nitrification seems to be essential for obtaining a floc that will settle readily and that is amenable

[1] In the strictest sense reduction of nitrates and nitrites with the loss of nitrogen, but used by Buswell and Neave to imply also assimilation of nitrates and ammonia nitrogen by living organisms.

to dewatering. In connection with the latter, Ardern (55) reviews his experience with sludge-dewatering by an Oliver filter as follows:

> Evidence is available to show that sludge resulting from the treatment of raw sewage with the production of nitrified effluents is suitable for dewatering by the vacuum-filtration process after chemical pretreatment. In the case of Manchester (Davyhulme) sewage, however, sludge of this type is not readily obtainable, owing to the frequent inhibition of the nitrification process by doses of trade waste.
>
> So far as this investigation has proceeded it would appear as if, in the case of Manchester sewage (Davyhulme Works), sludge of a suitable character and condition for dewatering by vacuum treatment is procurable only by treatment of:
>
> *a.* Raw sewage, so as to obtain a clarified rather than a nitrified effluent; or, preferably,
>
> *b.* Sedimentation tank liquor to well advanced nitrification.

Winter Operation.—In extremely cold weather at Milwaukee, Copeland (12) found that the free ammonia was not reduced and nitrates were not formed in material quantity, 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 adsorb colloidal matter. This condition could be limited, in part at least, by increasing the air supply by about 25 per cent.

Cold weather was found by Pratt and Gascoigne (56) to have no appreciable effect on the process at the Cleveland testing station, where clarification instead of stability was the operating aim. The Cleveland experimental plant was placed in operation on Feb. 10, 1916, when the temperature was 22°F. and the bottoms of the tanks were covered with ice. According to Pratt and Gascoigne,

> . . . within 10 days activated sludge was produced—during a time when the temperature of the atmosphere varied from 14 to 43°F. and that of the treated sewage from 54 to 59°. The sludge was typical in appearance and settled as rapidly as any produced since then. There appears to be no reason why an activated-sludge plant cannot be placed in operation during the coldest months of the year.

Lederer (57) has suggested that in cold weather the turbidity of the effluent is a good index of its quality. At such times he found, in the treatment of sewage from the stockyards and Packingtown in Chicago, that the stability 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.

Ardern and Lockett (4) state that, generally speaking, temperatures below 50°F. have been shown to exert a marked retarding effect upon oxidation, while temperatures above 90°F. cause an initial lag in nitrification with a subsequent increase in the rate. It has been suggested that sewage could be warmed by heating the air used for aeration and some have feared that the process of aeration would cool the sewage sufficiently to cause cessation of action in winter. Neither of these thoughts is well founded, as can be proved by a consideration of the specific heat and weight of air.

Eddy and Fales (58) have explained the matter as follows:

The specific heat of air is approximately 0.238 (water = 1), whence 4.2 times as much air as water by weight is required to produce a given result in heating or cooling. The specific gravity of air equals 0.00129+ (water = 1), whence 1 volume of water weighs the same as 775 volumes of air. The relative volumes of air and water at a given temperature, to have the same calorific power, are therefore as 3250:1. On the basis of the above computations, 100 times the actual quantity of air being used for 20 hours' aeration would be required at 90°F. to raise the wastes from 50 to 70°F. in 20 hr. If the air were heated to 212°F., 500 times the quantity of air being used per hour would be required to raise the temperature of the wastes from 50 to 70°F. in 1 hr.

Effect of Industrial Wastes.—In the activated-sludge process, as in other treatment processes, the presence of industrial wastes may have an appreciable bearing upon the operation and design of the works. In activated-sludge plants any increase in loading due to putrescible organic wastes requires consideration, as well as direct interference with biological activity, aeration and sludge disposal. Wastes containing copper from copper-working mills, or arsenic and other metallic poisons from paint industries, have interfered seriously with the process, although there is a limited tolerance to such substances, as there is to phenolic wastes. In concentrations of 3 per cent or more, acids and alkalies are also detrimental to the activating organisms. Tar and oil give rise to an important problem by preventing effective aeration and affecting the sludge.

The results obtained at a number of activated-sludge plants are given in Table 102. The removal of suspended solids ranges from 85.5 per cent at the Irwin Creek plant in Charlotte, N. C., to 96.5 per cent at San Antonio, Tex. Bacterial removal at Indianapolis is 97.6 per cent, at Milwaukee, 97.5 per cent, and at Charlotte, 98 per cent, making a favorable comparison with the standards quoted above. The removal of five-day biochemical-oxygen demand ranges from 87.3 per cent at the Sugar Creek plant in Charlotte to 95.3 per cent at Milwaukee.

The effect of carbohydrate wastes upon the process has been discussed previously in connection with the phenomenon of sludge bulking. Some industrial wastes, on the other hand, are believed to improve oxidation by conveying oxygen to the bacteria or otherwise acting as catalysts.

It is in connection with the control of noxious industrial wastes that the use of equalizing tanks has been advocated, in order to reduce the concentration of interfering substances to the limit of tolerance or "threshold" dose, by distributing their discharge more or less uniformly over the 24 hr. of treatment. Preliminary treatment by submerged contact aerators or trickling filters is also considered advantageous.

Effect of Liquors from Sludge-treatment Processes.—Some decrease in the efficiency of activated-sludge units has been observed because of the addition to the sewage-treatment process of liquors displaced from separate sludge-digestion tanks or discharged from centrifugal machines used for sludge dewatering. The experience of McConnell with digestion-tank overflow liquor at the Sugar Creek plant in Charlotte, N. C., has been discussed in Chap. XVII.

Similar experiences are reported by Fischer (59) for the Salem, Ohio, plant. Operating difficulties were overcome by connecting the return-sludge line to the sedimentation-tank influent and discharging the excess activated sludge through this line into the plant influent. The mixture of primary sludge and excess activated sludge was pumped to the digester at frequent intervals for short periods of time. A relatively clear digester effluent was secured by this means, whereas direct discharge of excess activated sludge into the digester produced unsatisfactory results.

Experiments were made at Milwaukee with dewatering the excess activated sludge by a centrifuge and discharging the effluent of the centrifuge, which was still high in suspended solids, into the aeration plant. Copeland (60) reported that

> . . in every case where such procedure was kept up for a considerable time we found,
>
> First, the volumes of waste sludge increased day by day.
>
> Second, that the numbers of bacteria and quantity of suspended matter carried by the Demonstration Plant effluent increased.
>
> Third, that the stability of the effluent of the demonstration plant decreased.
>
> After a week of such treatment the plant effluent would not meet our standard of purification.

Degree of Purification Secured.—An important characteristic of the activated-sludge process is the high degree of purification which may be obtained.

So large a proportion of the suspended solids and colloids may be removed that under ordinary conditions the settled effluent will be clear and contain but little suspended matter and comparatively few bacteria.

TABLE 102.—RESULTS OF ANALYSES OF SEWAGE AND EFFLUENT FROM ACTIVATED-SLUDGE PLANTS

	Charlotte, N. C.		Chicago, Ill.	Houston, Tex.		Indianapolis, Ind.	Milwaukee, Wis.	Pasadena, Cal.	Peoria, Ill.	San Antonio, Tex.	Springfield, Ill.
	Irwin Creek plant	Sugar Creek plant	North Side plant (battery "B")	North Side plant	South Side plant						
Year	1931	1931	1930	1923	1923	1931	1933	1931	1932	1933	1932
Period covered by analyses	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.	1 yr.
Suspended solids, p.p.m.											
Influent	213	223	147	272	166	330	269	294	210	359	162
Effluent	31	20	15	34	9	32	18	22	15	12.6	21
Per cent removed	85.5	90.9	89.8	87.5	94.5	90.3	93.3	92.5	93.9	96.5	87.0
Biochemical-oxygen demand, 5 days, p.p.m.:											
Influent	200	212	107	120	.....	256	268	201	190	214	168
Effluent	24	27	8.2	8.2	.....	20	12.6	13	12.9	12.9	17
Per cent removed	88.0	87.3	92.2	93.1	.....	92.2	95.3	93.5	93.9	94.1	90.0
Dissolved oxygen, p.p.m.:											
Influent	.....	.....	.....	0.0	.....	....	.....	.....	0.3		
Effluent	.....	.....	.....	3.9	.....	....	.....	.....	3.1	2.0	
Nitrogen as free ammonia, p.p.m.:											
Influent	.....	.....	.....	19.2	16.5	....	39.7	19.0	12.6	20.9	
Effluent	.....	.....	.....	7.2	5.0	....	14.6	9.7	6.5	4.3	
Per cent removed	.....	.....	.....	62.5	69.7	....	83.2	48.9	48.4	79.3	
Nitrates in effluent, p.p.m.	.....	.....	.....	4.1	11.3	0.34	2.4	4.91	.....	6.9	
Stability, days	5+	5+									
Bacteria on agar at 20°C. in 48 hr., 1000 per cc.:											
Influent	.....	.....	.....	.....	.....	2400					
Effluent	.....	.....	.....	.....	.....	58					
Per cent removed	98	98	.....	.....	.....	97.6	97.5				

The reduction in bacteria, however, is accomplished by physically removing them, rather than by destroying them as in the case of disinfection. As far as pathogenic organisms are concerned, actual destruction probably takes place, owing chiefly to the presence of predatory protozoa, food rivalry of other bacteria and the time factor.

In many cases color may be removed, probably by action similar to that of mordants.

Odor-producing substances may be oxidized directly or they may be absorbed, the effluent being comparatively free from odors noticeable to the ordinary observer.

Oxidation of organic matter in sludge occurs coincidentally with its 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.

As previously stated, however, the presence in the effluent of large quantities of nitrogen in these forms may encourage the development of plant life to such degree as to create objectionable conditions in the waters receiving the treated sewage.

As early as 1917, Hatton determined upon the following standard as a measure and limit of the improvement 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

According to Fuller (19), the Wards Island plant for New York City . . . will produce a well-clarified effluent from which not less than 90 per cent of the total suspended matter, not less than 95 per cent of the total bacteria and from 85 to 90 per cent of the organic impurities are removed.

Bibliography

1. Stein, M. F.: "Water Purification Plants and Their Operation," 2d edition, 1923.
2. Buswell, A. M., and H. L. Long: *Jour.*, Am. Water Works Assoc., 1923; **10**, 309.
3. Parker, P. A. M.: "Control of Water," 1913.
4. Ardern, Edward, and W. T. Lockett: *Jour.*, Soc. of Chem. Industry, 1914; **33**, 523.
5. Harris, F. W., T. Cockburn and T. Anderson: *Water Works*, 1927; **66**, 24.
6. Ann. Rept., Milwaukee Sewerage Comm., 1915.
7. *Bull.* No. 14, Ill. State Water Survey, 1916.
8. Parsons, A. S.: *Water Works and Sewerage*, 1929; **76**, 397.

9. PARSONS, A. S., and H. WILSON: *Surveyor*, 1927; **72**, 221.
10. *Eng. News-Rec.*, 1923; **90**, 835.
11. GRANT, S., E. HURWITZ and F. W. MOHLMAN: *Sewage Works Jour.*, 1930; **2**, 228.
12. Ann. Rept., Milwaukee Sewerage Comm., 1916.
13. ARDERN, EDWARD: *Jour.*, Soc. of Chem. Industry, 1917; **36**, 822.
14. BELL, H. D.: *Water Works and Sewerage*, 1930; **77**, 51.
15. *Surveyor*, 1927; **71**, 309.
16. GOUDEY, R. F.: *Water Works and Sewerage*, 1932; **79**, 177.
17. FRIES, FRANZ: *Gesundheits-Ingenieur*, 1931; **54**, 681.
18. *Surveyor*, 1933; **83**, 152.
19. *Eng. News-Rec.*, 1929; **103**, 1005.
20. GOUDEY, R. F.: *Eng. News-Rec.*, 1931; **106**, 443.
21. *Eng. News-Rec.*, 1930; **102**, 111.
22. BENDER, E.: *Public Works*, 1930; **61**, no. 9, p. 33.
23. *Sewage Works Jour.*, 1934; **6**, 159.
24. MARTIN, A. J.: *Surveyor*, 1929; **76**, 336.
25. WHITEHEAD, H. C.: *Surveyor*, 1925; **68**, 449.
26. Ann. Rept., Mass. State Bd. of Health, 1913.
27. MARTIN, A. J.: "The Activated Sludge Process," 1927.
28. *Surveyor*, 1923; **64**, 67.
29. EDWARDS, G. P.: *Sewage Works Jour.*, 1930; **2**, 8.
30. O'SHAUGHNESSY, F. R.: *Sewage Works Jour.*, 1930; **2**, 255.
31. HATFIELD, W. D.: *Sewage Works Jour.*, 1931; **3**, 621.
32. GREELEY, S. A., and W. D. HATFIELD: *Trans.*, Am. Soc. C. E., 1930; **94**, 544.
33. HATFIELD, W. D.: *Jour. Ind. Eng. Chem.*, 1928; **20**, 832.
34. KENNEDY, C. C.: *Sewage Works Jour.*, 1929; **1**, 622.
35. MOHLMAN, F. W., and C. E. WHEELER: *Sewage Works Jour.*, 1930; **2**, 529.
36. SIERP, F.: *Bauamt und Gemeindebau*, Sept. 5, 1930.
37. GOUDEY, R. F., and S. M. BENNETT: *Water Works and Sewerage*, 1933; **80**, 179.
38. O'BRIEN, W. S.: *Sewage Works Jour.*, 1934; **6**, 42.
39. RIDENOUR, G. M., and C. N. HENDERSON: *Sewage Works Jour.*, 1934; **6**, 36.
40. *Pub. Health Repts.*, 1935; **50**, 143.
41. Rept. on Industrial Wastes from Stockyards and Packingtown in Chicago, vol. 2, 1921.
42. Ann. Rept., Milwaukee Sewerage Comm., 1920.
43. *Eng. News-Rec.*, 1928; **101**, 469.
44. Ann. Rept., Manchester Rivers Dept., 1928.
45. AGERSBORG, H. P. K., and W. D. HATFIELD: *Sewage Works Jour.*, 1929; **1**, 417.
46. CRAMER, ROBERT: *Jour. Ind. Eng. Chem.*, 1931; **23**, 309.
47. MORGAN, E. H., and A. J. BECK: *Sewage Works Jour.*, 1928; **1**, 46.
48. *Pub. Health Bull.* 132, U. S. Pub. Health Service, 1923.
49. DONALDSON, WELLINGTON: *Sewage Works Jour.*, 1932; **4**, 48.

50. Mohlman, F. W.: *Sewage Works Jour.*, 1934; **6**, 119.
51. Richards, E. H., and G. C. Sawyer: *Jour.*, Soc. of Chem. Industry, 1922; **41**, 62 T.
52. Bull. No. 18, Ill. State Water Survey, 1922.
53. Ardern, Edward, Clarence Jepson and Percy Gaunt: *Jour.*, Soc. Chem. Ind., 1923; **42**, 230T.
54. Reddie, J. A.: *Proc.*, Sewage Works Managers' Assoc., 1925, p. 94.
55. Ann. Rept., Manchester Rivers Dept., 1926.
56. Pratt, R. W., and G. B. Gascoigne: *Eng. News*, 1916; **76**, 1061.
57. Lederer, Arthur: *Eng. News*, 1916; **75**, 932.
58. Eddy, H. P., and A. L. Fales: *Jour. Ind. Eng. Chem.*, 1916; **8**, 648.
59. Fischer, A. J.: *Sewage Works Jour.*, 1931; **3**, 30.
60. Ann. Rept., Milwaukee Sewerage Comm., 1921.

CHAPTER XXV

GENERAL DESIGN OF ACTIVATED-SLUDGE PLANTS AND DETAILS OF MECHANICAL AERATION

There are certain general conditions which affect the design of activated-sludge units. Many of these are common to all types of sewage-treatment works, but the nature of the activated-sludge process introduces other conditions that are not so important in other types of sewage treatment. These conditions may be listed as follows:

1. Quantity, strength and character of sewage to be treated, including the effects of preliminary treatment and the influence of industrial wastes.
2. Quality of effluent desired, including consideration of separate sludge re-aeration and partial treatment by activated sludge with subsequent treatment by other oxidation devices.
3. Quality of sludge to be produced, including evaluation of its influence upon the treatment process itself and upon subsequent treatment, disposal and commercial utilization of the sludge.

These general conditions influence in one way or another a number of design and operating factors, many of which are mutually dependent. Among the design and operating factors which will be considered in subsequent sections are: period of aeration; functions of aeration and circulation; and proportion of return sludge. Variation in one of these factors may be offset, within limits, by suitable adjustment of the others.

In considering the design and operating factors, reference will be made chiefly to the requirements of the normal process as established by American experience up to the present time. Departures from it, particularly as employed abroad, will receive only passing attention.

Period of Aeration.—The *period of aeration* is generally expressed in hours and corresponds to the detention period of sedimentation units, allowance being made for the volume of return sludge. The period of aeration in hours is obtained, therefore, by dividing the tank capacity, in cubic feet or gallons, by the quantity of sewage and return sludge, in cubic feet or gallons per hour.

The progressive nature of the changes which take place in the quality of the sewage during aeration is shown in Table 103, giving the results of applying a continuous supply of 160 cu. ft. of air a minute to sewage which originally had 235 p.p.m. of suspended matter (1). These tests were made in 1915 at the Milwaukee testing station by Copeland. The data in this table indicate that all that is necessary to obtain an effluent of the desired quality is to select the appropriate detention period for the operating conditions. This, however, is not so simple as it appears, for these conditions, which are determined by factors such as the design of the tanks, the rate of application of the air, the character of the sewage and the quality and volume of the return sludge, may vary greatly and continuously in the same plant.

TABLE 103.—RESULTS OBTAINED BY AERATION OF SEWAGE FOR VARIOUS PERIODS OF TIME

Aeration, hours	0	1	2	3	4	5
Cu. ft. of air per minute	0	160.00	160.00	160.00	160.00	160.00
Cu. ft. of air per gallon	0	0.67	1.32	1.98	2.64	3.31
Appearance of settled liquor[1]	Turbid	Clear	Clear	Clear	Clear	Clear
Stability, hr	0	2.00	33.00	120+	120+	120+
Bacteria removed, per cent	0	52.00	81+	92+	95+	98+
Free ammonia, p.p.m	22.00	17.00	15.00	11.00	7.00	5.00
Nitrites, p.p.m	0.08	0.00	0.95	1.75	2.20	2.50
Nitrates, p.p.m	0.08	0.04	0.70	2.80	5.60	8.20
Dissolved oxygen, p.p.m	0.00	0.30	1.90	4.30	5.90	6.70

[1] The suspended matter carried by the sewage averaged 235 p.p.m. and the supernatant liquor after 1 hour's aeration contained not more than 10 p.p.m.

Tests made at the Houston activated-sludge experiment station during 1915, the results of which are given in Table 104, indicated that a relatively high degree of clarification and purification could be secured with 1 hour's aeration (2). The experimental aeration tank was 7.5 ft. deep and the ratio of tank area to diffuser area was 12.25 to 1. The free air supplied was 0.437 cu. ft. an hour per gal. of sewage.

The tank operated with 25 to 30 per cent of return sludge. All results are averages from a large number of samples. The relative stability at 20°C. was never less than 16 days after 1 hour's aeration.

For American municipal sewages the economical period of aeration by diffused air appears to lie between 2 and 6 hr., depending upon the kind of preliminary treatment, the degree of purification desired and the use of re-aeration. At Milwaukee, where fine screens precede aeration and it is desired to produce a suitable sludge for dewatering, drying and sale as fertilizer, a 6-hr. aeration period was provided for in the design. The same period was provided for in the design of the North Side plant at Chicago, after partial clarification of the sewage in settling tanks with a ½-hr. detention period. The design of the Wards Island plant at New York provides for an aeration period of 5¼ hr. after

TABLE 104.—RESULTS OF OPERATION OF EXPERIMENTAL AERATION TANK AT HOUSTON, TEXAS, 1915

	Crude sewage, p.p.m.	Aeration period							
		1 hr.		2 hr.		3 hr.		4 hr.	
		P.p.m.	Per cent removal	P.p.m.	Per cent removal	P.p.m.	Per cent removal	P.p.m.	Per cent removal
Total organic nitrogen	5.5	2.0	64	1.9	65	1.87	66	1.8	67
Free ammonia	6.32	2.50	60	1.44	82	0.63	90	0.32	95
Total organic matter	299	137	40	134	41	134	41	129	44
Oxygen consumed	139	18	87	17	88	15	89	14	90
Dissolved oxygen consumed	103	3	97	2	98	2	98	2	98
Suspended solids	253	5	98	5	98	5	98	4	98
Nitrites and nitrates	Trace	4	....	8	....	10	....	11	
Bacteria	2,800,000 per cc.	176,000 per cc.	93.7	118,000 per cc.	95.8	117,000 per cc.	95.8	116,500 per cc.	95.8

sedimentation for 1 hr. under average conditions of flow. The North Toronto plant at Toronto, Ont., provides for settling the sewage 2 hr. prior to aeration for 5.75 hr. A 4-hr. period of aeration is allowed for at Elyria, Ohio, following ½ to 1 hour's sedimentation in Imhoff tanks. Operation of the Indianapolis plant calls for fine screening of part of the flow and rapid settling of the remainder. The designed aeration period is 4.7 hr. The North Side plant at Houston and the plant at Pasadena provide aeration periods of 2.0 and 4.4 hr. without presedimentation but employ re-aeration periods of 1.0 and 2.0 hr., respectively. For aeration periods in bioflocculation tanks see Chap. XXIV.

In determining upon the allowance for aeration period, consideration should be given to the variations in dry-weather flow of sewage, the maximum rate of dry-weather or storm flow to be treated in the aeration units and the variations in rate of return sludge. A daily peak rate of sewage flow of 130 to 140 per cent of the average is common, and it may be much higher in small plants. A maximum month may have a sewage flow 20 to 25 per cent greater than the annual average. The daily 4-hour peak flow during the maximum month may be 150 per cent or more of the annual average rate. It is common practice in design to allow for treating in the activated-sludge process sewage and storm flows up to 150 per cent of the average sewage flow. The rate of return sludge varies over a wide range as between different plants and frequently at any particular plant. Rates from 20 to 60 per cent of the average sewage flow have been employed. With an aeration period of 6 hr. based on the annual average sewage flow, including 25 per cent return sludge, the aeration period with a sewage flow 50 per cent greater than the annual average and 30 per cent return sludge would be 3.85 hr. Correspondingly, an aeration period of 4 hr. for average flow and 25 per cent return sludge would be reduced to 2.57 hr. for the maximum flow condition assumed above. The effect of the shorter aeration periods during maximum rates of flow to be treated may be a vital factor in the basic allowances, if it is necessary to obtain at all times a high degree of purification.

The normal detention period for which mechanical aeration tanks are designed is commonly longer than for diffused-air tanks. The Simplex plant at Princeton, Ill., provides a detention period of 8 hr. during the 16-hr. period of maximum flow and that at Lawton, Okla., has a tank capacity of 8 hr. with 4.8 hr. for the re-aeration unit. The aeration period at Sheffield, England, is 15 hr., while tests of an experimental Link-Belt unit at the Des Plaines treatment works at Chicago indicated a required period of 6 hr.

At Essen-Rellinghausen, Germany, the aeration units provide for 3½ hr. of flow in treating a settled, relatively weak sewage by a combination of diffused air and mechanical agitation. A similar experimental unit at the Calumet works of Chicago has yielded satisfactory results with a 5-hr. detention period.

The period of aeration must naturally be longer when a highly nitrified effluent is to be produced than when clarification is the criterion of treatment.

Functions of Aeration and Circulation.—According to Buswell (3),

> When air is blown into the aeration chamber of an activated-sludge plant it does three things: (1) It maintains the sludge in suspension. (2) It maintains aerobic conditions. (3) It stirs up the mixture, allowing fresh liquor to come in contact with the sludge. We are not able to tell, therefore, which one of these three factors determines the critical minimum air requirement.

By eliminating the first and third functions of the air in an experimental unit in which circulation was maintained by stirring, Buswell was able "to produce an abundant growth of sludge, very good clarification and a substantial purification in from 3 to 6 hr. when using as low as 0.002 cu. ft. of air per gallon" and came to the following conclusions:

> Stirring apparently is very much more important than is oxygen. Stirring will sweep away the saturated film from the air surface and bring it in contact with the activated sludge particles. It will move the dissolved and colloidal organic matters about so that they come in contact with the floc and will sweep away the metabolic products of the sludge organisms.

Oxygen Requirements.—It follows from Buswell's experiments that 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 (4) 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. By combining mechanical agitation with diffused-air aeration, Imhoff (5) obtained 30 per cent utilization of oxygen at Essen-Rellinghausen, Germany. The quantity of oxygen taken up by the floc in the activated-sludge process has been studied more recently by Grant, Hurwitz and Mohlman (6), who draw the following conclusions from experiments with activated sludge and sewage from the North Side treatment works of Chicago.

> 1. The rate of absorption of oxygen by activated sludge, in the amounts and periods of aeration commonly used, is fairly uniform, and varies directly in proportion to the weight of sludge.
>
> 2. The rate of absorption of oxygen varies, further, in proportion to the content of organic (volatile) matter in the sludge.
>
> 3. The rate of absorption of oxygen by North Side activated sludge averaged approximately 7.0 mg. of oxygen per gram of sludge per hour.

4. Mixtures of sewage and sludge absorb oxygen in proportion to the weight of sludge, but with increasing amounts of sludge most of the oxygen-demanding material of the sewage is coagulated or adsorbed by the sludge, so that very little oxygen is required for the unadsorbed material in colloidal or true solution. The subsequent rate of oxidation of the coagulated material in the sludge is slow and comparatively uniform. The sludge passes through many cycles of aeration before it is removed, and during these successive aerations the organic matter is slowly oxidized.

5. Based on reduction of B.O.D. our results so far do not indicate that the period of aeration and amount of sludge vary inversely (that is, that the period of aeration $\times$ per cent of sludge = constant) as claimed by Harris, Cockburn and Anderson (7). Small amounts of sludge reduced the B.O.D. of the sewage almost as rapidly as large amounts. Re-aeration of sludge is indicated, however, when small amounts are used because of the shorter period of retention.

6. Adjustment of the rate of air supply should be governed by the amount of sludge in the aeration tanks. With large amounts a fairly uniform rate is desirable. Rates of aeration in proportion to the B.O.D. of sewage, as recommended by Jenks and Levine (8), are greatly modified by the oxygen requirements of the sludge.

Velocity of Circulation.—The velocity necessary to maintain circulation of sludge in the aeration units depends to a considerable extent upon the design of the tanks. It depends, too, upon the relative absence of heavy or gritty substances which weigh down the sludge or settle by themselves. If pocketing is prevented, the velocity of the main body of sewage may be made lower than when there is a chance for the establishment of eddy currents and dead areas. Velocities along the tank bottom are particularly important. Hurd found at Indianapolis that with spiral flow, minimum velocities of ½ ft. a second prevented deposition of solids. Design values vary from this up to 1½ ft. a second, the figure used at Sheffield, England. The Sheffield value, however, represents the average channel velocity and the bottom velocity is probably appreciably lower. The velocity of circulation may differ considerably from the rate of longitudinal travel.

Quantity of Air Required in Diffused-air Aeration.—The volume of air required in diffused-air aeration may be stated in two ways for purposes of comparison: as cubic feet per gallon of sewage treated and as cubic feet an hour per square foot of tank surface. Since the degree of compression varies for different plants, "free air conditions" are commonly specified, *i.e.*, air at the prevailing atmospheric conditions of pressure, temperature and moisture, commonly 30 in., 60°F. and 50 per cent saturation, respectively. Both methods of statement yield significant information as regards plant design and operation, the first being used more widely than the second. To change from cubic feet of air per gallon of sewage to cubic feet of air an hour per square foot of tank surface, multiply 7.48 times the former value by the tank depth in feet and divide by the period of aeration in hours. Thus 1.0 cu. ft. a gallon in a 10-ft. tank with a period of aeration of 6 hr. equals

$$\frac{1.0 \times 7.48 \times 10}{6} = 12.5 \text{ cu. ft. an hour per square foot.}$$

Air used for re-activating the sludge and pumping it back into the aeration tanks commonly is included when computing the quantity of air required. The quantities used for purposes other than supplying air to the aeration units usually are kept separate, however, in order to permit determining the air requirements for the different purposes that the air may serve.

Among the factors whose relation to the quantity of air needed has been studied or otherwise recognized are the required degree of purification; the kind of preliminary treatment; the means of air diffusion and the arrangement of plates; the dimensions of aeration units, and the strength of the sewage.

The required degree of purification in some measure determines the quantity of air needed. This is illustrated by data in Table 103. With 0.67 cu. ft. of free air per gallon of sewage, not much but clarification was obtained; with 1.98 cu. ft., fairly good purification resulted; and with 3.31 cu. ft., still further improvement took place.

As already pointed out, removal of solids by fine screening may be of substantial aid in the prevention of clogging of diffuser plates or the settling of heavy solids in the tank and may reduce the quantity of air otherwise required. With preliminary sedimentation, which is much more efficient in the removal of solids than fine screens, the air supply may be appreciably reduced from that required in the absence of presedimentation units.

The means of diffusion may have an important bearing upon the quantity of air required, for some systems are more wasteful than others.

Apparently the quantity of air required is not affected appreciably by the depth of tank. From a study of the records of parallel tests on aeration tanks of the ridge-and-furrow type, 10 and 15 ft. deep, at the Milwaukee demonstration plant in 1919, Eddy, Fales, Copeland, Ferebee and Hatton (9) reported as follows:

> There was a slight difference in favor of the 15-ft. depth of tank not taking into consideration the extra cost of compressing the air for a 15-ft. depth of liquor over that for a 10-ft. depth.

As a result of operation of the experimental plant of the ridge-and-furrow type at Houston, Texas, Sands (2) came to the following conclusion:

Tests using different depths of sewage in the tanks demonstrated the fact that the purification accomplished is not a function of the number of cubic feet per gallon of sewage, but is a function of the number of cubic feet of air per unit of time per square foot of tank surface.

The strength of sewage is of particular significance in connection with air requirements, as a material increase in the period of aeration, rate of air supply, or both, may be needed as the strength of sewage increases. Experiments with two widely divergent sewages will illustrate this point. At Houston, Texas, tests with a domestic sewage of average strength indicated that it was possible by 1-hr. aeration, using 0.44 cu. ft. of air per gal. of sewage, to secure a stability of more than 16 days. At the Decatur testing station, on the other hand, tests with a strong sewage, affected by wastes from starch works, indicated that about 16 hours' aeration and 4.0 cu. ft. of air a gallon might be required for complete treatment after preliminary sedimentation, in order to turn out an effluent containing sufficient nitrates to insure stability (10).

The air consumption of a number of American plants is shown in the following tabulation:

City	Year	Type of aerators	Preliminary treatment	B.O.D. of activated-sludge influent, p.p.m.	Aeration period, hr.	Air used in tanks and channels, cu. ft. per gallon of sewage
Chicago, Ill.[1]	1930	Spiral flow	Sedimentation, 0.5 hr.	84	5.0	0.50
Indianapolis, Ind.	1931	Spiral flow	[2]	225	8.4	1.15
Milwaukee, Wis.	1933	Ridge and furrow	Fine screens	268	5.8[3]	1.56
Pasadena, Cal.	1932	Ridge and furrow	Fine screens	201	4.9	1.64[4]
Peoria, Ill.	1932	Spiral flow	Sedimentation, 2.8 hr.	165	9.6	0.98
San Antonio, Tex.	1933	Spiral flow	Sedimentation, 1.0 hr.	160[5]	7.7	0.97[4]
Springfield, Ill.	1932	Spiral flow	Sedimentation, 1.0 hr.	150	5.0	0.91
Toronto, Ont.[6]	1932	Spiral flow	Sedimentation, 1.8 hr.	154	4.9	1.01

[1] Battery B, North Side plant.
[2] 37 per cent of flow treated by "rapid settlers."
[3] Including detention period in aerated channels, the total aeration period was 6.7 hr.
[4] Including re-aeration of return sludge.
[5] Based on 25 per cent reduction in B.O.D. of raw sewage by 1 hour's sedimentation.
[6] North Toronto plant.

Most of the values approach 1 cu. ft. per gallon of sewage. Where preliminary sedimentation is employed, there is a tendency to provide somewhat less air as well as shorter detention periods. The air requirements of plants employing re-aeration do not seem to be lower in American experience than those of works not so operated. Partial treatment naturally decreases the air consumption considerably from that required for complete treatment of the same sewage.

As the cost incident to the use of a large quantity of air is burdensome in operating expense and in fixed charges upon the installation, it is necessary to reduce it to the minimum practical limit. Determination of this limit requires careful consideration, not only of the degree of purification secured, but also of the properties of the sludge produced, taking into account its concentration, effectiveness as return sludge and, in certain cases, its dewatering characteristics and its fertilizing value. Hatton (11) has given the following opinion on the subject:

> From the author's somewhat limited knowledge of the methods so far pursued in England in developing the activated sludge process, he has the opinion that the aim has been to produce a clarified and stable effluent with the least expenditure of air and without much attention being given to the character of the sludge produced. From the author's viewpoint both sides of the problem must be solved together, that is the character of the sludge produced is of equal importance to the engineer as is the character of effluent produced, and if by using more air or longer contact a sludge can be produced which is higher in available nitrogen or which can be dewatered at less expense, then the additional air and greater period of contact should be used. It follows naturally that these additions also produce a higher quality of effluent.

Diffused Air versus Mechanical Aeration.—Advocates of mechanical means of aeration have contended that the aerating value of air bubbles is not an essential feature of activated-sludge tanks employing diffused air and that the absorption of atmospheric oxygen which takes place at the surface of diffused-air units is sufficient to maintain the oxygen requirements of the process. While the latter is probably so, recent experiments seem to show that air bubbles do give up their oxygen effectively during passage through the sewage and that the claim is untrue, or only partly true, that a film of liquid travels with the bubbles or colloidal matter in the sewage, forms a layer around each bubble and prevents, to some extent, the passage of oxygen into the liquid (12) (13). These experiments support the contention of Hurd that the advantage of air circulation over common methods of mechanical agitation is that, with equal opportunity for economy due to the surface absorption of oxygen, there is an assurance of always having a sufficient quantity of entrained air to give positive oxidation throughout the sewage mass (14). In this connection Whitehead has also stated that one cannot brand compressed air as mechanically inefficient; one great advantage that compressed air has over all means of mechanical agitation known at the present time is that it diffuses its power in a wonderfully microscopic and even manner throughout the whole mass (15).

Buswell (3), on the other hand, has expressed the following opinion:

Stirring by means of blowing air through sewage is, however, a rather inefficient process. We do not get as much stirring for the power input as we would if the power were applied in some other way. When air is blown into sewage, the activated sewage flocs, air bubbles and liquid are all driven in the same direction. The most efficient process would be one in which the air surface and flocs would be held stationary while the liquid is caused to flow through them.

The fact that a longer period of aeration is ordinarily required in mechanically aerated units than in diffused-air units, to secure equivalent results, is generally taken to be indicative that diffused air is a more effective agent than mechanical equipment.

Diffused Air Combined with Mechanical Aeration.—Experimental evidence on the efficiency of mechanical agitation plus diffused air has been gathered at Chicago since 1923. Analytical and operating results, obtained by Mohlman and Wheeler (16) in a large-scale test at the Calumet plant, are presented in Tables 105 and 106. According to these workers the analyses indicated that after April, 1929, a satisfactory effluent was produced while treating 1.5 m.g.d. with an aeration period of 5.0 hr. and an air consumption of 0.20 to 0.25 cu. ft. a gallon. Com-

TABLE 105.—OPERATING DATA ON USE OF DIFFUSED AIR PLUS MECHANICAL AGITATION AT CALUMET TREATMENT PLANT, CHICAGO

Month, 1929	Sewage treated, m.g.d.	Aeration period, hr.	Return sludge, per cent of sewage	Air consumption (free air)		Suspended solids in aeration tank, p.p.m.	Settling rate in Dorr tank, gal. daily per sq. ft.
				Cu. ft. per gal.	Cu. ft. per plate per min.		
Feb.	1.78	4.21	15.8	....	....	....	1571
March	1.67	4.67	19.1	0.27	1.71	....	1474
April	1.81	4.03	19.0	0.26	2.07	1982	1597
May	1.79	4.15	17.1	0.21	1.63	3016	1582
June	1.53	4.77	19.7	0.25	1.64	4683	1352
July	1.44	5.04	19.6	0.25	1.57	4456	1272
Aug.	1.44	5.05	19.4	0.23	1.43	4039	1270
Sept.	1.44	5.06	19.5	0.19	1.16	3511	1267
Oct.	1.44	5.05	18.8	0.21	1.34	2976	1270
Nov.	1.46	5.01	19.1	0.21	1.33	3262	1286
Dec.	1.46	5.01	18.8	0.21	1.31	2629	1286

TABLE 106.—ANALYSES OF EFFLUENT FROM TANK USING DIFFUSED AIR PLUS MECHANICAL AGITATION AT CALUMET TREATMENT PLANT, CHICAGO

Parts per Million

Month, 1929	Nitrogen as				Dissolved oxygen	5-day B.O.D.	Suspended solids			Relative stability, days
	Organic	Ammonia	Nitrite	Nitrate			Total	Volatile	Fixed	
Feb.	4.2	5.3	0.1	0.2	9.3	63.0	80	47	33	3.5
March	3.7	3.9	0.2	1.5	9.1	38.0	68	41	27	6.0
April	2.7	3.0	0.5	1.5	7.4	13.0	58	29	29	9.5
May	2.5	3.7	0.0	0.4	3.5	8.3	29	16	13	7.8
June	2.2	3.5	0.2	0.4	4.3	8.1	35	20	15	8.3
July	1.5	0.6	1.2	0.7	4.4	6.0	18	10	8	9.8
Aug.	1.4	2.3	0.2	0.8	3.4	9.0	16	10	6	9.2
Sept.	1.8	4.1	0.3	0.4	3.5	10.9	12	8	4	9.4
Oct.	1.7	4.6	0.3	0.5	4.1	9.8	14	8	6	9.5
Nov.	1.5	4.9	0.1	0.4	3.6	7.7	18	11	7	9.4
Dec.	2.6	4.6	0.1	0.4	3.7	11.0	28	19	9	7.4

paring the results for mechanical aeration plus diffused air with observations on a similar tank operating during 1927 with similar sewage but with diffused air alone, Mohlman and Wheeler state that their best judgment indicates "that it would require 0.80 cu. ft. of air per gallon with air alone and a 5-hr. aeration period to produce an effluent comparable with that produced in 1929 by paddle-wheel plus air, with 0.20 cu. ft. per gal. and a 5-hr. period."

At Essen-Rellinghausen, Germany, Imhoff (17) has operated since 1926 a plant in which paddle-wheels are opposed by diffused air. An air consumption of 0.08 to 0.14 cu. ft. per gallon of sewage and a power requirement of 7 to 8 hp. per mil. gal. are reported.

According to Besselievre, Dorrco aerators, covered by patents granted to Imhoff with added improvements in design, have been installed at Phoenix, Ariz., Los Angeles, Cal., Escanaba and Muskegon

TABLE 107.—OPERATING RESULTS OF COMBINED PADDLE-WHEEL DIFFUSED-AIR AERATORS

	Escanaba, Mich.	Muskegon Heights, Mich.	Newark, N. Y.	Phoenix, Ariz.
Number of tanks	3	4	2	6
Length by width, ft.	68 × 15	91 × 14	75 × 27	330 × 27
Depth, ft.	14	14	14	14
Time of test[1]	July 13–21, 1932	Aug. 1–31, 1932	April 1–30, 1933	Sept. 1–30, 1932
Sewage flow:				
Design basis, m.g.d.	1.00	1.67	1.50	12.0
During test, m.g.d.	0.69	0.78	0.90	9.5
During test, per cent of design basis	69.0	46.7	60.0	79.2
Suspended solids:				
Plant influent, p.p.m.	373.0	313.0	296.0	106.0
Final sedimentation-tank effluent, p.p.m.	24.0	12.0	10.0	29.0
Removal, per cent	93.5	96.2	96.8	72.6
Biochemical-oxygen demand, 5 days:				
Plant influent, p.p.m.	393.0	655.0[2]	......	174.0
Final sedimentation-tank effluent, p.p.m.	17.0	5.0	......	17.0
Removal, per cent	95.7	99.2	......	90.2
Dissolved oxygen, p.p.m.:				
influent	0.0	0.0	......	0.0
Effluent	3.8	3.4	......	2.4
Air consumption, cu. ft. per gal.	0.25	0.647	0.60	0.268
Power input to air compressors, hp. per mil. gal.	7.8	17.2	14.2[3]	6.0
Power input to revolving paddles, hp. per mil. gal.	11.0	15.8	13.1	11.3
Total power input to aerators, hp. per mil. gal.	18.8	33.0	27.3[3]	17.3

[1] All determinations based on hourly readings and hourly samples composited daily.
[2] High B.O.D. due to tannery wastes.
[3] Power for air compressor in general derived from sludge gas.

Heights, Mich., and Newark, N. Y. The distinctive principle of the Imhoff design is the creation of a rotary motion of the sewage, mechanically induced by paddles, which is countercurrent to the upward flow of air bubbles injected through a row of diffuser plates, located along one side of the tank only. The operating results from four installations of Dorrco aerators are given in Table 107 (18).

Proportion of Return Sludge.—An ample supply of activated sludge in the aeration tanks is essential to the success of the process. The proportion required depends upon the character of the sewage, the degree of purification to be secured, the period of aeration, the concentration of solids in the return sludge, the concentration of solids desired in the mixed liquor and other considerations.

There appear to be three different concepts of the proportion of return sludge: the percentage ratio of the volume of return sludge, as measured by a venturi meter or other measuring device, to the volume of incoming sewage; the percentage or p.p.m. by weight of suspended solids in the mixed liquor at the aeration-tank inlet; and the percentage volume of sludge deposited in a settling glass during a given length of time, as, for example, ½ hr., 1 hr. or 2 hr., from a sample of mixed liquor taken at the aeration-tank inlet. The first and third concepts should not be confused. The first should be referred to as the "per cent of return sludge"; the third as the "per cent of sludge in aeration tanks, determined by ½, 1 or 2 hours' settling," as the case may be.

A knowledge of the volume of return sludge is required in the design of the sludge-return equipment,—pipes or channels and pumps or air lifts. It is for this purpose that the first conception of the proportion of return sludge is useful. If the volume and suspended-matter content of the return sludge and incoming sewage are known, the percentage weight or parts per million of suspended solids in the mixed liquor can be computed. Naturally, this value can also be determined directly from the dry weight of suspended solids in the mixed liquor. The result obtained constitutes the second concept of the proportion of return sludge and defines an important operating feature of the activated-sludge process, namely, the opportunity for contact between activated sludge and fresh sewage. Similar to this second concept, but quantitatively less reliable, is the third, which has the advantage, however, of being obtained by a simple and expeditious test, described later in this chapter.

To illustrate these three concepts, let it be assumed that 250,000 gal. of return sludge containing 2 per cent solids are mixed with 1,000,000 gal. of sewage containing 300 p.p.m. of suspended solids. The propo tion of return sludge then is: $250{,}000/1{,}000{,}000 \times 100 = 25$ per cent by volume of the sewage treated; $\frac{25 \times 2 + 100 \times 0.03}{125} = 0.402$ per cent or 4020 p.p.m. by weight of the mixed liquor, or, from experience with this type of sludge, about 20 per cent by volume of the mixed liquor, based upon a settling period of 1 hr.

With reference to the percentage of sludge in the aeration tanks settling from the mixed liquor in 1 hr., the literature shows that in general the quantity of 98 per cent sludge required for the treatment of moderately strong sewage is equivalent to about 25 per cent of the volume of sewage treated. This quantity may be stated also as about $25/1.25 \times 0.02 = 0.4$ per cent of dry suspended solids by weight, referred to the total weight of mixed liquor.

Certain observers have found little difference in results obtained with sludge volumes varying from 7 to 30 per cent of the volume of mixed liquor, but the consensus of opinion seems to be that there is an optimum ratio, which depends upon the character of the sewage and sludge, the time of aeration and the volume of air supplied. With strong sewage or with activated sludge of high moisture content, proportions as high as 50 per cent may be required, particularly when the sewage contains much colloidal matter. The volume of return sludge usually is kept as low as possible, to secure economy of pumping and tank capacity and because an unnecessarily large volume of sludge requires more air to keep it active. On the other hand, the larger the proportion of sludge in the aeration tank, the greater is its reserve capacity to deal with variations in the quantity and quality of the incoming sewage. The relation between proportion of return sludge and aeration period has been studied particularly by Harris (7) and his collaborators at Glasgow, Scotland, working with a large-scale unit. Keeping the air supply and agitation constant, they determined the minimum sludge ratio and contact period to produce equal purification. The minimum effective proportion of sludge was found to be 8 per cent with a contact period of 4.2 hr. The product of these two values, 33.6, was called the *coefficient of interfacial contact*. The use of this coefficient is said to have been of practical value at the Shieldhall works in Glasgow, in making it possible to maintain good operating conditions and increased plant capacity.

As already indicated, there is an intimate relation between the period of aeration, the quantity of air required and the proportion of activated sludge. Variations in one of these factors may be offset, within limits, by appropriate adjustment of the others. It appears to be important to secure the most effective and economical regulation of these quantities. In this connection Fugate (19) states:

> . . the strength and concentration of sewage to be treated, the time of aeration, the quantity of air and percentage of sludge carried, are shown to be closely related. Considerably more air and time are required to obtain the same results with percentages of sludge of 25 in the aeration channel,

and 75 in the re-aerating channel, than with 10 and 30, respectively, although the ratio of returned liquid is the same. The best results are noticed when the percentage of sludge does not exceed 15 and 35, all other factors being equal.

Attention is called to the fact that due allowance for the return sludge is necessary in stating and determining upon aeration and final-sedimentation periods or capacities. The quantity of air supplied, however, commonly is expressed in terms of sewage flow only.

The ratio of return sludge to sewage flow utilized in the operation of several activated-sludge plants is given in the following tabulation:

City	Year	Return sludge, per cent of sewage flow		
		Maximum month	Minimum month	Average for year
Chicago, Ill.[1]	1930	28	20	22
Indianapolis, Ind.	1931	63	30	38
Milwaukee, Wis.	1933	34	15	24
Peoria, Ill.	1932	42	20	25
San Antonio, Tex.	1933	58	45	52
Springfield, Ill.	1932	29	15	20
Toronto, Ont.[2]	1932	22	17	19

[1] Battery B, North Side plant.
[2] North Toronto plant.

Quantity of Sludge to Be Handled.—The sludge deposited in the sedimentation units following aeration tanks consists, broadly speaking, of the activated sludge which has been discharged into the influent, augmented by the materials removed from the flowing sewage. A large part of this sludge is again discharged into the influent of the aeration units as return sludge and a small part is disposed of as excess sludge. Both usually require handling by pumps. Limited aeration of raw sewage with activated sludge produces a clear effluent after sedimentation and the quantity of sludge is probably as great as, or greater than, that yielded by carrying treatment to a point where a thoroughly nitrified effluent is obtained. The quantity of excess sludge depends primarily upon the quantity of suspended and colloidal matter in the sewage, which varies considerably, particularly when the sewage is subject to preliminary clarification or when it contains storm water or certain classes of industrial wastes, such as iron salts and organic matter from packinghouses and tanneries.

Experience indicates that a short period of aeration with good activated sludge, followed by sedimentation, may reduce the suspended matter carried by the influent by about 90 per cent. A portion of the suspended matter thus removed is converted into gaseous and dissolved substances by oxidation, the remainder forming sludge. The proportion of settleable solids thus removed by oxidation is subject to considerable doubt. Values as high as 50 per cent and as low as 4 per cent have been reported, the lower values being more probable under average conditions of operation. The proportions, character and volumes of sludge handled at a few American plants are given in Table 108, together with a hypothetical case, which will illustrate the theoretical relations. No allowance has been made in the hypothetical case for "bacterial digestion of sludge," which is discussed in Chap. XXIV.

It should be noted that preliminary sedimentation is not provided in any of the cases cited, except at Indianapolis, where a portion of the flow is treated by a short settling period. If 50 per cent of the suspended solids were removed by preliminary sedimentation, the volume of sludge produced per mil. gal. would be about one half as large in each case but the volume of return sludge might remain the same.

TABLE 108.—QUANTITY OF SLUDGE HANDLED IN ACTIVATED-SLUDGE PLANTS

	Milwaukee	Pasadena	Indianapolis	Hypothetical case
Year	1928	1928–29	1927	
Quantity of sewage treated, m.g.d.	82.16	7.7	15.4	1.0
Water content of sludge, per cent	98.46	99.18	98.3	98
Specific gravity of sludge	1.00	1.003	1.00[7]	1.005
Weight of dry solids produced, lb. per mil. gal.	2,085	2,270	1,950	2,250[1]
Volume of excess sludge produced, gal. per mil. gal.	16,550	33,000	13,700	13,500[2]
Volume of return sludge, gal. per mil. gal.	196,000	450,000	217,000	250,000[3]
Total sludge from sedimentation tanks, gal. per mil. gal.	212,550	483,000	230,700	263,500
Return sludge, per cent of total	92.2	93.2	94.1	94.9[4]
Excess sludge, per cent of total	7.8	6.8	5.9	5.1[5]
Excess sludge, per cent of sewage treated	1.66	3.3	1.37	1.35[6]

[1] On the assumption of 90 per cent removal of 300 p.p.m. suspended solids, $0.9 \times 300 \times 8.33 = 2250$ lb.

[2] $\dfrac{2250}{8.33 \times 0.02 \times 1.005} = 13{,}500$ gal.

[3] Assumed at 25 per cent of sewage treated.

[4] $250{,}000/263{,}500 \times 100 = 94.9$.

[5] $13{,}500/263{,}500 \times 100 = 5.1$

[6] $13{,}500/1{,}000{,}000 \times 100 = 1.35$.

[7] Assumed in reporting dry solids.

As the density of the sludge may vary widely, the volumes handled may change considerably and approximately inversely as the solids content of the sludge.

Measurement of Return Sludge.—The quantity of return sludge is such an important matter in this process of sewage treatment that its accurate measurement is essential. As stated on page 610, there are three yardsticks that may be employed to determine the quantity of sludge which should be recirculated: the ratio of return sludge to the volume of sewage treated; the percentage by weight or p.p.m. of suspended solids in the mixed liquor; and the percentage by volume of sludge deposited in a settling glass during a given length of time from a sample of mixed liquor. While there is a tendency to stress the use of the first two methods in careful studies of plant performance, the third test, employing a convenient uniform settling period, may serve as a valuable means of regulating and controlling the process.

One of the most convenient means of measurement is a 1000-cc. graduated glass cylinder, about 2¼ in. in diameter and 16 in. high. By its use the percentage of sludge can be read directly. For determining and regulating the proportion of activated sludge present in the aeration tanks and sedimentation tanks, a period of sedimentation in the measuring glass of 1 hour commonly is used. At Milwaukee a period of ½ hr. proved to be adequate and its use was advantageous, as the results of tests could be obtained with relative promptness. Shorter periods of settling are subject to too great error and variation.

Sludge-blanket Depth Detectors and Indicators.—The depth of sludge accumulated in final-sedimentation tanks of the activated-sludge process is subject to change, due to variations in quantity and volume of sludge discharged into the tanks and changes in the rate of sludge withdrawal. It has been found advantageous in plant operation to be able to detect changes in depth of sludge blanket promptly, in order to apply corrective measures, particularly when sludge is accumulated to such a depth as to cause either undue discharge of suspended solids over the effluent weirs or deterioration of sludge, since activated sludge decomposes rapidly in storage.

Several means have been devised for the detection, observation or sampling of the sludge accumulated in final tanks. One such device, illustrated in Fig. 170, was developed at the North Side plant in Chicago and installed subsequently at the North Toronto plant in Toronto, Ont., and at Lancaster, Pa., among other places. It comprises a number of sampling pipes with open tops about 1 in. above the liquid level and terminating at various depths. By introducing compressed air into these pipes about 2 ft. below the surface, the liquid or sludge is drawn from the various levels and forced out of the top of the pipes, where it may be observed and sampled. Another device which has proved effective in revealing the depth of sludge blanket and quality of sludge at the North Toronto plant consists of a glass tube, 2 in. in diameter, which is slowly lowered vertically into the final-sedimentation tank to the full depth. The top of the tube is then closed with a stopper and the tube, when withdrawn, affords a typical cross section of the sludge blanket.

Vermilye (20) has described a sludge-blanket detector developed at the Tenafly, N. J., plant, which consists of a 1-in. galvanized pipe, one

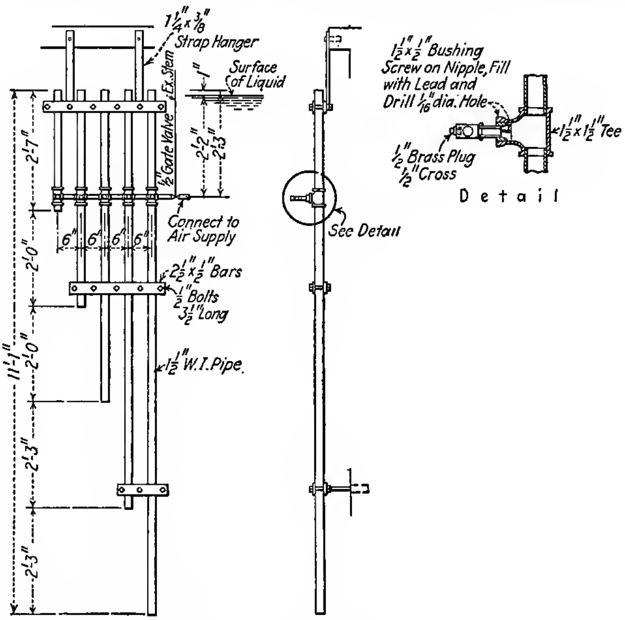


FIG. 170.—Sludge-level indicator used in final-sedimentation tanks, North Side plant, Chicago.

end of which is closed with glass. A flash-light socket and bulb are attached outside the pipe beyond the glass and furnished with power from a battery, attached to the other end of the pipe. In use, the detector is lowered, bulb end down, into the tank. The operator looks through the pipe and observes the level at which the sludge blanket can be seen around the lighted bulb.

Activated-sludge Piping, Pumping and Flow Control.—Activated sludge will flow readily in channels or pipes. In fact, computations of the size of conduits to transport activated sludge employ coefficients of discharge but slightly lower than for water. The hydraulics of sludge flow and pumping are discussed in Chap. XXVII.

The velocity in sludge conduits is usually maintained at 2 ft. a second or more, in order to avoid deposition of solids. Where the velocity is less than this in open channels, diffuser plates are commonly provided, to keep the sludge in suspension. They may be employed also to maintain and sometimes increase sludge activation. Where sludge is aerated thus during flow, an additional allowance in the hydraulic gradient may be desirable on account of interference with flow by cross-currents.

Activated sludge may be raised by centrifugal and other types of pumps, by air lifts, and by compressed-air ejectors. It is important to avoid violent agitation, which will break up the floc and reconvert it into a colloidal state. This is of importance not only in pumping return sludge but also in pumping waste sludge. In discharging excess activated sludge into primary-sedimentation units, as is done at the North Toronto plant in Toronto, Ont., if the floc is broken up in high-speed centrifugal pumps, the sludge will not settle in the tanks and may cause excessive accumulation of scum. Centrifugal pumps, initially provided for pumping excess activated sludge to vacuum filters at Milwaukee, so broke up the floc that difficulties with filtering were encountered and the centrifugal units were replaced by plunger pumps. There is only moderate danger of breaking up the floc in slow-speed centrifugal pumps of suitable design and even less in plunger pumps, air lifts and ejectors. Where the last-named are used, it is desirable to design the ejector pot so as to prevent forming a deposit on the bottom of the pot. The flow of sludge may be controlled by the pumping equipment or, where the sludge is drawn under hydrostatic pressure, by throttling a valve or by changing the hydrostatic head by means of a swiveled or telescoping draw-off pipe. At the North Toronto plant the flow of return sludge from the final-sedimentation tanks is governed by rate controllers of the venturi type. Hoffman (21), at Morristown, N. J., using a photo-electric cell, has devised an arrangement which automatically opens or closes valves regulating the sludge level. Return sludge may be discharged either to re-aeration tanks, to mixing channels or directly to the aeration tanks.

Air lifts have been used in some small plants, including those at Lodi, Cal., Gastonia, N. C., Mamaroneck, N. Y., East York and York Township, Ont., and Houston, San Marcos, Sherman, and Waco, Tex. With air lifts the flow of return sludge is controlled readily by throttling the air supply. At the North Toronto plant, during 1930, the average volume of air used in the air lifts was 585 cu. ft. a day, with an average sewage flow of 5 m.g.d. and an average rate of return sludge of 1.67 m.g.d. The air lifts at this plant were replaced in 1933 with centrifugal pumps.

The return-sludge pumps at the North Side plant in Chicago, shown in Fig. 171, are horizontal-shaft, volute type, single-stage, centrifugal pumps, direct-driven by induction motors (22). There are four pumps with 30-in. suction and 24-in. discharge. Each pump will deliver approximately 13,500, 8100 and 5400 gal. per minute at 10, 7 and 6 ft. total dynamic head respectively, when operating at approximately 408, 288 and 238 r.p.m., respectively. A dynamic suction head of

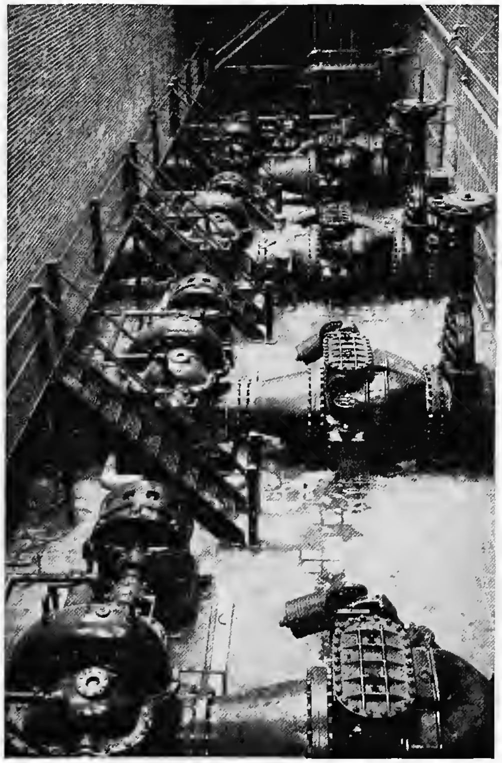


Fig. 171.—Return-sludge pumps at North Side plant, Chicago.

approximately 10 ft. will prevail over the usual range of head and capacity conditions. The four motors are 440-volt, three-phase, 60-cycle, induction motors rated at 50 hp. at 438 r.p.m., with variable-speed control. These motors are controlled by a float-operated liquid-level controller.

Two 350-g.p.m. centrifugal pumps are provided at the Pomona, Cal., plant for pumping the return sludge, one being motor-driven and the other, a stand-by unit, driven by an oil engine.

Excess or waste activated sludge is usually pumped by centrifugal or plunger pumps. At the North Toronto plant in March, 1931, the waste activated sludge was pumped for a few minutes each hour to the influent of the primary tanks at the rate of about 225 g.p.m., by means of a horizontal centrifugal pump of the Wood trash type. Two motor-driven, plunger sludge pumps, each of 50-g.p.m. capacity, are supplied at the Irwin Creek plant in Charlotte, N. C., for pumping the waste activated sludge to Oliver filters for dewatering. At the Sugar Creek plant in the same city the excess sludge is discharged to digestion tanks by means of a compressed-air sewage ejector. The ejector capacity is 50 g.p.m., but this capacity may be lowered by decreasing the flow of sludge to the ejector pot, as the discharge cycle commences automatically when the pot is full.

In Chicago, the excess activated sludge at the North Side plant is collected in a waste-sludge well, the rate of waste being governed by rate controllers of the venturi type. The sludge from the primary-sedimentation tanks is also conveyed to this waste well. All the waste sludge from the North Side plant is pumped through a 14-in. cast-iron force main, approximately 17 miles long, to the West Side plant for digestion in Imhoff tanks, together with the sludge from the latter plant. Three horizontal, motor-driven, single-suction, centrifugal pumps, each with a capacity of 1,000 g.p.m. when pumping against a total head of 180 ft., including a suction lift of 8 ft., are provided for pumping the waste sludge. The motors are 440-volt, three-phase, 60-cycle, squirrel-cage motors, rated at 100 hp. at 1765 r.p.m.

Both return sludge and excess sludge are commonly confined within pipes, except in plants where large volumes of sludge are handled. Here open channels may be provided to carry them. These channels are usually equipped with diffuser plates and aerated so as to keep the sludge solids in suspension and maintain the sludge in a condition favorable to the activated-sludge process. The flow of return sludge is commonly measured by venturi meters, so as to provide the necessary information for governing the rate of flow and for keeping operation records.

In order to mix the return sludge with the incoming sewage, the sludge is usually discharged into an aerated mixing channel, before distribution to the aeration tanks. At Elyria, Ohio, on the other hand, return sludge is discharged at the influent end of the individual aeration units. The aerated mixing channel at the North Toronto plant is 50 ft. long, 5 ft. wide and about 5.5 ft. deep, providing about 2 minutes' detention period.

A comparison between air lifts and centrifugal pumps for handling return sludge has been made at the Charlotte plants. The results are summarized by McConnell (23) as follows:

Each plant is provided with air lifts for returning sludge but these were used only a few months before being abandoned for slow speed centrifugal pumps. Experiments were conducted by both methods with apparently no decided difference in quality of sludge or ill effects upon the process. The extra cost of air lifting is certainly not compensated for. Sludge has been returned for nearly two years at each plant by centrifugal pumping very successfully. There is evidence to indicate that air lifted sludge is actually injured by the agitation of the air lift while a properly selected type of centrifugal pump moves the column of sludge forward in a steady stream. No experiments were conducted with a plunger type pump as one was not available. There are reasons to believe that this type pump might be the better type for this purpose.

Final Sedimentation.—The design of sedimentation tanks in which activated sludge is removed from the aeration-tank effluent is based on the principles outlined for the design of plain-sedimentation tanks in Chap. XII, due provision being made for the nature of the sludge to be settled out. The flocculent nature of the sludge requires consideration, suitable allowances being made for variations in its density. Since activated sludge decomposes rapidly, excessive sludge-storage periods are not desirable. The volume of return sludge is sometimes varied to meet fluctuations in rate and strength of sewage flow. The volume of sludge may be 15 to 60 per cent of the sewage flow and liberal allowances are required in the tank capacity, to permit its settling in the tanks and to prevent excessive quantities from being discharged in the effluent. At Milwaukee, Wis., and North Toronto and West York, Ont., difficulty has been encountered, particularly during the colder months, in securing a well clarified effluent at the time of daily peak flows. The aeration and sedimentation periods are shortest during such flows and the sludge is not in such good condition for settling as at other times.

After three years of experimentation on final-sedimentation tanks at the Milwaukee testing station, Copeland (24) concluded as follows:

1. That when activated sludge is well aerated it settles readily.
2. That the settled sludge must be removed as fast as it collects.
3. That clarification depends upon area rather than depth.
4. That in order to secure complete removal of the sludge the flow of liquor through the tank must not be allowed to exceed three linear feet per minute.

Final-sedimentation tanks with relatively deep sludge hoppers have been provided at some plants. Concerning operation of this type of tank Fugate and Stanley (25) state:

It has long been realized that the settling tank which must be used after aeration of the sludge is the critical point and controls the successful operation of an activated sludge plant. With deep hopper-bottomed tanks as used at Houston, this control is greatly complicated by the inability to prevent a concentrated sludge from decomposing in the settling tanks and thereby losing its aerobic condition.

The bottom slopes in these tanks were 1¾ vertical on 1 horizontal. When the North Side plant at Houston was remodeled during 1929, the hopper-bottom sedimentation tanks were converted into aeration tanks and octagonal sedimentation tanks, 65 ft. across, equipped with Fidler-type spiral sludge-removal equipment, were constructed (26). Slopes at the South Side plant were changed from 1¾ on 1 to more than 2½ on 1.

At San Marcos, Tex., the sedimentation tanks were remodeled to give bottom slopes of 2 to 2½ on 1. Pearse (27) found that the slopes should be about 2 vertical on 1 horizontal.

Vertical-flow tanks of the Dortmund type are employed extensively abroad. In these, it is customary to keep the vertical velocity below 30 ft. an hour. Horizontal-flow tanks equipped with sludge-removal devices, such as the Dorr, Link-Belt and Fidler mechanisms, are used most commonly in this country. Detention periods of 1½ to 2 hr. are common.

A device for the removal of activated sludge has been developed at Milwaukee by Townsend and Brower (28). It is known as the Rex Tow-Bro sludge remover and consists of radial pipe arms with multiple inlets, centrally supported as in the case of the Dorr sludge plows. Pressure, obtained either hydrostatically or by pumping, acts upon the inlets and the sludge is drawn into the radial arms as the mechanism revolves. The sludge then flows inside the pipe to either the return-sludge line or the waste-sludge line. The sludge is "picked up" by the mechanism at the point where it is deposited, much as a vacuum sweeper picks up dirt. An illustration of the Rex Tow-Bro sludge remover is given in Fig. 172. Equipment of this type has been installed at Milwaukee, Wis., and Escanaba, Mich.

As the area of a sedimentation tank with respect to the volume of mixed liquor which it can successfully handle is an important consideration in dealing with flocculent activated sludge, it has been found convenient to make use of the unit of horizontal area in computations relating to such tanks. As a result of experiments at Milwaukee, the sedimentation tanks were designed to provide 1 sq. ft. of tank area for each 1600 gal. of mixed liquor to be treated daily, 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 surface for each 850 gal. of sewage, based upon the average daily flow. Values of 500 to 1400 gal. daily per square foot of tank surface have commonly been employed.

The extent of treatment in the aeration tanks has an important effect upon the 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 sludge which was reasonably dense. In such 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 the activated-sludge process, the design of satisfactory means of introducing influent into final-sedimentation tanks and removing effluent from them offers special problems. It is essential to prevent

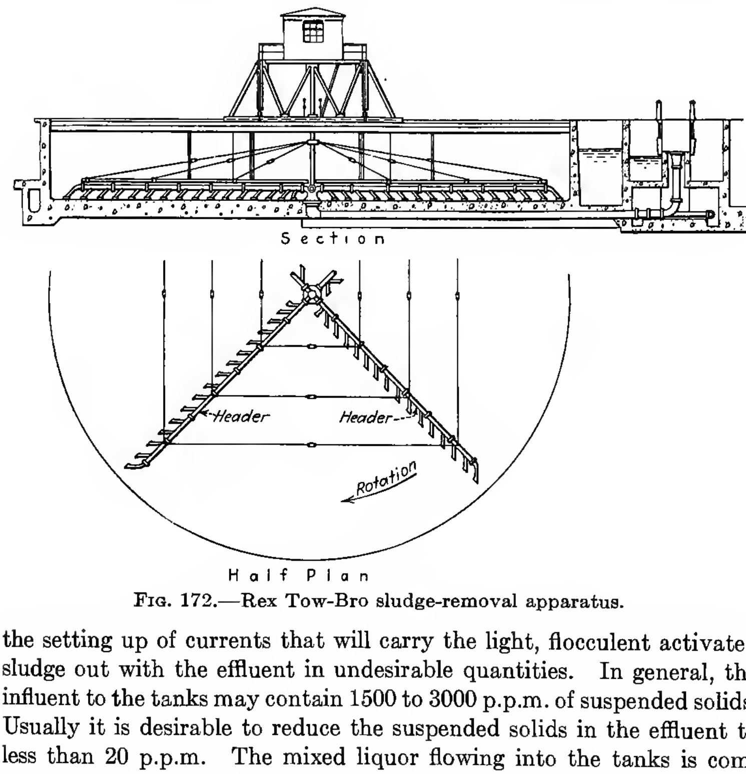


FIG. 172.—Rex Tow-Bro sludge-removal apparatus.

the setting up of currents that will carry the light, flocculent activated sludge out with the effluent in undesirable quantities. In general, the influent to the tanks may contain 1500 to 3000 p.p.m. of suspended solids. Usually it is desirable to reduce the suspended solids in the effluent to less than 20 p.p.m. The mixed liquor flowing into the tanks is commonly diffused by multiple ports, which may be baffled. In small plants the effluent is generally discharged over weirs at the ends of the tanks. The larger plants, such as those at Chicago and Milwaukee, have collecting troughs suspended in the tanks. Tests at Springfield, Ill., Toronto, Ont., and elsewhere indicate that multiple collecting troughs are much more efficient in rectangular, activated-sludge sedimentation tanks than the ordinary types of single, end-wall effluent weirs.

The characteristics of the sedimentation units constructed in connection with a number of American plants are shown in Tables 109 and 113. Since activated sludge produced in the bio-aeration process possesses essentially the same characteristics as that resulting from diffused-air aeration, the design of final settling tanks is governed by the same principles in both types of plants.

MECHANICAL-AERATION PLANTS

As previously stated, the facts that but a small proportion of the oxygen contained in the air blown into sewage is actually utilized and that one of the important functions of the air is to keep the sludge in suspension were utilized first by Haworth at Sheffield, England. He developed a mechanical device designed to perform the same functions as the air, by permitting absorption of oxygen from the atmosphere in contact with the sewage surface, while inducing sufficient velocity of sewage in the tank to keep the sludge suspended and moving through the liquid. Experiments at Glasgow, Scotland, have indicated that in bio-aeration tanks, to produce sufficient change of surface to ensure an adequate solution of oxygen, a minimum surface velocity of 1 ft. a second must be maintained.

Systems of Mechanical Aeration.—A number of different systems of mechanical aeration have been devised, among them being the Haworth or Sheffield system; the Hartley or Birmingham system; the Bolton or Simplex system; the Link-Belt system; the Kessener system; and the system of stream-flow aeration. Reference already has been made to the combined use of diffused air and mechanical aeration, as developed in the Sanitary District of Chicago, by Imhoff at Essen-Rellinghausen, Germany, and by the Dorr Co. from the Imhoff patent.

In America only the Simplex, Link-Belt and Dorrco-Imhoff systems have so far found application in municipal installations. The Sheffield and Hartley systems have been employed in England. Stream-flow aeration is still in the course of experimental development and the Kessener system has been applied only in Holland, in small industrial plants and experimentally in municipal works. In order to round out the discussion of the activated-sludge process, brief attention will be given, before proceeding to American practice, to the design features of such of these systems as have been in operation in municipal installations abroad.

The Sheffield System.—As employed at Sheffield, the aeration tank is divided by walls or round-the-end baffles into a series of long and relatively narrow and shallow channels, giving a length of travel of about a mile. The channel farthest from the inlet feeds a transverse channel, from which the sewage overflows into settling tanks. A return channel leads from this transverse channel to the inlet end and returns a small

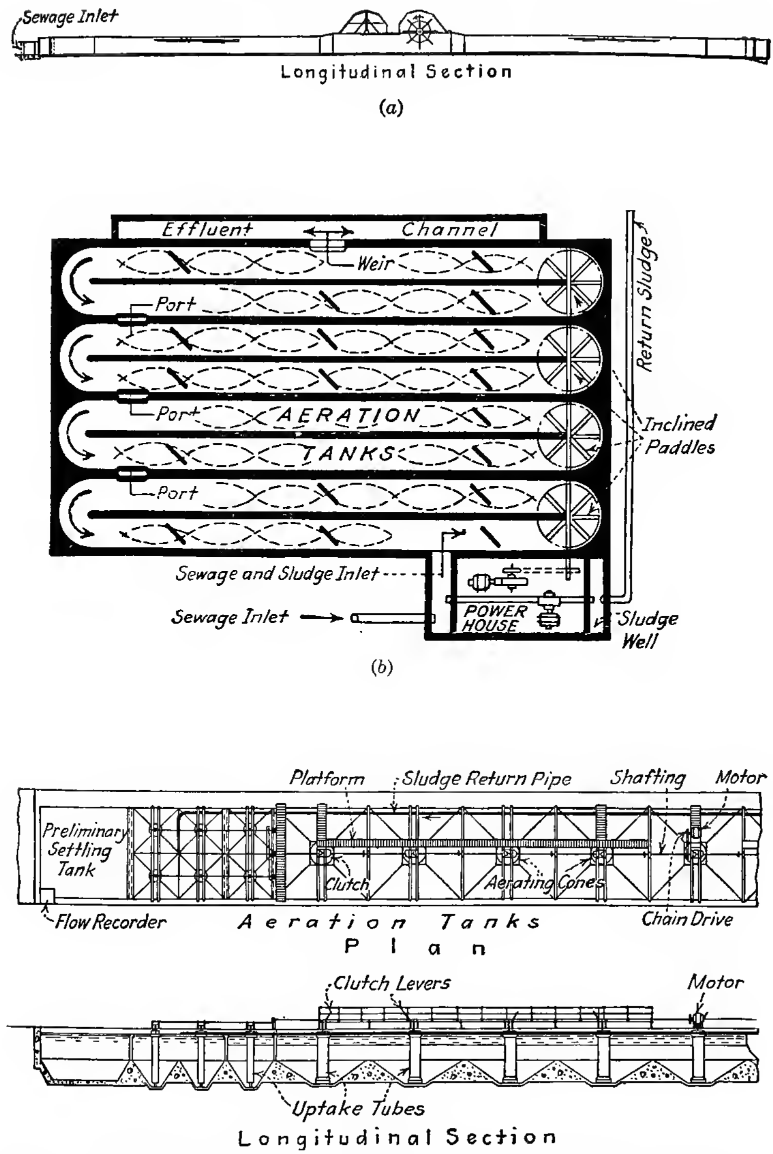


FIG. 173.—Sheffield, Hartley and Simplex systems of mechanical agitation. *a*, Sheffield system; *b*, Hartley system; *c*, Simplex system.

portion of the tank effluent to the incoming sewage. By this means the volume of return sludge pumped is reduced somewhat.

Aeration takes place at the sewage surface, new surfaces of liquid being brought into contact with the air and circulation of the sewage being maintained by two rows of paddles operated from horizontal shafts crossing the channels about midway of their length, as shown in Fig. 173*a*. The paddles are staggered and consist of two sets of radial arms forming a rimless wheel. Short waves are created at the sewage surface. The ends of the channels are rounded and the dividing walls terminate in pear-shaped enlargements. This feature prevents deposition of sludge at this critical point and preserves the waves.

The design characteristics of the Sheffield plant are given in Table 109.

The Hartley System.—In the Hartley system, employed at Birmingham, England, paddle wheels mounted on shafts, which are inclined at a small angle from the vertical, are provided at the return bends of long, narrow and shallow channels, as shown in Fig. 173*b*. The channels are endless, as at Sheffield, but are broken up into several units operated in series. The straight part of the channel contains a number of baffle plates, which can be moved diagonally to the flow to cause spiral motion, which also is induced in the sewage by the inclination of the paddle wheels. This motion is said to aid surface aeration and keep the sludge in suspension at lower velocities.

In 1928, Whitehead (29) equipped the Birmingham units with air diffusers. The Birmingham tanks are bioflocculation units with a detention period of 1 hr. and a reactivation period of 18 hr. There are two types of reactivation tanks, diffused-air tanks and Simplex units.

The Simplex System.—The Simplex system was developed at Bury, England. Relatively deep, hopper-bottom tanks are employed in this system, which is illustrated in Fig. 173*c*. The aerating mechanism consists of a revolving cone suspended above a vertical uptake tube, centrally situated in each hopper. The tube is anchored by tie rods to the tank sides and is set on legs so as to clear the tank bottom by 6 in. The cone draws the mixed liquor through the uptake tube and is fitted with vanes so designed as to throw upper and lower films of liquid over the tank surface.

Aeration is thus secured and the sewage circulates, moving upward through the tube and downward outside. Since the sewage leaves the vanes at an angle, a spiral movement is set up. When more than one unit is required, alternate aerators are rotated in opposite directions to prevent short-circuiting. The tank contents, it is stated, are circulated two or three times an hour. An air seal is provided between the cone and the uptake tube, so that all liquid is drawn from the tank bottom.

The cones spin at a rate of about 60 r.p.m. and may be driven through a worm-gear or bevel-gear drive by individual motors or by a line-shaft arrangement with individual clutches. Power consumption is affected by the position of the aerating cone relative to the sewage surface and provisions for raising and lowering the mechanism with varying flows may be advantageous.

Simplex units have been operated in parallel, in series, or in parallel series, and on the fill-and-draw principle. Plants are frequently designed to permit the use of one or more aerating units for re-aeration of the sludge.

Among the American municipalities employing Simplex aerators are Dunsmuir, Cal., Christopher, Princeton and Woodstock, Ill., Rochester, Minn., and Lawton, Okla. The Princeton plant employs preliminary settling tanks with 1-hr. detention. The total aerating period in the 5 aeration units is 8 hr. during the 16 hr. of maximum flow. One unit can be employed for re-aeration of the sludge. The aeration units are almost square, being 22.00 ft. by 22.25 ft. in plan. The maximum effective depth is 12 ft. and the bottoms of the hoppers are inclined about 35 deg. from the horizontal. The uptake tubes are 2½ ft. in diameter and flare out to 4 ft. at the bottom. The power requirement at rated capacity is 17.5 hp. per mil. gal.

At Lemoore, Cal., four units are employed for partial treatment of the sewage preceding trickling filters.

The Link-Belt System.—In the case of the Link-Belt aerator, shown in Fig. 174, surface aeration and spiral motion are induced by a paddle mechanism, which operates partially submerged on a horizontal shaft running along one side of the tank. Circulation of the tank contents is aided by a longitudinal baffle, placed 18 in. from the wall adjacent to the aerator and penetrating to within 18 in. of the tank bottom. Rotation of the paddle mechanism causes the sewage to rise in the narrow division partitioned off from the main tank and induces a return current in the wider portion of the tank. Thus spiral flow is established. At the same time the surface of the sewage is agitated and surface absorption of atmospheric oxygen takes place. Transverse baffles are placed in the wide portion of the tank, to prevent short-circuiting. The aerating mechanism was originally similar in design to a ribbon conveyor. More recently it has been changed in construction to resemble an open metal lattice. The paddles rotate at a rate of 40 to 43 r.p.m. and commonly are driven by individual electric motors through a reduction gear with chain drive to a sprocket wheel on the horizontal shaft.

Among the American municipal installations employing Link-Belt aerators are Barrington and Collingswood, N. J. The Barrington plant employs preliminary settling tanks with 1-hr. detention. The total aerating period is 6 hr. for the average flow of 0.6 m.g.d. The aerating units are rectangular in plan, 60 ft. long, 12.5 ft. wide and 12.5 ft. deep.

The power requirement is 1.75 kw. per unit of 0.2 m.g.d., equivalent to 8.75 kw. per mil. gal. daily.

The aeration units at Collingswood are 14.2 ft. wide, 8 ft. deep and 120 ft. long. The capacity of each of the 4 units is 0.33 m.g.d. and the power required is 4 kw. per unit or 12 kw. per mil. gal. daily.

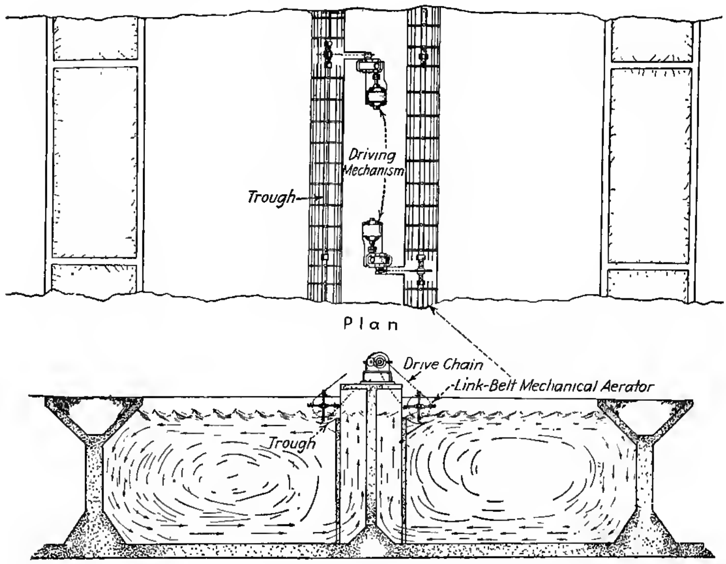


FIG. 174.—Link-Belt system of mechanical agitation.

Paddle-wheel and Diffused-air System.—As developed at Essen-Rellinghausen, Germany, aeration units employing both diffused air and mechanical agitation are 20 ft. wide and 10 ft. deep, as shown in Fig. 175 (5). There are two longitudinal rows of diffusers, separated by a ridge and spaced centrally in the tank. There are two submerged paddle mechanisms, 10 ft. apart, operated from a horizontal shaft running the length of the tank. The tank is thus divided into two operating sections. The paddles make 7 r.p.m. and tend to beat down the air rising from the diffuser plates. Air consumption is 0.08 cu. ft. per gallon of sewage. The detention period is 3½ hr. and the entire power consumption is 7 hp. per mil. gal. daily. When operated with diffused air alone, 0.7 to 1.0 cu. ft. of air per gallon of sewage was required and a power consumption of 22 hp. per mil. gal. daily was registered. The sewage treated is relatively weak for German conditions, averaging 130 gal. per capita daily, and passes through Imhoff tanks with a 20-min. detention period prior to aeration. The quantity of return sludge in the tanks is 8 per cent of the tank volume, as measured after 1 hour's settling. For other design values see Table 109.

Combined paddle-wheel and diffused-air aeration has been used since 1928 in a tank at the Calumet plant in Chicago. The results of operation during 1929 are shown in Tables 105 and 106. As described by Mohlman and Wheeler (16), the tank is 103 ft. long, 34 ft. wide and 14.5 ft. deep, with a net capacity of 360,000 gal. The paddle wheels are 12 ft. in diameter, with a 2- by 10-in. plank at the end of each radial arm. There are two paddle wheels in each tank, on long horizontal shafts, driven by a motor with reduction gears, sprockets and chain.

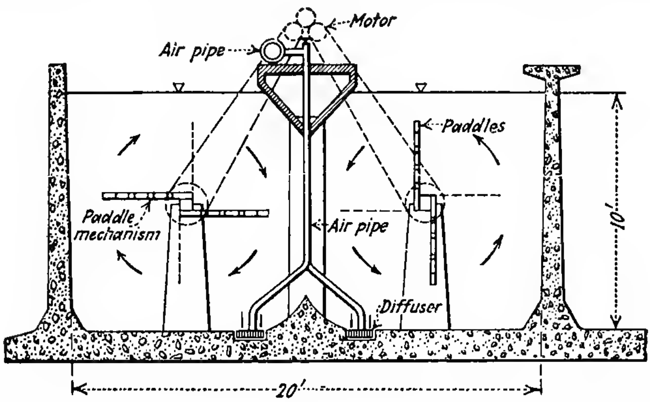


FIG. 175.—Aeration tank equipped with paddle-wheel and diffused-air system Essen-Rellinghausen, Germany.

A diagram of the cross section of the tank is shown in Fig. 176. The paddles revolve in opposite directions at 3.6 r.p.m., giving a peripheral speed of 2.25 ft. a second.

From Feb. 20, 1929, to Jan. 10, 1930, the Filtros plates were set directly under the paddles when the paddle arm was in a horizontal position. Thus the rising current of air was directly opposed to the motion of the paddles, as in the Essen-Rellinghausen installation. Tests were then made, starting Mar. 9, 1930, with the diffuser plates directly under the shaft. Indications were that equal results were obtained in both cases and that the total power consumption was reduced from 13.2 hp., required with the plates at the sides, opposing the motion of the paddles, to 10.1 hp. per mil. gal. daily of sewage treated with the plates beneath the shaft, aiding the circulation of the paddles. In the latter case, the paddles required 3.7 hp. and the air, at 0.20 cu. ft. a gallon, 6.4 hp.

Combined paddle-wheel and diffused-air aerators of the Dorrco-Imhoff type have been installed at the Phoenix, Ariz., plant, which has a design capacity of 12 mil. gal. daily. There are five aeration tanks and one re-aeration tank. Each tank is 330 ft. long and 27 ft. wide, with an effective depth of 14 ft. The base of each tank is divided into two compartments with rounded cross sections at the bottom. A single row of porous plates for diffused air is placed on the dividing wall between compartments, 4 ft. above the lowest level of the tank floor. Each tank has 260 sq. ft. of diffuser plates. Agitation is supplied by mechanically operated paddles, extending the length of the tank, in each compartment. These paddles revolve against the air stream at a speed of about 4 r.p.m. The operating results during September, 1932, are given in Table 107 and the design data are summarized in Table 109.

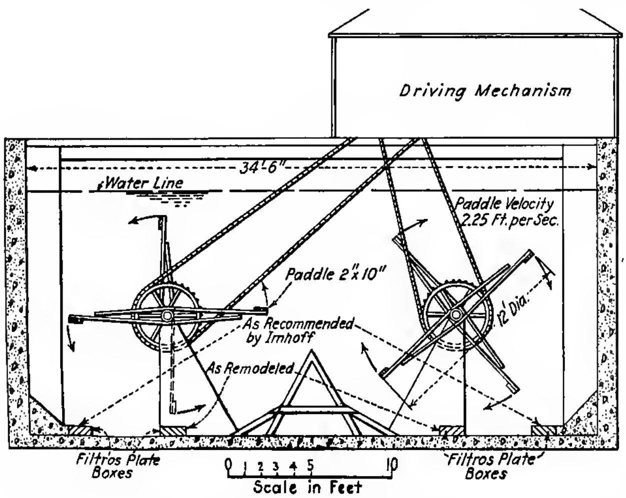


FIG. 176.—Aeration tank equipped with paddle-wheel and diffused-air system, Calumet treatment works, Chicago.

Several possible advantages of the Dorrco type of aerator have been pointed out by Besselievre (18): During the night, when the sewage flow is low and relatively weak, the air-compressor plant may be shut down for about 8 hr.; The operating paddles prevent deposition of solids and the agitation induced by the paddles may maintain suitable conditions throughout the night run; and since the paddles prevent the segregation of solids, the quantity of air introduced at different points in the tank may be varied. For example, it may be advantageous to admit air at a maximum rate near the influent end and reduce the rate gradually toward the effluent end.

Statistics of Mechanical-aeration Plants.—Statistics of a number of activated-sludge plants employing mechanical agitation are given in Table 109. Mechanical aerators are generally patented and the

TABLE 109.—DESIGN CHARACTERISTICS OF TYPICAL ACTIVATED-SLUDGE PLANTS EMPLOYING MECHANICAL AERATION

	Barrington, N. J.	Chesterfield, England	Collingswood, N. J.	Phoenix, Ariz.	Lawton, Okla.	Lemoore, Cal.[2]	Princeton, Ill.	Sheffield, England
Date of construction	1929	1926	1929	1931	1929	1928	1927	1930
Population served	3,000	66,450	13,000	....	15,000	....	3,500	530,000
Average sewage flow, m.g.d.	0.6	2.1	1.33	12.0	1.0	0.38	0.5	
Character of sewage	....	Combined	....	Separate	....	Separate	Combined	Combined
Preliminary treatment:								
Grit chambers	Yes	Yes	Yes	No	....	....	Yes	Yes
Racks, clear opening, in.	Yes	....	....	1	Yes	3/4	Yes	
Fine screens	None	....	....	None	....	....	None	None
Sedimentation tanks, type	Hopper bottom	Hopper bottom	Link-Belt	Dorr	Hopper bottom	....	Link-Belt	Plain
Detention period hr.	....	5	....	1.0	1	....	1	12
Aeration tanks:								
Type	Link-Belt	Paddle wheel	Link-Belt	Dorrco	Simplex	Simplex	Simplex	Sheffield
Number of units	3	3	4	5	9	4	5	21
Length, ft.	60	242	125	330	24	22	22.2	265
Width, one channel, ft.	12.5	6	14.5	27	24	22	22	6
Effective depth, ft.	12.0	4	8.5	14	18	12	12	4.4
Number of passes per tank	1	14	1	1	1	1	1	21
Period of aeration, hr.	6	24	6	6.6	8	5.5	8	17
Air consumption, cu. ft. per gal.	....	....	....	0.27				
Power for mechanical equipment, h.p. per m.g.d.	11.7	....	16.0	15[1]	18	19	17.5	35
Return sludge, per cent of sewage flow	25	....	25	....	20	40	18	15–20
Final-sedimentation tanks:								
Type	Link-Belt	Hopper bottom	Link-Belt	Dorr	Dorr	....	Link-Belt	Dortmund
Number	3	15	4	1	1	1	2	168
Inside dimensions, ft.	36 × 12	24 × 24	36 × 14.5	140 × 140	45 × 45	25 × 23	48 × 9.5	25 × 25
Effective depth, ft.	12	22	1	11.5	13.5	10	11.8	22.5
Detention period, hr.	....	7.7	....	2.1	2.5	2	2	6
Flow of effluent, gal. daily per sq. ft.	....	202	....	613	500	608	470	310
Re-aeration tanks:								
Number	None	None	None	1	1	None	None	None
Period of reaeration, hr.	....	....	....	....	4.8	....	....	....

[1] Including air compression.
[2] Biofloculation plant; effluent treated on 6-ft. trickling filters at rate of 4.25 m.g.d. per acre.

aeration tanks are designed to conform to the requirements of the equipment.

Bibliography

1. Ann. Rept., Milwaukee Sewerage Comm., 1915.
2. Ann. Rept. of City Engr. on Sewage Disposal, Houston, Tex., 1916.
3. *Eng. News-Rec.*, 1923; **90,** 835.
4. Crawford, F. N., and Edward Bartow: *Jour. Ind. Eng. Chem.*, 1916; **8,** 646.
5. Imhoff, Karl: *Eng. News-Rec.*, 1926; **97,** 298.
6. Grant, S., E. Hurwitz and F. W. Mohlman: *Sewage Works Jour.*, 1930; **2,** 228.
7. Harris, F. W., T. Cockburn and T. Anderson: *Surveyor*, 1926; **70,** 30.
8. Jenks, H. N., and Max Levine: *Eng. News-Rec.*, 1928; **100,** 808.
9. Ann. Rept., Milwaukee Sewerage Comm., 1919.
10. Greeley, S. A., and W. D. Hatfield: *Trans.*, A. S. C. E., 1930; **94,** 544.
11. Hatton, T. C.: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 88.
12. Fowler, G. J., and S. N. Chatterjee: *Surveyor*, 1927; **72,** 139.
13. Parsons, A. S., and H. Wilson: *Surveyor*, 1927; **72,** 490.
14. Hurd, C. H.: *Eng. News-Rec.*, 1923; **91,** 260.
15. *Surveyor*, 1929; **75,** 81.
16. Mohlman, F. W., and C. E. Wheeler: *Sewage Works Jour.*, 1930; **2,** 529.
17. Imhoff, Karl: *Eng. News-Rec.*, 1927; **99,** 790.
18. Besselievre, E. B.: *Water Works and Sewerage*, 1933; **80,** 364.
19. Rept. of Operation of Sewage Pumping and Disposal Plants, Houston, Tex., to Dec., 1923.
20. Vermilye, S.. *Water Works and Sewerage*, 1933; **80,** 34.
21. *Munic. Sanitation*, 1933; **4,** 374.
22. Eng. Works, Sanitary District of Chicago, Aug., 1928.
23. McConnell, E. G.: *Proc.*, N. J. Sewage Works Assoc., 1931, p. 16.
24. Ann. Rept., Milwaukee Sewerage Comm., 1917.
25. Fugate, G. L., and W. S. Stanley: *Sewage Works Jour.*, 1928; **1,** 70.
26. Hildreth, T. N.: *Am. City*, 1929; **41,** no. 4, p. 117.
27. Rept. on Industrial Wastes from Stockyards and Packingtown in Chicago, vol. 2, 1921.
28. Townsend, D. W., and James Brower: *Eng. News-Rec.*, 1930; **105,** 63.
29. *Surveyor*, 1928; **74,** 114.

CHAPTER XXVI

DIFFUSED-AIR ACTIVATED-SLUDGE PLANTS

The early experiments with the activated-sludge process were made in tanks operated by the fill-and-draw method. Ardern and Lockett (1) obtained poor results with continuous flow of sewage in the first tests in which they employed this method of operation. They attributed the results obtained to "short-circuiting." At Milwaukee, however, Hatton (2) soon reported that there was no difference in results obtained by the two methods, with respect to volumes of air and activated sludge required, cost of aeration and degree of purification secured, as indicated by clarification, nitrates, stability and removal of bacteria and organic matter.

Almost all activated-sludge plants in America and abroad are now operated on the continuous-flow principle. Short-circuiting has been combated by suitable arrangement of air diffusers and in some cases by baffling. The general arrangement of aeration units is commonly governed by the size of the plant, the type of air diffusion, the space available for the tanks and other elements of design. As with other sewage-treatment processes, it is desirable to provide two or more aeration units, so that tanks may be cut out of operation for repairs without entirely disrupting the process or the operation of the plant. Most aeration units are rectangular in plan and generally also in vertical cross section. As shown in Table 113, flow may be in one direction, the outlet being placed at the opposite end of the tank from the inlet, or the sewage may make several passes through the unit, each tank being divided into two, three or four compartments by longitudinal, round-the-end baffles. Units are generally operated in parallel, series operation being considered only in connection with the stage process, as discussed in Chap. XXIV, in which case settling units are placed between aeration units.

Departures from rectangular tanks are few. The small aeration unit at Mercedes, Tex., shown in Fig. 181, surrounds the final-sedimentation and sludge-digestion tanks, which together are circular in plan (3). The aeration unit is irregularly U shape in cross section.

Activated-sludge Patents.—Patents on the activated-sludge process and upon the spiral-flow type of aeration tank have been taken out in the United States by Activated Sludge, Inc. This company has instituted suits against Milwaukee, Chicago and other cities, charging infringements of patents. In the case against Milwaukee a decision was handed down on Feb. 7, 1933, by Judge F. A. Geiger of the U. S. District Court of Milwaukee, upholding the validity of certain of the patents. The case was appealed and the validity of four patents was affirmed in a decision made by the U. S. Circuit Court of Appeals for the Seventh Circuit, Mar. 2, 1934. On Oct. 8, 1934, the U. S. Supreme Court denied a request by the city of Milwaukee to review the decision of the Court of Appeals.

In the case against Chicago, the same four patents were upheld by Judge W. C. Lindley of the District Court. Two other patents were involved at Chicago, which did not apply at Milwaukee. The fifth patent was similar to the four and was held to be infringed. The sixth patent, on the baffling of the tank to facilitate spiral flow, was held invalid, as it covers only "the exercise of mechanical skill in adapting a structure to such form as will best promote general and complete circulation with the least possible friction." However, Judge Lindley refused to issue an injunction, on the ground that to enjoin the operation would work an irreparable damage upon the health and lives of the people served.

Certain, and perhaps the more important, activated-sludge process patents held by Activated Sludge, Inc., expired Nov. 20, 1934.

Systems of Air Diffusion.—One of the most important construction details is the means of diffusing air through the sewage in the aerating tanks. Much study has been given to the subject of air diffusion, with a view to securing a diffuser that will meet the following requirements: deliver the air to the sewage in a state of fine division; operate without becoming clogged; and offer only a small frictional resistance to the passage of the air.

Porous plates are most commonly used at present. Such plates, composed of fused crystalline alumina, are manufactured by the Carborundum Co. and the Norton Co., the product of the former being called Aloxite brand and that of the latter, Alundum. Other plates, composed of silicious sand and known as Filtros, are manufactured by Filtros, Inc. The Filtros plates may be described as artificial porous stones and are made of high-silica sand, bonded by a synthetic silicate, fusing at temperatures over 2000°F. The plates are kiln-burned until hard and strong. The Alundum plates are made of fused alumina grains, bonded in a kiln at a temperature of approximately 2400°F. with a high-aluminum glass. The Aloxite diffuser material is an aluminous earth, known as bauxite ore, which is calcined to remove a water content of 30 per cent and then melted at 2000°C. with coke and iron turnings, to produce chemically a highly refined crystalline alumina. The fused alumina is crushed and screened for grading and the selected grains are bonded together with a special clay bond, molded under pressure and finally vitrified at high temperature.

In England, Jones and Attwood diffusers, made by Activated Sludge, Ltd., are generally employed. They are made by a secret process

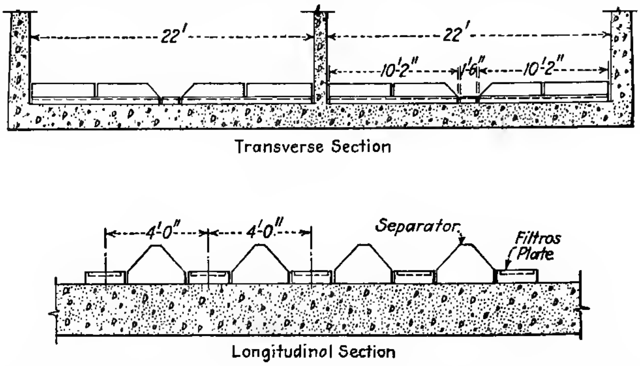


FIG. 177.—Ridge-and-furrow system of air diffusion at Milwaukee, Wis.

and it is stated that they are neither baked nor fired. The characteristics of diffuser plates will be discussed later.

At San Marcos, Tex., air is now applied to the sewage in the aeration tanks by means of numerous open-end pipes, although Filtros plates

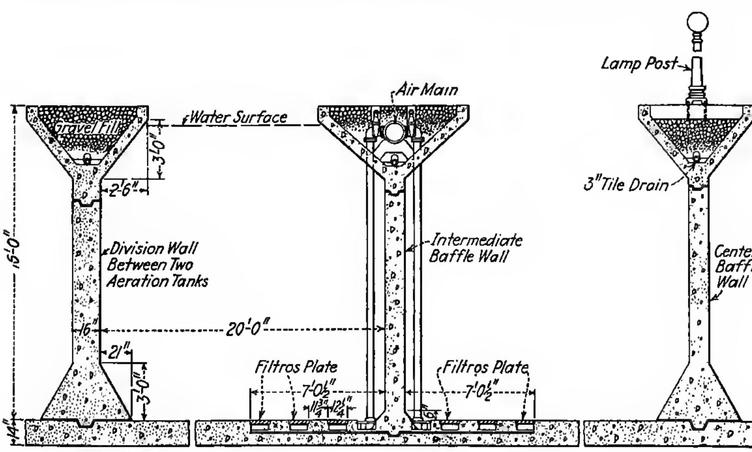


FIG. 178.—Cross section of aeration tank at Indianapolis, Ind., showing spiral-flow system of air diffusion.

were installed when the plant was placed in operation in 1916. The San Marcos plant is said to have been the first activated-sludge plant in this country to serve an entire city (27).

The arrangement of air diffusers in aeration tanks is important. Three general arrangements of plate diffusers have been developed thus far. These have given rise to three systems of design known as the ridge-and-furrow system, the spiral-flow system and the longitudinal-furrow system. The three systems involve not only different positions of the plates in the tanks, but also certain structural differences in floors and walls, as illustrated in Figs. 177, 178 and 179. The spiral-flow system is most commonly used in America.

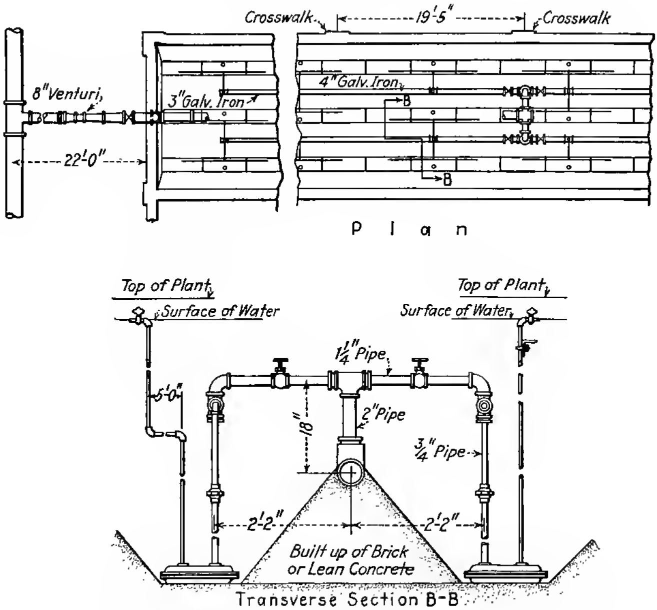


FIG. 179.—Longitudinal-furrow system of air diffusion at Irwin Creek plant, Charlotte, N. C.

Kusch (4) has described an experimental aeration unit at Oranienburg, Berlin, which consists of hopper bottoms in series. In the centers of these hoppers, vertical pipes rise, much as in the Simplex system of mechanical aeration, but only to within a few inches of the surface. Air, blown into each pipe through a connection 32 in. below the flow line, induces an upward flow through the pipe and a downward return current in the tank. The rising mixture of air and sewage impinges upon a submerged deflector, suspended above the pipe, and is deflected outward and downward. Pronounced rippling of the sewage surface is stated to be induced. A somewhat similar arrangement was employed in early tests at Milwaukee (5).

Ridge-and-furrow System.—In this country the ridge-and-furrow system was first employed at the experimental plants in Milwaukee and Cleveland and at the Houston and Milwaukee treatment works. In ridge-and-furrow tanks the diffusers are situated in depressions created by sawtooth construction of the tank bottom. The concrete wedges constructed between the plates are known as spacers and are given sufficient slope to prevent deposition of solids upon their surfaces. The plates are placed in rows across the tanks at right angles to the direction of flow, thus causing the sewage to be subjected to successive impacts of air, sometimes referred to as "air baffles" or "curtain baffles."

At the North Side plant in Houston, Filtros plates, each 1 ft. square, are set in containers forming rows of diffusers across the tank. These rows are spaced 5 ft. apart and produce a ratio of diffuser area to tank area of about 1:7. At Milwaukee, the plate containers are 4 ft. apart and are separated by slightly truncated wedges of concrete, forming an angle somewhat greater than 45 deg. with the horizontal. A longitudinal furrow equipped with diffusers passes through the center of each of the two compartments of the aeration units. Drainage of the tanks is facilitated by this furrow. The arrangement of diffusers at Pasadena is similar to that at Milwaukee. The longitudinal drainage furrow, however, is not equipped with plates. The ratio of plate area to tank area at Pasadena is 1:6. An attempt has been made at this plant to reduce short-circuiting by placing transverse wooden baffles in the tanks, dividing them in plan into six 10⅓- by 10-ft. and one 5¼- by 10-ft. compartments. The baffles rise 2 in. above the sewage surface and reach to within 1½ in. of the top of the spacers.

Transverse baffles are common in English tanks, communicating openings either being situated at the bottom of the baffles or alternating between top and bottom of successive baffles. The latter are 35 ft. apart in the Withington works at Manchester and 9 ft. apart at Stamford.

Spiral-flow System.—The spiral-flow system was first employed at Manchester, England, but was further developed at Indianapolis. In this system the diffusers are placed in single or multiple rows running along one side of the tanks. The air rises vertically along this side and induces a rotation of the sewage which combines with the longitudinal displacement to produce helical motion. Geometrically, the designation "spiral flow" is incorrect; but it is so universally employed that it would be useless to attempt a change. An appreciable quantity of air is carried across the tank and some of it passes downward with the sewage on the opposite side. In American plants, deflectors are generally provided at the sewage surface and the corners between the floor and side walls are beveled or rounded, to aid in establishing and maintaining spiral motion. Figure 180 shows a model of the spiral-flow system in operation (6).

Hurd (7) believes there is a real distinction between the spiral-flow system developed at Manchester and the American system developed by his experiments at Indianapolis. In the Manchester system comparatively narrow tanks are used, in which the principal velocities are up and down. Observations of experiments in England indicated that curved baffles caused a reduction in velocity near the floor of the tank. Sandford (8) states that,

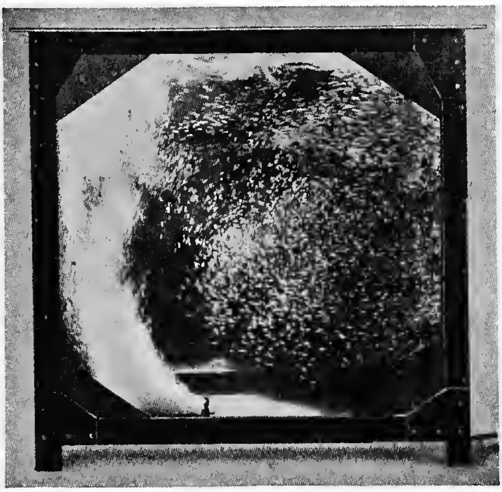


FIG. 180.—Operating model of spiral-flow system of air diffusion.

. taking this into consideration, together with the fact that atmospheric surface was appreciably reduced, resulting in a reduction in interfacial contact between the sewage and atmospheric air, it was finally concluded that the tank which gave the best results was the open tank without baffles.

Hurd's experiments (8) indicated that a material saving in air could be secured by spiral circulation in comparatively wide tanks provided with suitable deflecting baffles. In Hurd's words,

. . . Properly designed angular baffles, in correct relation to air diffusers, accelerate circulatory velocities without perceptibly reducing surface atmospheric contact. It is believed that curved baffles . . . are less effective.

With deflecting baffles set at an angle of about 45 deg. and extending laterally about 15 per cent of the width of the tank on each side, the velocity was increased practically 100 per cent in tests at Indianapolis. Concerning spiral circulation Hurd has expressed the following views (9):

The effectiveness of this method, in addition to the use of deflecting baffles, depends on the arrangement of the air diffusers. The whole design might differ with unscreened and unsettled sewage. The arrangement of diffusers depends upon whether it is necessary to sweep the bottom with high velocity to prevent settling of the heavier solids or whether air might be used more effectively to give more complete diffusion.

In the design of a 25 per cent addition to the Indianapolis plant, Hurd (9) states that

. . . we have brought the diffusers even more to the center of the tank, which is one of the original and important features of spiral flow as distinguished from other systems with air eccentrically applied. In addition to this, we have provided diffusers which are set in a series of V shaped formations to break up uniformity and to prevent any possibility of short circuiting.

Experiments at Indianapolis have shown that as little as 0.6 cu. ft. of air per gallon of sewage is required to prevent sludge accumulations upon the bottom of tanks 30 ft. wide and 15 ft. deep. Tests at Chicago indicate that horizontal bottom velocities of 1 ft. a second will keep the sludge in suspension. Hurd, as stated in Chap. XXV, reports that velocities equal to or in excess of ½ ft. a second are satisfactory.

The formation of a central, more or less quiescent, core in spiral-flow tanks has been recognized as producing some short-circuiting. According to Makepeace (10), "the spiral-flow method is too perfect in that a good deal of quiescence is found in the center of the cone." At English plants this tendency has generally been offset by throwing transverse baffle walls across the tanks at intervals. A compartment length of 16 ft. is provided at the Davyhulme works in Manchester and 17½ ft. at Reading. Rectangular openings in the lower corners of the baffles above the diffusers maintain communication between the compartments. The area of the openings is about 4 per cent of the effective cross-sectional area of the tank at Davyhulme and 3 per cent at Reading. At Tenafly, N. J., air baffles are produced by transverse rows of diffusers.

The ratio of plate area to tank area is commonly smaller in spiral-circulation than in ridge-and-furrow tanks. At Indianapolis three rows of diffusers are set in recesses in the floor, provision being made for the subsequent addition of a fourth row if needed. With three rows the ratio of diffuser area to tank area is 1:13.3. When the tanks were placed in operation, it was found that only two rows of diffusers were necessary in practice, thus changing the ratio to 1:20. The center line of the first row of diffusers is 12½ in. from the wall, that of the second 17⅞ in. from the first, and that of the third and fourth 25½ in.

from the preceding ones. At the North Side plant in Chicago there are two rows of plates and the plate ratio is 1:9.4. The center line of the first row of diffusers is 24 in. from the wall and the distance between center lines of diffuser rows is 33 in. The aeration tanks at North Toronto, Ont., have two rows of diffusers and the plate area is 10.7 per

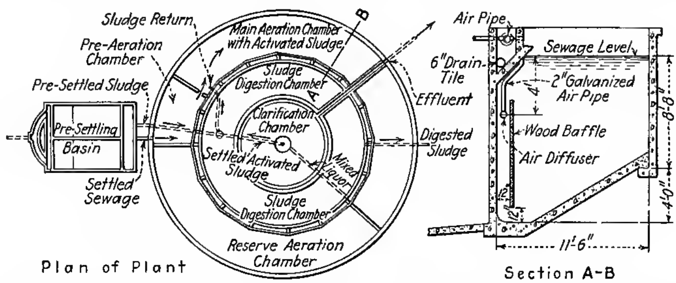


FIG. 181.—Plan and sectional detail of activated-sludge plant at Mercedes, Tex.

cent of the tank area. The distance of the center line of the first row of diffusers from the wall is 12.5 in. and that of the second row is 19 in. from the first.

A variation of the spiral-flow principle, as developed by Elrod (3) and employed at Mercedes, Texas, and elsewhere, is illustrated in Fig. 181. Air is diffused into the sewage through 3-in. porous tubes

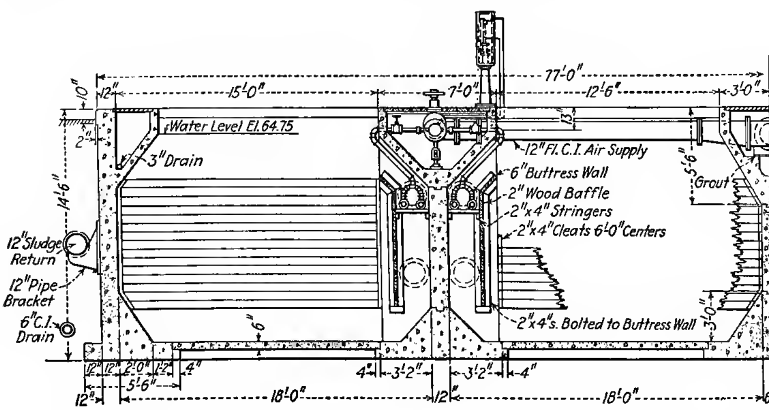


FIG. 182.—Cross section of aeration tank at Woonsocket, R. I.

suspended 4 ft. 2 in. below the flow line. To obtain spiral flow, the tubes are hung along the inner wall of the tank, which is ring shaped in plan, and a vertical baffle is placed so that a pronounced upward current is induced.

The Elrod system has been installed at Woonsocket, R. I. Here there are four aeration tanks, each 180 ft. long, 18 ft. wide and 12 ft.

deep below the flow line. Longitudinal vertical baffles of wood extend from 30 in. above the bottom of the tank to 3 ft. below the water surface at a distance of 42 in. from the side wall. The porous air-diffuser tubes are suspended 6 ft. below the water surface in the narrow section between wall and baffle. Wooden cross baffles, with bottom and top at the same elevations as those of the longitudinal baffle, are provided in the aeration tanks, spaced 45 ft. apart. A typical tank section is shown in Fig. 182 (11).

Longitudinal-furrow System.—The longitudinal-furrow or split-circulation system was developed at Manchester, England. This system is incorporated in the plants at Gastonia and Charlotte, N. C. It resembles both the ridge-and-furrow and spiral-circulation system, the former in construction and the latter in operation. Transverse travel of the sewage and air is induced in two directions by each row of diffusers. Two or more longitudinal rows of diffusers may be employed. The rows are separated by ridges, as in the case of the ridge-and-furrow system, and beveled corners are employed, as in spiral-flow units.

At Gastonia there are two rows of diffusers and the ratio of plate area to tank area is 1:5; at Charlotte there are three rows and the ratio is 1:6. The system installed at the Irwin Creek plant in Charlotte is illustrated in Fig. 179.

Comparison of Diffused-air Systems.—The ridge-and-furrow and spiral-flow systems were compared by the Chicago Sanitary District in experiments on corn-products wastes. From 4.0 to 4.7 cu. ft. of air per gallon were required for the former against 3.0 to 3.5 cu. ft. for the latter. Results obtained with spiral flow, furthermore, were found to be more uniform and the working of the tank was considered more reliable (12). Advantages claimed for spiral-flow over ridge-and-furrow units are: a reduction in air requirements, a longer path of travel secured for both air bubbles and sewage, greater absorption of atmospheric oxygen at the tank surface and reduced short-circuiting. It has been computed that the length of travel of the sewage in the Indianapolis tanks, which are 15 ft. deep and 20 ft. wide and consist of four compartments 238 ft. long, is over 7 miles. The pitch of the helix is estimated at 1.75 ft.

Comparing ridge-and-furrow aeration with spiral flow, Mohlman and Wheeler (13) state:

> In the earlier plants using diffused air, the duffusion was accomplished through the ridge-and-furrow type of tank bottom. Although this scheme gave very thorough aeration and agitation, the motion of the liquid was not uniform and the air bubbles rose directly from the plates to the surface as rapidly as possible.
>
> The introduction of the spiral-flow type of aeration, as exemplified on the largest scale at Indianapolis and the North Side Treatment Works, is

considered by us to be a distinct improvement in efficiency of aeration. The uniform motion of the liquid makes it possible to keep the sludge in suspension with less air and in our opinion the time of contact of the air bubbles with the sewage is longer than in the ridge-and-furrow system, when the bubbles rise directly to the surface. Our studies indicate a decided economy in air requirements when the spiral-flow type of design is correctly interpreted. Its efficiency depends upon the proper placing of the diffuser plates, the proper rate of air through the plates, the tank dimensions, the use of baffles and other variable factors. If these factors are not considered properly, all the benefits of this improvement may not be realized.

Tank Dimensions.—The capacity of aeration units is determined by the period of aeration, allowance being made, as previously stated, for the return sludge as well as the incoming sewage. The choice of tank dimensions, depth, width and length, depends upon the system of air diffusion, power consumption, local subsoil conditions and the area available.

Deep tanks conserve area but require more power for compressing the air. At Milwaukee, where a depth of 15 ft. was adopted, the controlling consideration was the impracticability of obtaining sufficient land for shallower tanks. Experiments at the Milwaukee demonstration plant with the ridge-and-furrow type of tank indicated a slight difference in favor of the 15-ft. depth of tank, not taking into consideration the extra cost of compressing the air for the deeper tank. Deep tanks give a longer period of contact between air and sewage than do shallow tanks. The air bubbles become relatively large, however, as they approach the surface, and there is a tendency for large bubbles to coalesce. There is some ground for the opinion, therefore, that the relative efficiency of surface and subsurface aeration thus may be reduced. Parsons and Wilson (14), however, report experiments on fill-and-draw tanks 6½ and 22½ ft. deep, in which they secured equal oxygen concentration in the sewage and equal purification, while using less air for complete nitrification of the sewage in the deep than in the shallow tanks. Some difficulty was experienced with the rising after apparent sedimentation of the sludge produced in the deep tanks.

In connection with tank depth, tank width and velocity of circulation, as well as the characteristics of air compressors, require consideration.

The effective depth of aeration units in large American plants is commonly 15 ft., depths of about 10 ft. being most in favor for smaller installations. Utilization of existing structures in English plants has resulted in the use of lesser depths in many installations in that country.

Long tanks reduce short-circuiting, but the choice of length is influenced also by the desirable width and depth. Width is greatly affected by requisites of circulation and diffuser arrangement. As shown in

Table 113, widths varying from 23 to 2 times the depth have been employed in American plants of the ridge-and-furrow type. The tanks at Milwaukee were made 22 ft. wide, in order to secure an economical arrangement of air piping and diffuser plates. The number of plates fed by a single line was restricted to nine 1-ft. square plates and an air main was installed along the center line of the tank to supply them and the central line of diffusers. The width of spiral-flow tanks varies from a value about equal to the depth to more than twice the depth. The longitudinal-furrow tanks at Gastonia and Charlotte, N. C., possess widths that are 0.8 and 1.36 times the depth, respectively.

The spiral-flow tanks at Indianapolis, Ind., are 20 ft. wide and 15 ft. deep. At the North Side plant in Chicago the tanks are 16.18 ft. wide by 15 ft. deep. The aeration tanks at North Toronto, Ont., are 13 ft. wide and 10.5 ft. deep.

Roe (15) has reported that, "as an experiment, a division wall was removed between two aeration tanks at the North Side plant at Chicago making the tank approximately 33 ft. wide with a 15-ft. liquor depth. This has been successfully operated using only the two rows of diffusers formerly employed in one 16 ft. wide tank." Since this experiment there has been a tendency to install wider tanks. The aeration tanks at the Easterly plant in Cleveland are 27 ft. wide by 15 ft. deep.

The desired length may be secured, where necessary, by dividing the tank into several compartments by means of longitudinal, round-the-end baffles. As previously noted, one to four passes have been employed in American designs.

Deflectors.—The surface deflectors commonly employed in spiral-flow tanks in some cases are made an integral part of the structural design, as at Indianapolis and North Toronto, and in other cases they are bracketed to the walls, as at the North Side plant in Chicago and at Elyria, Ohio. At Indianapolis the slope of the deflectors is six vertical on seven horizontal above the diffusers and six vertical on five horizontal on the opposite side, as shown in Fig. 178. They occupy 18 and 12.5 per cent, respectively, of the clear width of the tank. The large V-shape trough, created by this design above the wall adjacent to the diffusers, carries the air-supply piping. The troughs on all the walls are filled with gravel and are drained. At North Toronto the deflectors form an angle of about 42 deg. with the horizontal and occupy about 15 per cent of the width of the tank on each side. The top of the wall between faces of the deflectors is filled with concrete.

At the North Side plant the deflectors form an angle of 40 deg. with the horizontal and occupy about 21 per cent of the total width of the tank on each side, as shown in Fig. 183. The angle was chosen after full-scale experimentation. The deflectors are formed by concrete slabs placed in brackets that are integral with the tank walls and the walks in which the tops of the walls terminate. The deflectors at the Elyria plant are quadrants of circles. They are made of Armco iron and are bracketed to the wall, a clear space of about 2 in. being left between the bottom of the deflector and the wall, to prevent quiescence of sewage between the deflector and the wall.

The spiral-flow type of tank is commonly equipped with a fillet or bevel at the junction of the tank wall and bottom. At Indianapolis the slope of the finished surface of the fillet on the wall opposite the diffusers is 12:7 and begins 3 ft. above the bottom. At North Toronto the fillet is set at 45 deg. and is 12 in. high. At the North Side plant the fillet is set at 45 deg. and is 18 in. high. The fillet adjacent to the diffusers is generally somewhat smaller, as indicated in Fig. 183.

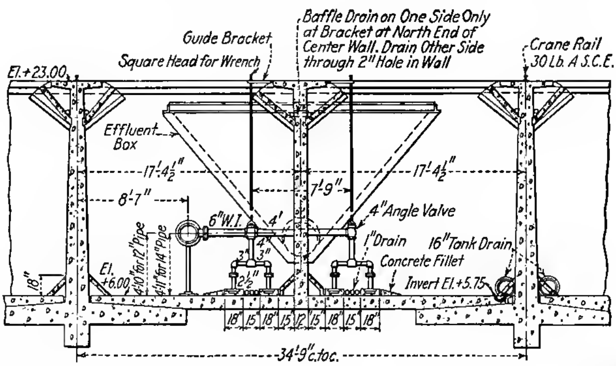


FIG. 183.—Cross section of aeration tanks at North Side works, Chicago.

Characteristics of Plate Diffusers.—The requirements for air diffusers have been stated on page 632 and the different types of diffusers in common use have been described. The design of activated-sludge units employing diffused air entails a knowledge of the normal quantities of air that can be handled by different diffusers and the pressure losses involved.

Diffuser plates are commonly chosen on the basis of permeability rating. This rating is the number of cubic feet of air per minute which will pass through 1 sq. ft. of diffuser, when tested dry at 70°F. under a pressure equivalent to 2 in. of water. A method of testing diffuser plates for permeability rating, developed by Hatton, is illustrated in Fig. 184 (16). In plant design the characteristics of the plates when wet are the determining factor, due allowance being made for the clogging to which they are subject.

Holmstrom (17) has reported on the selection of plates with regard to permeability as follows:

In determining the specifications for porous plates and porous plate holders in aeration or preaeration tanks, the problem of cleaning plates after installation is an important factor in the final decision.

In selecting permeability of porous plates, a second factor is the size of bubble. Early practice considered that the smaller the bubble size, the greater was the oxidation that takes place as the bubbles of air move up through the sewage. Based on this factor alone, a plate of low permeability was selected. The difficulty that was found, however, in selecting porous plates of a low permeability, was that the plates clog too quickly which means that the cost of operation is increased. Accelerated clogging tests which have been conducted also prove that clogging occurs less rapidly with higher permeability than with lower permeability. Early installations contained plates of permeabilities of less than 10. Practically all these

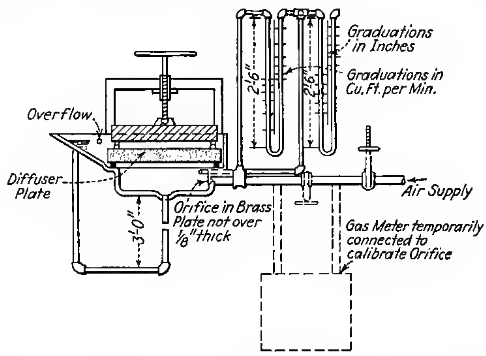


FIG. 184.—Apparatus for testing porosity of diffuser plates.

plates as well as some of even higher permeability have been removed because of the problem of keeping these plates clean.

The tendency at present is to allow the cleaning factor to decide the permeability rather than the bubble size. In reality, changing to higher permeabilities will not affect the pore size as much as might be imagined. In a 20 permeability plate, the average diameter of pores is 0.21 mm. and in 40 cu. ft. plate 0.30 mm. Further, it has also been found that the major part of the oxidation takes place at the surface of the sewage and consequently the efficiency of the aeration tanks is not reduced to any appreciable degree by the larger bubble size which results from using plates of higher permeability.

A third factor which should be considered in the selection of plates is the friction loss through the plates. . . . The wet pressure loss of a 40 permeability plate is slightly over one third of the wet pressure loss at 10 permeability and less than one half the wet pressure loss at 20 permeability. . . .

In actual practice in recent installations, we find that permeabilities as high as 50 are in use.

Holmstrom further reports that the Wards Island plant at New York will use plates of 20 permeability; that Pasadena, Cal., uses 50 permeability; that Birmingham, England, uses 45, and that Indianapolis has found 35 successful.

The manufacturers of Aloxite diffusers state that diffuser plates with a permeability rating of 30 to 36 are now recommended. The manufacturers of Filtros plates have reported that the general tendency is toward adopting a much coarser plate than was originally used in the activated-sludge process and that ratings of 16 to 20 are within the most popular range.

It is desirable that diffuser plates be tested for permeability at the point of manufacture and that the rating to the nearest 0.1 cu. ft. be stamped or stenciled thereon. Not less than two plates out of each hundred should be tested for uniformity and pressure loss. All plates controlled by any one air valve through the aeration tanks or channels should be of the same permeability rating, within 0.5 cu. ft. Not less than 6 in. of clean water should be maintained over the plates while the tank is not in service.

Filtros Plates.—The standard Filtros plate is 12 in. square and 1½ in. thick and weighs about 12 lb. The plates are manufactured in seven grades that differ in porosity. The manufacturer's rating of Filtros plates is shown in Table 110.

TABLE 110.—AVERAGE CHARACTERISTICS OF FILTROS PLATES

Grade	Porosity, per cent	Modulus of rupture, lb. per sq. in.		Breaking load,[1] lb. per sq. in.	Permeability[2]	Air resistance of water-saturated plate, in. of water when passing per sq. ft. per min.[3]		
		Dry plate	Wet plate			2 cu. ft.	4 cu. ft.	8 cu. ft.
A. Extremely coarse	36.5	381	341	6.7	60	7–8	8–9	12–13
B. Coarse	33.8	517	412	9.3	40	7–8	8–9	12–13
C. Fairly coarse	30.2	644	499	11.3	24	8–9	9–10	13–14
R. Medium	33.7	568	545	12.3	15	9–10	10–11	14–15
S. Fairly fine	31.1	856	645	14.6	10	10–11	11–12	15–16
E. Fine	28.8	734	712	16.1	6	11–12	13–14	17–18
H. Dense	26.0	1284	1247	28.2	1.5	13–15	16–18	22–24

[1] Computed from wet modulus for 1½-in. plate; load uniformly distributed on 11½-in. span.

[2] Cu. ft. of air per minute per square foot of plate area under a pressure of 2 in. of water. Plate 1½ in. thick.

[3] Plate 1½ in. thick.

The results of tests of the loss of head through various grades of Filtros plates, as well as other types of plate diffusers, are presented in Fig. 185 (18). The manufacturer comments as follows upon the capacities and air resistances of dry and water-saturated Filtros plates:

With dry plates the volume of air passing appears to be directly proportional to the pressure, but with the plates water-saturated, this does not seem to hold true, probably because the water in the plate is gradually forced out of the pores. With a fixed volume of air passing, the pressure resistance gradually decreases, or with a fixed pressure the volume passing gradually increases. In other words, the resistance of the water-saturated plate is not constant, hence the latitude in the figures dealing with the ratings of these plates.

Alundum Plates and Tubes.—The standard Norton plate is 12 in. square and 1 in. thick. The plates weigh about 11 lb. and there are

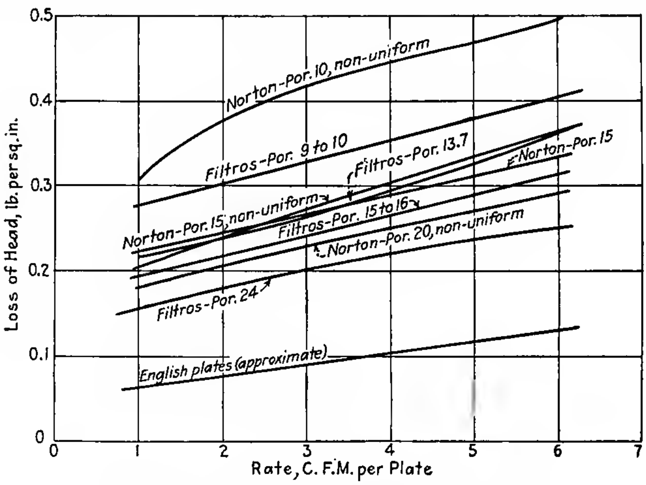


FIG. 185.—Loss of head through diffuser plates.[1]

three standard grades, coarse, medium and fine. The manufacturer's rating of Norton Alundum plates is shown in Table 111. Porous Alundum tubes are available in various diameters. At Woonsocket, R. I., tubes 2 ft. long, with an internal diameter of 4½ in. and a wall thickness of ⅝ in., have been used.

Aloxite Plates and Tubes.—The standard Aloxite plate also is 12 in. square and 1 in. thick. The standard tube is 2 ft. long, with an internal diameter of 3 in. and a wall approximately ⅝ in. thick. The porosity of the medium grade ranges from 34 per cent for plates of high permeability to 38 per cent for plates of low permeability. Plates and tubes may be obtained with permeability ratings ranging from a minimum of 4.0 cu. ft. a minute to a maximum of 60.0 cu. ft. a minute. The relation of pressure loss to air flow for plates within the range of ratings usually specified for sewage aeration is given in Table 111.

[1] Plate box full at start; constant head maintained for 20 min.; then drained. Tests made under water head of 14.5 ft. and with free air at 14.7 lb. per sq. in. absolute at 62°F.

TABLE 111.—CHARACTERISTICS OF ALUNDUM AND ALOXITE DIFFUSER PLATES

Grade	Porosity, per cent	Average modulus of rupture lb. per sq. in.		Average breaking load, lb. per sq. in.[1]	Permeability[2]	Water-saturated plate	
		Dry plate	Wet plate			Pressure loss, in. of water	Volume of free air, cu. ft. per sq. ft. per min.
Alundum plates							
Coarse.....	34	2684	2235	22.6	40.5	5	3.48
						6	6.60
Medium....	35	3045	2644	26.7	22.5	6	0.40
						7	2.69
						8	4.28
Fine.......	36	3770	3716	37.5	4.3	21	3.09
						23	4.18
						25	5.95
Aloxite plates							
					34.5	5	2.55
						6	6.35
					25.4	6	1.00
						7	4.00
						8	7.00
					14.7	10	2.75
						11	5.50
						12	8.15

[1] Computed from wet modulus for 1-in. plate; load uniformly distributed on 11½-in. span.

[2] Cu. ft. of air per minute per square foot of plate under a pressure of 2 in. of water. Plate 1 in. thick.

Jones and Attwood Plates.—The Jones and Attwood diffuser, as generally employed, consists of a porous tile set in a shallow cast-iron box, of which it forms the cover, this box being connected to the air supply. The sizes vary from a few inches to 4 ft. 6 in. in length and from 1 in. to 24 in. in width. It is difficult to obtain diffusers which are uniformly porous and, as generally used, each diffuser or pair of diffusers is supplied by a separate down-pipe with a valve for regulating the air supply. The plates are usually designed to pass 2 cu. ft. of free air a minute per square foot of surface with a head equal to 3 in. of water, producing bubbles of less than 1/8 in. in diameter.

Clogging and Cleaning of Diffuser Plates.—Provision is commonly made for thoroughly cleaning the air before passing it through diffuser plates. Nevertheless, some trouble has been reported due to the clogging of the bottoms of plates. At Houston, Tex., Filtros plates, clogged by iron rust from cast-iron holders, were cleaned by immersing them in a 10 per cent solution of hydrochloric acid for a few hours. On the other hand, no trouble from clogging by iron rust has been reported from the many plants in Canada and England using the Jones and Attwood diffuser plates, which are set in cast-iron containers.

Copeland (19) has stated that inspection of Filtros plates after 4 years of operation in experimental aeration tanks at Milwaukee disclosed that a gelatinous coating, containing clots of bacteria and filamentous algae, practically covered the bottoms of the plates. This coating had increased the air pressure from 6.5 to 7.8 lb. per square inch.

At Pasadena, Cal., plates that had been set unsuccessfully in asphalt were cleaned by washing them with carbon bisulfide, coal oil and water. In spite of this, they lost about 25 per cent in efficiency.

Clogging of plates at Indianapolis has been relieved by taking a tank out of operation, drawing off the liquor and resting the tank.

Clogging of the upper surfaces of diffuser plates has been experienced at a number of plants. This has been relieved, in part at least, by scrubbing the plates with compressed air and by washing them with a weak solution of hydrochloric acid. Grinding with emery wheels and sand blasting have been tried with comparatively poor results.

According to Townsend (20), a blow-torch method utilized at Milwaukee has restored to their original state of porosity Filtros plates which were clogged at the surface but were free from clogging on the bottom. This method consists of applying the blow-torch flame to the surface of the plate, until the temperature results in cracking off the entire top surface. The thickness of film removed varies from extreme thinness to 1/8 in. The scale is swept off and the plate is rubbed smooth with a carborundum block, applied for a few minutes. Air is blown through the plate continuously during the cleaning. The cost of the work at Milwaukee was about 10 cents a plate.

The tendency toward selection of plates that have greater permeability than those at first installed, in order to reduce clogging, has been pointed out.

Plate Containers.—Diffuser plates are commonly set in precast concrete containers, that are placed on the floor or sunk into recesses in the floor. An air line generally leads to each container. When the plates are spaced close together, they are commonly supported on galvanized-steel angles. When sufficient space intervenes, reinforced-concrete construction may be employed throughout.

At Milwaukee the largest containers hold 9 plates, as shown in Fig. 186; at the North Side plant in Chicago a 10-plate container similar in construction has been employed. At the Wards Island plant in New York precast concrete plate boxes are used, the double-row boxes

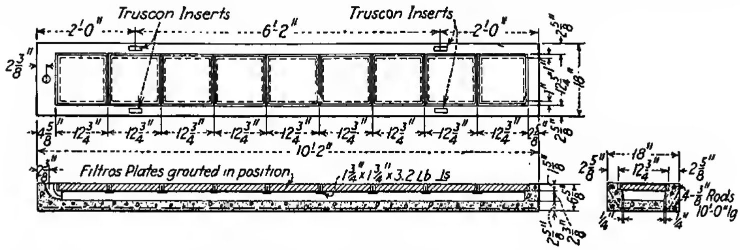


Fig. 186.—Concrete plate container at Milwaukee sewage-treatment plant.

containing 14 plates and the single-row containing 10 plates each. At Indianapolis the plate containers are an integral part of the floor construction, as shown in Fig. 178, and the recesses are interconnected by 4-in. vitrified-tile pipe laid in the floor. Fifty plates are operated from each air-control valve.

Cast-iron plate containers are subject to rust formation and have given trouble from this cause at Houston, Tex. Nevertheless they

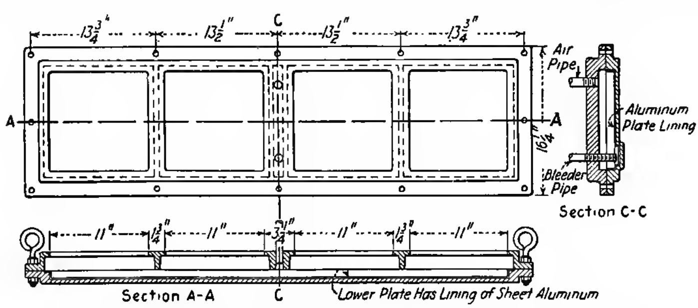


Fig. 187.—Cast-iron plate container at Irwin Creek plant, Charlotte, N. C.

are used widely in England. A cast-iron container protected by an asphalt coating and a lining of sheet aluminum, each container holding four plates, has been employed with apparent success at Charlotte and Gastonia, N. C. One of the Charlotte containers is shown in Fig. 187 (21). In Durham, N. C., the plates have been placed in vertical holders made of galvanized cast iron, as developed by Piatt, who also developed the containers used at Charlotte and Gastonia.

Care is required to prevent the escape of air through the joints around the plates. The authors have investigated a number of grades of asphalt and pitch but have found none which would not allow the air to blow through it under the excess pressure required to force the air through the Filtros plate—about ¾ lb. per square inch—at a temperature of 80°F. As a result of this investigation they have used Portland cement mortar for filling the joints. This has proved impervious to air and otherwise satisfactory, except that it is difficult to cut it out and remove the plates without damaging them.

Lately, in some cases, the joints have been made by filling the bottom half with oakum and the top half with a Portland cement mortar. This is said to make it easier to remove the plates for cleaning or replacement than is the case with the full cement-mortar joint.

Recently various forms of containers have been devised which allow ready removal of the plates for cleaning. Metal containers of this type are relatively expensive and may not find wide application. The Norton Co. has developed a precast concrete frame to be set in a channel, cast in the concrete floor of an aeration unit. The plates are held in place by means of bolts and clamps which may be easily removed. The manufacturer's estimate that the cost of a frame for four plates will be about 63 cents a plate.

Air Piping.—Since air is an elastic fluid, it may be conveyed through pipes at high velocities without running the risk of placing excessive strains on the piping, due to the sudden checking of flow. To secure equality of air distribution in activated-sludge tanks, however, the friction loss is usually kept low enough to avoid using an excessively large number of regulating valves. At Milwaukee, with 2514 Filtros plates to a tank, one valve was provided to control 1000 plates. At North Toronto each valve governs the air flow to four 8- by 35-in. diffuser plates. In England, on the other hand, separate regulating valves for each diffuser seem to be preferred. In designing the system of air piping, it is necessary to balance economy of materials against power consumption. Fuller and McClintock (22) state that air velocities should generally not exceed 2000 to 3000 ft. a minute and that the distribution loss, including the plate loss, should usually be below 1.0 to 1.5 lb. per square inch. The pressures carried in the air mains, therefore, usually vary from 5 to 8 lb. per square inch above atmospheric pressure for tanks 10 to 15 ft. deep.

The air mains are commonly constructed of cast iron, the submains of cast iron, steel or wrought iron, and the diffuser supply pipes of wrought iron or steel. Since rust would quickly clog the diffuser plates, cast-iron mains are generally coated with some bituminous material and wrought-iron and steel pipes are usually galvanized. Aluminum and its alloys are used extensively at the Easterly plant in Cleveland on small air piping, to resist corrosion.

Design of air piping involves a study of the velocity heads and friction losses in pipes and fittings.

Pipes.—Estimates of pipe losses can be made by a number of formulas. The Sanitary District of Chicago has tested and employed the Fritzsche formula, which may be stated as follows:

$$s = \frac{1.268Q^{1.852}t}{10{,}000pd^{4.973}}$$

where s = drop in pressure per 100 ft. of pipe, pounds per square inch
Q = cubic feet of free air per minute at 60°F.
t = absolute temperature in degrees Fahrenheit = recorded temperature in degrees Fahrenheit +459.6
p = absolute pressure in pounds per square inch = gage pressure +14.7
d = diameter of pipe, inches

Special allowances must be made for losses due to bends, entrance, venturi meters, valves and the like. The Fritzsche formula was developed from extensive tests in which air velocity varied from 8.2 to 190 ft. a second, temperature from 57 to 239°F., and pressure from 2.9 to 164 lb. per square inch absolute. Only small pipes were studied, however.

Morrill (18) has prepared a chart, shown in Fig. 188, for the solution of this formula (23). Tests of 10- and 24-in. cast-iron pipes and a 4-in. steel pipe by Bushee and Zack of the Chicago Sanitary District indicate that, for 10- to 24-in. cast-iron, bell-and-spigot pipe, the friction loss will not be far from actual conditions, if computed by the Fritzsche formula and multiplied by 1.25, and that this formula is practically suitable for wrought-iron and steel pipe and does not give too low results for galvanized-iron pipe. Air temperatures need to be carefully selected, allowances being made for the rise in temperature due to compression and the subsequent temperature drop in the piping due to external cooling, particularly in submerged pipes, and due to expansion cooling when the mains carry a higher pressure than needed at the point of discharge; for example, when shallow channels carrying mixed liquor or sludge are supplied with air from mains leading primarily to deep aeration units.

Air Meters.—A study of losses in venturi air meters by the Chicago Sanitary District has shown that the losses vary approximately with the differential head and are somewhat higher in small meters than in large ones, as recorded in the schedule (18) at the bottom of page 651.

The accurate measurement of air is important, for upon the correct application of air depends much of the success of the activated-sludge process.

For measuring and indicating or recording the flow of air, several types of equipment are available. The most common arrangement

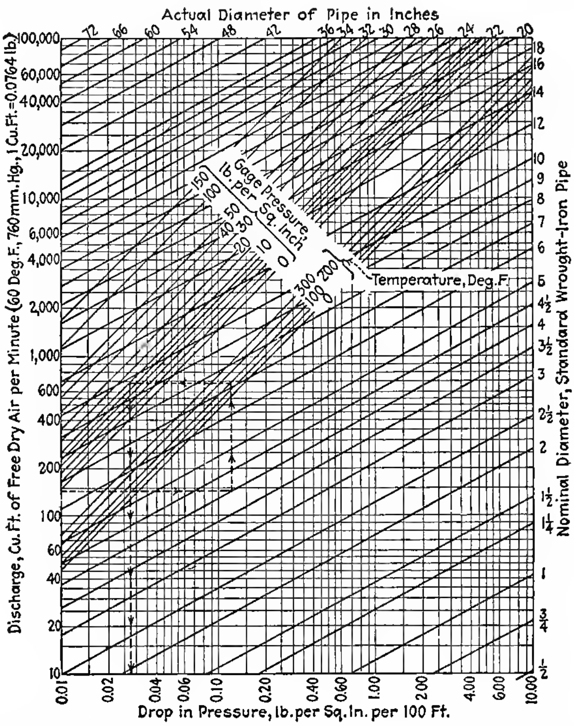


FIG. 188.—Flow of air in circular pipes, based on Fritzsche formula.

on the main supply or for large flows is the venturi air meter, shown in Fig. 189, in conjunction with an indicating and recording register. For

Size of meter, in.	Air pressure, lb. per sq. in.	Air temperature, °F.	Loss of head	
			Per cent of differential head	Lb. per sq. in. per ft. differential
10 × 5	21.5	86	18	0.078
5 × 2	22.0	80	22	0.095
4 × 2	18.7	86	21	0.091
3 × ¾	21.5	78	24	0.104

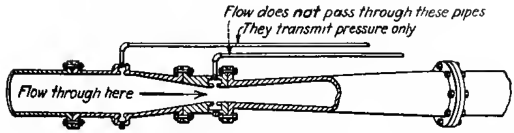


FIG. 189.—Venturi air meter.

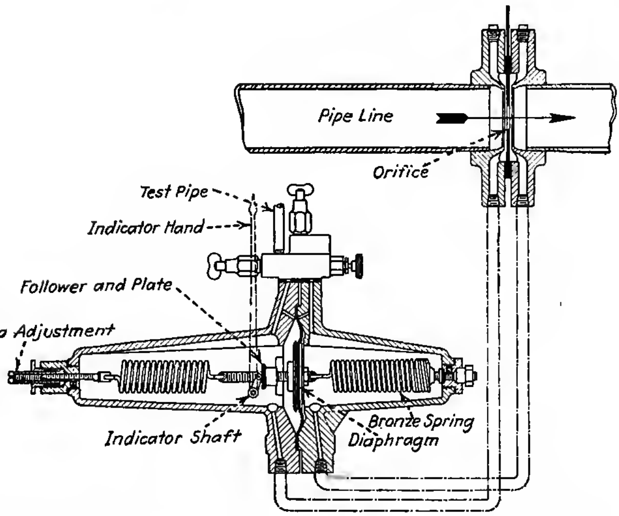


FIG. 190.—Orifice plate and "Oriflo" diaphragm meter. (*Builders Iron Foundry.*)

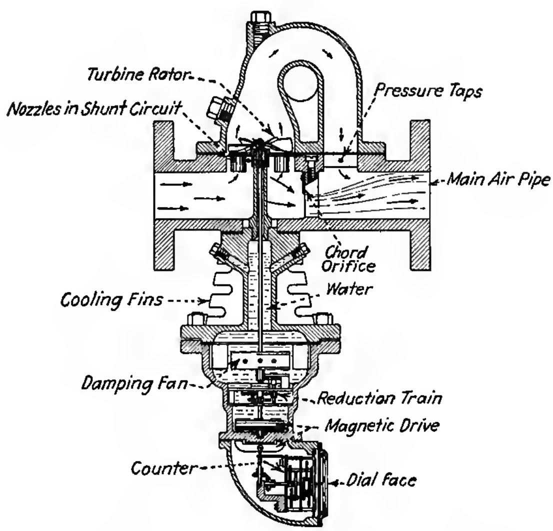


FIG. 191.—"Shunt" air meter. (*Builders Iron Foundry.*)

small flows or divisions of the main flow, when the recording of the flow does not warrant the installation of a register, indicating manometers are sometimes employed. Orifice plates may be used, in conjunction with register-indicator recorders, manometers, or "Oriflo" diaphragm meters. The last-named device is illustrated in Fig. 190. The "Shunt" meter, shown in Fig. 191, recently has been developed for measuring the flow of air through small pipes.

Most designs include two or more sizes of meters, a large one on the main supply and smaller ones on the lines leading to aeration units, sludge and mixed-liquor channels, re-aeration and sludge-conditioning tanks, and air lifts. Bushee and Zack (18) have made the following statement in regard to the selection of air meters:

> Meters should be selected to give the minimum differential head consistent with accurate readings. To keep down the loss it is desirable to keep the differential head in a venturi air meter less than a foot of water, and for most types of recorders the accuracy is not good for differentials less than an inch of water. Differential heads between a foot and an inch give a ratio of maximum to minimum air flow of 3.5 to 1.0. By properly selecting the meter tubes the total loss in two tubes could be kept to 0.1 lb. per square inch.

Bends and Valves.—Martin has worked out from formulas given by Hurst (24) the length of pipe, in feet, equivalent to a square elbow and to a 90-deg. bend, whose radius is equal to a pipe diameter. His results are shown in the following schedule:

Diameter, in.	1	2	3	4	6	8	10	12	14	16	18	20	24
Equivalent length of pipe, ft.:													
Elbow	1.5	4.9	9.4	14.5	25.9	38.0	50.7	63.7	76.7	90.1	104	117	144
Bend	0.23	0.74	1.41	2.2	3.9	5.7	7.6	9 6	11.5	13.5	15.5	17.5	21.6

Martin states further that the head due to a square elbow is approximately equal to one velocity head and that the loss of pressure at a screw-down valve is about 1½ times that due to a square elbow. Saunders (25) recommends the following formulas:

1. For globe valves:

$$\text{Additional length of pipe in ft.} = \frac{11.4 \times \text{diameter of pipe in in.}}{1 + (3.6 \div \text{diameter})}$$

2. For elbows and tees:

$$\text{Additional length of pipe in ft.} = \frac{7.6 \times \text{diameter of pipe in in.}}{1 + (3.6 \div \text{diameter})}$$

Elbow losses are sometimes assumed to equal the loss in a pipe of like size approximately 30 diameters in length. In testing a 10-in.

aluminum-flap check valve, to which an elbow was attached, at the Des Plaines works of Chicago, Bushee and Zack assumed this value and found a loss of about 0.06 lb. per square inch for normal air flows.

In most designs two or more check valves are employed, one on the main following the blower and the others on the takeoffs to the individual units. In some of the recent designs, check valves on the lateral air lines to the individual aeration tanks have been omitted. At Springfield, Ill., the check valves originally installed on the air lines to the aeration tanks have been removed, with a resultant reported saving, due to decreased friction, of about 3 per cent in power requirements for air compression.

Power Required for Compressing Air.—The pressure to which the air must be raised is equal to the pressure due to the head of sewage in the tanks plus distribution losses in piping and plates, with a reasonable factor of safety.

Since heat is generated in compressing air, compression is not *isothermal* but *adiabatic*, the heat not carried off swelling the volume of the air and increasing the power expended by an amount corresponding to the increase in volume compressed.

TABLE 112.—POWER REQUIRED FOR COMPRESSING AIR

Final pressure of air, lb. per sq. in.	Theoretical work to compress 1 mil. cu. ft. of free air, hp.-hr.	Theoretical power to compress 100 cu. ft. of free air a minute, hp.	Theoretical cost of compressing 1 mil. cu. ft. of free air with elec. power at 1 ct. a kw.-hr.
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, as is the practice in nearly all blower work, from atmospheric pressure and an initial temperature of 60°F. Weight of 1 cu. ft. of free air = 0.0764 lb. Slightly lower power consumption may be obtained with good water-jacketed reciprocating compressors.

The power required and its cost can be estimated from data in Table 112, 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 60 to 70 per cent.

A convenient approximate unit of power to bear in mind is that 30 hp. per hour will be required for furnishing 1.0 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 15 ft. deep.

Air Compressors.—A number of types of air compressors have been used in activated-sludge plants, among which the following may be mentioned: single-stage compressors; positive-pressure blowers; centrifugal compressors; and Hytor compressors. The last three mentioned are illustrated in Fig. 192. The first has not been used much in this country, but a number of installations have been made in Canada and England.

In selecting the type of compressors, 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, requires consideration. The initial operating pressure may be increased from time to time on this account. It is also important to provide a flexible compressor plant, capable of adjustment in air supply to the volume and quality of the sewage and to other conditions and requirements subject to considerable change. In large plants variation in volume of air can be secured to a reasonable degree by throwing in or out of service one or more compressors. In smaller plants it may be of importance to select compression units with which the volume of air furnished by each unit can be varied without wasting power or material loss in efficiency. The several types of compressors vary greatly in size and weight, so that the type of equipment selected affects materially the design of the structural details of the compressor plant.

Four types of prime movers may be employed, namely, electric motors, Diesel engines, gas engines and steam engines. In selecting the type of power, consideration is generally given to practical operating conditions, the general power requirements of the plant being taken into account. Power may be needed for sewage and sludge pumps, screens, sludge conveyors, dryers and other mechanical equipment that may constitute part of the treatment works. Where sewage solids are digested, utilization of the gases of decomposition sometimes is adopted. Some of the questions to be considered are discussed in Chap. XVIII, "Pumps and Pumping Stations," of Volume I of this work.

Piston-type Compressors.—At a number of Canadian and English plants, high-speed, single-stage piston compressors are in use, generally driven by electric motors, either directly or with belt connection. This type is moderately efficient but has the serious disadvantage that it is difficult to keep cylinder oil out of the compressed air. The compressors

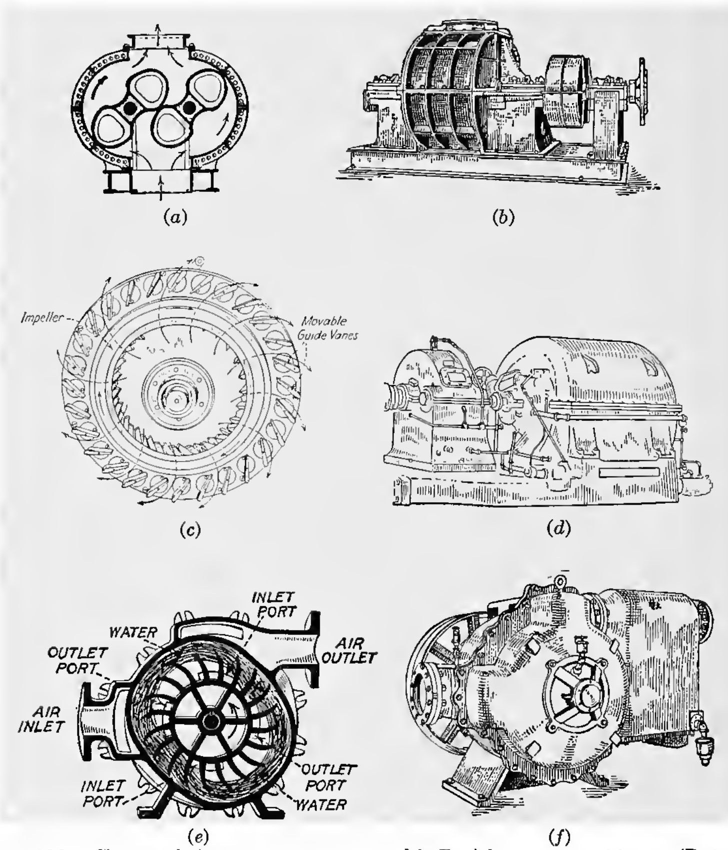


Fig. 192.—Types of air compressors. *a* and *b*, Positive-pressure blower (Rotary type); *c* and *d*, centrifugal compressor; *e* and *f*, Hytor compressor.

are heavy and cumbersome, as compared with other types for supplying air at the relatively low pressures needed for activated-sludge plants. The quantity of air supplied per unit cannot be varied easily to meet changing conditions. Three Belliss and Morcom reciprocating compressors, each of about 3000-c.f.m. capacity at a pressure of 6 lb., are provided at the North Toronto sewage-treatment plant. Air is conditioned by filters of the Midwest type.

Positive-pressure Blowers.—Positive-pressure blowers of the rotary type have been used for a number of medium-size installations. They are built with capacities ranging from 100 to 15,000 cu. ft. of free air a minute and for pressures up to 10 lb. to a square inch. The speed varies from 1000 r.p.m. or more, for small blowers of 100-c.f.m. capacity, to 300 r.p.m. or less, for large blowers of 5000-c.f.m. capacity. These blowers operate at relatively high efficiency, 75 to 80 per cent in the larger sizes, and are of comparatively simple and reliable design. The space and foundation requirements are nominal. They may be driven direct-connected to steam or oil engines or slow-speed electric motors. Chains, belts or gears are commonly used for connection with high-speed electric motors. These blowers do not introduce any oil or dirt into the air, if suitable stuffing boxes are provided.

At a given speed, a practically constant volume of air is furnished by positive-pressure blowers, 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 diffuser plates and pipe lines. On the other hand, the capacity cannot readily be varied to conform to varying air requirements, unless provision is made for economically varying the speed.

According to Fuller and McClintock three positive-pressure blowers of 3200-c.f.m. capacity against 5 lb. pressure have operated satisfactorily for several years at Houston (22). Two are connected to 150-hp. electric motors and one to a 150-hp. Diesel engine. The overall efficiency of these units from the switchboard, referred to adiabatic air horsepower, is 50 to 55 per cent.

At Springfield, Ill., three positive-pressure Connersville blowers are installed, having capacities of 2200, 3300 and 4500 c.f.m. against a head of 8.5 lb. per square inch. The 2200- and 4500-c.f.m. units are driven by direct-connected synchronous motors. The 3300-c.f.m. unit is direct-connected to a 180-hp. gas engine. Test data on the operation of the latter unit are given in Chap. XIV. With this unit it is possible to change the engine speed and blower output, in order to match the air requirements in the activated-sludge process.

Hytor Compressors. Hytor compressors are manufactured in various sizes, ranging in capacity from 30 to 3000 c.f.m. and adapted to pressures up to 15 lb. per square inch, but are most efficient when delivering against pressures of 8 to 12 lb. The speed varies from 1600 r.p.m. for the smallest size to less than 250 r.p.m. for the largest. The power requirements are 30 to 40 per cent greater for Hytor compressors than for other types. It is otherwise a simple, reliable and efficient machine. The air is quite thoroughly washed during compression and contains no oil. A suitable separator is furnished with the compressor to remove water entrained in the compressed air.

The compressor is quiet in operation and produces a steady flow of air, free from pulsation, without the use of a receiver. The operating characteristics with regard to pressure and variable capacity are similar to those of the positive-pressure blower.

Four motor-driven Hytor compressors, developing a pressure of 8 lb. per square inch, are provided for the aeration tanks in the Calumet sewage-treatment plant at Chicago. Two units, driven by 150-hp. motors, have a capacity of 2500 c.f.m. each, another, driven by a 75-hp. motor, has a capacity of 1000 c.f.m. and another, driven by a 45-hp. motor, has a capacity of 450 c.f.m.

Six motor-driven units of this type, having a combined capacity of 11,450 c.f.m., are installed in the Des Plaines sewage-treatment plant at Chicago. Screens and an air washer are provided for cleaning the air at the latter plant and screens and an air filter are employed at the Calumet plant.

Centrifugal Compressors.—Centrifugal compressors, otherwise known as turbo-compressors, in sizes of 2500 to 40,000 c.f.m., are well adapted to the requirements of activated-sludge plants. Flexibility can be secured and the volume of air furnished can be varied by opening or closing the blast gate in the suction pipe. The power required varies approximately with the volume of air used, when operating at constant speed. However, this type of compressor furnishes air at a uniform pressure, which can be varied only by changing the speed of the impeller. With certain types of driving apparatus, this is not practicable. In such cases, the pressure necessarily is made high enough to meet all emergencies. During a large part of the time, therefore, there is a waste of power in compressing the air to a higher pressure than is necessary.

Centrifugal compressors in the larger sizes operate at comparatively high speeds, adapted to direct connection to steam turbines or high-speed electric motors. In some units, a single-stage centrifugal compressor is driven through increasing gears, so as to operate at high speed. This is claimed to be preferable to a low-speed machine which has two or more stages in series, being lighter, just as reliable, much more efficient, more accessible, occupying less floor space and costing less. Unless it is possible to adjust the speed, as in the case of a steam turbine, it is essential to know closely the maximum pressure against which centrifugal compressors will be called upon to operate.

At the Irwin Creek plant in Charlotte, compressed air is furnished at a pressure of 8 lb. per square inch by three 170-hp., motor-driven, 3000-c.f.m., single-stage, General Electric compressors, illustrated in

Fig. 193. The motor speed is 1750 r.p.m., with increasing gears to raise the speed of the compressors to 13,650 r.p.m.

The North Side plant at Chicago is equipped with seven turbo-compressors, driven by direct-connected synchronous motors at 3600 r.p.m. A view of the interior of the compressor house is shown in Fig. 194. All blowers are designed for a rated pressure of 7.75 lb. to a square inch

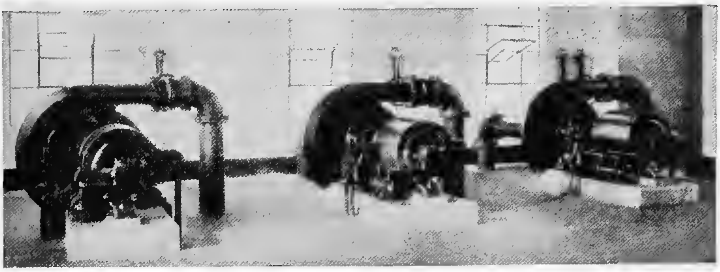


Fig. 193.—Centrifugal air compressors geared to induction motors at Irwin Creek sewage-disposal plant, Charlotte, N. C.

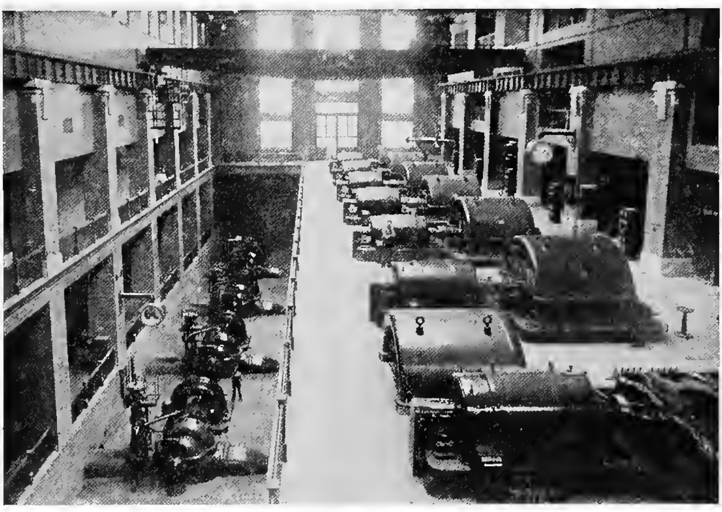


Fig. 194.—Compressor plant at North Side treatment works, Chicago.

at 85°F. Three units, driven by 1650-hp. motors, have a rated capacity of 30,000 cu. ft. of free air a minute and four units, each driven by 2160-hp. motors, supply 40,600 cu. ft. of free air a minute. The air-conditioning equipment includes air heaters, washers and oil-coated air filters.

The Milwaukee plant has four centrifugal compressors of 35,000-c.f.m. capacity at a gage pressure of 10 lb. to a sq. in., driven by steam turbines.

With steam at 200 lb. pressure and 100°F. superheat, these units showed a steam consumption of 10.98 lb. per brake-hp., including auxiliaries, or 10.11 lb. per 1000 cu. ft. of air. The test efficiency of the compressors alone was 66.3 per cent.

Air is furnished at the Sugar Creek plant in Charlotte by three De Laval, three-stage, centrifugal air compressors. Two of these compressors have a capacity of 3000 c.f.m. at a pressure of 8 lb. per square inch and are driven by 150-hp. motors. The motor speed is 1750 r.p.m. with increasing gears to give a compressor speed of 5850 r.p.m. The third unit is similar, except that it has a capacity of 3500 c.f.m. and is driven by a 225-hp. gas engine. This engine has a normal speed of 1200 r.p.m. with increasing gears to give a compressor speed of 5850 r.p.m. The engine may be adapted to use either gasoline or the gas collected from the separate sludge-digestion tanks at the plant.

Cleaning Air.—To prevent the clogging of plate diffusers, the air generally is cleaned thoroughly before being compressed. The means for accomplishing this are generally the same as those employed in the ventilation of office buildings, assembly halls and factories. They may be classified as filters, air washers, and oil cleaners.

Canton flannel and 10- or 12-oz. duck were employed as filtering media at a few of the early activated-sludge plants. At the Des Plaines works of Chicago the duck is supported on wooden slats and each square foot of cloth passes about 4 cu. ft. of air a minute. An examination of the air-cleaning equipment of American plants shows that air filters of cloth are not employed so widely as air washers or oil cleaners.

In air washers the air is drawn through a spray chamber in which it encounters a fine spray of water discharged from numerous atomizing nozzles. Leaving the spray chamber the air passes through an eliminator, consisting of narrow vertical passages formed by corrugated metal sheets, so formed as to give the air several changes of direction. The first corrugations are wet with water thrown out of the air by its change of direction. The dirt and other solid matter, which have been wetted by the water sprays, are thrown against these sheets and washed to a settling tank below. The last corrugations appear dry but accomplish some removal of free or unabsorbed moisture and solid matter. The water is strained and recirculated by a pump.

Air washers are employed at Indianapolis and at the North Side works of Chicago. The Indianapolis equipment is guaranteed to remove from the air 99 per cent of all dust and floating particles. Troubles with air washers due to freezing were encountered at Milwaukee. Heaters have been employed at Chicago for the purpose of avoiding such troubles. The frictional resistance of air washers generally is about ¼ to ½ in. of water, when passing air at rated capacity.

Oil cleaners of the contact type consist of cells filled with a metal medium, coated with a light oil. The air passages are tortuous and the impurities are removed on the contact surfaces. From time to time the cells are flushed with oil which is circulated by a pump from a reservoir. Oil cleaners are employed at the North Side works in Chicago, as well as at Milwaukee, North Toronto and Pasadena. During 1931 filters, in which the filtering medium is paper, were installed at the Milwaukee plant, to follow the oil cleaners. When the filtering medium becomes dirty, it is thrown away and new material is put in its place.

The Midwest air filter is composed of units 20 × 20 in. in area and 4 in. deep. Each cell passes 700 to 1000 cu. ft. of air a minute with a resistance of 0.25 in. of water. The guaranteed efficiency is 96 to 99 per cent removal of dust. It is guaranteed also that not more than 0.05 grain of dust per 1000 cu. ft. of air will remain in the cleaned air.

Re-aeration Units.—As shown in Table 113, re-aeration tanks were incorporated in the design of some of the early American works, such as the North Side plant at Houston and the plant at Pasadena. Re-aeration tanks were installed also at Indianapolis for emergency use. In general, the design of re-aeration tanks follows that of the aeration units themselves. At Houston the re-aeration tanks have one half the capacity of the aeration units. Of the air supplied at Houston about 2 or 3 per cent is used for air lift, two thirds of the balance for aeration and one third for re-aeration. The total volume of air used per gallon of sewage is 1.85 cu. ft. (26).

At Pasadena the re-aeration capacity is one sixth of the aeration capacity. During the year ending June 30, 1929, the average aeration period at Pasadena was 4.4 hr., the estimated average re-aeration period was 2.0 hr. and the total air supply per gallon of raw sewage treated was 1.9 cu. ft.

Statistics of Diffused-air Plants.—Statistics of a number of diffused-air plants are given in Table 113.

Costs of Construction, Operation and Maintenance of Diffused-air Plants.—The construction cost of diffused-air activated-sludge units, based on prices prevailing from 1925 to 1929 and including only aeration and final-sedimentation tanks, operating gallery, blower plant and return-sludge pumping plant, varies from $25,000 to $40,000 per million gallons daily capacity and from $2.50 to $5.00 per capita for large plants. The cost may be appreciably higher for plants with an average capacity of less than 5 m.g.d. The usual range of construction costs of complete treatment plants employing the activated-sludge process has been given in Chap. II.

Operating charges are primarily for power, labor and supplies. Power is employed principally for compression of air used in the aeration

TABLE 113.—DESIGN CHARACTERISTICS OF TYPICAL ACTIVATED-SLUDGE PLANTS EMPLOYING DIFFUSED AIR

	Charlotte, N. C., Irwin Creek plant	Chicago, Ill., No. Side plant	Elyria, Ohio	Gastonia, N. C.	Houston, Tex., No. Side plant	Indianapolis, Ind.	Milwaukee, Wis.	New York, N. Y., Wards Island plant
Date of construction	1928	1928	1928	1921	1917, 1929	1924	1925	1931
Population provided for	50,000	800,000	36,000	....	....	360,000	575,000	1,230,000
Average sewage flow, m.g.d	6.0	175	3.6	1.0	12.5	50	85.2	180
Character of sewage	Separate	Combined	Combined	Separate	Combined	Combined	Combined	Combined
Preliminary treatment:								
Grit chambers	None	Yes	Yes	None	Yes	Yes	Yes	Yes
Racks, clear opening, in	3/8	1	1½	¼	1	Yes	Yes	Yes
Fine screens, type	....	....	....	....	None	Link-Belt	Link-Belt	None
Size of opening, in	....	....	....	....	....	0.033		
Sedimentation tanks, type	....	Dorr	Imhoff	....	None	Link-Belt	....	Dorr
Detention period, hr	....	0.5	0.5 or 1.0	....	....	0.8	....	1.0
Aeration tanks:								
Type	Long. furrow	Spiral flow	Spiral flow	Long. furrow	Ridge and furrow	Spiral flow	Ridge and furrow	Spiral flow
Number of units	6	36	4	4	5	6	24	16
Length, ft	200	420	80	66	386	238	236	345
Width, one channel, ft	15	16.2	10	8.5	18	20	22	22
Effective depth, ft	11	15	15	11	9.8	15	15	15.2
Number of passes per tank	1	2	2	1	1	4	2	4
Period of aeration, hr	5	6.3	4	3.3	3.7	5.1	6	5.6
Air consumption, cu. ft. per gal	....	0.8	....	1.8–3.6	....	1	1.5	0.8
Return sludge, per cent of sewage flow	....	20	20	....	15–25	20	25	25
Type of return-sludge pump	Centrifugal	Centrif.	....	Air-lift	Centrif.	Centrif.	Centrif.	Centrif.

Mixed liquor:								
Suspended solids, p.p.m.	....	....	....	....	....	....	3500	
Settling solids, per cent	....	....	....	....	20[2]	....	20(½ br.)	
Ratio of area of diffusers to area of tank[1]	1:6	1:9.4	1:14.3	1:5	1:7	1:13	1:4	1:13.7
Final-sedimentation tanks:								
Type	Link-Belt	Dorr	Dorr	Link-Belt	Fidler	Hopper bottom	Dorr	Scraper conveyor
Number	6	30	2	4	4	12	11	32
Inside dimensions, ft.	52 × 15	77 × 77	45 × 45	30 × 8.5	65, octagonal	78 × 42	98 × 98	179 × 42.5
Depth at side wall, ft.	13	13	12	8	....	13	14	12.5
Detention period, hr.	1½	2	2	1	2.0	1.6	1.7	2
Flow of effluent, gal. daily per sq. ft.	1280	980	890	980	870	1380	905	740
Re-aeration tanks:								
Type	None	None	None	None	Ridge and furrow	None	None	None
Number	....	....	....	....	5			
Inside dimensions, ft.	....	....	....	....	386 × 9			
Effective depth, ft.	....	....	....	....	9.8			
Period of re-aeration, hr.	....	....	....	....	1			
Air consumption, cu. ft. per gal. of sewage	....	....	....	....	10			

[1] Below deflectors, if used. [2] In 1921.

TABLE 113.—DESIGN CHARACTERISTICS OF TYPICAL ACTIVATED-SLUDGE PLANTS EMPLOYING DIFFUSED AIR.—(*Continued*)

	Pasadena, Cal.	Peoria, Ill.	Pomona, Cal.	Providence, R. I.	San Antonio, Tex.	Toronto, Ont., No. Toronto plant	Waco, Tex.
Date of construction	1924	1931	1926	1931	1930	1929, 1933	1927
Population provided for	120,000	140,000	20,000	....	400,000	100,000	75,000
Average sewage flow, m.g.d.	7.7[2]	22	1.5	52	30	9	4.5
Character of sewage	Separate	Combined	Separate	....	Combined	Combined	
Preliminary treatment:							
Grit chambers	None	....	None	Yes	Yes	Yes	Yes
Racks, clear opening, in.	....	....	....	3/4	1/2	1	Yes
Fine screens, type	Dorr	....	....	......	....	None	None
Size of opening, in.	1/16 × 2						
Sedimentation tanks, type	....	....	Imhoff	Dorr	Dorr	Dorr	Plain
Detention period, hr.	....	1.5	0.8	0.75	0.5	2.0	0.7
Aeration tanks:							
Type	Ridge and furrow	Spiral flow	Spiral flow	Spiral flow	Spiral flow	Spiral flow	Spiral flow
Number of units	30	4	2	16	6	8	3
Length, ft.	67.5	189	100	115	150	163	98
Width, one channel, ft.	10	22.7	16	20	20	13	20
Effective depth, ft.	15	15	15	15	15	10.5	12.5
Number of passes per tank	1	3	2	3	3	2	2
Period of aeration, hr.	4.4[2]	10.5[3]	9.6	5	4	5.75	4.7
Air consumption, cu. ft. per gal.	1.6[2]	4.5 max.	1.0	....	1.5	1	
Return sludge, per cent of sewage flow	30–40[2]	20	20	25	20	20	
Type of return-sludge pump	Centrifugal	....	Centrifugal	....	Triplex	Centrifugal	Air-lift

Mixed liquor:							
Suspended solids, p.p.m.	...	...	...	...	...	3000	
Settling solids, per cent	23(½ hr.)[2]	...	...	...	...	20(½ hr.)	
Ratio of area of diffusers to area of tank[1]	1:6	...	1:10	...	3:20	1:10	1:13
Final-sedimentation tanks:							
Type	Fidler	...	Dorr	Dorr	Fidler	Fidler	Dorr
Number	5	4	2	5	4	5	3
Inside dimensions, ft	50 × 50	70 × 70	30 × 30	100 × 100	90, diam.	65 × 65	35, diam.
Depth at side wall, ft	12.4	15	12	10.8	15	11.2–15.5	12.5
Detention period, hr	2.5[2]	...	2.1	1.5	2	4.75	1.6
Flow of effluent, gal. daily per sq. ft	616[2]	600[3]	830	1040	1600	426	1560
Re-aeration tanks:							
Type	Ridge and furrow	None	Spiral-flow	None	Spiral-flow	None	None
Number	5	...	1	...	1		
Inside dimensions, ft	67.5 × 10	...	35 × 11.5	...	150 × 20		
Effective depth, ft	15	...	15	...	15		
Period of re-aeration, hr	2.0[2]	...	1.0				
Air consumption, cu. ft. per gal. of sewage	0.3[2]	...	0.15				

[1] Below deflectors, if used. [2] In 1928–1929. [3] For 11 m.g.d.

tanks and channels, with some additional power required for pumping return sludge and operating mechanical equipment in the final-sedimentation tanks. Power costs for large plants may range from $3 to $10 per mil. gal. of sewage treated. Such costs may be considerably greater for small plants treating less than 5 m.g.d. Including labor and supplies, the operating charges may be in the vicinity of twice the power costs. The cost of operating activated-sludge units seldom is separated from that of operating the entire plant for the treatment and disposal of sewage and sludge. Treatment requirements, unit air supply, care given to maintenance and laboratory control vary greatly among plants, so that average unit costs are of little value.

Bibliography

1. Ardern, Edward, and W. T. Lockett: *Jour.*, Soc. of Chem. Ind., 1914; **33**, 1122.
2. Ann. Rept., Milwaukee Sewerage Comm., 1915.
3. Steel, E. W.: *Eng. News-Rec.*, 1929; **102**, 761.
4. Kusch: *Gesundheits-Ingenieur*, 1929; **52**, 644.
5. Hatton, T. C.: *Civil Eng.*, 1930; **1**, 31.
6. Hurd, C. H.: *Eng. News-Rec.*, 1923; **91**, 258.
7. *Eng. News-Rec.*, 1929; **103**, 738.
8. *Eng. News-Rec.*, 1929; **103**, 1021.
9. Hurd, C. H.: *Sewage Works Jour.*, 1929; **1**, 578.
10. *Sewage Works Jour.*, 1929; **1**, 583.
11. Streander, P. B.: *Water Works and Sewage*, 1930; **77**, 339.
12. Rept. of Chief Engr., Sanitary District of Chicago, 1926.
13. Mohlman, F. W., and C. E. Wheeler: *Sewage Works Jour.*, 1930; **2**, 529.
14. Parsons, A. S., and H. Wilson: *Surveyor*, 1927; **72**, 490.
15. Roe, F. C.: *Sewage Works Jour.*, 1933; **5**, 813.
16. Filtros, Inc. "Activated Sludge Process of Sewage Treatment," 3d ed., 1930.
17. Holmstrom, A. B.: Paper before N. Y. State Sewage Works Assoc., Jan. 16, 1934.
18. Bushee, R. J., and S. I. Zack: *Eng. News-Rec.*, 1924; **93**, 823.
19. Ann. Rept., Milwaukee Sewerage Comm., 1922.
20. *Bull.* of Filtros, Inc., 1932.
21. McConnell, E. G.: *Public Works*, 1928; **59**, 268.
22. Fuller, G. W., and J. R. McClintock: "Solving Sewage Problems," 1926.
23. McMillan, H. L.: *Eng. News-Rec.*, 1923; **91**, 178.
24. Martin, A. J.: "The Activated Sludge Process," 1927.
25. Saunders, W. L.: "Compressed Air Data," 3d edition, 1927.
26. Rept. to N. Y. Bd. of Estimate and Apportionment on Methods of Sewage Treatment in Use in Other Cities, 1928
27. Steel, E. W.: *Eng. News-Rec.*, 1935; **115**, 222.

CHAPTER XXVII

UTILIZATION AND DISPOSAL OF SEWAGE SLUDGE

In most cases the solids removed in one way or another from the liquid sewage in sewage-treatment works constitute the most important by-product of the treatment processes. These solids include grit, screenings and sludge, the last being by far the largest in volume. The disposal of screenings and of grit has been taken up in Chaps. X and XI, respectively, and it has been pointed out that screenings are sometimes disposed of together with sludge.

Since the daily volume of sewage handled in treatment works of other than rural communities commonly runs into millions of gallons and the sewage contains substantial quantities of sludge-forming substances, the daily volume of sludge produced generally runs into thousands of gallons. The problem of dealing with such quantities of material is naturally one of considerable magnitude, especially when it is remembered: that the sludge is made up in considerable proportion of substances which are responsible for the offensive character of untreated sewage; that, except in the case of well-digested sludge, these substances are still capable of rapid decomposition in the state in which they are produced by the sewage-treatment process; and that only a small part of the sludge is solid matter.

The dewatering of sludge on drying beds, so as to prepare it for disposal without creating offense, will be taken up in Chap. XXVIII. The dewatering and drying of sludge by mechanical means will be considered in Chap. XXIX. The character of sewage sludge, how to dispose of the treated or untreated sludge as economically as possible, and its utilization as a fertilizer where circumstances justify it, are matters to be considered in this chapter.

The various means of sludge disposal are presented in the following outline.

Methods of Sludge Disposal

A. Disposal of wet sludge
- 1. Disposal in water
  - *a*. Dumping at sea
  - *b*. Dumping in rivers during flood
  - *c*. Dumping in lakes

2. Disposal upon land
   a. Flowing on land
   b. Covering in furrows and trenches
   c. Lagooning
   d. Use as fertilizer

B. Disposal of dewatered sludge
1. Disposal in water
   a. At sea
   b. In lakes
   c. In rivers
2. Disposal upon land
   a. For filling
   b. As fertilizer
3. Disposal by other less well-established methods, including incineration

C. Disposal of dried sludge
1. As fertilizer
2. As fertilizer base

In order to avoid repetition, this outline will not be followed in detail in subsequent sections of this chapter. The disposal of wet and partially dewatered sludge, for example, will be discussed jointly.

Quantity of Sludge Produced by Different Sewage-treatment Processes.—The quantities and characteristics of sludge produced by different processes of sewage treatment have been discussed in preceding chapters, together with a consideration of the factors influencing its production. For purposes of comparison and reference a summary of typical yields is given in Table 114.

Appreciable departures from these values are to be expected, as shown by referring back to the figures of actual experience previously recorded. A comparison of the actual sludge yield of the same sewage when subjected to three different methods of treatment may be obtained from experiments by Eddy and Fales (1) made at Worcester, Mass., on a large scale under practical working conditions.

Their results are summarized in Table 115.

Statistics of sludge quantities may be reported in several different ways. For purposes of comparison the quantities are generally expressed in terms of the sewage flow or the tributary population. Gallons of sludge produced per million gallons of sewage treated is a unit in common use by plant operators. Engineers use in addition cubic feet, cubic yards, pounds and tons per million gallons; cubic feet, cubic yards and pounds per capita or per 1000 population daily; and tons per capita or per 1000 population yearly. Sludge volumes may be compared also on the basis of 90 per cent water content.

Expressing results in terms of the quantity of sewage treated is generally less informative than basing them on the tributary population.

TABLE 114.—NOMINAL QUANTITIES AND CHARACTERISTICS OF SLUDGE PRODUCED BY DIFFERENT TREATMENT PROCESSES[1]

Treatment process	Normal quantity of sludge: Gal. per mil. gal. of sewage	Normal quantity of sludge: Tons per mil. gal. of sewage	Normal quantity of sludge: Cu. ft. per 1000 persons daily	Moisture, per cent	Specific gravity	Dry solids: Lb. per mil. gal. of sewage	Dry solids: Lb. per 1000 persons daily
Plain sedimentation:							
Undigested	2950	12.5	39.0	95	1.02	1250	125
Digested in separate tanks	1450	6.25	19.0	94	1.03	750	75
Digested in Imhoff tanks	860	3.75	11.5	90	1.04	750	75
Digested and dewatered on sand beds	......	0.94	5.7	60	.....	750	75
Digested and dewatered on vacuum filters	......	1.36	4.3	72.5	1.00	750	75
Trickling-filter humus tanks	745	3.17	9.9	92.5	1.025	476	48
Chemical precipitation	5120	22.0	68.5	92.5	1.03	3300	330
Dewatered on vacuum filters	......	6.0	19.3	72.5	.....	3300	330
Preliminary sedimentation and activated sludge:							
Undigested, as drawn from preliminary sedimentation tanks	6900	29.25	92.0	96	1.02	2340	234
Undigested and dewatered on vacuum filters	1480	5.85	20.0	80	0.95	2340	234
Digested in separate tanks	2700	11.67	36.0	94	1.03	1400	140
Digested and dewatered on sand beds	......	1.75	18.0	60	.....	1400	140
Digested and dewatered on vacuum filters	......	3.5	11.7	80	0.95	1400	140
Activated sludge:							
Wet sludge	19,400	75.0	258.0	98.5	1.005	2250	225
Dewatered on vacuum filters	......	5.62	19.0	80	0.95	2250	225
Dried by heat dryers	......	1.17	3.0	4	1.25	2250	225

[1] Based on a sewage flow of 100 gal. per capita daily and 300 p.p.m., or 0.25 lb. per capita daily, of suspended solids in sewage.

TABLE 115.—SLUDGE OBTAINED FROM WORCESTER SEWAGE BY DIFFERENT TANK TREATMENTS, 1902 AND 1903

	Tons of dry suspended matter per mil. gal. of sewage: Effluent	Sludge[1]	Total	Sludge, gal. per mil. gal. of sewage
Untreated sewage	1.247	.....	1.247	
Chemical precipitation	0.250	1.485	1.725	4872
Plain sedimentation	0.601	0.580	1.181	2544
Septic tank	0.840	0.161	1.001	548

[1] Including scum.

This fact is illustrated in Table 116, which gives the unit quantities of sludge removed by horizontal-flow sedimentation tanks in a Massachusetts city of about 12,000 population, producing practically no industrial wastes. The tanks were cleaned every three weeks. The difference between maximum and minimum unit quantities is 185 per cent of the average value, on the basis of sewage flow, and only 12 per cent when referred to the population.

TABLE 116.—QUANTITY OF SLUDGE PRODUCED AT A MASSACHUSETTS CITY, 1912

Period	Flow of sewage, gal. per capita daily	Quantity of sludge		
		Gal. per mil. gal. of sewage	Cu. yd. per mil. gal. of sewage	Cu. ft. per capita daily
Maximum, Oct. 4–Oct. 25	29.2	4271	21.2	0.0165
Minimum, May 10–May 31	112.6	908	4.5	0.0146
Average, Nov. 7, 1911–Nov. 22, 1912	64.1	1822	9.0	0.0162

Character of Sludge Produced by Different Sewage-treatment Processes.—As the moisture content of sludge is reduced, it changes its state from that of *watery sludge*, first to *sludge paste*, next to *sludge cake* and finally to *sludge grains*. Watery sludge contains so much water that the sludge flows by gravity and is pumped readily, cohesion of the particles being slight. The point at which the sludge will no longer flow varies for different sludges. Digested sludge from Imhoff tanks, for example, will flow when it contains 80 per cent moisture, while activated sludge of this water content is distinctly a paste. Sludge with a water content low enough to enable it to be handled with a shovel is called *spadable*. When the water content of Imhoff sludge is reduced to 50 or 60 per cent, it can be handled readily with a fork and is called *forkable*. The sludge at the North Toronto plant, resultant from the digestion in separate sludge-digestion tanks of solids removed from primary tanks together with excess activated sludge, at times has been so heavy as not to flow readily even though the moisture content is above 90 per cent. This sludge has been forkable and has been removed from the drying beds while still containing 80 per cent moisture. Sludge cake contains less water than is required to fill the natural pore spaces between the particles. The volume then no longer depends upon the water content. The sludge finally becomes granular when its moisture is reduced below 10 per cent, a requirement which generally must be met when the sludge is to be marketed as a commercial fertilizer.

From the standpoint of sludge treatment and disposal, factors other than volume and water content enter into the problem. Among them may be the appearance and putrescibility of the sludge, its digestibility, dewatering characteristics and fertilizing value. Appearance, putrescibility and dewatering characteristics, particularly air drying, must usually be considered when the sludge is to be disposed of without treatment, other than perhaps drainage to a spadable condition by running it on to beds of sand or other porous material. Digestibility is of importance if the sludge is to be digested, either alone or mixed with other sludges in two-story or separate digestion tanks. Dewatering characteristics, with special reference to centrifugalizing, filtration and heat drying, generally become of moment when the moisture content is to be reduced by mechanical means, particularly in connection with the preparation of the sludge, notably activated sludge, for commercial utilization of its fertilizing constituents.

The character of different sludges, as measured by appearance, putrescibility, digestibility and air-drying qualities, is discussed below. Their properties as regards mechanical dewatering and drying will be discussed in Chap. XXIX and their use as fertilizers will be considered in subsequent sections of the present chapter.

Activated sludge generally has a brown flocculent appearance. If the color is quite dark the sludge may be approaching a septic condition. If the color is lighter than usual there may have been underaeration with a tendency for the solids to settle slowly. The sludge, when in good condition, has an inoffensive earthy odor, but sometimes has a tendency to become septic rather rapidly and then has a disagreeable odor of putrefaction. It will digest readily alone or mixed with fresh sewage solids. If the undigested sludge is flowed over a sand bed to a depth greater 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 by evaporation.

Sludge from chemical-precipitation tanks is usually black in color, though its surface may be red if it contains much iron. 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. 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. When the sludge is spread on drying beds, the water will gradually drain away or evaporate, leaving after several weeks a stratum of slimy sludge of about the consistency of lard, containing 70 to 80 per cent of water.

Sludge from plain-sedimentation tanks is usually gray in color, slimy in consistency and, in most cases, possesses an extremely offensive odor.

TABLE 117.—COMPOSITION OF SEWAGE SLUDGE

Plant	Condition of sludge	Specific gravity	Moisture, per cent	Per cent, dry basis					Remarks
				Volatile solids	Total nitrogen as N	Phosphates as P_2O_5	Potash as K_2O	Ether-soluble matter	
Primary sludge:									
Brockton, Mass.	Undigested	.....	93.8	84.0	....	....	....	....	Annual Rept. of Sewer Comm., 1929
Cleveland, Ohio	Undigested	1.033	87.0	50.0	1.60	1.37	...	12.8	Experimental plant
Columbus, Ohio	Undigested	.....	87.2	39.8	....	....	...	....	Experimental plant
Fostoria, Ohio	Undigested	.....	94.9	74.3	....	....	...	....	Sept., 1930
Grand Rapids, Mich.	Digested and dried	.....	85.7	40.8	2.58	1.84	0.3	....	Annual Rept., Treat. Works Div., 1933–1934
Worcester, Mass.	Undigested	.....	....	51.0	3.05	1.71	...	....	Experimental plant
Chemical-precipitation sludge:									
New Britain, Conn.	....	1.025	95.0	73.5	3.10	....	...	30.3	Miles acid process—Jackson and Doman (3)
Worcester, Mass.	....	.....	....	47.3	2.77	0.47			
Septic-tank sludge:									
Akron, Ohio	Digested	1.070	84.0	43.8	0.96	....	...	6.5	Experimental plant
Baltimore, Md.	Digested	.....	79.2	73.8	2.64	....	...	9.0	Schaetzle (4)
Cleveland, Ohio	Digested	1.038	89.2	40.8	1.50	1.20	...	9.0	Experimental plant
Columbus, Ohio	Digested	1.081	83.4	26.8	1.34	....	...	6.8	Experimental plant
Lindsay, Cal.	Digested	.....	....	42.9	1.83	0.89	...	....	Lipman and Burgess (5)
Philadelphia, Pa.	Digested	1.048	86.9	49.0	1.35	....	...	7.3	Allen (6)
Reading, Pa.	Digested	.....	91.8	47.0	....	....	...	....	Allen (6)
Waterbury, Conn.	Digested	1.025	85.8	45.5	1.35	....	...	10.8	Allen (6)
Worcester, Mass.	Digested	.....	....	43.9	3.01	1.85	...	....	Experimental plant
Imhoff sludge:									
Allentown, Pa.	Digested	1.035	90.2	45.0	....	....	...	....	July–Sept., 1930
Alliance, Ohio	Digested	1.012	78.4	52.1	2.50	1.90	0.4	8.5	Wagenhals, Theriault and Hommon (7)
Atlanta, Ga. Peachtree Creek plant	Digested	1.016	92.2	47.3	2.30	1.60	0.5	13.2	Wagenhals, Theriault and Hommon (7)
Baltimore, Md.	Digested	1.017	92.3	62.7	0.75	0.58	0.2	....	Average for 9 years—Schaetzle (4)
Canton, Ohio	Digested	0.990	86.5	37.8	1.60	1.20	0.2	13.9	Wagenhals, Theriault and Hommon (7)
Cleveland, Ohio Westerly plant	Digested	1.032	91.3	50.4	2.17	1.05	...	8.4	Average for 5 years, 1925–1929
Columbus, Ohio	Digested	0.997	96.2	....	2.80	1.60	0.6	24.4	Wagenhals, Theriault and Hommon (7)
Fitchburg, Mass.	Digested	1.049	87.0	42.3	....	....	...	....	Annual Rept., Dept. of Pub. Works, 1929
Lexington, Ky.	Digested	1.014	92.0	51.7	3.00	2.00	0.8	9.2	Wagenhals, Theriault and Hommon (7)
Rochester, N. Y. Brighton plant	Digested	1.015	85.5	32.3	1.90	0.80	0.2	6.0	Wagenhals, Theriault and Hommon (7)

Irondequoit plant	Digested	1.031	87.4	47.6	2.20	0.80	0.6	8.0	Wagenhals, Theriault and Hommon (7)
Worcester, Mass.	Digested	1.038	93.2	45.2	2.73	....	...	....	Ann. Rept., Supt. of Sewers, 1929
Trickling-filter humus sludge:									
Brockton, Mass.	Undigested	.....	93.0	60.0	....	....	...	....	Annual Rept. of Sewer Comm., 1929
Cleveland, Ohio	Undigested	1.040	89.0	37.0	1.30	1.20	...	5.9	Experimental plant
Columbus, Ohio	Undigested	.....	90.2	27.9	....	....	...	....	Experimental plant
Fitchburg, Mass.	Undigested	1.021	93.5	44.9	....	....	...	....	Average for 14 years, 1915–28
Fostoria, Ohio	Undigested	.....	96.0	62.2	....	....	...	....	Sept., 1930
New Britain, Conn.	Undigested	1.030	93.8	48.5	2.88	....	...	22.8	Jackson and Doman (3)
Worcester, Mass.	Undigested	.....	....	51.0	2.97	....	...	....	Experimental plant
Primary and humus sludges:									
Baltimore, Md.	Digested	.....	65–70	50–65	1.80	0.50	0.2	9.8	After drying on sand beds
Fostoria, Ohio	Digested	.....	92.9	50.4	....	....	...	....	Sept.–Oct., 1930
Activated sludge:									
Chicago, Ill.									
Des Plaines plant	Undigested	.....	....	55.0	5.00	4.00	...	....	Pearse (8)
Cleveland, Ohio	Undigested	.....	97.5	49.0	3.21	1.73	0.3	6.8	Pratt and Gascoigne (9)
Houston, Tex.	Undigested	.....	....	44.7	3.00	2 10	0.4	5.0	Wagenhals, Theriault and Hommon (7)
Indianapolis, Ind.	Undigested	.....	97.0	67.1	6.09	3 16	...	8.9	After drying in heat dryers; Ann. Rept. of Sanitary District, 1931
Milwaukee, Wis.	Undigested	.....	98.5	69.3	5.78	....	..	....	Average for 1928
Milwaukee, Wis.	Digested	.....	95.5	57.1	3.46	....	...	5.1	Rudolfs and Heisig (10)
New Britain, Conn	Undigested	1.004	99.6	64.6	4.10	....	...	....	Jackson and Doman (3)
Pasadena, Cal.	Undigested	.....	....	65.0	5.00	2.40	0 3	...	Hatch (11)
Sherman, Tex.	Undigested	.....	....	76.6	5.80	2.80	0.6	10.4	Wagenhals, Theriault and Hommon (7)
Urbana, Ill.	Undigested	.....	....	....	5.10	3.00	...	....	Pearse (8)
Primary and activated sludges:									
Elyria, Ohio	Digested	1.040	92.7	42.0					
Peoria, Ill.	Undigested	.....	96.5	71.7	4.38	....	.	...	
Peoria, Ill.	Digested	.....	95.9	56.1					Ann. Rept. on Plant Operation, 1932
Peoria, Ill.	Digested and dried	.....	53.2	53.9	3.54	2 41			
San Antonio, Tex.	Undigested	.....	95.4	77.5	....	....	.	....	
San Antonio, Tex.	Digested	.....	95.0	60.3					Average for 1932
San Antonio, Tex.	Digested and dried	.....	57.5						
Springfield, Ill.	Undigested	.....	96.1	55.5	....	....	...	...	
Springfield, Ill.	Digested	.....	91.5	41.1					
Springfield, Ill.	Digested and dried	.....	54.1	36.1	2.07	1.49	...	4.6	Ann. Rept. on Plant Operation, 1931–1932
Toronto, Ont.									
No. Toronto plant	Undigested	1.013	96.1	58.2	....	....	...	...	Average for 1930
No. Toronto plant	Digested	1.019	93.9	42.4					

It will digest readily under suitable conditions of operation and can be dried by spreading it on porous beds, but it must be spread thinly in order to enable the water to percolate away rapidly.

Sludge from single-story septic tanks is black and, unless well digested by long storage, is offensive on account of the hydrogen sulfide and other gases it gives off. The sludge can be dried on porous beds, if spread out in thin layers, but objectionable odors are to be expected while it is draining, except when the sludge is well digested.

Digested sludge is dark brown to black in color and contains an exceptionally large quantity of gas. When thoroughly digested it is not offensive, its odor being relatively faint and like that of hot tar, burnt rubber or sealing wax. When drawn off on porous beds in layers 6 to 10 in. deep, the solids first are 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 to the bed. As the sludge dries the gases escape, leaving it more or less spongy and with an odor resembling that of garden loam.

Trickling-filter humus is brownish, flocculent and relatively inoffensive when fresh. It generally undergoes decomposition more slowly than other undigested sludges, but when it contains many worms it may become offensive quickly. It is readily digested, but when this is not done it is like activated sludge in being difficult to dry on porous beds.

At Providence, R. I., the sludge resulting from sedimentation of chlorinated sewage did not possess a particularly offensive odor when observed during the summer of 1912. Similarly the sludge produced by the septic process at Birmingham, England, is said to produce little odor because of the presence of copper compounds. At New Haven, on the other hand, Winslow and Mohlman (2) ascribed the offensive condition of Imhoff sludge, obtained from an experimental plant on the Boulevard sewer, to the antiseptic action of copper wastes.

Composition of Sewage Sludge.—The analysis of sewage sludge has been discussed in Chap. IV. As there shown, the characteristics of sewage sludge may be measured by a number of different tests, selection of which depends upon the information desired. Frequent reference has been made in previous chapters to these tests that are of general interest in the management of sewage works. Tests that are valuable in connection with the disposal of sewage sludge are enumerated below.

Of evident significance in the disposal of sewage sludge are the tests for specific gravity, moisture and volatile and fixed solids, together with such qualitative observations as color, consistency and odor. Where sludge is to be discharged into water, the B.O.D. test yields valuable information. Other tests are of importance when sludge is to be treated prior to disposal. Among them may be mentioned measurements of hydrogen ion concentration, nitrogen in different forms, cellulose, fats and composition of sludge gases. Tests for fertilizer constituents

TABLE 118.—MINERAL ANALYSES OF SEWAGE SLUDGE

Plant	Type of tank	Results in per cent of sludge dried at 100°C.								Total fixed solids
		SiO_2 and HCl-insoluble residue	Al_2O_3 and FeO_3	P_2O_5	SO_3	CaO	MgO	K_2O	Na_2O	
Alliance, Ohio....	Imhoff	35.8	8.2	1.9	0.9	1.8	0.3	0.4	2.0	51.3
Atlanta Ga......	Imhoff	26.8	18.5	1.6	1.4	3.6	0.8	0.5	1.7	54.9
Baltimore, Md...	Imhoff	22.6	6.4	0.8	1.0	2.4	0.3	0.2	1.9	35.6
Baltimore, Md...	Separate digestion	17.7	3.4	1.0	2.3	1.7	0.1	0.5	0.4	27.1
Canton, Ohio....	Imhoff	44.3	11.0	1.2	1.5	3.9	0.3	0.2	1.7	64.1
Columbus, Ohio..	Imhoff	22.1	4.2	1.6	2.2	4.5	0.5	0.6	0.8	36.5
Fitchburg, Mass..	Imhoff	47.8	9.0	1.6	0.6	0.8	0.3	0.6	0.5	61.2
Houston, Tex....	Activated sludge	42.4	6.5	2.2	0.3	1.3	0.4	0.4	0.6	54.1
Lexington, Ky....	Imhoff	31.1	6.1	2.0	1.0	5.0	0.4	0.8	0.7	47.1
Rochester, N. Y. Brighton plant.	Imhoff	58.1	5.2	0.8	0.6	2.7	0.5	0.2	2.0	70.1
Irondequoit plant........	Imhoff	40.8	3.6	0.8	1.0	2.5	0.3	0.6	1.2	50.8
Sherman, Tex....	Activated sludge	9.3	2.9	2.8	1.0	2.5	0.2	0.6	3.3	22.6

TABLE 119.—MINERAL ANALYSES OF DRIED SLUDGES FROM CHEMICAL PRECIPITATION, PLAIN SEDIMENTATION AND SEPTIC-TANK TREATMENT AT WORCESTER, MASS.

	Chemical precipitation, per cent	Plain sedimentation, per cent	Septic-tank treatment, per cent
Fixed solids......................	52.74	48.96	56.06
Silica (SiO_2)....................	25.46	28.59	20.41
Iron sulfide (FeS)................	0.02	0.57	16.58
Iron, not as sulfide................	5.80	2.45	2.98
Sulfur, not as sulfide..............	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
Carbon (C)......................	28.60	31.26	23.95
Hydrogen (H)....................	4.21	4.46	3.64
Nitrogen (N)....................	2.77	3.05	3.01

finally may be added, when the sludge is to be used for agricultural purposes. Mineral analysis of sludge is sometimes undertaken, but yields information of practical value only under special circumstances and for purposes not ordinarily encountered in sewage-works practice. The determination of grease is sometimes made, to estimate its recovery value.

The composition of the sludge produced by different treatment processes in a number of American municipalities is shown in Table 117. Mineral analyses of some of these sludges, as reported by Wagenhals, Theriault and Hommon (7), are recorded in Table 118.

Table 119 gives a mineral analysis of the sludge produced at the chemical-precipitation plant in Worcester, Mass., together with mineral analyses of sludges produced during experiments on plain sedimentation and septic-tank treatment.

UTILIZATION OF SLUDGE

Economic Values in Sludge.—The constituents of sewage sludge that have commercial value are: fertilizing ingredients, including nitrogen, phosphorus and potassium compounds, together with humus; fats, including those combined as soaps; fuel values, including the calorific power not only of the combustible gases liberated during the digestion of sewage sludge but also of the sludge itself when burned or subjected to destructive distillation. The fertilizing value and fat content of sewage sludge will be discussed in this chapter; the value of the gases of decomposition has been noted in Chap. XIV, while the fuel value of sludge obtained by burning or destructive distillation is given on page 700 of this chapter.

Fertilizing Ingredients Entering Sludge from Sewage.—A comparison of Table 117 with Tables 29 to 33 will show that only a relatively small proportion of the nitrogen content of sewage enters the sludge produced by different sewage-treatment methods, the remainder very largely passing off in the effluent. About half the nitrogen in sewage is present in readily soluble form and, therefore, will not find its way in appreciable quantities into the sludge, unless physically absorbed or assimilated by the organisms living in the sludge. To a large extent, therefore, soluble nitrogen is lost in the effluent. In addition, some insoluble nitrogen will go into solution, as a result of the activities of living organisms, and both soluble and insoluble nitrogen may be lost to the atmosphere as nitrogen gas, ammonia or volatile amines.

For purposes of comparison and reference a summary of typical recoveries of nitrogen in sewage sludge is presented in Table 120.

Substantial departures from these values are to be expected, as shown by referring back to the figures of actual experience.

TABLE 120.—NOMINAL QUANTITIES OF NITROGEN IN SEWAGE SLUDGE PRODUCED BY DIFFERENT TREATMENT PROCESSES

	Treatment process							
	Activated sludge		Chemical precipitation	Plain sedimentation	Septic tanks	Imhoff or separate digestion tanks	Trickling-filter humus tanks	
	Undigested sludge	Digested sludge					Undigested sludge	Digested sludge
Dry solids, lb. per 1000 persons daily[1]	225	150	330	125	81	75	48	23
Nitrogen content, per cent	4.5	3	1.5	3	2	2	4.5	3
Lb. per 1000 persons daily	10	5	5	4	1.6	1.5	2.2	0.7
Per cent of sewage nitrogen in sludge[2]	30	15	15	12	5	4	7	2

[1] See Table 114.
[2] On the basis of 15 gm. per capita daily or 33 lb. per 1000 persons daily.

The nitrogen balance in the activated-sludge process has been discussed in Chap. XXIV. Experimental results obtained by Buswell and Neave (12) at Champaign, Ill., and by Pearse and Mohlman (13) at the Stockyards testing station in Chicago are summarized in Table 121. Attention has previously been called to the fact that the Stockyards station was operated with greater unit quantities of air than the Champaign plant, in order to care for the strong packing-house sewage. This may explain the loss of nitrogen experienced.

TABLE 121.—RESULTS OF EXPERIMENTS ON NITROGEN BALANCE IN THE ACTIVATED-SLUDGE PROCESS

Test	Nitrogen, lb. per mil. gal.			Per cent loss (−) or gain (+)	Per cent of sewage nitrogen in sludge
	Influent	Effluent	Sludge		
Chicago, Ill.:					
Aug. 1 to Oct. 22, 1916	469	183	90	−41	19
Oct. 23, 1916 to Mar. 26, 1917	592	324	130	−23	22
Mar. 27 to Nov. 14, 1917	380	161	85	−35	22
Champaign, Ill.:					
May 3 to Dec. 28, 1921	329	247	88	+ 2	27

Digestion of activated sludge results in a loss of nitrogen, as is to be expected. Rudolfs and Heisig (10), experimenting at Milwaukee, recorded a loss of 32 per cent by weight of nitrogen introduced into the digestion unit, the reduction in solids being 28 per cent. Sludge from trickling-filter humus tanks is generally similar to activated sludge in its nitrogen relations.

Chemical-precipitation sludge, although, owing to the precipitation of finely divided solids, commonly including a larger percentage of the incoming nitrogen than the sludge produced by any other method of treatment, except the activated-sludge process, has a small percentage nitrogen content, because the sludge is diluted, so to speak, with the precipitant. The Miles acid process, however, yielded experimentally a sludge containing 3.3 to 3.6 per cent of nitrogen when applied to the strong Boston sewage (14), but only 1.6 to 3.0 per cent for the weaker sewage of New Haven, Conn. (2).

The nitrogen content of primary sludge seldom exceeds 4 per cent and generally lies somewhere near 2 or 3 per cent. These values are reduced by digestion. Rudolfs (15) reports an average nitrogen content of 3.62 per cent for the fresh solids at Plainfield, N. J., which is reduced by digestion to 2.70 per cent, a decrease of 25 per cent. His analysis of 18 digested sludges from 13 American plants yielded an average nitrogen content of 2.28 per cent. Generally speaking, the better digested the sludge the lower is its nitrogen content, owing to the driving-off of nitrogen during the process of digestion. Combining different sewage-treatment processes will generally yield additive results on a weight basis.

The phosphate and potash content of sewage sludge is, in general, of less economic significance than its nitrogen content. According to Brown (16),

> Nitrogen is the constituent most demanded in fertilizers, for it is the first element to become deficient in the soil. This is due to its ready conversion into ammonium compounds which are soluble and drain away. Potash and phosphorus occur more regularly in sufficient amounts than nitrogen, and are more stable in soils, consequently they do not need as frequent application or in as large amounts for average grain, fruit or vegetable crops. Thus the application of nitrogen in available form usually produces a more direct and favorable response than that of the other two elements. It has been computed that over 57 per cent of the money paid for fertilizers is paid for nitrogen and about 25 per cent for potash and phosphoric acid.

Another reason for the lower value of potash and phosphates in sewage sludge, however, is the almost negligible quantity of potash and relatively small quantity of phosphates commonly contained in it. As shown in Tables 117 and 118, the potash content as K_2O varies from about 0.2 to 0.8 per cent, with no marked superiority of any one of the sludges produced by different treatment methods. The phosphate content as P_2O_5 is appreciably greater, varying from about 0.5 to 4.0 per cent, in much the same manner as the nitrogen content of sludges resulting from different treatment processes.

Availability of Fertilizing Ingredients.—The analysis of sewage sludge for fertilizing ingredients may involve the determination of the availability of the nitrogen, phosphates and potash present, as well as their

TABLE 122.—ANALYTICAL RESULTS OF THE FERTILIZING VALUE OF SEWAGE SLUDGE

	Type of sludge			
	Activated	Fresh solids	Partly digested solids	Fairly well digested solids
Reference No.	(18)	(15)	(15)	(15)
Municipality	Milwaukee, Wis.[3]	Plainfield, N. J.[4]	Plainfield, N. J.[4]	Plainfield, N. J.[4]
Percentage composition				
Moisture	4.08			
Nitrogen:				
Total	5.42	4.81	3.98	3.55
Insoluble	5.12	4.14	3.61	3.41
Soluble	0.30	0.67	0.37	0.14
Active insoluble	4.41[1] 3.22[2]	2.98	1.89	1.57
Inactive insoluble	0.71[1] 1.90[2]	1.16	1.71	1.84
Availability of insoluble nitrogen	86.1[1] 62.9[2]	72.0	52.3	46.1
Phosphoric acid:				
Total	3.08			
Insoluble	0.65			
Available	2.43			
Ether extract	....	15.11	7.79	6.83
Crude fiber	....	6.01	5.55	5.50
Ash	....	25.70	37.90	42.62

[1] Neutral permanganate.

[2] Alkaline permanganate.

[3] Composite sample representing 145 carloads of Milorganite shipped Feb. 1 to Sept. 1, 1927.

[4] Sample shows high initial nitrogen content. Average content of Plainfield sludge is 3.62 per cent for fresh solids and 2.70 for digested sludge.

total quantities. Some information upon this phase of the problem of sludge values is presented in Table 122, availability being measured by the chemical tests commonly employed in fertilizer analysis. Commenting upon these procedures, Bartow and Hatfield (17) remark that they do not even approximate the conditions in the soil and are used because there has been no better method available.

A compilation of the data given by Lipman and Burgess (5) and the analyses made by Lipman (19) using activated sludge are given in Table 123.

TABLE 123.—PARTIAL COMPOSITION OF AIR-DRIED SLUDGES AND THEIR AVAILABLE NITROGEN COMPARED WITH THAT OF COMMERCIAL ORGANIC FERTILIZERS

	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	25.70	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
Activated sludge	75.00	25.00	6.2	0.000	2.9	15.8	2.9	6.5
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

Bartow and Hatfield (20) comment as follows:

These results show that the nitrogen in activated sludge is more easily nitrified, under the conditions of these experiments, than that in dried blood, high-grade tankage, fish guano, cottonseed meal and goat manure; but that it is not so available as that of septic and Imhoff tank sludges. The experiments with wheat in pot cultures using activated sludge, dried blood and other ammoniates, reported by us in 1916 and later in this thesis, confirm the above comparison between dried blood and activated sludge. On the other hand, we have obtained much better results with activated sludge than with Imhoff and septic tank sludges. [See Table 124.]

TABLE 124.—RESULTS OF EXPERIMENTS ON THE USE OF SLUDGE AS FERTILIZER, 1916

Fertilizer	Quantity used		Bushels of wheat per acre			Tons of straw per acre		
	Grams	Tons per acre	Yellow soil	White soil	Brown soil	Yellow soil	White soil	Brown soil
Check	0	0	10.7	17.0	30.6	1.1	1.5	3.0
Imhoff sludge	244	19.5	18.8	13.5	21.3	1.9	1.6	4.0
Septic-tank sludge	147	11.8	23.8	22.3	27.3	2.7	2.5	4.6
Activated sludge	49	3.9	33.8	27.4	41.3	3.2	3.3	5.1
Dried blood	15	1.2	15.6	17.0	41.4	1.1	1.6	5.0
Milwaukee activated sludge[1]	47.25	3.8	14.5	16.8	27.3	1.9	1.6	3.0

[1] Results not comparable with others because of late planting.

In addition to the fertilizing constituents of sewage sludge, its physical condition is an essential factor in its usefulness as a fertilizer. Thus Bach and Frank (21) have shown that the grease and fibrous material in fresh sludge render the soil fertilized with sludge impervious to rain. This is not desirable. Digested sludge, on the other hand, is not so fibrous and contains less fat, which, furthermore, is finely divided and uniformly distributed through the sludge, leaving it porous. Bartow and Hatfield (17) concluded from crop-culture experiments that the fats in activated sludge, too, are finely divided and uniformly distributed, so that extraction of grease in preparing activated sludge for fertilizer use is unnecessary.

In Bartow and Hatfield's experiments activated sludge that had been acidified, dried and extracted gave much better results than untreated sludge. These workers expressed the opinion, however, that this was due to a beneficial change in the nature of the sludge brought about by the acid treatment, rather than to the removal of the fats.

Of further importance in considering the fertilizing values in sludge are the nature of the products of decomposition of the fertilizing constituents, the availability of the constituents with respect to the period of growth and the presence of organic matter which will support a favorable bacterial flora. These factors are gaged best by crop culture. In America, important experiments of this kind have been conducted, especially since the introduction of the activated-sludge process, by

Bartow and Hatfield (17) (20) at Urbana, Ill.; Nasmith and McKay (22) at Toronto, Ont.; Brown (16), also at Toronto; Hatton, Kadish and their coworkers (18) (23) at Milwaukee, Wis.; Pearse and others (24) at Chicago, Ill.; Jareo and Domogalla (25) at Madison, Wis.; and Keefer and Armeling (26) at Baltimore, Md.

Bartow and Hatfield's experiments (17) were conducted on pot and plot cultures during the years 1915–1917. Several types of sludge were employed, but chiefly activated sludge. Their observations are summarized by them as follows:

In the pot culture experiments of 1915, activated sludge was found to be superior to dried blood, gluten meal, and inorganic nitrogenous fertilizer, when applied to sand. In garden experiments of the same year, activated sludge produced a still larger increase in the growth of lettuce and radishes.

Pot experiments using three types of soil fertilized with septic tank sludge, Imhoff sludge, activated sludge and dried blood proved that sewage sludge has a fertilizer value and that activated sludge nitrogen again gave better results than dried blood nitrogen. Experiments on another type of soil showed that 1.5 tons of activated sludge gave the best result in growing sweet corn. Garden experiments in 1916 produced an increase of 71 per cent in weight of cucumbers due to activated sludge.

Sand cultures in 1917 showed that, in sand, applications of dried blood greater than one-half to three quarters of a ton were toxic to wheat. Increasing the application of activated sludge on gray silt loam did not prove toxic to the growth of foliage, though in applications greater than one to two tons, it decreased the yield of grain.

The application of sewage sludges the second year to yellow silt loam did not prove toxic but did not give as large a yield as was obtained the year before. The addition of phosphorus to the white silt loam proved that phosphorus was the limiting element in this series; activated sludge being the only fertilizer which produced a better yield than the check pot.

Garden experiments in 1917 proved that both wet and dried activated sludges were valuable as fertilizers, the wet sludge giving better results.

Nitrogen in activated sludge is present largely in the form of nucleo[1] protein nitrogen and its hydrolytic products, which have been shown to be beneficial to plant growth. Pot culture experiments show that sodium nucleinate[1] is readily available to the plant and that uric acid, though it is at first toxic, in time is nitrified or decomposed so that its nitrogen is also available. Egg albumin was highly toxic to young wheat plants, showing that the intermediate hydrolytic products of albumin are toxic. Activated sludge which had been acidified gave much better results than unacidified sludge. This is probably due to some hydrolysis taking place in drying the acidified sludge.

Good results are obtained with dried blood if the application is in small enough amounts that the concentration of toxic substances is low.

[1] *Nuclein* is a constituent of the nuclei of all cells. It contains phosphorus, nitrogen and sulfur.

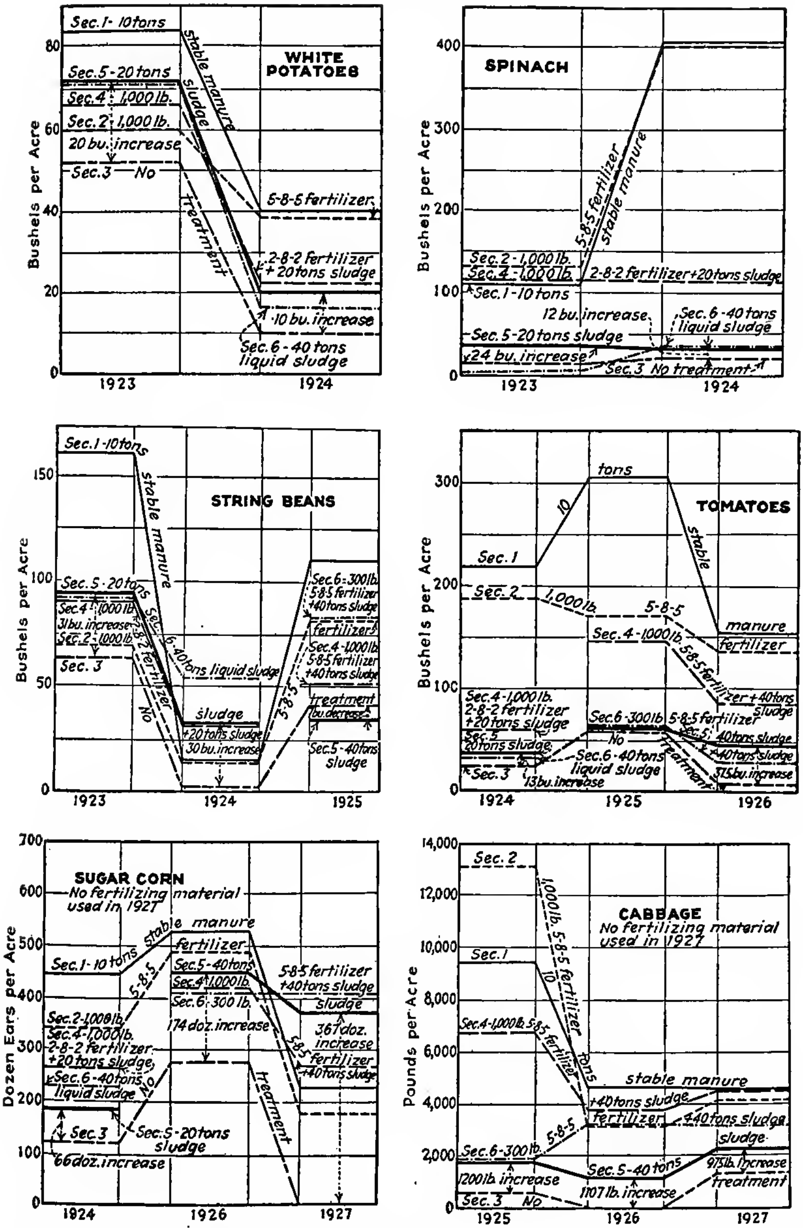


FIG. 195 —Experimental results of fertilizing six kinds of crops with sludge from Baltimore sewage-treatment plant.[1]

[1] All sludge was air-dried on open beds, except where liquid sludge is specified. All quantities of fertilizing material are given in tons per acre.

The work at Baltimore dealt with sludge from settling tanks preceding and following trickling filters, the sludge being more or less digested in separate digestion tanks. Less than 2 per cent of the sludge came from the humus tanks. The air-dried sludge contained 65 to 70 per cent moisture, 50 to 65 per cent volatile matter and 5 to 6 per cent sand. A typical analysis of the sludge shows 1.8 per cent N, 0.5 per cent P_2O_5, 0.2 per cent K_2O and 9.8 per cent fats. Experiments were carried out on plot cultures from 1923 to 1926. The results obtained are shown graphically in Fig. 195 (26). The customary method of indicating the fertilizing constituents of the mixture employed is used in these diagrams. For example a 5-8-5 fertilizer contains 5 parts of ammonia, 8 parts of phosphoric acid as P_2O_5 and 5 parts of potash as K_2O. Keefer and Armeling draw the following conclusions from the Baltimore studies:

1. Neither wet nor air-dried sludge was detrimental to the land. On the other hand, they contributed fertilizing substances to the soil and increased the yield of the six crops planted.
2. Air-dried sludge, when applied in amounts of 20 tons per acre, gave better results than did 40 tons of wet sludge per acre except when used to fertilize sugar corn. Such was to be expected, since, volume for volume, the former material contained from three to five times more nitrogen as ammonia than the wet sludge.
3. Spinach and tomatoes responded least to the use of air-dried sludge.
4. Of the six crops planted, white potatoes, cabbage and sugar corn thrived best. The increase in yield of sugar corn was particularly noticeable.
5. The results obtained with corn and cabbage indicated that sludge continued to give plant food to the soil during the second year after its application. No corresponding observations were made for the four other crops.

Fats in Sewage Sludge.—The quantity of ether-soluble matter in sewage and sewage sludge depends greatly upon the nature of the community and its industries. Different processes of sewage treatment, furthermore, include somewhat different proportions of fats in the sludge, as may be seen from Table 117. The greatest inclusion of fats in the sludge is apparently associated with the precipitation of the sewage solids by an acid such as H_2SO_4 or by SO_2 gas which hydrolyzes as H_2SO_3. Thus Pearse (27) reports the following removal of ether-soluble matter from sewage by different sedimentation processes:

	Imhoff tank	Dortmund tank	Chemical precipitation	Acid precipitation
Ether-soluble matter in influent, p.p.m.	157	157	157	135
Removal, per cent	55	47	66	69
Fat retention in sludge and scum, lb. per mil. gal.	718[1]	618	866	

[1] Not digested.

Higher removal of fats by acid precipitation is reported by Dorr and Weston (28) in experiments at Boston, the ether-soluble matter in the sewage representing 303 lb. per mil. gal. and in the sludge 430 lb. per mil. gal. This increase is attributed to the decomposition of soaps by the acid. At New Haven, Conn., Winslow and Mohlman (2) record a removal of approximately 100 per cent.

Rudolfs (15) has shown that the ether-soluble matter in sewage sludge is greatly reduced by digestion. For Plainfield, N. J., sewage sludge he gives percentage values of 15.11 for fresh solids, 7.79 for partly digested solids and 6.83 for fairly well digested solids. The reduction in the weight of grease remaining in the sludge per million gallons of sewage treated is naturally even greater than represented by these figures. Neave and Buswell (29) have shown that the destruction of grease is greatest during acid digestion but remains appreciable also during alkaline digestion of fresh sewage solids. For activated sludge, Rudolfs and Heisig (10) record a percentage grease content of 6.29 in the activated sludge employed in their digestion experiments at Milwaukee and of 5.09 in the digested material. This difference represents, for

	East St. sewer			Boulevard sewer, grease analysis
	Analysis of dried degreased sludge	Grease analysis	Analysis of dried sludge	
	Percentage composition			
Moisture	.....	.....	28	
Tankage[1]	.....	....	46	
Nitrogen as N	3.91			
Phosphoric acid as P_2O_5	0.96			
Ash	51.88			
Grease	.....	....	24	
Moisture and volatile matter	.....	11.0	..	0.5
Unsaponifiable material	.....	21.1	..	15.7
Free fatty acids, by weight	.....	40.2	..	41.5
Rosin	.....	14.4		
Actual free fatty acid	.....	25.8		
Neutral grease	.....	22.3		
Insoluble and metallic soap	.....	3.3	..	1.3

[1] "Tankage" is used here to mean the dried material in the sludge, other than grease.

the digested sludge, a reduction of 36 per cent in the weight of grease in the fresh sludge.

The composition of grease extracted from sewage sludge was studied by Winslow and Mohlman (2) at New Haven. Miles acid sludge, analyzed for these workers by Wells, yielded the results as shown in the table on page 685.

Monetary Value of Fertilizing Ingredients and Grease in Sewage Sludge.—The advisability of preparing sewage sludge for the market depends upon whether or not the fertilizer and grease obtainable possess sufficient value to pay the cost of recovery, including maintenance of plant, interest and depreciation on equipment and the cost of marketing the products. Fluctuations in market prices, governed by the law of supply and demand, also require consideration before a large investment is made in a recovery plant.

The monetary value of organic nitrogen fertilizers is commonly given in terms of nitrogen, an average figure being 20 cents a pound. The phosphate content of fertilizers as P_2O_5 is commonly valued at 3.5 cents a pound and the investigations at New Haven of the value of sewage grease led Winslow and Mohlman (2) to the adoption of a value of 5 cents a pound.

In utilizing sludge, the full value of the fertilizing ingredients and grease cannot be made available. There will be losses, especially in the recovery of grease, which will tend to reduce the return actually derived from the sludge below the theoretical return. A return from the recovery of grease can be expected only in exceptional cases.

The fertilizing value of sewage sludge was summed up as follows by the Metropolitan Sewerage Commission of New York in 1914 (30):

> In a general way it may be said that under favorable conditions as to transportation, a sludge containing 50 per cent moisture, whose dried material contains 3 per cent of ammonia and less than 10 per cent grease, might be further dried, ground and sold as a filler for fertilizer with some slight profit in the case of large works; but that no other than an occasional and uncertain offset to a part of the cost of operation can be looked for, even under favorable circumstances, from the sale of sludge in the form of crude cake or containing over 30 or 35 per cent of moisture.

Experience during the years that have elapsed since this statement was made does not seem to have changed the ideas of engineers materially, although the introduction of the activated-sludge process has greatly stimulated interest and practice in securing monetary returns from sewage sludge.

With reference to grease Winslow and Mohlman (2) state as follows:

> The usual limit for unsaponifiable matter in grease to be used for soap making is about 5 per cent, and unless grease containing 10–20 per cent of

material of this kind could be economically distilled it could be used only as wool grease, which is worth half as much as garbage grease or 5–6 cents a pound according to the high prices of 1918. Samples of sludge obtained from the New Haven sewage were submitted to Colgate and Company and the Cobwell Corporation of New York and to Swift and Company and Armour and Company of Chicago and the chemists of all these concerns, after extracting the grease and studying it, were of the opinion that in its crude state the material was of practically no value to a soap-maker. If such grease is to be utilized it must be first freed from its impurities by distillation.

Present Status of Sludge Utilization in America.—The marketing, on a commercial scale, of sewage sludge as a fertilizer as yet has been carried on by only a few sewage-treatment works, notably Milwaukee, Wis., and Pasadena, Cal. A larger number of plants, however, have been successful in creating a local demand for partially or completely dried sludge and is securing some revenue from this source.

In the country as a whole, the agricultural use of sewage sludge has not been developed extensively. This is clearly shown in Table 125, which presents a compilation of available data on sludge use. There is at present no municipal installation in which grease is being recovered from sewage sludge.

The experience of a number of different plants, both large and small, will illustrate the status of sludge utilization.

Imhoff Sludge.—Keefer and Armeling (26) have summarized as follows the experience with sludge utilization at Baltimore, Md.:

For the first few years after 1912, when the Baltimore sewage-works was put in operation, the disposal of the sludge was a question of little importance. The sewage flow was small, and there was sufficient capacity in the digestion tanks to retain the sludge for many months. Some of the material in both the wet and the dry forms was sold to neighboring farmers. The remainder, a small amount, was used for filling low-lying ground at the plant. By 1915 it became apparent that some definite means should be adopted to dispose of the sludge. A contract was therefore made with a reduction company, which erected a plant at the sewage works. From 1916 to the early part of 1923 it heat-dried a portion of the sludge and sold it as a filler for fertilizer (38). Since 1923 the principal method of disposal has been to give it to neighboring farmers. They haul it to their fields in trucks and wagons, which are loaded by means of a 5-ton locomotive crane. Efforts are continually being made to stimulate the farmer's interest in the use of this material.

For the five-year period from 1923 to 1927, 100,000 cu. yd. of sludge was removed from the drying beds, of which amount 45,000 cu. yd. was given away for fertilizing purposes. During the past three or four years considerable quantities of sludge have been used on the lawns at the sewage works. About 10 tons (9 cu. yd. per acre) is distributed over the ground in the early

TABLE 125.—USE OF SEWAGE SLUDGE IN AMERICAN MUNICIPALITIES

Municipality	Year of record	Type of plant	Extent of sludge drying	Annual production of dried sludge	Method of utilization	Revenue	Reference
Alliance, Ohio	1921	Imhoff tank, trickling filter	Dried on sludge beds	....	Given to farmers	....	(7)
Baltimore, Md	1923–27	Plain sedimentation, separate digestion	Dried on sludge beds	100,000 cu. yd.	45,000 cu. yd. given for fertilizing purposes	....	(26)
Canton, Ohio	1930	Imhoff tank, trickling filter	None	....	Wet digested sludge flowed on to municipal farm	....	(31)
Dayton, Ohio	1931	Imhoff tank	Dried on sludge beds to 63 % moisture Dried to 10 % moisture	11,000 cu. yd. 475 tons[1]	Sold to consumers	$4793.35[1]	(36)
Grand Rapids, Mich	1932	Plain sedimentation, separate digestion	Ground and dried	495 tons	Sold to consumers	$4331.16	(37)
Lexington, Ky	1921	Imhoff tank, trickling filter	Dried on sludge beds	....	Given to farmers	....	(7)
Marion, Ohio	1929	Imhoff tank, trickling filter	Dried to 5 % moisture Dried on beds to 65–70 % moisture	....	Sold to consumers	$23 a ton $2 a cu. yd.	(32)
Milwaukee, Wis	1930	Activated sludge	Dried to 4 % moisture	36,000 tons	Sold to fertilizer makers and consumers	$575,000	
Pasadena, Cal	1930	Activated sludge	Dried to 4 % moisture	2,879 tons	Sold to consumers	$63,000	(33)
Rochester, N. Y	1929	Imhoff tank	Dried on beds to 57 % moisture	6,904 cu. yd.	1,281 loads sold	$640.50	(34)
Schenectady, N.Y	1930	Imhoff tank, trickling filter	Dried on sludge beds	3,320 cu. yd.	Sold to farmers	25¢ a load	(35)

[1] Total sales June, 1931, to May 10, 1932; sludge not artificially dried was placed in sludge dump.

winter. In the spring that which remains is disintegrated by hauling a lapboard drag over the lawns. Applied in this quantity, the total output of air-dried sludge, amounting to about 20,000 cu. yd. annually, would fertilize 2200 acres. This method of treating lawns has been so successful that a considerable quantity of the sludge is used in the city parks and at the water-works filtration plant. Since Jan. 1, 1928, more than 8000 cu. yd. has been hauled a distance of 8 miles at a total cost of 95 cents a cubic yard to two city parks, where it has been spread over denuded ground to take the place of top soil.

Tatlock (6) has reported upon the preparation and sale of Imhoff sludge for commercial fertilizer at Dayton, Ohio.

Some of the sludge is dried to 10 to 15 per cent moisture on glass-covered beds. A hammer mill, equipped with a cyclone blower and sacker, was purchased for grinding and bagging the dried sludge. With the aid of second-hand equipment, a drying plant was built with a capacity of 1–1½ tons an hour, taking sludge at 40 to 45 per cent moisture and turning it out at 8 to 10 per cent. The capital investment was $1069.78.

The retail price of the sludge is $1.00 per 100 lb., or $15.00 per ton in lots of 1000 lb. or more. The wholesale price to agents is $8.50 per ton, subject to 5 per cent discount in 30 days. The total sales to May 10, 1932, were $4793.35 for 475 tons. The cost of production was made up as follows: maintenance and supplies, $1789.50; labor, $1566.68; total, $3356.18. An additional cost of removing sludge from the beds of $471.60, if included, would bring the total operating cost to $3827.78.

Tatlock states that there have been large repeat orders from tobacco growers who found their 1931 crop, fertilized with Dayton sludge, to have the qualities looked for in a high-grade tobacco, resulting in its sale at a premium.

Experience at the Irondequoit treatment plant in Rochester, N. Y., with the utilization of Imhoff sludge for fertilizer after dewatering on drying beds is reported by Ryan (34) as follows:

The dried sludge has a ready sale, particularly during the fall and winter months. It is hauled for distances of fifteen miles. . . . The sludge is applied to the greens and fairways of local golf courses and as a top dressing for lawns. Nearby fruit growers purchase a large quantity each season and state that its use increases the yield of fruit, the sludge being spread only around the trees. During the winter months, truck gardeners haul the sludge and spread it on their fields. In the spring it is plowed under the surface. The gardeners claim it is an excellent fertilizer for corn, celery, cabbage and bean crops; in fact, for any vegetable which grows above ground.

Data on the revenue from sale of sludge at Rochester during 1929 are included in Table 125.

Browne (32) records his experience at Marion, Ohio, as follows:

Imhoff sludge is sold as it is removed from the drying beds or sludge dump, and from a pit in which the sludge weathers to something like leaf mold, at \$2.00 per cubic yard at the plant site, or \$4.00 per cubic yard delivered in Marion. When dried to about 5 per cent moisture on the drying beds, pulverized and sacked, it is sold at \$1.15 per 100 lb. or \$20 per ton in small lots, and correspondingly lower prices for larger lots to a minimum of \$14 per ton.

Several golf clubs have used the fertilizer on fair-ways and greens with considerable success, florists and truck gardeners have found it a valuable addition to existing garden soils, and it has been used by residents of Marion as a turf starter or for accelerating the growth of lawn grass with equal success. However, the market is limited pretty well to the immediate vicinity of Marion or those communities that have no sewage plants, as every sewage plant treating domestic sewage is a possible source of a low grade fertilizer for its own vicinity. Newspaper advertising and the selection of demonstration plots in locations that were easily observed by passers-by informed the public of the value of Marion sludge as a substance supplying both plant food and organic matter, so that at the present time repeat orders for sludge are frequent. However, attention is directed at present towards selling the entire output of pulverized sludge to fertilizer mixing plants as a filler for more concentrated fertilizers in place of sand or bean meal. Used as such, sludge supplies the extra bulk needed and also organic matter in place of inert sand.

At Schenectady, N. Y., Cohn has been successful in interesting farmers in the agricultural utilization of Imhoff sludge. In 1932 he made the following report (39):

The sale of sludge becomes more active from year to year. The past year is the ninth consecutive year in which all sludge produced has been disposed of to farmers and local householders. This year the storage piles in the fields were all cleaned up by the end of September, due to an unprecedented heavy demand starting in August. From that time on, the purchasers obtained sludge from the drying beds, being loaded by our crew when they were working there and loading themselves when the men were not assigned to cleaning.

It was possible to discontinue the use of the city team for several weeks during the middle of the season and a full month earlier than usual in the fall. During this time, sludge has been removed by the farmers at an important saving to the city in labor and team hire. In addition, the sale of sludge at rates of from 25 cents to 75 cents per load, depending on size, netted the city over \$200.

Sludge is being used successfully on all types of crops, on lawns and flower beds. The largest individual sale was 100 loads to a cemetery for grassing sandy soil. All customers still receive handbills, warning that sludge be kept from all crops that grow in or on the ground and go to the table uncooked.

The Canton, Ohio, sewage-treatment plant is located on a municipal farm comprising nearly 600 acres of rolling land, 200 acres of which are either wooded or too rough to permit of economical cultivation (31). The farm is operated as a part of the sewage-treatment program, for the express purpose of providing a means of disposing of the sewage sludge. The treatment plant is of the Imhoff-tank, trickling-filter type, serving a population of about 100,000 in 1930.

The wet digested sludge from the Imhoff tanks is pumped through a force main, terminating on the top of a knoll approximately 300 ft. above the surrounding territory. From this high point the sludge flows in open ditches to fields as much as two miles distant. The sludge is spread out over the land by overflows from the ditches. Experience has shown that best results are obtained if the acreage is fertilized every four or five years. The city farm, together with the farms of abutting property owners, who desire sludge to enrich their acreage, apparently provides adequate area for the disposal of all the wet sludge produced. The common crops raised are wheat, corn, oats, hay and potatoes.

Sludge from Separate Digestion Tanks.—The utilization of digested primary sludge at Grand Rapids, Mich., is described by Rumsey (37) as follows:

Sludge cake, ground, dried, screened and bagged, is sold under the name of "Rapidgro," as a low-grade organic fertilizer and soil builder. It was marketed this past season to the extent of about 990,000 lb. in 110-lb. bags, of which about 90 per cent was sold at retail to about 3000 Grand Rapids citizens.

In 1932, 56 tons were furnished the Park Department, 22 tons were experimentally used at the airport, and about 100 tons were used on our own property.

The production cost for the past year amounted to $1575.51, and was distributed as follows: payroll $842.99, bags $453.67, delivery books and postage $133.78, machinery and repairs $133.88, and miscellaneous expense $11.19. Receipts for the year amounted to $4331.16, and with an inventory of $3000 the net return to date is $5755.65.

Our price for 1931 and up to September 15, 1932, was 50 cents per bag of 110 lb. It was then changed to 65 cents per bag delivered, 55 cents per bag at the plant, $10.00 per ton in ton lots and $9.00 per ton in 5-ton lots or more.

We are now rebuilding the equipment in the fertilizer plant to double our output, bringing it up to about 1000 tons per season, which is approximately one third of the total sludge cake available per year.

Activated Sludge.—Whereas Imhoff and other sludges resulting from plain sedimentation have found only a local agricultural use, activated sludge is being marketed as a fertilizer on a truly commercial scale at Milwaukee, Wis., and Pasadena, Cal.

At Milwaukee, Hatton, Kadish and Heisig (40) report experience with the sale of dried activated sludge as a fertilizer as follows:

The approximate gross return for the Milorganite has been between $15.00 and $16.00 per ton of 2000 lb., which includes the phosphoric acid (P_2O_5) in the mixture. Milorganite contains from 6.5 to 7.0 per cent of Ammonia (NH_3), and from 2.25 to 2.75 per cent of available P_2O_5. The manufacturer buys it upon the basis of units of NH_3 and P_2O_5 it contains. The other customers buy it by the ton regardless of the ingredients.

In order to show the relative use of this material, the following sales analysis is here given:

User	Tonnage[1]	Percentage
Fertilizer manufacturer	10,080	80
Special markets		
Golf courses and florists	2,150	17
Local sales		
Lawns and gardens	370	3
Total	12,600	100

[1] Shipments Jan. 1 to Sept. 1, 1927.

The quantity and character of the Milorganite produced during 1928, 1929 and 1930 are given in Table 126. The average revenue from its sale during 1930 was $16 a ton.

TABLE 126.—MILORGANITE PRODUCED AT MILWAUKEE SEWAGE-TREATMENT PLANT

	1928	1929	1930
Tons a day, average	88.4	95.6	98.5
Tons a year	32,350	34,900	35,950
Moisture, per cent	2.97	3.97	3.87
Ash, per cent	30.53	33.47	32.89
Nitrogen as NH_3, per cent	6.83	6.29	6.46
Ether-soluble matter, per cent	6.08	6.08	6.46

During the latter part of 1930 certain industrial wastes, such as acid pickling liquors, were diverted from the Milwaukee sewers, because of their destructive effect upon the sewers. As a result, the proportion of nitrogen in the Milorganite during 1931 was somewhat greater than during the three preceding years.

At Pasadena, during the period from July 1 to Sept. 30, 1928, 645 tons of dried sludge were produced, an average of 7 tons a day (41). The revenue from the sale of the sludge as a fertilizer was $25.25 a ton. In 1930, 2879 tons of fertilizer were produced, an average of about 7.9 tons a day, for which the city received about $22.00 a ton.

Hygienic Considerations in Sludge Utilization and Disposal.—The disposal of sewage sludge and, more particularly, its use for fertilizing purposes raise the question of the danger of spreading typhoid and other diseases by this means. Tests by Kligler (42) showed that the typhoid bacillus may survive in solid feces for about 10 days and in liquid feces for about 6 days. In moist soil the time varied from 30 to 70 days, depending on the pH value of the soil, being shorter for acid soils than for alkaline ones. In dry soil the viability was reduced to 20 days and the time of survival in septic tanks varied from 5 to 14 days. Other intestinal bacteria were shown to be less resistant.

Some of Kligler's conclusions follow:

> It would appear, then, from these experiments that the gram-negative bacilli of the typhoid-dysentery group die out rapidly in septic material. The typhoid bacillus may survive for about five days, the Flexner type of dysentery about three days, while the Shiga bacillus succumbs most rapidly. If the alkalinity of the fluid is low, in other words, if the tank is not "ripe," the organisms may survive for a much longer period. The germicidal power of the effluent from a ripe tank is probably due both to the alkaline reaction and to the presence of the antagonistic products of metabolism.
>
> It would seem that the speed with which typhoid and dysentery bacilli die off in soil would depend on a number of factors. Chief among these are the moisture content and reaction of the soil. The nature and abundance of other flora might play a secondary part. The important fact is that in dry or acid (moist or dry) soils most of the pathogenic bacteria would die off within ten days.

Other investigators have reported that the typhoid bacillus survives for much longer periods in soil. Wolman (43) comments on this matter as follows:

> The literature dealing with the viability of typhoid fever bacilli and similar organisms in sewage is extensive but not altogether in complete agreement. Prof. C. W. Stiles of the Hygienic Laboratory of the U. S. Public Health Service has kindly supplied me with a very complete bibliography of experimental work upon the life of typhoid fever in sewage and in soil. This bibliography covers almost ninety references. As a result of the experiments noted in this bibliography, it is reported that *Bacillus typhosus* has been recovered in varying forms of human sewage in anywhere from 6 hr. to 365 days, and in various characters of soil in anywhere from several to 540 days. It is pertinent to comment upon these findings that all of the studies which report a survival of typhoid bacillus of more than one week

were carried out prior to 1912 or more than 11 years ago. Most of the determinations were performed by investigators more than 20 years ago. These facts are pointed out at this point, since they have considerable to do with the emphasis which should be placed upon such findings, because of the improvements in methods of differentiation of pathogenic organisms during the last decade or two. The long period of survival noted above may also be questioned when it is found that more recent experimental evidence shows that the life of such organisms as the typhoid fever bacillus, under the unsatisfactory influences usually found in digesting sludge, is well below those indicated by some of the earlier investigators.

A study of available data indicates, as would be expected, that, generally speaking, fresh solids are more dangerous than well digested sludge and wet solids more so than partially dried solids. Dry solids seem to be relatively safe and heat-dried solids entirely so. Hygienic considerations require, therefore, as in the case of sewage farming, that sewage sludge, except, perhaps, heat-dried sludge, should not be brought into contact with produce which is consumed without cooking.

Wolman (43), after reviewing the information on the hygienic aspects of sludge utilization, proposed the following regulations for the disposal of wet sludge to farmers for fertilizing purposes, at the Baltimore, Md., sewage works:

1. Only such sludge shall be delivered to farmers as has been undergoing digestion for at least ten days.
2. The wet sludge shall be carted in vehicles that are watertight.
3. Sludge shall be used on ground only before crops are planted. It shall not be sprinkled over or brought in direct contact with growing vegetables.
4. A permanent record shall be kept by Baltimore city of all persons obtaining wet digested sludge, date, location of farm and quantity of sludge taken.
5. The city reserves the right to discontinue at any time the distribution of wet sludge to any farmer.
6. Any person found violating any of the above regulations shall not be allowed to take wet sludge from the disposal plant or to obtain it in any other manner.

FINAL DISPOSAL OF SLUDGE

Generally speaking, there are two broad methods of final sludge disposal which correspond to the disposal of liquid sewage by dilution and irrigation. They are disposal in water and disposal on land. Dewatering may or may not precede final disposal.

Disposal in Water.—The disposal of the sludge of sea-coast towns by carrying it out to sea and dumping it in deep water is practiced by a number of cities in the British Isles, notably London, Manchester, Dublin, Southampton and Glasgow. In this country, the city of Providence,

R. I., and the sewerage district of the Passaic Valley, N. J., conform to this practice and plans for the Wards Island treatment works of New York City and for the plant at Elizabeth, N. J., to serve twelve municipalities call for its adoption. Treatment of the sludge prior to disposal is not essential, except for the purpose of reducing the volume to be handled.

At Providence, the sludge from chemical-precipitation tanks has been treated with lime and dewatered to a moisture content of about 75 per cent. The sludge cake is placed in a double-bottom scow which has a capacity of 850 cu. yd. and is towed to the disposal area about 14 miles from the plant. During 1928 the Providence plant produced 21 m.g. of wet sludge from 13 m.g.d. of sewage contributed by a population of 248,000. The filter presses delivered 20,668 tons of cake containing 5167 tons of dry solids.

According to the 1926 report of the Passaic Valley Sewerage Commissioners, the sludge settling in the hoppers of plain-sedimentation tanks at Newark, N. J., is discharged through 2000 ft. of 20-in. sludge main into sludge barges at Newark Bay front. The barges are then towed out to the disposal area at sea designated by the Federal government and located five miles southeast of Ambrose Lightship and eight miles southeast of Scotland Light. The report states:

> The amount of sludge discharged at sea averages 3000 tons per week in winter, spring and fall months, and 5000 tons per week during the hot weather.
>
> The work is done under yearly contract and at the present time is costing $0.34 per ton of sludge taken from the Commissioners' bulkhead at Newark Bay.

The location of the sludge-dumping grounds is shown in Fig. 196.

The Wards Island project of New York City contemplates the following arrangements for sludge disposal (44):

> Excess activated sludge . . . will be delivered by the pumps to four main sludge-storage tanks on the dock, each with a capacity of 45,000 cu. ft. These tanks will be of steel, of suspension coal-bunker type. Each tank will have provision for decanting the supernatant liquid. Besides the sludge tanks there will be two 5000-gal. tanks for receiving the grease and skimmings from the presedimentation tanks. All six tanks will be arranged to discharge by gravity into ocean-going sludge boats.
>
> The boats for sea disposal of sludge will be three in number (unless one is cut out on account of the fertilizer plant, in case that is built). Each boat will have a capacity of 1500 tons and will be 250 ft. long, 43½ ft. beam and have a molded depth of 16 ft., and draw 11 ft. of water when loaded. Each boat will have twin screws driven by two 600-hp. diesel engines.
>
> Assuming 90 per cent removal of suspended matter, there will be about 140 tons daily of dry solids for disposal. It is estimated that of this 60 tons

(dry basis) will be from presedimentation tanks. This will have about 95 per cent water content, making 1200 tons on a wet basis. As this will be low in organic nitrogen, it is considered best to send it to sea, regardless of whether or not the fertilizer plant is built. The sludge potentially available for fertilizer will thus amount to 80 tons a day. The report recommends that the fertilizer plant with a capacity of 50 tons a day be built—apparently meaning 50 tons of finished product after the addition of chemicals to the sludge.

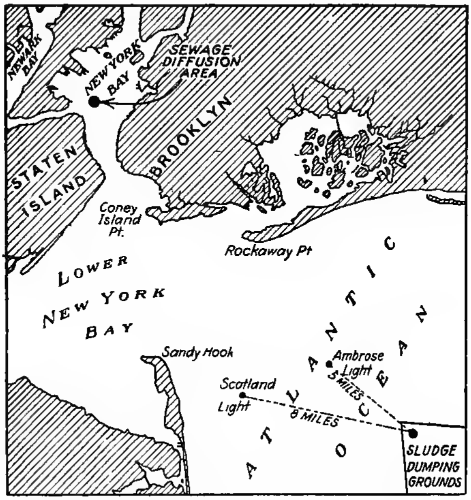


FIG. 196.—Location of sewage-diffusion area and sludge-dumping grounds of Passaic Valley sewerage system.

Potter (45) reports the method of sludge disposal proposed for the Joint Meeting plant at Elizabeth, N. J., to serve 12 municipalities, to be as follows:

Ultimate disposal of sludge at sea is to be adopted. Barge loads will be in the neighborhood of 3000 tons which, for the early years of operation, will represent about two weeks' sludge accumulation. Sludge-storage tanks will provide a capacity of about 150 per cent of a barge load to provide for a possible interruption of barging service and to assure the economy possible only with a full loading of the barges on each trip. The tanks will be provided with proper decantation valves and equipment.

At the East St. sewage-treatment plant in New Haven, Conn., sludge produced by plain sedimentation is stored in four steel tanks, similar to those planned for the Wards Island plant, before being barged to sea for disposal.

At Cleveland, Ohio, Imhoff sludge is pumped into Lake Erie, and several American communities discharge sludge into inland streams when they are in flood. Among the treatment plants following the latter practice may be mentioned the plant at Sherman, Tex., and the Peachtree Creek plant at Atlanta, Ga.

Disposal on Land.—Wet sludge is disposed of on land by flowing it on to the land, running it into trenches that are covered after being filled, or discharging it into lagoons. The latter may be used for temporary or permanent storage of sludge. Dried sludge, commonly press cake or air-dried sludge, may be used either for filling waste or low-lying areas or for fertilizer purposes. The agricultural utilization of dried sludge has been previously discussed.

Flowing and Trenching.—Disposal of sludge by flowing it on to the land or running it into trenches and covering it requires a relatively large disposal area. Use of these methods, sometimes called irrigation with sludge, is reported favorably from England, but in this country it is restricted more or less to an emergency measure. Disposal of undigested sludge in this manner is not regarded with favor, on account of the odors produced and the sanitary hazards involved, especially when crops are raised on the disposal area.

For San Marcos, Tex., Eggert and Cohen (46) report the satisfactory disposal of excess activated sludge by discharging it through an 8-in. cast-iron pipe on to a disposal area six acres in extent. The tributary population is about 6000 and the sewage flow about 0.35 m.g.d. The land is plowed in furrows through which the sludge flows. The requisite furrows receive sludge for one day, after which they lie idle until thoroughly dry. The surface crust is then broken by a single plowing down the center of the furrow and the sand is again put into service. Absence of odors is reported. Similar disposal of activated sludge in sandy fill during the summer season has been employed at Mamaroneck, N. Y. (47). At Atlanta, Ga., most of the Imhoff sludge is dried on prepared sand beds, but some is run on to fields and low-lying areas adjacent to the plant (7). Similar emergency disposal of sludge to relieve overloaded drying beds is practiced at Canton, Ohio.

Trenches about 3 ft. wide and 18 in. deep, which are covered after receiving sludge, have been employed at certain English disposal works. The area required has been found to vary from about ¼ to 1 acre or more per 1000 long tons of sludge. As the land once used in this way must lie idle for an appreciable length of time, a large tract is required. More recent English practice is given for Wolverhampton by Clifford (48) and for Blackburn by Eaton (49).

Lagooning.—Lagooning of wet sludge is employed widely in this country and abroad, for all types of sludge. A *lagoon* is a basin formed either by a natural depression, often a sand pit, clay pit or quarry, or by surrounding a tract of land with a dike of earth. Lagoons vary greatly in area and depth depending upon local conditions. The sludge is left to dry by evaporation and by such losses of water through the soil as may take place naturally. Drying may require 2 to 6 months and is frequently accompanied by offensive odors when undigested sludge is disposed of in this way. Digestion naturally takes place in the basins. Lagoons are particularly useful for emergency storage or disposal of sludge. In some cases they are worked until they are filled completely with sludge but in other cases the drained and digested material is removed from time to time and used to fill low-lying areas in the neighborhood. Winter lagoons may be employed for the storage of sludge from heated, separate sludge-digestion tanks, the winter's accumulation being removed to drying beds during the drying season. Lagoons are also used for the storage of sludge for discharge into streams when they reach flood stage. The progressive drying of fresh and digested sludges from plain-sedimentation tanks in lagoons is illustrated by the results obtained at the Philadelphia sewage testing station in 1909. These are shown in Table 127 (50).

TABLE 127.—RESULTS OF AIR-DRYING SLUDGE IN EARTH LAGOONS AT PHILADELPHIA TESTING STATION, 1909

Source of sludge	Time, days	Depth, inches	Quantity in lagoon, cu. yd.	Moisture in sludge, per cent	Rain-fall, in.	Sludge per acre, cu. yd.
Sedimentation tank 12; influent, screened sewage	0 26 49	12.20 7.67 3.50	3.60 2.50 1.04	82.8 57.0 51.6	0 0 0.43	1600 1000 470
Sedimentation tank 12; influent, screened sewage	0 62	13.50 7.00	4.00 2.10	90.1 61.0	0 3.14	1800 950
Contents of the sludge-digestion tank in first experiment	0 23 44	12.00 2.67 2.67	3.50 1.80 1.80	96.5 60.4 51.6	0 0.43 0.82	1600 360 360
Sedimentation tank 12; influent, crude sewage	0 59	12.00 4.70	3.50 1.40	88.7 62.8	0 2.59	1600 640

In 1926 sludge from Imhoff tanks at Dallas, Tex., was lagooned because of insufficient drying-bed capacity during the rainy season (51).

The lagoon was used for skimmings and excess sludge during 1926 and, as no odors were noticed and costs were reduced, all the sludge has been taken care of in this way since 1927.

At Toronto, Ont., sludge lagoons receive undigested sludge from plain-sedimentation tanks. After becoming filled with sludge, the lagoons are allowed to rest for about a year, after which the sludge is pumped out by a suction dredge and removed as fill to an adjacent area in the bay.

Lagoons have been used for emergency disposal of sludge at Fitchburg, Mass., Rochester and Schenectady, N. Y., and Houston and San Marcos, Tex. A cross between lagooning and bed drying has long been employed at Reading, Pa., where a large, low-lying sand area receives sludge from septic tanks and humus tanks. The area is cleaned about once a year.

Lagooning of excess activated sludge has not been satisfactory at Houston. The experience there is summarized by Fugate and Stanley (52) as follows:

> Up to 1920, the only means of disposal of excess activated sludge was lagooning. It has been repeatedly observed that when the pH of sludge in lagoons falls below 7.0 a thick crust of undigested sludge solids forms on the surface, accompanied by a highly offensive odor. This can be remedied in most case by hosing the lagoon with fire hose. Aggravated cases were known to persist for ten days or more, and were alleviated only by persistent stirring and hosing. The addition of lime would doubtless help but the cost of such procedure makes its use out of the question.
>
> The failure of lagooning made it necessary to turn toward a method free from its objectionable features.

Considerable difficulty was experienced with the lagooning of activated sludge at Pasadena, Cal., before a successful mechanical dewatering and drying plant was placed in operation. Objectionable conditions, due to odors as well as to flies and other insects, were created. Goudey (53) reports that "at one time a comprehensive survey indicated that odors from lagooned sludge traveled 7000 ft. covering an area of 3100 acres and affecting 10,000 people."

At Syracuse, N. Y., sludge from plain-sedimentation basins is removed continuously by Dorr mechanisms and pumped to a disposal area about 2 miles distant, where it is lagooned together with many times its volume of industrial wastes from the Solvay Process Co. The resulting mixture is sterilized completely by the chemicals in the Solvay wastes and the overflow from the lagoons flows into Onondaga Lake.

Disposal of Dried Sludge.—The sludge removed from drying beds or lagoons, as well as the press cake from sludge filters, which is not hauled away by farmers—and commonly the bulk of the sludge is not disposed of in this way—usually is placed in sludge dumps or fills in low-lying land adjacent to the disposal works. Where industrial railway tracks are provided for sludge removal, they generally terminate at the sludge dump. At the Calumet works of Chicago, for example, a dump about 700 ft. long was built up on a clay spoil bank left from the plant construction (54). Tracks are laid the long way of the dump and, as the sludge is deposited next to the track, it is pushed over the edge of the bank by a bulldozer. The distribution thus secured avoids moving the track more than once a year.

Other Methods of Sludge Disposal.—Two other methods of sludge disposal are of interest, incineration and destructive distillation. In both of them an attempt is made to utilize the calorific value of the sludge in its disposal.

Incineration.—The burning of sludge was attempted at a few places in the United States in the early days of sewage disposal: notably at Worcester, Mass., in 1891, and at Coney Island, N. Y., for a number of years prior to 1892 (55) (56). This method of sludge disposal, however, has not been employed to any great extent.

Investigations carried on at the Philadelphia experiment station showed that, although sludge could be burned readily with the addition of somewhat less than an equal weight of coal, the full calorific value could not be utilized, because of the quantity of water which had to be evaporated in burning it (50).

The improved methods of dewatering fresh, activated and digested sludges by vacuum filtration and improved furnace equipment have revealed new possibilities for final disposal by incineration.

Fair and Moore (57) have studied the fuel values of raw and digested sewage solids and compared their findings with the results obtained by Wisner and Pearse (27), Bach (59) and Pearse and Mohlman. From the results of these studies they have developed the formula,

$$Q = CP^{1.333}$$

where Q = calorific value in B.t.u. per pound of solids, dry basis
P = volatile solids in per cent, dry basis
C = a coefficient, which varies from 25 for activated sludge, raw or digested, to 29 for primary sludge, raw or digested.

The average fuel value of sewage sludge, based on the formula developed by Fair and Moore, is given in Table 128.

Rudolfs (60) gives the following as average B.t.u. values for the different types of sludge: fresh solids, 6500 to 8500; activated sludge, 6000 to 8000; and digested sludge, 4800 to 7200.

Rumsey (37) reports as follows upon experiments in the use of sludge as fuel:

Of the tangible by-products there seem to be some possibilities in using excess digested sludge cake as a fuel. Our own experience this past year

TABLE 128.—FUEL VALUE OF SEWAGE SLUDGE

Volatile matter in sludge, per cent	Fuel value, B.t.u. per pound of solids, dry basis		
	Primary sludge[1]	Activated sludge[1]	Mixed sludge[2]
85	10,840	9450	10,100
80	10,000	8620	9,300
75	9,200	7910	8,450
70	8,360	7210	7,800
65	7,570	6530	7,050
60	6,810	5870	6,330
55	6,060	5220	5,650
50	5,450	4610	4,980
45	4,650	4000	4,330
40	3,970	3420	3,690
35	3,320	2860	3,090
30	2,710	2330	2,520

[1] Raw or digested.
[2] Mixed raw or digested primary sludge and activated sludge.

was in firing a boiler with about 45 to 50 tons of this material. The cake had a heat value of 4300 B.t.u. per dry pound, and an ash residue of about 50 per cent. We find that this fuel, when having more than a 50 per cent moisture content, will not burn successfully under a boiler. At 30 per cent moisture a good fire is maintained by somewhat frequent rocking of the grates to remove the excess ash.

Sludge cake in the bin, at our Market Avenue Pumping Station, cost about 66 cents per ton, and assuming 3 tons of cake equivalent to 1 ton of slack coal, it gives us a relative value of $2.00 per ton for the sludge cake as compared to $4.75 for coal to produce a like number of B.t.u.'s. In order to utilize any quantity of excess cake for fuel purposes, it would be necessary to provide a good size storage, as this material is quite bulky.

At Dearborn, Mich., sludge from the chemical-precipitation plant, now being mechanically dewatered by vacuum filtration, is being incinerated in a unit constructed and ready for initial operation early in 1935.

At the West Side plant in Chicago, an experimental incinerator plant with a capacity of 20 to 30 tons of sludge daily was built in 1932. Rotary dryers were employed to reduce from 80 to 20 per cent the moisture content of the sludge cake from vacuum filters. Different types of grates and different temperatures were under investigation. These studies resulted early in 1934 in replacing the plant with a new demonstration unit having a daily capacity of some 20 tons. In operation, the sludge, after being dosed with ferric chloride, goes to a rotary vacuum filter, where the moisture content is reduced from 98 per cent to approximately 80 per cent (61). The sludge is dried and pulverized in kiln mills and delivered to the furnace, where it is burned in suspension. The hot gases from the furance are used in air preheaters, in which fresh air is heated and used for drying the sludge cake and for combustion in the furnace. Thus, the moisture-laden air from the kilns passes through the high-temperature zone of the furnace and it is said that complete destruction of odors takes place before the gases are released to the stack. Cyclone separators are used for separating fly ash from the flue gases. Although the heat from the sludge burning has been demonstrated to be nearly sufficient for predrying and combustion, a small quantity of supplementary fuel in the form of pulverized coal is required from time to time, in order to obtain the desired temperature. Testing under a variety of operating conditions, men, organized in an "odor patrol," are said to have established that the system is entirely odorless.

The success of the demonstration plant has resulted in a program for construction of similar units at the Calumet treatment plant, as well as extension of the West Side incinerator plant.

Distillation.—The destructive distillation of sludge cake has been experimentally studied both in this country and abroad. In 1908 experiments were conducted by the Massachusetts State Board of Health on sludge from plain-sedimentation tanks, septic tanks, chemical-precipitation tanks and trickling-filter humus tanks and, for comparative purposes, on peat, sawdust, wood pulp, waste sulfite-pulp liquor, soap, grease and bituminous coal. In these investigations about 400 gm. of dried sludge were placed in a cast-iron retort, which was heated slowly to bright-red heat. The results were reported as follows (62):

The volume of gas produced from different samples of the same kind of sludge and from sludges of different kinds has varied considerably. The amount of gas produced per ton of dry sludge averaged about 6600 cubic feet for settled sewage sludge, 8100 cu. ft. for chemically precipitated sewage sludge, 4900 cu. ft. for septic tank sludge, and 6000 cu. ft. for sediment from the effluents from trickling filters. The residue from the evaporation of sulphite pulp liquor produced about 11,000 cu. ft. of gas per ton. In comparison about 12,000 cu. ft. of gas were obtained from both sawdust and wood pulp, about 8400 cu. ft. from peat and about 5300 cu. ft. from soap grease, while the different kinds of soft coal yielded from 8600 to 12,900 cu. ft. of gas per ton. The composition of the gases from different sources was quite different. In general, the gases from sludge contained a much larger proportion of CO_2, of CO and of the so-called illuminants than the gas produced from coal, while the proportions of hydrogen and methane were correspondingly less. Gases of similar composition are used successfully in many places, however, as sources of light and heat. The coke resulting from the distillation of the various sludges amounted to 45 to 65 per cent of the weight of the dry sludge, and, in spite of the fact that it was usually very soft and friable and that it contained a large proportion of mineral matter, this could undoubtedly be burned on properly constructed grates, and the heat used either for making steam or for drying the wet sludge. Analyses of the coke also showed it to contain from 1.1 to 1.7 per cent of available P_2O_5 and on an average about 22 per cent of the nitrogen of the sludge, and it is possible that it might profitably be used as a basis for the manufacture of fertilizer. The per cent of P_2O_5 might be increased by burning the sludge more completely, but this would cause the removal of the nitrogen, which is a desirable ingredient if the coke is to be used for this purpose. Much of the fats in the sludge distilled over with the tars, resulting in a light, greasy tar. This by-product of the process could readily be disposed of by mixing it with the coke and burning, or if it were formed in sufficient amounts it could be burned directly, in the same manner as water gas tars are utilized. The average results of the investigation are shown in Tables 129 and 130.

TABLE 129.—ANALYSES OF SLUDGES USED FOR DESTRUCTIVE DISTILLATION AND CHARACTERISTICS OF COKE PRODUCED

	Composition of sample before distillation, per cent			Coke produced, per cent of dry sludge	Percentage of nitrogen[1]		Available P_2O_5 in coke, per cent
	Total nitrogen	Loss on ignition	Fats		Found in coke	As NH_3 in washer	
Lawrence sludge[2]	3.36	36.8	12.8	63.5	.11	.586	1.33
Andover sludge[2]	2.14	46.6	27.5	59.5	.67	.226	1.33
Clinton sludge[2]	2.36	74.4	7.7	44.5	.72	.404	1.44
Brockton sludge[2]	1.76	46.0	6.2	60.5	.94	.137	1.17
Worcester sludge[3]	1.19	44.5	3.2	54.0	.09	.544	1.67
Septic-tank sludge	2.46	47.9	8.3	68.5	.27	.497	1.15
Trickling-filter sludge	2.10	48.3	4.9	62.0	.56	.809	1.31
Sludge from evaporation of sulfite-pulp liquor	....	87.3	....	32.0			
Peat	2.54	92.0	....	49.0	.70	.700	0.31
Sawdust	0.00	....	....	25.0	...	.000	
Wood pulp	0.00	....	....	25.0	...	.000	
Soft coal[4]	....	96.8	....	77.3	...	.222	

[1] Per cent by weight of total sludge taken.
[2] Settled sewage sludge.
[3] Chemically precipitated sludge.
[4] Average of four kinds of steam and gas coal.

More recently, Thompson (63) at Leeds, England, has conducted tests on both the burning of sludge and the carbonization of sludge cake. At this plant, limed press cake not sold to farmers is dumped on low-lying ground and ignited after drying for about a year. The brown ash produced, containing 45 per cent calcium carbonate on a dry basis, is sold to farmers as a substitute for lime.

TABLE 130.—RELATIVE VOLUME AND COMPOSITION OF GASES PRODUCED BY DESTRUCTIVE DISTILLATION OF SLUDGE

	Cubic feet of gas per ton of sample	Per cent						
		CO_2	Illuminants	O	CO	H	CH_4	N
Lawrence sludge[1]	4,900	4.4	2.2	0.3	30.7	34.9	18.6	9.1
Andover sludge[1]	6,400	7.4	15.1	0.6	14.3	22.9	34.3	5.4
Clinton sludge[1]	9,100	8.3	6.7	0.0	20.4	33.2	24.5	7.0
Brockton sludge[1]	6,000	16.5	21.4	0.2	10.3	22.6	29.1	0.2
Worcester sludge[2]	8,100	14.2	4.9	0.3	29.8	32.6	16.2	2.2
Septic-tank sludge	4,900	7.5	1.0	0.1	24.3	44.0	13.0	10.2
Trickling-filter sludge	6,000	20.2	17.4	0.3	6.6	32.7	22.8	0.0
Sludge from evaporation of sulfite-pulp liquor	11,000	21.6	2.1	0.0	20.0	42.0	12.0	0.3
Peat	8,400	39.0	4.7	0.2	11.0	28.0	17.1	0.0
Sawdust	12,700	18.4	4.8	0.3	28.2	26.9	19.1	2.3
Wood pulp	12,000	23.5	1.4	0.0	16.4	44.5	13.3	0.9
Soap grease	5,400	6.8	44.5	0.0	6.2	15.8	26.7	0.0
Soft coal[3]	10,200	1.6	2.0	0.1	5.2	62.3	25.7	3.2
Lawrence illuminating gas[4]	......	3.4	9.1	0.0	21.5	42.5	19.7	3.8

[1] Settled sewage sludge.
[2] Chemically precipitated sludge.
[3] Average of four kinds of steam and gas coal.
[4] From gas company pipes at experiment station.

CONVEYING SEWAGE SLUDGE

In modern sewage-treatment works the sludge produced may be conveyed from point to point in a number of different ways, that may vary greatly with the method of sewage and sludge treatment and the manner of final disposal of the sludge. Several methods of conveying sludge have been mentioned in the preceding sections of this chapter and in connection with the various sewage-treatment processes described in previous chapters.

Watery sludge is commonly caused to flow by gravity in open or closed conduits, or it is lifted by different kinds of pumps in pipe lines. Questions of sludge pumping and sludge flow in pipes will be considered in the succeeding sections of this chapter. Sludge paste, according to experience at Milwaukee, is best transported on belt conveyors. At this plant trouble was encountered with the use of screw conveyors handling activated sludge, the moisture content of which had been reduced to about 80 per cent by vacuum filtration. The sludge tended to cake on the sides of the conveying machinery and to form balls that could not be dehydrated successfully in the heat dryers. Sludge cake is commonly transported in wheelbarrows, carts or industrial dump-cars, as described in Chap. XXVIII. Granular sludge, finally, may be conveyed on belt or screw conveyors.

The most elaborate development of sludge-conveying mechanisms is naturally associated with large activated-sludge plants producing a commercial fertilizer from sewage sludge. The magnitude of the problem is emphasized by the following schedule of average quantities handled daily during 1928 at Milwaukee:

Volume of sewage, m.g.d.	81.8
Excess activated sludge, 98.5 per cent moisture, tons	5670
Sludge paste from filters, 84.5 per cent moisture, tons	553
Granular sludge from dryers, 2.97 per cent moisture, tons	88.4

Flow of Sludge in Pipes and Other Conduits.—The design of pipes and other conduits for conveying watery sludge requires a knowledge of the characteristics of sludge flow. Unfortunately sewage sludge does not constitute a homogeneous fluid and the well-known laws of hydraulics apply only approximately. The problem is further complicated by the variable nature of sludge, as regards both the character of the solids carried in suspension and the moisture content. Although some excellent work has been done, information is still too incomplete to permit of broad generalization for design purposes.

The friction of sewage sludge in pipes has been studied by a committee of the American Society of Civil Engineers whose conclusions follow (64):

1. Sludge is neither a viscous nor a homogeneous material, but is variable in character.
2. The usual analytical tests do not define its physical qualities, but it seems to behave more like suspended matter.
3. Below the critical velocity[1] sludge has a different friction factor from that above. As yet the coefficient of flow below the critical velocity cannot be concisely stated. Above the critical velocity, it can only be expressed in ranges.
4. Sludge friction losses increase with a decrease of moisture content.
5. Sludge friction losses tend to increase with lower temperatures.
6. Sludge friction losses for high velocities (from about 5 to 6 ft. per second, or more), tend to follow more nearly the characteristic laws for the flow of water.
7. Friction losses for fresh or undigested sludge and for sludge from combined sewage are more erratic and the determination of a friction factor is correspondingly more difficult to obtain.
8. Within the limits investigated no law of flow has been found.

[1] The velocity at which the type of motion changes from straight-line to turbulent flow is called the *critical velocity*.

According to the information so far available, there seem to be four possible states of flow proceeding from low to high velocities as follows: flow at "sedimentation velocity," at which sludge will settle to the bottom of the pipe and the sedimentation velocity will be maintained in the remaining cross section, thus producing a change in calculated velocity, as defined by discharge divided by full pipe area, but no change in lost head; "streamline, nonsinuous or laminar flow," in which state the friction loss of homogeneous fluids varies with the first power of the velocity and the loss is greatly affected by the kinematic viscosity of the fluid, or the absolute viscosity divided by the density; "transitional flow," in which the state of flow changes to turbulence; and "turbulent or sinuous flow," in which state the friction loss of homogeneous fluids varies as some power of the velocity, approaching or reaching the square, and the loss is no longer affected much by the kinematic viscosity of the fluid.

While the American Society of Civil Engineers' committee did not believe that a general law for the flow of sewage sludge could be derived from the information available, Clifford has pointed out that an analysis of sludge flow is possible, if sewage sludge is considered a pseudohomogeneous fluid and if Stanton and Pannall's approach is used in the solution of the problem (65) (66). As a first approximation, Clifford has suggested a value of 0.001055 for the kinematic viscosity of a sludge with 90 per cent moisture. The value of the Reynolds criterion for a fluid of this character indicates that laminar flow obtains in a 4-in. pipe up to a velocity of about 6 ft. a second and that the flow probably becomes turbulent at about 10 ft. a second; in an 8-in. pipe these limiting velocities are respectively 3 and 4 ft. a second, in a 12-in. 2 and 3 ft. a second, and in a 24-in. 1 and 1.5 ft. a second. An examination of Table 131 will show that, judged on this basis, the velocities of flow employed in sludge lines are generally such as to result in laminar or transitional flow. Comparisons of friction losses of sludge with those of water do not appear to be well conceived, because water is flowing turbulently at these velocities. Nevertheless Clifford (65) has stated the following in summarizing his studies:

> For practical purposes, and until more accurate data are available, the friction head or pressure drop for 90 per cent sludge per 1000 ft. of pipe line, may be taken as approximately one and a half times that of water. Ample allowance should be made for valves, bends and junctions, and a further provision for those gritty sludges which appear from time to time. The average friction head from all sources, of four sludge mains in use, is about four times that due to water flowing in straight lengths of asphalted cast iron pipes, under the same conditions of diameter and average velocity.

The characteristics of a number of long pipe lines conveying sewage sludge are shown in Table 131. Part of them have been built in Great

TABLE 131.—CHARACTERISTICS OF LONG PIPE LINES CONVEYING SEWAGE SLUDGE

	Baltimore, Md.	Birmingham, England	Bradford, England	Canton, Ohio	Chicago, Ill.	Glasgow, Scotland	Manchester, England, Davyhulme works	Pittsfield, Mass.	Syracuse, N. Y.	Toronto, Ont.	Wolverhampton, England
Diameter, in.	12	12	8	6	14	9	10	24	5	12	4
Length, ft.	2252	21,000	21,800	5500	74,000	32,736	3960	10,850	12,000	400	591–3651
Ave. pressure, lb. per sq. in.	20 (max.)	50	60	....	80 (max.)	70	45	18	40–130	20	34–85
Discharge, c.f.s.	7.0	1.33	1.09	0.67	2.14	0.57	1.52	10.0	0.2–0.8	....	0.22
Ave. velocity, f.p.s.	....	1.70	3.12	3.5	2.0	1.29	2.79	3.2	2.5	....	2.5
Type of sludge	[2]	....	....	[3]	[1] and [2]	....	....	[2]	[2]	[2]	
Moisture, per cent.	92.8–98.5	90	80	....	....	88	89	....	98+	91–93	90
Friction loss, ft. per 1000 ft.	....	5.29	8.18	....	....	4.75	17.17	2.0	....	....	27.9
Type of pump	Cent.	Rams / Cent.	Air	Triplex plunger	Cent.	Rams	....	Cent.	Triplex plunger	Cent.	Air
Number of pumps	1	....	....	2	3	....	....	....	4	1	
Capacity per unit, g.p.m.	3200	....	....	300	1000	....	....	4500	92		
Discharge head, ft.	50	....	....	....	180	....	....	41	462	44	
Suction head, ft.	....	....	....	....	8	....	....	10–17			

[1] Activated sludge. [2] Primary sludge. [3] Imhoff sludge.

Britain. The following description of some of these lines is taken largely from the report of the American Society of Civil Engineers' committee (64):

Line and Grade:

Birmingham, England: Not laid on a grade; inlet and outlet practically level.

Glasgow, Scotland: Practically level.

Bradford, England: Line laid on a grade with an open end for free discharge.

Wolverhampton, England: Pipes laid on surface of ground without a regular gradient.

Syracuse, N. Y.: Line laid on uniform grades with blow-offs at depressions; insulated wherever exposed.

Baltimore, Md.: Gradient depends upon topography of the ground and necessary connections to other structures.

Chicago, Ill.: Line laid on regular grades between summit and valley as far as practicable.

Toronto, Ont.: Gradient for 12-in. line is 1.25 per cent.

Maintenance:

Birmingham, England: Access boxes and provisions for flushing under pressure are provided, but have not been needed; line flushed twice each year by pumping sewage at 3 or 4 ft. a sec. for 12 hr.

Glasgow, Scotland: Two scour valves at excessive dips are tested regularly; after pumping sludge of 88 per cent moisture, a thinner sludge is used as a precaution.

Bradford, England: When sludge pumping is stopped a few days the line is blown out with compressed air.

Syracuse, N. Y.: Manholes with hatch boxes every 500 ft.

Chicago, Ill.: Manholes with hatch boxes every 3260 ft., more or less.

Air Valves:

Birmingham, England: Provision for valves at intermediate summits, but valves removed.

Chicago, Ill.: At all summits.

Stoppages:

Birmingham, England: Have occurred, although rarely, when excessive grit is present or when pumping is irregular.

Glasgow, Scotland: Checking has occurred perhaps twice, but a steady pressure of 90 to 100 lb. per square inch cleared it readily.

Syracuse, N. Y.: Three have occurred and were readily located by pressure gage at hatch boxes.

Preparation of Sludge:

Birmingham, England: Screened before entering pumps.

Syracuse, N. Y.: Drawn from bottom of Dorr clarifiers and screened twice.

Baltimore, Md.: Drawn from sedimentation tanks and passed through ⅞-in. bar racks.

Chicago, Ill.: Mixture of activated sludge and the product of 20- or 30-min. settling in Dorr clarifiers after passing screen.

Method of Pumping:

Birmingham, England: Two types of pumps.[1]

Glasgow, Scotland: Rams, 10 by 5 ft. in diameter, of 187-cu. ft. capacity; rate, 1 ton per minute at 100 lb. per square inch; working pressures, 60 to 80 lb. per square inch.

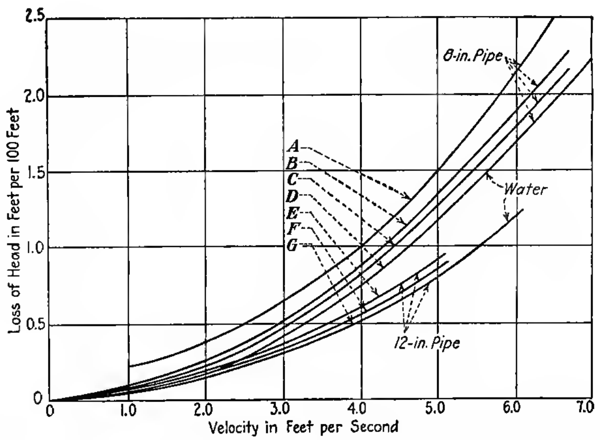


FIG. 197.—Average friction curves for activated sludge at the Calumet sewage-treatment plant, Chicago.

Bradford, England: Plugs of sludge are followed by compressed air at 50 to 60 lb. per square inch.

Wolverhampton, England: Compressed air.

Syracuse, N. Y.: Four 6- by 10-in. stuff pumps; 5-in. bronze ball-valves; 15-hp., slip-ring motors; capacity, 92 gal. a minute against 462-ft. head; two pumps for ordinary service at 80-ft. head.

Baltimore, Md.: Centrifugal pumps.

Chicago, Ill.: Centrifugal pumps of special design, to pass solids and keep runners clean; capacity, each 1000 gal. a minute against 188-ft. dynamic head; 100-hp. squirrel-cage motors at 1765 r.p.m.

Toronto, Ont.: 18-in. centrifugal pump through a 44-ft. lift to the entrance of the line.

[1] One is a direct-acting, duplex, steam-driven pump, with a piston speed limited to 4 ft. a minute. There are also electrically operated, single-acting, triplex pumps. Both kinds of pumps work well on sludge containing 90 per cent water. Sludge containing more than 95 per cent water is pumped by centrifugals, of the stereophagus or unchokable type for sizes of 6 in. and less and of ordinary types for larger sizes.

Miscellaneous:

Birmingham, England: No trouble has been experienced at the joints.

Bradford, England: Numerous summits tend to break up the sludge and air pockets.

Baltimore, Md.: This is one of several lines; the longest is more than 5000 ft.

Chicago, Ill.: Pipe was shop-tested to 300 lb. per square inch and tested in place to 175 lb. per square inch.

Toronto, Ont.: In addition, another line of 400 ft. of 10-in. spiral-riveted pipe was used in the 1919 and 1929 tests.

A few tests have been made of the friction of activated sludge in 8-in. and 12-in. pipes at the Calumet plant in Chicago. The results of these tests are shown in Fig. 197, together with the corresponding curves for water (64). Conditions under which the curves were derived are described below:

Curve, Fig. 197	Temperature, °F.	Moisture, per cent
A	72	96.50
B	42.5	98.00
C	42	98.64
D	....	100.00
E	42.5	98.00
F	42	98.64
G	....	100.00

The friction of Imhoff sludge compared with that of water in the two sizes of pipe tested at the Calumet plant is shown in Fig. 198 (64). The curve drawn for water in each case is based on actual tests in the pipe. Some of the tests indicate a friction loss for sludge less than that for water. The committee states that "the only explanation appears to be the effect of gas in the sludge. There is a possible lubrication of the pipe wall by the sludge and a sliding of the sludge as a unit, due to the slippery pipe and cohesive characteristics of the sludge. All these conditions would tend to decrease friction."

There are no test results available on the flow of sewage sludge in open channels. In the Emscher District of Germany the sludge channels transporting Imhoff sludge have a slope of 1:40. At Akron, Ohio, the sludge channels to the drying beds have a slope of 1:83. The open sludge channel at the North Toronto plant in Toronto, Ont., carrying digested primary sludge and excess activated sludge, has a slope of 1:80. On rare occasions the sludge does not flow readily in this channel and must be assisted by means of shovels.

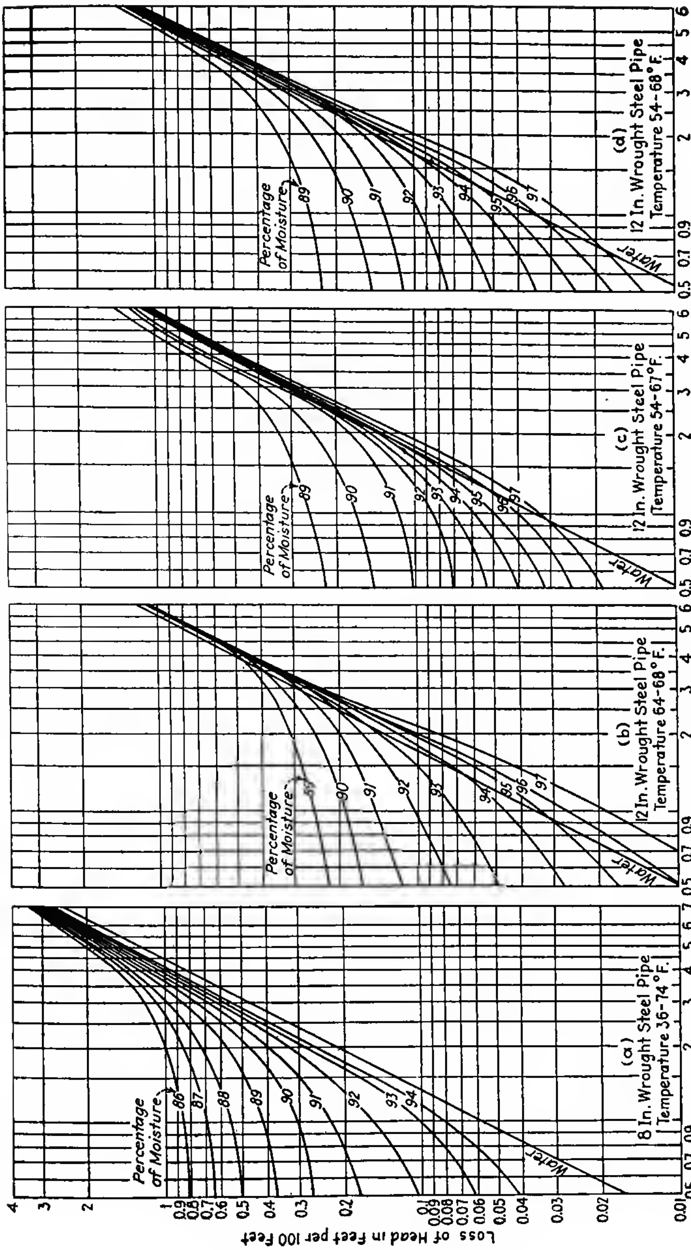


FIG. 198.—Average friction curves for Imhoff sludge at the Calumet sewage-treatment plant, Chicago.

Sludge Pumps.—The type of equipment suitable for the pumping of sludge depends upon the character of sludge, the desired rate of discharge, the suction and discharge heads and the method of treatment, if any, of the sludge following pumping.

In selecting equipment consideration is usually given to the maximum size of objects in the material, determined in large part by the clear openings in racks or fine screens; the type of tank from which the sludge is derived; the source of the sewage, as from separate or combined sewerage systems; and the predominance of any particular kind of industrial wastes. The rate of sludge pumping is of importance, because as the rate decreases the difficulty of securing non-clogging pumps increases. Centrifugal pumps smaller than 4 in. in size are seldom used for sludge pumping. Plunger and diaphragm pumps and compressed-air ejectors are commonly used where the desired rate of discharge is small. If continuous or semicontinuous pumping is required, spare units are necessary, whereas with intermittent pumping, as for wasting humus sludge, one unit frequently serves.

Air lifts are commonly useful for sludge pumping only when the lift is small. The head to which sludge can be lifted by this means is limited by the depth of submergence of the air-eduction pipe. The submergence usually is made 65 per cent or more of the total length, in order to secure moderate efficiency. Thus, with an Imhoff tank 30 ft. deep a lift of about 15 ft. is feasible. Plunger and centrifugal pumps are suitable for high lifts. The centrifugal pumps for wasting sludge at the North Side plant in Chicago provide for pumping against a dynamic discharge head of 180 ft. and a suction head of 8 ft. The plunger pumps at Syracuse provide for a maximum head of 462 ft., as described in the preceding section. Pistons and other appropriate parts of these pumps are continuously washed with clean water during service, the water being furnished by an auxiliary, high-pressure, triplex plunger pump.

At Canton, Ohio, two 10- by 10-in. triplex plunger pumps, equipped with 40-hp. motors, are provided for pumping the digested Imhoff sludge through 5500 ft. of 6-in. force main. The capacity of each pump is 300 gal. per minute at 30 r.p.m.

Sludge settling in plain-sedimentation tanks at Pittsfield, Mass., is pumped through a 24-in. force main, about 10,850 ft. long, on to sand beds reserved for sludge drying. The pump used for discharging sludge is a 10-in., horizontal, split-casing, double-suction, open-runner, centrifugal pump, driven by a 100-hp. slip-ring induction motor. The pump is protected by two sets of racks, one with 1-in. and the other with ¾-in. clear spacing between bars. The capacity at the minimum suction head of 10.2 ft. is 6.5 m.g.d. and at the maximum suction head of 17.2 ft. the discharge is 5.3 m.g.d. The average efficiency is 52.2 per cent. The dynamic discharge head is about 41 ft. The pumps are primed by a priming pump, equipped with a vacuum breaker.

Bibliography

1. Eddy, H. P., and A. L. Fales: *Jour.*, Assoc. of Eng. Socs., 1906; **37,** 67.
2. Sewage Disposal Problem of New Haven, 1918.
3. Jackson, J. F., and Joseph Doman: *Jour.*, Boston Soc. C. E., 1922; **9,** 37.
4. Schaetzle, T. C.: *Eng. News-Rec.*, 1921; **87,** 97.
5. *Bull.* 251, Cal. Agricultural Experiment Sta., 1915.
6. Elsner, A., F. Spillner and Kenneth Allen: "Sewage Sludge," 1912.
7. *Pub. Health Bull.* 132, U. S. Pub. Health Service, 1923.
8. Pearse, Langdon: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 126.
9. Pratt, R. W., and G. B. Gascoigne: *Eng. News*, 1916; **76,** 1124.
10. Rudolfs, Willem, and H. M. Heisig: *Sewage Works Jour.*, 1929; **1,** 146.
11. Hatch, J. N.: *Public Works*, 1929; **60,** 129.
12. *Bull.* No. 18, Ill. State Water Survey, 1922.
13. Rept. on Industrial Wastes from Stockyards and Packingtown in Chicago, vol. 2, 1921.
14. *Am. Jour. Pub. Health*, 1916; **6,** 334.
15. Rudolfs, Willem: *Soil Science*, 1928; **26,** 455.
16. Ann. Rept., Ontario Bd. of Health, 1920.
17. *Bull.* No. 16, Ill. State Water Survey, 1920.
18. Kadish, V. H.: *Jour. Ind. Eng. Chem.*, 1928; **20,** 9.
19. Lipman, C. B.: *Jour. Ind. Eng. Chem.*, 1917; **9,** 189.
20. Bartow, Edward, and W. D. Hatfield: *Jour. Ind. Eng. Chem.*, 1916; **8,** 17.
21. Bach, Hermann, and L. C. Frank: *Eng. Rec.*, 1913; **68,** 331.
22. *Jour. Ind. Eng. Chem.*, 1918; **10,** 239.
23. Kadish, V. H.: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 134.
24. *Am. Jour. Pub. Health*, 1926; **16,** 39.
25. Jareo, J. W.: *Am. City*, 1928; **39,** no. 2, p. 111.
26. Keefer, C. E., and G. K. Armeling: *Eng. News-Rec.*, 1928; **101,** 837.
27. Rept. on Industrial Wastes from Stockyards and Packingtown in Chicago, 1914.
28. Dorr, E. S., and R. S. Weston: *Jour.*, Boston Soc. C. E., 1919; **6,** 145.
29. Neave, S. L., and A. M. Buswell: *Jour. Ind. Eng. Chem.*, 1928; **20,** 1368.
30. Rept., N. Y. Met. Sew. Comm., 1914.
31. Hommon, C. C.: *Am. City*, 1931; **44,** no. 2, p. 110.
32. Browne, F. G.: *Public Works*, 1929; **60,** 449.
33. McDonnell, R. E.: *Munic. Sanitation*, 1931; **2,** 114.
34. Ryan, W. A.: *Am. City*, 1930; **43,** no. 1, p. 87.
35. Ann. Rept., Schenectady Bureau of Sewage Disposal, 1930.
36. Tatlock, M. W.: *Sewage Works Jour.*, 1932; **4,** 519.
37. Rumsey, J. R.: *Water Works and Sewerage*, 1933; **80,** 241.
38. Keefer, C. E.: *Eng. News-Rec.*, 1926; **96,** 238.

39. Ann. Rept., Schenectady Bureau of Sewage Disposal, 1932.
40. Hatton, T. C., V. H. Kadish and H. M. Heisig: *Jour.*, Inst. of Sanitary Engrs., 1927; **31**, 121.
41. Wyman, A. W.: *Sewage Works Jour.*, 1929; **1**, 339.
42. Rockefeller Monograph No. 15, 1921.
43. Wolman, Abel: *Eng. News-Rec.*, 1924; **92**, 198.
44. *Eng. News-Rec.*, 1929; **103**, 1005.
45. Potter, Alexander: *Sewage Works Jour.*, 1934; **6**, 58.
46. Eggert, E. G., and Chester Cohen: *Eng. News-Rec.*, 1929; **102**, 830.
47. Donaldson, Wellington: *Nation's Health*, 1926; **8**, 1.
48. *Surveyor*, 1921; **56**, 498.
49. *Surveyor*, 1922; **61**, 487.
50. Rept. of Bureau of Surveys, Comprising Work at Sewage Experiment Sta., Philadelphia, 1910.
51. Whedbee, Edgar: *Proc.*, 12th Tex. Water Works Short School, 1930; p. 153.
52. Fugate, G. L., and W. S. Stanley: *Sewage Works Jour.*, 1928; **1**, 70.
53. Goudey, R. F.: *Western Construction News*, 1926; **1**, 27.
54. Goodman, A. H., and C. E. Wheeler, Jr.: *Sewage Works Jour.*, 1929, **1**, 444.
55. *Eng. News*, 1892; **28**, 77.
56. *Eng. News*, 1892; **28**, 368.
57. Fair, G. M., and E. W. Moore: *Sewage Works Jour.*, 1932; **4**, 242.
58. Bach, Hermann: *Technisches Gemeindeblatt*, 1930; **33**, 108.
59. Bach, Hermann: *Gesundheits-Ingenieur*, 1930; **53**, 790.
60. Rudolfs, Willem: *Sewage Works Jour.*, 1933; **5**, 409.
61. *Am. City*, 1934; **49**, no. 12, p. 97.
62. Ann. Rept., Mass. State Bd. of Health, 1908.
63. *Surveyor*, 1923; **64**, 466.
64. *Proc.*, A. S. C. E., 1929; **55**, 1773.
65. Clifford, William: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 59.
66. *Proc.*, A. S. C. E., 1930; **56**, 821.

CHAPTER XXVIII

SLUDGE-DRYING BEDS

The wet sludge produced by sewage treatment in America is most commonly run onto specially prepared sand beds, on which it is air-dried. Part of the water passes from the sludge into the sand and percolates through it to underdrains, while part evaporates. Climatic conditions and the air-drying characteristics of different sludges are important considerations in providing sludge-drying areas.

Characteristics of Sludge-drying Beds.—The behavior of different sludges when placed on sand beds has been indicated in Chap. XXVII, from which it appears that drying beds are particularly suitable for dewatering well-digested sludge. The following detailed description of the behavior of Imhoff sludge during air-drying is taken from Gault's report (1) of Fales' experiments at Worcester, Mass:

When first drawn onto the drying bed, the Imhoff-tank sludge became completely covered with gas bubbles, due to the release of pressure on drawing from the tank. The escaping gas lifted the solids, leaving a comparatively thin liquid at the bottom, just as in the case of the glass cylinders, where, inside of 24 hr., from 15 to 30 per cent of clear liquid remained at the bottom. This clear liquid filtered away within a day or two, reducing the volume of the sludge correspondingly. Under favorable weather conditions, the surface of the sludge on the second day presented a thin unbroken crust. When the sludge was stirred up, however, much gas escaped, making a sound like a man puffing on his pipe, and within a few minutes the surface where it was broken looked as if it were strewn with blackberries, due to the gas bubbles in the sludge. The entire mass where disturbed, except for color, now resembled soft dough in which yeast is very active. Within a few hours after this condition, cracks began to appear on the surface and gas escaped through these cracks, causing the sludge to become a seething mass. The evolution of gas soon subsided, although the sludge retained much gas for several days, rendering the sludge porous and spongy. The cracks gradually increased in number and deepened, thereby facilitating the drying. The bottom portion was the last to dry, the time required to dry out depending upon the depth of sludge.

In the experiments at Worcester it proved practicable to remove the sludge from the drying beds while it contained as much as 70 per cent of water, although the volume and weight of such moist sludge can be greatly reduced by further drying. The weight of the sludge removed varied from 60 lb. per cubic foot, when it contained 68 per cent of water, to 31¼ lb. per cubic foot with a water content of 8 per cent. The latter grade was produced by drying a thin layer of sludge for 4 weeks during extremely favorable weather. The average loss in volume at Worcester was 70 to 80 per cent and the average loss in weight was 75 to 85 per cent.

Weather conditions are of great importance. If rain falls within 24 hr. after running digested sludge on a bed, it tends to liberate the gas, making the sludge more compact and less porous, so that it does not dry out so rapidly as otherwise. If rain falls while the surface of the sludge is cracking, the water is retained near the surface and retards the rate of drying, but if the sludge is nearly dry, so that the cracks extend through the mass, the water soon drains away and the sludge quickly dries again.

Drying sludge generally becomes spadable when its moisture content is reduced to 50 or 60 per cent and it is commonly removed from the drying beds when it has reached this condition. At the Calumet treatment works of Chicago, for example, Goodman and Wheeler (2) have recorded the average yearly water content for the years from 1923 to 1928 as varying from 87.2 to 89.1 per cent in the Imhoff sludge applied to the beds, while the sludge removed averaged 39.3 to 55.1 per cent water.

At the North Toronto plant in Toronto, Ont., it has been possible to remove well-digested primary sludge and excess activated sludge from the glass-covered drying beds with forks, when the moisture content was 80 to 85 per cent. The sludge so removed, however, was soggy and slumped in the sludge dump.

In connection with the experiments at Worcester, comparative tests were made of the behavior during drying of Imhoff, chemical-precipitation and primary sludges. The last-named was coarse and contained a large quantity of hair from a local tannery. It formed a mat over the surface, which did not crack readily, so that the air failed to penetrate it thoroughly. For this reason it did not dry so completely as the Imhoff sludge, except when exposed in thin layers, and rain falling while the sludge was drying was likely to undo the effect of 2 or 3 weeks' exposure. The chemical-precipitation sludge formed a multitude of cracks, which gave it an advantage over the Imhoff sludge when applied in a layer not over 6 in. thick. It did not change from its liquid state as quickly as the Imhoff sludge, but the cracks caused it to reach a condition suitable for removal quite as soon.

At Tenafly, N. J., activated sludge is allowed to flow gradually over sand beds under glass covers. Donaldson and Kurtz (3) report that

. . most of the water quickly separates from the solids and runs ahead into the sand before it is sealed by the solids. This is continued until the entire bed is covered, precaution being taken not to allow too heavy a blanket to be formed near the feed point. The dosing operation takes about 30 min. and in about 2 hr. all free water has disappeared from the surface. For removal of the sludge layer the sludge is marked into squares the size of the lifting fork.

The solids content of the liquid sludge is about 2.5 per cent and is increased to 20–25 per cent in 4 or 5 days. The cake is ¾ to 1 in. thick.

At Brockton, Mass., sludge from plain-sedimentation tanks flows in a layer about 2 in. thick on to drying beds, formerly used as intermittent sand filters. The sludge applied contains about 90 per cent water and the dried sludge about 35 to 40 per cent. At Gloversville, N. Y., sludge from plain-sedimentation and humus tanks contains about 95 per cent water and is reduced in volume about 85 per cent during drying.

The drying of sludge from plain-sedimentation tanks may be accompanied by the liberation of objectionable odors, making this method impracticable unless the drying area is well isolated. The drying beds at Brockton are remote, so that little offense is caused.

Rudolfs, Downes and Campbell (4) concluded from full-scale experiments performed at Plainfield, N. J., during April, May and June, 1923, that air-drying of digested sludge can be greatly accelerated by the addition—with a minimum of stirring, to secure adequate mixture—of a solution of alum[1] to the sludge, just prior to running it on to the drying area. A dosage of 1⅓ lb. of alum per cubic yard of sludge from the Plainfield tanks, which have at times produced poorly digested sludge, increased the number of dryings from 6 to 11 during 7 months. The drying of well-digested sludge, however, was not greatly improved by the addition of alum. The latter is in accord with experience at Schenectady, N. Y.

The effluent from drying beds may require treatment with the raw or clarified sewage, if the sludge liquid is of questionable nature and there is no opportunity for filtering or otherwise treating it independently of the sewage. Although generally clear, the effluent may be high in bacteria and may have a considerable biochemical-oxygen demand. At Baltimore the effluent from the sand beds, upon which partially digested sludge dries, passes through 4 in. of sand and 12 in. of gravel (5). The effluent is clear but has an average B.O.D. of 365 p.p.m.

Drying Area Required.—The drying area required varies with the following items: the quantity of solids removed from the sewage; the character of the sludge to be dried; local atmospheric conditions, such as rainfall, relative humidity, temperature and duration of period when drying on open beds is feasible; the degree of protection from the

[1] For the principles of sludge conditioning see Chap. XXIX.

elements; and, if the area is covered, the efficiency of the heating and ventilating system. Allowances are generally expressed in square feet of drying area per capita, the number of applications of sludge yearly and the average depth of application also being given at times. The loading may be expressed in pounds of dry solids in the sludge applied to the beds yearly per square foot of bed area.

There is a marked seasonal variation in the yield of air-dried solids, even with glass-housed sludge beds. The authors have determined monthly yields by computing the weighted averages based on the number of days the sludge was on each bed during the month. Sludge beds must be adequate to handle the sludge as produced unless sufficient tank storage is afforded to take care of variations in sludge bed yields.

The removal of solids from the sewage depends on the strength of sewage and the type and degree of treatment provided. The percentage of the total suspended solids in the raw sewage commonly removed as sludge in various treatment processes is as follows:

	Per Cent of Total Suspended Solids in Raw Sewage Removed as Sludge
Primary sedimentation	40–60
Sedimentation following trickling filters	25–40
Activated-sludge process	80–90
Preliminary sedimentation and sedimentation following trickling filters	65–85
Preliminary sedimentation and activated-sludge process	80–90

The above values are reduced 35 to 50 per cent on the average by digestion in Imhoff tanks or separate digestion tanks. Well-digested sludge drains and dries more rapidly than poorly digested sludge, so that the efficiency of the digestion process influences the capacity of the drying-bed area. The percentage moisture in the sludge, the depth to which it may be placed on the beds and dewatered effectively, and the facility with which it drains and dries to a condition suitable for removal, all influence the area requirements.

For use in connection with Imhoff tanks treating solids removed from sewage primary sedimentation only, open drying areas of ⅓ to 1 sq. ft. per capita generally have been provided. After a survey of operating characteristics of sludge beds treating Imhoff sludge at a number of American plants, engineers of the United States Public Health Service reported (6) that

. . . it would appear that under American conditions the majority of the plants require a minimum of 0.5 sq. ft. per capita, and that 0.8 or even 1 sq.

ft. per capita is a safer design figure, especially where facilities for lagooning or wasting in low lands are not available, and entire reliance must be placed on the beds . . No part of a treatment plant is less expensive than the sludge beds. Where area is available, the cost of a liberal bed installation is but little more than the minimum requirements.

The reduction in area made possible by the use of glass enclosures and the relative economy incident to it have not as yet been adequately established. Goodman and Wheeler (2) comment as follows upon the operation of open and covered beds at the Calumet works of Chicago:

Experimental work has been carried on at this plant to compare drying on open beds and drying on glass-covered beds. The glass enclosures are of standard greenhouse construction built over two of the 17 ft. 7 in. by 66 ft. 4 in. drying beds. Two adjacent open beds of the same size are used for controls. With only the natural ventilation that could be obtained by the side and top ventilators, tests indicated that there was practically no improvement over the rate of drying on open beds. After this had been demonstrated a Buffalo Forge Co. fan was installed for forced ventilation to replace the moisture-laden air above the sludge with air from the outside. In extreme winter conditions the air is heated in an attempt to keep the sludge from freezing, but it has been impossible to keep the temperatures in the greenhouse enclosure above freezing at all times with the facilities available. During a year's operation with circulation of air and heating in the cold months 8 dosings and removals were made on the open beds while 12 were obtained on the covered beds. The open and covered beds were operated under parallel conditions with regard to quality of sludge, depths on the beds and moisture content at removal. Dried sludge was removed from the covered beds and controls as soon as practicable. Average daily temperatures inside the covered beds in the winter period did not go below freezing, although there were several occasions when the minimum daily temperature was below freezing.

Browne and Jones (7), on the other hand, claim that their Ohio experience shows that covered beds need have only ⅓ the area of open beds. They estimate the glass enclosure to cost $1.60 a square foot and the bed proper, with industrial railway track, $1.00 a square foot, thus indicating an advantage for covered beds. Maintenance costs, however, are not included in these figures. With regard to the area of glass-covered beds required, they comment as follows:

Experiments conducted at Marion, Ohio, at various times during the past year [1925] on glass-covered beds, indicate that 6 in. of 92 per cent moisture sludge can be dried to 65 per cent moisture in 4 days, 8 in. of 94 to 91 per cent in 9 days, and 10 and 12 in. of 93 per cent in about 15 days.

At Alliance, Ohio, using glass-covered sludge drying beds with a partially digested sludge of rather poor quality, the following results were obtained on 10,240 sq. ft. of drying area, dosed to an average depth of 6 in.:

	Cu. yd. of wet sludge applied	Number of fillings
1920	2525	14
1921	3560	23
1922	3015	19.5
1923	2910	19.5
Average....	3000	19.0

In 1930 the average drying period in the glass-covered beds at Dayton, Ohio, was 11 days during July and August, when the beds were being operated at about a maximum rate, and 16 days during June, September, October, November and December, when they were idle about 10 days, on the average, between the time of cleaning and time of application of the next dose. It is believed that, if necessary, at least 20 applications a year could have been secured under 1930 weather conditions.

The covered beds at Marion, Ohio, were designed on a basis of 0.314 sq. ft. per capita. The plant employs Imhoff tanks and trickling filters. With 0.5 sq. ft. of drying area available per capita during 1929, 20 complete dryings of sludge were obtained in 320 days on two beds that were set aside for the purpose of establishing performance (8). The loading was 15.5 cu. ft. of 92.5 per cent sludge per square foot of drying area and the sludge cake, as removed from the bed, contained 45 to 79 per cent moisture, with an average of 65 per cent. The most satisfactory sludge depth was found to be 10 in. The usual drying period was 7 to 10 days for the whole area, except in winter, when the sludge remained on the beds for 30 days. During the summer months it was possible to dry the sludge to 10 per cent moisture. Much more extensive drying areas are commonly required for undigested than for digested sludges. It is still questionable if sludge from chemical-precipitation or activated-sludge tanks can be dried economically on open beds, except in small plants or in dry climates. Both classes of sludge must be spread in thin layers, in order to drain and dry, and their volume is so great that a relatively large area of drying beds is required and the cost of operation is considerable. Areas of open beds varying from 3 to 7 sq. ft. per capita are mentioned in the literature for drying undigested activated sludge and sludge from chemical-precipitation tanks. An area of 1.0 sq. ft. per capita is provided at Tenafly, N. J., for dewatering activated sludge under glass covers.

The loading in pounds of solids dried and removed per square foot of bed area annually is probably the most useful expression of the work performed by a drying-bed area thus far developed, although it has not been employed widely.

The loadings of sludge-drying beds during operation at various plants, in pounds per square foot annually, are summarized in Table 132. Data are not available to show how efficiently the drying-bed areas were operated in all cases. The loading of 69 lb. per square foot annually at Marion, Ohio, was obtained during a test period of one year, when 10 beds were operated at capacity.

TABLE 132.—DATA ON LOADING OF SLUDGE-DRYING BEDS

Municipality	Year	Type of sludge	Loading, lb. per sq. ft. annually[1]
Open drying beds:			
Akron, Ohio	1930	Imhoff	26.7
	1931	Imhoff	35.4
Baltimore, Md	1930	Separate digestion, primary	25.7
Chicago, Ill., Calumet plant.	1926	Imhoff	27.1
	1927	Imhoff	29.0
Fitchburg, Mass	1922 to 1928	Imhoff	Ave. 41 Max. 51 Min. 32
Flint, Mich	......	Imhoff	28.4
Peoria, Ill	1932	Separate digestion, mixed primary and waste activated	18.5
Rochester, N. Y., Irondequoit plant.	1921-1930	Imhoff	36.6
San Antonio, Tex	1931-1932	Separate digestion, mixed primary and waste activated	24.4
Schenectady, N. Y	1932	Imhoff	32.3
Covered drying beds:			
Dayton, Ohio	1931	Imhoff	52.5
Marion, Ohio	1929	Imhoff	62[2]
Tenafly, N. J	......	Undigested waste activated	35
Toronto, Ont., North Toronto plant.	1932	Separate digestion, mixed primary and waste activated	34.3

[1] Dry-solids basis.

[2] Operation of beds for one year during test.

While the unit loads vary widely, there appears to be rather consistent performance, which may be valuable as a guide in future designs. For drying digested sludge from Imhoff tanks on open beds, the data in

TABLE 133.—SLUDGE-DRYING BED AREA PROVIDED IN DESIGN OF VARIOUS PLANTS

	Location of plant	Type of bed	Area, sq. ft. per capita
Undigested sludge from primary-sedimentation tanks.	Brockton, Mass.	Open	2.0
	Marlborough, Mass.	Open	3.0
	Pittsfield, Mass.	Open	6.0
Imhoff sludge........................	Albany, N. Y.	Open	0.44
	Allentown, Pa.	Open	0.77
	Atlanta, Ga., Peachtree Cr. plant	Open	0.39
	Chicago, Ill., Calumet plant	Open	0.43
	Columbus, Ohio	Open	0.38
	Dallas, Tex.	Open	0.47
	Framingham, Mass.	Open	1.00
	Hamilton, Ont.	Open	0.55
	Lakewood, Ohio	Open	0.57
	Philadelphia, Pa.	Open	0.41
	Rochester, N. Y., Charlotte plant	Open	0.33
	Irondequoit plant	Open	0.38
	Trenton, N. J.	Open	0.58
	Alliance, Ohio	Glass-housed	0.33
	Dayton, Ohio	Glass-housed	0.42
	Marion, Ohio	Glass-housed	0.29
Activated sludge, undigested..........	Tenafly, N. J.	Glass-housed	1.0
Imhoff sludge and digested humus sludge...	Akron, Ohio	Open	0.63
	Fitchburg, Mass.	Open	0.33
	Bloomington, Ill.	Open	0.54
		Glass-housed	0.23
Sludge from primary sedimentation, digested in separate tanks.	Birmingham, Ala.	Open	0.40
	Hartford, Wis.	Open	0.58
	Lincoln, Neb.	Open	0.65
	Boonton, N. J.	Glass-housed	0.57
Primary sludge and activated sludge, digested in separate tanks.	Lancaster, Pa., North plant	Open	1.40
	South plant	Open	0.75
	Peoria, Ill.	Open	0.76
	Elyria, Ohio	Glass-housed	0.33
	Lima, Ohio	Glass-boused	0.50
	Toronto, Ont., No. Toronto plant	Glass-housed	0.60

Table 132 indicate that a safe loading would be 25 to 30 lb. per square foot annually, based on dry solids. During favorable years, beds designed on this basis might have surplus capacity. For drying digested, mixed primary and excess activated sludge on open beds, the data from Peoria, Ill., and San Antonio, Tex., indicate that a loading of 20 to 25 lb. per square foot may be handled annually. Apparently covered beds may be operated with 50 to 100 per cent greater loadings than open beds, depending upon the characteristics of the sludge.

The unit sludge-drying-bed areas provided in the designs of various plants are given in Table 133. The average unit area provided for drying sludge from Imhoff tanks, when solids from preliminary sedimentation only are digested, is 0.52 sq. ft. per capita for the 13 plants with open beds and 0.34 sq. ft. per capita for the three plants with glass-housed beds. These figures are design allowances, which have not proven adequate in all cases. It is known that the areas at Columbus, Rochester, Atlanta and Fitchburg are already inadequate.

Construction Features of Drying Beds.—In Massachusetts, where intermittent sand filtration is much in vogue, sludge is dried on sand beds similar in construction to filter beds and sometimes even on one of the latter reserved for this purpose. This is done, for example, at Marlborough and Pittsfield with primary sludge, at Brockton with primary and humus sludges and at Worcester with Imhoff and humus sludges. At Brockton the beds were formerly used as intermittent sand filters, but 2 ft. of fine sand were replaced by coarse sand in the case of those beds which handle primary sludge.

In the rest of the United States, however, sludge beds of special design are generally employed. The filtering material in these beds is seldom more than 18 in. deep and commonly consists of 2 to 6 in. of sand overlying graded layers of gravel or stone. In Europe cinders and other porous materials have been widely used. The sand and gravel are generally laid on a natural-ground bottom, but puddled clay has also been employed, as at the Brighton plant in Rochester, N. Y. Open-jointed tile underdrains, 3 to 6 in. in diameter, laid in coarse gravel or stone, are generally provided and are commonly placed in trenches toward which the bottom of the bed slopes, as shown in Fig. 199. The spacing of the drains is usually adapted to the width of bed employed and varies from 5 to 20 ft. in practice. The authors have frequently laid underdrains for drying beds in the same manner as explained in Chap. XVIII for intermittent sand filters. According to Gregory and Keefer (9), trenches with side slopes of 1½ on 1 are preferable to vertical-wall trenches, since a tendency exists in the latter for dirt to be washed into the underdrains. Cleanouts sometimes are provided at the ends of underdrains, to permit flushing them with water.

Some of the construction features of drying beds for sludge in American plants are shown in Tables 135 and 136.

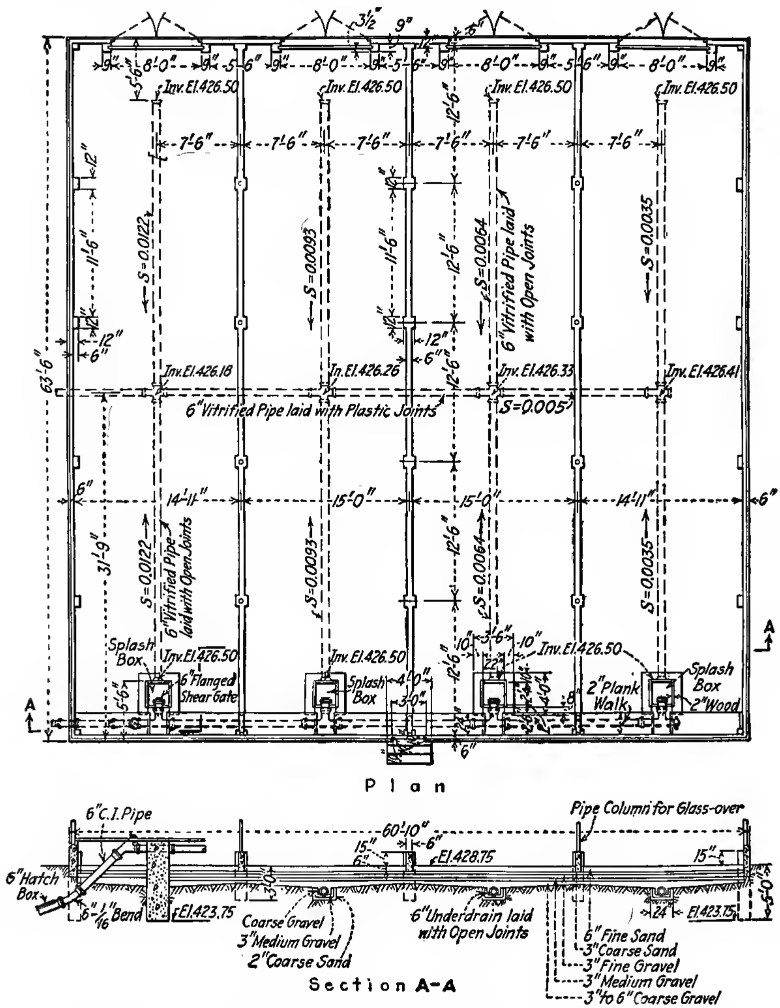


FIG. 199.—Plan and section of sludge-drying beds at Presque Isle, Me.

The drying area is commonly divided into several units or compartments, whose length and width are chosen so as to facilitate uniform distribution of the wet sludge and removal of the dried sludge. In small plants the width is frequently between 15 and 20 ft., a dimension which permits convenient loading of dump cars running on industrial tracks. In large plants employing mechanical sludge-removal devices the width may be much greater.

Beds over 100 ft. in length have been constructed to be fed from a single inlet at one end. In this connection Gregory and Keefer (9) state that " . . . if sludge is 12 in. deep at the inlet end of one of the beds constructed, which are 150 ft. long, it will often be found to be only about 8 in. deep at the other end." The beds constructed at Baltimore in 1923 were made 125 ft. long and 20 ft. wide. With an industrial car 4 ft. wide placed in the center of each compartment, the maximum distance for sludge to be thrown by a man standing 3 ft. from the wall would be 5 ft., which was believed to be readily attainable. The dimensions of sludge beds may be furthermore dictated by the quantities of sludge that can be conveniently drawn at one time.

The sludge-drying area and its subdivisions generally are surrounded by low wood or concrete walls. These are usually constructed of either creosoted cypress planks or precast reinforced-concrete slabs and are supported by precast concrete posts. At Baltimore the partitions built in 1923 were made 20 in. high, projecting 14½ in. above the sand surface.

Sludge may be distributed to drying beds through pipes or open channels. The hydraulics of sludge flow are discussed in Chap. XXVII. The sludge is commonly diverted to the bed to be dosed by shear gates. A splash board or box is generally placed under the outlet.

Covers for Drying Beds.—The idea of covering sludge beds with a glass structure similar to a greenhouse, in order to protect the drying sludge from rain and cold and to permit winter operation, originated at the Cleveland sewage-testing station in 1913 (6). Since then, glass-covered beds have come into relatively wide use, particularly in connection with heated separate sludge-digestion tanks and activated-sludge units. Improvement in the appearance of the plant is an advantage incidental to the use of glass enclosures. Figure 200 shows the glass-covered beds at the North Toronto plant in Toronto and Fig. 201 shows an interior view of one of the houses.

The framework for glass covers is designed to carry the weight of the glass together with a substantial allowance for wind and snow load. A common allowance for the latter is 10 lb. per square foot for steam-heated greenhouses, but for unheated sludge beds a larger allowance usually is made. A value of 30 lb. per square foot is used by some manufacturers of glass-overs. The framework, furthermore, generally is designed to be resistant to the action of the sludge gases. As a rule, the glass rests directly upon bars of galvanized iron or cypress and an acid-resisting paint, such as aluminum, is used on all woodwork and structural steel. The modulus of rupture of single-strength and double-strength window glass and 26-oz. clear sheet glass is indicated to be 7000 lb. per square inch by tests of the U. S. Bureau of Standards. Double-strength window glass, 16 by 24 in. in size, is commonly employed in glass covers.

Experience seems to show that the walls upon which the glass enclosures rest should not rise much above the projected sludge surface but

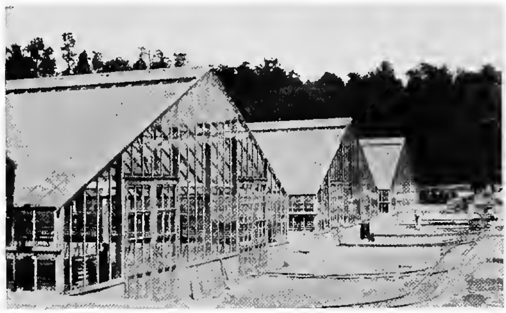


Fig. 200.—Glass-covered sludge-drying beds at North Toronto plant, Toronto, Ont.

should be sufficiently raised above the outside ground level to protect the glass against snow and ice. This will reduce to a minimum the sludge area that lies in the shadow of the walls.

Suitable ventilating facilities are an essential feature of glass-covered beds, since the moisture-laden air in the enclosure needs to be displaced

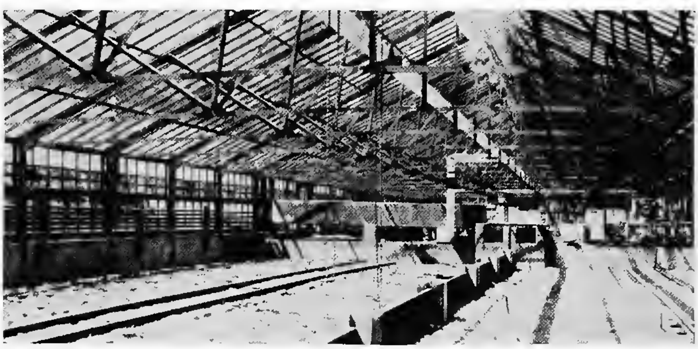


Fig. 201.—Interior of sludge-bed enclosure at North Toronto plant, Toronto, Ont.

as often as possible by air containing less water vapor. The air currents in sludge-bed enclosures have been studied by Laboon (10) at Torrance, Pa. By burning smoke candles outside of such a structure, the air currents were traced as shown in Fig. 202 under varying conditions of natural ventilation. Under the conditions of test number 6, measurements taken with an anemometer indicated about 18 complete air changes in an hour. Laboon draws the following conclusions from these tests, supplemented by information obtained in experiments at Greenwich, Conn., and Lodi, N. J.:

1. It has been demonstrated beyond question that, for a sludge bed enclosure of the simple double pitch roof type, the side wall sash should be hinged at the top to swing inward, thus deflecting the air currents to the areas immediately over the sludge bed. Since the vertical projected area of

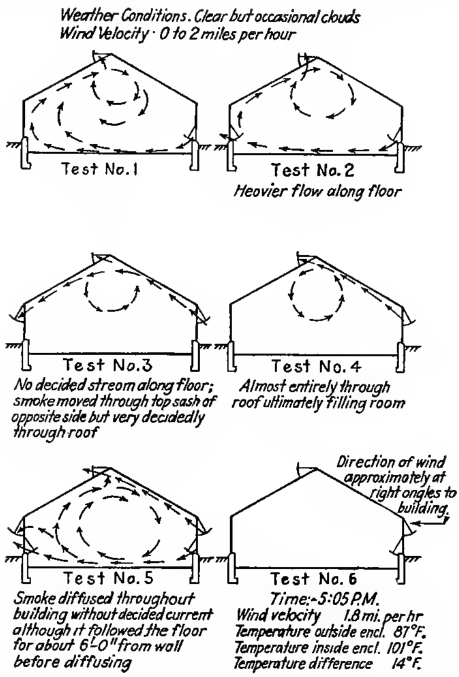


FIG. 202.—Air currents in sludge-bed enclosures, as determined by Laboon.

the opening of the side wall sash is greater in the case of the in-swinging sash than in the case of the usual out-swinging sash, greater advantage can be taken in the former arrangement of the increased ventilation due to wind from the outside entering the enclosure.

2. In sludge beds with outward opening side sash, best circulation of air is obtained with all ridge and side sash open.

3. In sludge beds with inward swinging side sash, best circulation of air should be obtained with all but the windward ridge sash open.

4. Glass side walls should be located as close to the sludge line as possible.

5. If side sash must be placed high above sludge line, improved ventilation is possible with blower fans and air ducts located near the sludge line, as a substitution for or adjunct to inward swinging side sash.

A serious objection to inward-swinging sash is the danger of discharge of large quantities of roof water into the beds, if the sash is not closed during storms. Each line of sash is generally controlled from a single operating point, which may be located either inside the building near the doors or outside the building. Much can be done by intelligent operation of the ventilation system toward securing rapid drying of the sludge and controlling odors, which, if produced, are usually associated with the first few hours after sludge discharge. Facilities for catching the moisture condensing upon the glass and for carrying it away are commonly provided.

Some engineers prefer small roof spans, necessitating sawtooth roof construction, while others employ relatively large spans and thus secure larger air volumes in the enclosure. An objection to sawtooth roofs is the accumulation of snow in troughs between roofs.

Forced ventilation by fans and heating of the air during cold weather have been employed experimentally at the Calumet works of Chicago (2). The best form of hot-air ventilation, whether by fan and heater units or by a central heating system and air ducts, remains to be determined. The glass-covered beds at the North Toronto plant have been heated by steam radiation when required to prevent the sludge from freezing.

The long sides of sludge compartments may be either parallel or perpendicular to those of the enclosure. In the former case the doors are placed in the gable ends of the enclosure, while in the latter case they are constructed in the sides.

The use of a roof or shelter of wood or corrugated iron, to prevent rain from falling on otherwise open beds during the spring and autumn, is reported by Emerson (11), who states that "such construction has been used with success on small beds, the roof being built in panels which can easily be removed in summer when not needed for protection against rain."

At Marion, Ohio, Browne (12) has recorded temperatures of the air outside and inside the glass-overs for the year 1927 as shown in Table 134.

In general, experience seems to show that the difference in maximum temperatures inside and outside the enclosures is about 30°F. and that the difference in minimum temperatures is only a few degrees.

As previously noted, glass-covered beds are employed particularly in connection with heated, separate digestion tanks, in which sludge is produced throughout the year and, therefore, requires drying in winter, unless storage space is provided for the wet sludge. Winter lagoons, or inexpensive earthen basins for winter storage, in some instances, however, have been resorted to during cold weather and sludge-drying operations have been confined to the warmer months.

TABLE 134.—TEMPERATURES OF AIR IN SLUDGE-BED ENCLOSURES AT MARION, OHIO, 1927

Month	Temperature, °F.				Number of fair days
	Outside enclosure		Inside enclosure		
	Max.	Min.	Max.	Min.	
Jan.	50	−2	86	14	14
Feb.	67	16	110	20	12
Mar.	72	17	90	20	12
Apr.	81	28	104	30	14
May	86	35	110	38	15
June	93	43	119	45	19
July	94	49	112	53	23
Aug.	91	45	120	47	21
Sept.	94	40	112	41	23
Oct.	88	34	114	34	23
Nov.	75	23	96	23	10
Dec.	62	4	85	8	16

Removal of Dried Sludge.—After sludge has dried sufficiently to become spadable, it is ready for removal from the drying area and for final disposal. In small plants the beds are generally stripped by hand, using shovels or forks, and the sludge is hauled away in wheelbarrows, dump cars or wagons. In large plants mechanical sludge-stripping may be employed, as at the Calumet and West Side plants in Chicago.

As shown in Tables 135 and 136, extensive use is made of dump cars running on industrial tracks. The gage of these tracks is 24 in. and $8\frac{1}{3}$-lb. rails are employed, attached to rolled steel ties. Steel rocker dump cars of 1-cu. yd. capacity are generally used. The cars are moved either by hand, by draft animals, by gasoline engines or by other power-driven equipment. In practice the tracks are commonly laid permanently in the drying beds. In wide beds more than one track to a bed may be employed. The spur tracks running into the beds are generally connected by switches to a main track laid outside the drying area and leading to the sludge dump or disposal site. Gregory and Keefer (9) have employed portable tracks at Baltimore, Md. Room is made for the tracks by removing from the sand a strip of sludge about 4 ft. wide and throwing the sludge to one side or the other. The temporary track is connected to the main track by a portable switch. Records covering the period from April to November, 1922, indicate that the cost of clearing sludge for the tracks and moving and laying them in beds 125 ft. long and 20 ft. wide was $3.54 for each cleaning, including interest and depreciation on equipment, as against an estimated annual cost of permanent tracks of $26.72 a bed. According to these figures each bed would have to be cleaned 7½ times or more yearly, in order to make the

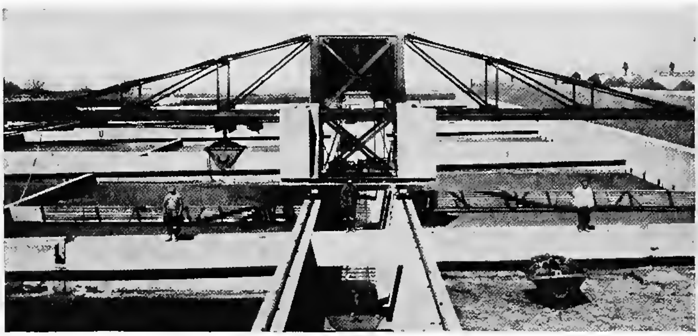


Fig. 203.—Equipment for cleaning sludge beds at Irondequoit plant, Rochester, N. Y.

use of permanent tracks economical at the Baltimore works. In 1921 and 1922 the drying beds received 3.02 and 5.46 fillings of digested sludge, respectively. Gregory and Keefer point out, however, that in small works with insufficient labor for handling portable track, permanent trackage may be more economical.

Fig. 204.—Machine used in stripping sludge from drying beds at Worcester, Mass.

At the Irondequoit plant in Rochester, N. Y., a crane traveling centrally over the whole drying area, as shown in Fig. 203, lifts dump buckets from trucks on industrial tracks beneath the crane and delivers them on to the beds. Later, after they have been filled by hand, the crane replaces them on their trucks.

At a number of plants glass-covered beds are equipped with overhead carriers and the sludge is loaded into ¼- or ½-yd. buckets. At Elyria, Ohio, beds so equipped are 17 ft. wide and 87 ft. long. The bucket carriers are propelled by hand either to a loading platform, whence the sludge may be hauled away by farmers, or to a dump near the works. The structural steel of the carrier may be incorporated in the roof trusses or else left independent. The latter scheme avoids vibrations and possible damage to the enclosure. Ordinarily wheelbarrows or industrial tracks are utilized in covered beds in much the same way as in open ones. In some plants the tracks are elevated above the surface of the bed. Some space is usually lost with this arrangement. Overhead monorails and bucket carriers have also been used with open beds, as at San Bernardino, Cal.

At Worcester, Mass., old intermittent sand filters serve as sludge drying beds for Imhoff-tank sludge and trickling-filter humus. There are 22 beds, each about an acre in area, providing a total drying area of 4.82 sq. ft. per capita. Sludge is stripped from these beds by a

Fig. 205.—Longitudinal type of sludge-stripping machine at West Side works, Chicago.

Barber-Greene loader, shown in Fig. 204, which is driven over the sand surface and fills horse-drawn dump wagons.

At the West Side treatment works in Chicago there are two types of sludge-stripping machine: one type, designed by the staff of the Sanitary District of Chicago, removes sludge from a strip approximately 80 ft. wide, as the machine moves longitudinally along the walls of the bed; the other type, designed by Evers and Sauvage, removes sludge from a strip 12 ft. wide, as the cleaning element moves transversely across the bed.

The Sanitary District type of machine, shown in Fig. 205, is equipped with a set of fork-like prongs or fingers, which pass under the sludge and break up the cake as the machine moves forward. The fingers are ½-in. round curved rods, tapered to a point in 4 in. and spaced 2⅝ in. center to center. The fingers are set at the bottom of an inclined steel apron, up which the broken cakes of sludge are pulled by a continuous drag scraper. At the top of the incline the sludge drops on to a traveling belt, which discharges it into standard-gage railway cars along the side of the bed. The cleaning devices are mounted on a framework of two main lattice girders, suitably cross-braced. The machine is operated by a gasoline-driven generator set, which drives various electric motors. It is designed to have a forward speed for cleaning of 3 to 6 ft. a minute, with a reverse speed of 90 to 100 ft. a minute. The designed capacity is about 4½ cu. yd. of sludge a minute.

In the Evers-Sauvage type of machine, shown in Fig. 206, the cleaning element consists of an adjustable frame carrying an endless chain of

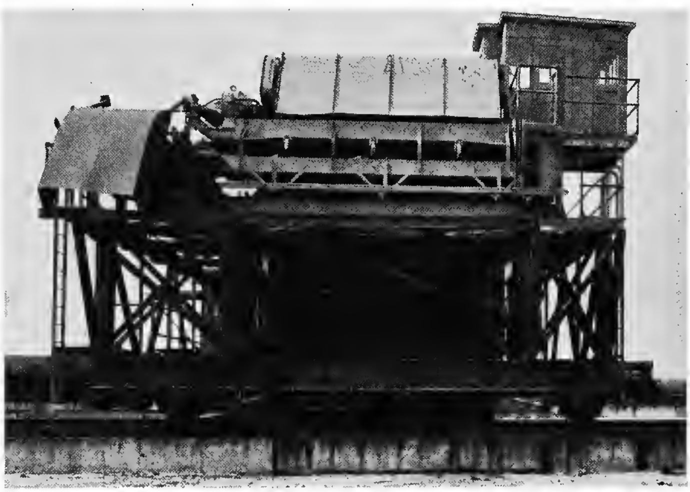


Fig. 206.—Transverse type of sludge-stripping machine at West Side works, Chicago.

buckets. The cutting edges of the buckets, which are equipped with cast-iron teeth, advance into the sludge layer where it rests on the sand and then turn upward, lifting the pieces of sludge and discharging them on to a belt conveyor. After the machine has cleaned a transverse strip of the bed, it moves sideways and makes a new cut. The speed of the cleaning element while removing sludge is 30 to 73 ft. a minute and on its return for a new cut the maximum speed is 173 ft. a minute. The bridge on which the machine is mounted has a designed working speed of 30 ft. a minute, with a speed of 100 ft. a minute on the return trip. This machine, like the Sanitary District type, is driven by a gasoline-motor generator set.

Statistics of Sludge-drying Beds.—A summary of the design characteristics of open sludge-drying beds at various plants is given in Table 135

TABLE 135.—DESIGN CHARACTERISTICS OF OPEN SLUDGE-DRYING BEDS

	Framingham, Mass.	Rochester, N. Y., Charlotte plant	Fitchburg, Mass.	Lincoln, Neb.	Allentown, Pa.	Trenton, N. J.	San Antonin, Tex.	Rochester, N. Y., Irndequoit plant	Akron, Ohin	Philadelphia, Pa., Northeast plant	Baltimore, Md.
Population provided for	10,000	10,000	55,000	80,000	128,000	150,000	181,400	200,000	260,000	300,000	590,000
Date of construction	1924	1921	1913	1923	1928	1927	....	1916	1927	1917	1909–1923
Character of sludge	Imhoff	Imhnff	Imhoff	Separately digested	Imhoff	Imhoff	Separately digested	Imhoff	Imhoff	Imhoff	Separately digested
Source of sludge	Preliminary sed.	Primary sed.	Preliminary sed. and humus	Primary sed.	Preliminary sed.	Primary sed.	Preliminary sed. and activated-sludge	Primary sed.	Preliminary sed.[1]	Primary sed.	Preliminary sed. and humus
Area, sq. ft. per capita	1.0	0.33	0.33	0.65	0.77	0.58	0.44	0.38	0.63	0.41	0.90
Total area of beds, sq. ft.	10,000	3,360	18,300	52,000	98,000	87,200	80,000	76,400	164,000	122,400	530,000
Number of beds	6	4	11	16	68	24	42	40	68	80	142
Length by width, ft.	115 × 14.5	{ 68 × 10 100 × 10	111 × 15	125 × 26	80 × 18	182 × 20	100 × 20	44 × 43	160 × 15	85 × 18	Various
Depth of bed, in.	12	14	....	12	22	14	12	19	18–24	12–15	12–15
Filtering material	Sand, 1 in. Fine stone, 1 in. Coarse stone, 10 in.	Sand, 4 in. Sand and gravel, 4 in. Broken stone, 6 in.	Natural sand deposit	Sand, 2 in. Gravel, 10 in.	Sand, 6 in. Gravel: ⅛–¼ in., 3 in. ¼–¾ in., 3 in. ¾–1½ in., 3 in. 1½–2½ in., 7 in.	Sand, 2 in. Gravel, 1/16–2½ in., 12 in.	Sand, 4 in. Gravel: ¾, 4 in. Coarse, 4 in.	Fine sand, 2 in. Coarse sand, 3 in. Fine gravel, 8 in. Broken stone, 6 in.	Sand, 6 in. Gravel: ⅛–¼ in., 3 in. ¼–¾ in., 3 in. ¾–1½ in., 3 in. 1½–2½ in., 3–9 in.	Sand, 2 in. Gravel: 1/16–¼ in., 2 in. ¼–¾ in., 4 in. ¾–1½ in., 4–7 in.	Sand, 4 in. Gravel: ⅛–⅜ in., 2 in. ⅜–1 in., 2 in. 1–2½ in., 4–7 in.
Underdrains:											
Diameter, in.	4	6	None	6	6	....	6	3	6	3	4
Spacing, ft.	6	7	....	20	18	....	20	8	15	9	20–28
Grade, per cent	....	2.08	....	....	0.35	....	....	2.08	0.35	1.0	0.5–1.0
Min. depth to invert, in.	....	....	....	....	30	....	....	....	30		
Partitions	Plank	Concrete	Plank	Plank	Plank	....	Earth	Concrete	Plank	Concrete	Plank and concrete
Max. depth of dose provided for, in.	12	....	12	....	18	....	....	15	15		
Sludge-removal equipment	....	....	Industrial track and cars	Industrial track and cars	Industrial track and cars	Industrial track and cars	Movable runners and trucks	Gantry crane, industrial track and cars	Industrial track and cars	Conveyor	Industrial track and cars

[1] Humus sludge may be discharged into Imhoff tanks.

TABLE 136.—DESIGN CHARACTERISTICS OF GLASS-COVERED SLUDGE-DRYING BEDS

	Alliance, Ohio	Elyria, Ohio	Marion, Ohio	Bloomington, Ill.	Toronto, Ont., No. Toronto plant	Dayton, Ohio
Population provided for	36,000	36,000	40,000	54,000	100,000	262,000
Date of construction	1928	1929	1924	1928	1929, 1933	1928
Character of sludge	Imhoff	Separately digested	Imhoff	Imhoff	Separately digested	Imhoff
Source of sludge	Preliminary sed.	Preliminary sed. and activated-sludge	Preliminary sed. and humus	Preliminary sed. and humus	Preliminary sed. and activated-sludge	Primary sed.
Area, sq. ft. per capita	0.33	0.33	0.31	0.77[1]	0.60	0.42
Total area of beds, sq. ft.	12,150	12,000	12,600	41,800	60,000	109,000
Number of houses	1	2	1	1	10	11
Length by width of house, ft	162 × 75	175 × 36	170 × 74	188 × 66	160 × 40	164 × 61
Number of beds	9	8	10	20	40	22
Length by width of bed, ft	75 × 18	87 × 17	74 × 17	94 × 22	77 × 19.5	163.5 × 30.4
Depth of bed, in	....	14	14–17	15	18–24	18
Filtering material	Sand, 12 in. Gravel	Sand, 12 in. Gravel, 2 in.	Sand, 12–15 in. Gravel, ¼ in, 2 in.	Sand and gravel	Sand, 6 in. Gravel: 1/16 in.–¼ in., 3 in. ¼ in.–½ in., 3 in. ½ in.–1 in., 3 in. 1–1½ in., 3–9 in.	Sand, 6 in. Gravel: ⅛ in.–¼ in, 3 in. ¼ in.–¾ in., 3 in. ¾ in.–1½ in., 3 in. 1½ in–2½ in., 3 in.
Underdrains:						
Diameter, in	6	....	6	....	6	6
Spacing, ft	18	17	14.6	....	19.5	15
Grade, per cent	....	....	1.0	....	1.0	0.35
Min. depth to invert, in	....	....		....	30	24
Partitions	....	....	....	Concrete	Plank	Plank
Max. depth of dose provided for, in	....	....	....	....	15	15
Sludge-removal equipment	Industrial track and cars	Bucket carriers on monorail	Industrial track and cars	....	Industrial track and cars	Industrial track and cars

[1] Comprising 0.23 sq. ft. per capita under glass cover and 0.54 sq. ft. open.

and a similar tabulation with respect to glass-covered beds is presented in Table 136.

Operation of Sludge-drying Beds.—Operation of drying beds which handle Imhoff sludge at the Irondequoit plant in Rochester, N. Y., has been described by Ryan as follows (13):

> The drying season usually extends from Apr. 15 to Nov. 15, the best drying weather occurring during the months of June, July and August. The number of removals possible varies from six to ten per season, depending on the precipitation. Also, the depth of wet sludge drawn to the beds is varied during the year. Eight inches appears to be the best depth from April to June, and September to November. During the midsummer months, the depth is increased to 10 in. and the last drawing of the year is made about 20 in., inasmuch as the sludge remains on the beds during the winter.
>
> With the adverse drying conditions, it is necessary to lagoon a portion of the sludge produced. . . At Rochester, the drying time is about 12 days in summer and 20 to 30 days during the spring and fall months.

A summary of operating data on sludge drying at the Irondequoit plant for the years 1924 to 1929, inclusive, is given in Table 137.

TABLE 137.—OPERATING DATA ON DISPOSAL OF IMHOFF SLUDGE, IRONDEQUOIT TREATMENT PLANT, ROCHESTER, N. Y.

	1924	1925	1926	1927	1928	1929
Wet sludge drawn from						
Imhoff tanks, cu. yd.	26,696	25,056	21,710	22,935	28,932	26,436
Drawn on drying beds, cu. yd.	24,174	22,230	14,400	19,584	18,294	24,096
Lagooned, cu. yd.	2,522	2,826	7,310	3,351	10,638	2,340
Moisture, per cent	90.0	90.9	90.8	91.9	93.5	92.7
Nitrogen, per cent, dry basis	1.99	2.05	1.99	2.05	2.21	2.33
Volatile solids, per cent, dry basis	44.2	44.7	47.2	43.1	48.2	47.9
Ether-soluble matter, per cent, dry basis	9.7	12.7	11.4	13.4	12.8	12.5
Dry sludge removed from beds, cu. yd.	8,294	6,923	5,831	7,648	5,612	6,904
Moisture, per cent	57.0	57.2	56.0	55.7	56.5	57.4
Number of removals	11	8.5	6	10	8	10
Reduction in volume on beds, per cent	66	69	59	61	69	71
Revenue from sale of sludge	$961.00	$391.50	$337.40	$253.00	$423.50	$640.50

The operation of sludge-drying beds which dewater digested sludge from Imhoff tanks at Akron, Ohio, has been summarized by Backherms (14) as shown in the table at the top of page 736 for the years 1930 and 1931.

A summary of operating data for the Fitchburg drying-bed area during the years 1923 to 1927, inclusive, as compiled from the annual reports of the Commissioner of Public Works, is given in Table 138.

	1930	1931
Sludge applied, cu. yd.	27,062	46,953
Sludge applied, tons, dry-solid basis	2,170	2,890
Moisture in wet sludge, per cent	91.3	92.9
Moisture in dewatered sludge, per cent	54.6	47.6
Average number of cleanings	5.5	10.3
Average time on beds, days	30	30
Loading on beds, lb., dry basis, per sq. ft. annually	26.7	35.4
Sludge removed, cu. yd.	7,000	13,325
Operating cost:		
Labor	$3,814	$6,435
Materials and supplies	258	299
Total	$4,072	$6,734
Per cu. yd. of sludge removed	$0.58	$0.50

The drying beds receive primary sludge and humus sludge after digestion in Imhoff tanks. The sludge which is not dried on the beds is discharged into a sludge lagoon.

TABLE 138.—DATA ON OPERATION OF SLUDGE-DRYING BEDS AT FITCHBURG, MASS.

	1923	1924	1925	1926	1927
Tributary population	38,900	38,900	40,000	40,000	40,000
Average number of times dosed	8.09	6.36	5.36	3	3.46
Volume of digested sludge applied, gal.	1,003,899	836,168	813,347	465,881	570,445
Depth of application, in.	10.94	11.89	11.04		
Percentage of sludge pumped, dried on beds	86	100	82	58.2	65.6
Specific gravity of sludge	1.024	1.061	1.053	1.049	1.052
Total solids, per cent	8.41	14.22	13.06	14.19	13.68
Dry solids applied, tons	365.62	469.82	436.11	291.06	345.21
Organic matter, per cent	51.94	38.72	35.78	35.36	33.70
Number of times beds cleaned	5.36	4.82	4.00	2.27	3.09
Volume of sludge removed, cu. yd.	1193.26	1498.29	1262.50	844.75	992.25
Total solids, per cent	48.28	58.24	54.64	50.03	55.07
Cost of operation and maintenance[1]	$1,079.49	$1,412.14	$904.43	$779.45	$1,601.53
Cost of cleaning:					
Total	$663.29	$881.64	....	....	$1,047.68
Per cu. yd.	$0.56	$0.59	....	....	$1.05
Area, sq. ft. per capita served	0.46	0.46	0.45	0.45	0.45

[1] Including administration, laboratory, care of grounds and miscellaneous expenditures not directly chargeable to any one account, divided pro-rata among all accounts.

The operation of the glass-covered sludge-drying beds at Dayton, Ohio, treating Imhoff sludge, is summarized for 1931 by Tatlock (15) as follows:

	Sludge added to beds	Sludge removed from beds
Total volume, cu. ft.	1,388,500	297,400
Moisture, per cent	95.2	63.0
Specific gravity	1.009	0.732
Total solids, dry basis, tons	2,515.6	2,108.7
Volatile solids, per cent	58	55
Ether-soluble matter, per cent	12	9
Total nitrogen, per cent	3.40	3.11
Depth, in.	9.0	1.6
Loading, lb. of solids per sq. ft. annually	52.5	
Cost of removal: Total	....	$4,110.30
Per cu. yd	....	$0.37
Per ton of solids, dry basis	....	$1.95

TABLE 139.—RESULTS OF OPERATION OF GLASS-COVERED SLUDGE-DRYING BEDS AT NORTH TORONTO PLANT, TORONTO, ONT.

Month	Sludge as applied to beds				Days until removed	Sludge as removed from beds			
	Average depth, in.	Cu. ft. daily	Moisture, per cent	Loading, dry basis, lb. daily		Average depth, in.	Moisture, per cent	Shrinkage, per cent	Yield, dry basis, lb. per sq. ft. daily
June, 1930	12.5	1321	91.7	7093	13.3	6.3	74.5	50.2	0.394
July	11.8	1106	91.7	4997	12.3	5.6	77.9	53.2	0.278
Aug	10.1	791	92.6	3823	15.3	5.2	72.0	48.5	0.212
Sept	8.1	856	93.4	3643	13.1	3.9	75.4	52.1	0.202
Oct	8.6	715	93.9	2377	13.6	4.1	82.3	54.9	0.132
Nov	7.7	846	95.4	2370	10.3	3.6	84.7	52.9	0.132
Dec	7.5	701	94.8	2315	14.0	3.2	85.0	45.0	0.129
Jan., 1931	6.9	406	95.3	1209	17.6	3.4	82.6	50.8	0.067
Feb	9.3	322	94.0	1236	22.6	5.2	81.3	51.5	0.069
Mar	8.8	658	93.3	2810	16.4	5.1	79.9	42.9	0.156
Apr	9.1	827	92.4	4060	16.9	5.6	78.3	38.9	0.226
May	10.3	1095	94.4	3916	14.3	5.2	81.7	45.1	0.218
Average	9.2	804	93.6	3321	15.0	4.7	79.6	48.8	0.185

The results of operation of the glass-covered beds, handling digested primary sludge and activated sludge, at the North Toronto plant for the year ending May 31, 1931, are given in Table 139.

At San Antonio, Tex., open drying beds are utilized for drying digested activated sludge and primary sludge. Data on the operation of this drying-bed area are given in Table 140 (16). In this table the daily average yield of the beds has been computed per square foot for each month, based upon the quantity of dry solids removed from the entire drying-bed area in that month.

Another method of computing the yield, which gives, perhaps, a more accurate measure of the work performed by the drying area in any month, takes into account the number of months or parts of months during which sludge has remained on the beds before rémoval. For example, if it is assumed that sludge applied to the beds in July is removed in September, instead of crediting to the month of September all the sludge removed in that month, it may be more accurate to credit a large portion of the drying of that sludge to the month of August and another portion to July, the respective portions being based on the percentage of the total drying period falling in each month. If the yield is computed in this manner, as has been done in Table 139, the results may vary considerably from those obtained by the method used in Table 140.

TABLE 140.—DATA ON OPERATION OF SLUDGE-DRYING BEDS AT SAN ANTONIO, TEX.

Month	Average temperature, °F.	Total rainfall, in.	Number of beds		Moisture, per cent		Depth, in.		Drying time, days	Yield, dry basis	
			Filled	Cleaned	Wet	Dewatered	Wet	Dewatered		Tons	Lb. per sq. ft. daily[1]
Apr., 1931	64.2	2.28	20	32	95.03	71.0	9.0	4.0	13.6	78.0	0.062
May	72.5	1.36	35	30	96.21	72.4	9.0	4.0	14.8	64.2	0.049
June	81.2	3.10	37	25	91.12	63.3	9.0	4.0	11.0	72.6	0.058
July	82.6	3.09	54	37	95.93	68.1	9.0	4.0	14.0	109.7	0.084
Aug	82.0	0.30	53	46	96.28	48.1	9.0	4.0	15.7	112.0	0.086
Sept	83.2	0.01	73	81	96.55	37.8	9.0	4.0	14.3	174.5	0.138
Oct	78.0	0.75	66	58	96.73	53.6	10.0	4.5	13.7	127.0	0.097
Nov	66.6	0.72	47	49	96.70	51.6	10.1	4.0	21.2	103.7	0.082
Dec	55.2	2.79	17	14	96.69	59.2	9.8	4.6	47.7	27.3	0.021
Jan., 1932	55.4	3.30	22	27	96.72	64.4	11.4	4.5	53.2	52.2	0.040
Feb	63.0	1.86	37	35	96.28	65.8	9.9	4.2	33.3	65.5	0.054
Mar	57.4	1.05	10	18	96.00	45.1	6.2	4.5	34.2	38.9	0.030
Total for yr.	....	20.61	471	452	.....	....	....	...	....	1025.6	
Average month	70.1	1.72	39.2	37.7	95.85	58.4	9.3	4.2	23.9	85.5	0.067

[1] Based on solids removed from beds and a total area of 84,000 sq. ft.

The drying area provided at Brockton, Mass., for primary and humus sludges is about 2 sq. ft. per capita, a total area of about 3 acres being reserved for sludge drying.

The operation of the Brockton sludge-drying beds may be summarized from the annual report of the Sewerage Commissioners. During 1929, the sludge removed from the primary tanks amounted to a total of 2,430,000 gal., equivalent to 2180 gal. of sludge per million gallons of sewage treated. The sludge contained 93.78 per cent moisture and 6.22 per cent solids, of which 84 per cent were volatile. The sludge after drying on sand beds contained 40.69 per cent moisture and 59.31 per cent solids, of which 23.3 per cent were volatile. The dried sludge removed from the beds amounted to 3165 cu. yd., a reduction of 73.5 per cent. The quantity of sludge removed from the humus tanks was 539,300 gal., or 585 gal. per million gallons of sewage treated in the trickling filters. This sludge had a moisture content of 93.03 per cent and the volatile solids amounted to 60 per cent on a dry basis. The humus sludge was dried on sand beds, the average percentage moisture of the dried sludge being 19.76 and the volatile solids comprising about 20 per cent on a dry basis. About 670 cu. yd. of dried sludge were removed from the beds. The sludge was largely disposed of by filling in swampy areas, although some of it was given to farmers for use as a fertilizer. The cost of removing sludge from the beds was $2900, equivalent to $0.76 per cubic yard of dried sludge removed.

Donaldson (17) asserts that the addition of 1 to 2 lb. of copper sulfate to each 1000 gal. of sludge assists materially in controlling odors from undigested activated sludge at Tenafly, N. J.

Costs of Construction, Maintenance and Operation of Sludge-drying Beds.—The construction cost of open sludge-drying beds is affected materially by the excavation necessary to level off the tract, the underdrainage system required, the depth of filtering material provided, the availability of filtering materials and the equipment provided for sludge removal. Based on prices prevailing from 1926 to 1929, inclusive, the cost for open beds varies from $30,000 to $50,000 an acre. Industrial locomotives cost $2,500 to $3,500, industrial cars $150 to $200, and industrial track about $1.00 a linear foot. The cost of sand and gravel may amount to 25 to 35 per cent of the total cost of open beds. This material may cost $2 to $5 a cubic yard in place.

The cost of standard greenhouse construction for sludge beds averages about $1.60 a square foot. The average total cost of covered beds, including trackage, is about $2.75 a square foot.

The principal maintenance cost of sludge beds is in the replacement of sand. At ½ in. a year and $5.00 a cubic yard, this cost would be about $350 an acre annually. No information is available on the cost of maintenance of sludge-bed glass-overs.

The main item in the operating cost of sludge beds is the removal of the sludge. For large plants with mechanical sludge-removal equipment, the cost of sludge removal may be $0.25 to $0.50 a cubic yard. Where sludge is removed by hand labor and deposited in industrial cars, the cost may be $0.50 to $1.00 a cubic yard. The sale of sludge as fertilizer, which may effect a reduction in the cost of sludge treatment, is discussed in Chap. XXVII.

Irving (18) has studied the costs of operating sludge-drying beds at a number of plants and reports that they vary from $1.54 to $4.21 per ton of sludge, on a dry basis. He estimates the total costs, including fixed charges, at $4.10 to $9.84 per ton of sludge, on a dry basis. With 60 per cent moisture in the dried sludge and a weight of 1600 lb. per cubic yard, the operating costs, based on Irving's figures, would be $0.49 to $1.35 per cubic yard of sludge removed from the beds, excluding fixed charges.

Bibliography

1. Ann. Rept., Supt. of Sewers, Worcester, Mass., 1912.
2. Goodman, A. H., and C. E. Wheeler, Jr.: *Sewage Works Jour.*, 1929; **1,** 444.
3. Donaldson, Wellington, and A. L. Kurtz: *Sewage Works Jour.*, 1929; **1,** 333.
4. *Eng. News-Rec.*, 1923; **91,** 888.
5. Keefer, C. E., and Herman Kratz, Jr.: *Sewage Works Jour.*, 1929; **1,** 120.
6. *Pub. Health Bull.* 132, U. S. Pub. Health Service, 1923.
7. Browne, F. G., and F. W. Jones: *Proc.*, N. J. Sewage Works Assoc., 1926, p. 13.
8. Browne, F. G., and F. W. Jones: *Public Works*, 1930; **61,** no. 7, p. 31.
9. Gregory, J. H., and C. E. Keefer: *Eng. News-Rec.*, 1924; **93,** 13.
10. Laboon, J. F.: *Public Works*, 1930; **61,** no. 1, 33.
11. Emerson, C. A., Jr.: *Sewage Works Jour.*, 1929; **1,** 231.
12. Browne, F. G.: *Public Works*, 1929; **60,** 449.
13. Ryan, W. A.: *Am. City*, 1930; **43,** no. 1, p. 87.
14. Backherms, A. B.: *Munic. Sanitation*, 1933; **4,** 229.
15. Tatlock, M. W.: *Sewage Works Jour.*, 1932; **4,** 519.
16. Stanley, W. S.: *Water Works and Sewerage*, 1933; **80,** 123.
17. Donaldson, Wellington: *Munic. Sanitation*, 1932; **3,** 116.
18. Irving, D. F.: *Water Works and Sewerage*, 1933; **80,** 403.

CHAPTER XXIX

MECHANICAL DEWATERING AND DRYING OF SEWAGE SLUDGE

Although sludge is most commonly dried on sand beds, other methods of dewatering sometimes are used, such as filtering with or without additional pressure or vacuum, dewatering in centrifuges, flotation and spray drying. After sludge has been partially dewatered, either on drying beds or by mechanical means, its moisture content is sometimes reduced further by the use of heat.

Conditioning Sludge for Mechanical Dewatering.—Before taking up the dewatering of sludge by filtering it through cloth, consideration will be given to some of the factors entering into the problem. These are illustrated by investigations conducted at Milwaukee, Wis., Chicago, Ill., Pasadena, Cal., and elsewhere, relative to the prefiltration treatment of sludge for the purpose of facilitating filtration.

The objects of *sludge conditioning* are to change the subdivision of sludge particles so as to cause the smallest particles to unite and form aggregates or large particles, that will not penetrate the interstices of the filtering medium and clog it and thus, as well as in other ways, to increase the drainability of the sludge. This clogging is known as "blinding" of the filter cloth. From a consideration of the principles of colloid chemistry there are several methods of sludge conditioning that tend to improve filtration, including changing the H ion concentration of the sludge, adding chemical coagulants to the sludge, heating the conditioned sludge and adding physical absorbents to the sludge.

Wilson, Copeland and Heisig (1) reported on sludge dewatering as follows:

> Careful studies of the effect of hydrogen ion concentration upon the rate of filtration of sludges obtained throughout the year made it evident that hydrogen ion control, tremendously important in itself, is not the only factor vital to the efficient operation of the plant. Sludge obtained during February, when the temperature of the raw sewage was 12°, required eighteen times as long to filter as sludge obtained during August, when the temperature had risen to 21°, even though each sample was brought to its optimum acidity before filtering.

The relative condition of the sludge at Milwaukee for filtering during the first eight months of 1921 is shown in Fig. 207. The curves cannot be used as a measure of the effect of temperature alone on the condition of the sludge for filtering, because of the changes in character of the sludge during the time of the tests, but they are indicative of the effect of temperature.

The use of chemicals for conditioning activated sludge prior to filtration was tried as early as 1917 at the Stock Yards testing station in Chicago (2). There the addition of sulfuric acid to the sludge was found to increase materially the yield of press cake. In 1923 alum was used for the same purpose in tests at the Des Plaines treatment works in Chicago.

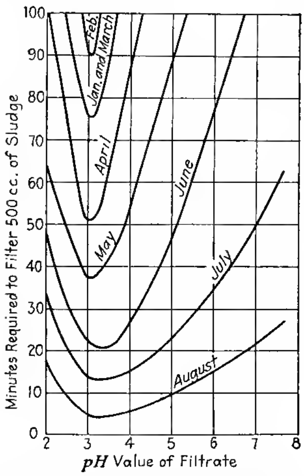


FIG. 207.—Changes in condition of activated sludge at Milwaukee from January to August, 1921.

Experience with the addition of chemical coagulants to the sludge at Milwaukee has been summarized by Hatton, Kadish and Heisig (3) as follows:

> Of the several conditioners which have been investigated, the following appear to give the best results and are most satisfactory from both a practical and economic standpoint:
>
> Sulfuric acid
> Aluminum sulfate
> Ferric chloride—sulfate solution
> Ferric chloride—commercial
> Sulfuric acid and heat.
>
> Each of these chemicals has been applied alone, or combinations have been used to meet the varied characteristics of the sludge. Certain definite optimum pH values have been established for each conditioner. These values must be maintained in order to get the greatest filtering efficiencies. The following optimum pH values maintain throughout the several yearly seasons:

Conditioner	Optimum pH value
Sulfuric acid	3.3
Aluminum sulfate	4.4
Ferric chloride—sulfate solution	3.3
Sulfuric acid—aluminum sulfate	3.9 to 4.1
Sulfuric acid—ferric chloride—sulfate	3.3

> Considerable work was also done by adding sulfuric acid to the raw cold sludge, then heating the mixture to from 120 to 180°F. Very satisfactory results were thus obtained in the laboratory and on the small preliminary testing unit, and from these results the main plant was provided with equipment to thus condition the sludge during the season of low temperatures.

This equipment failed to function satisfactorily, and after considerable experimentation upon other types of heating equipment, the idea of heating the sludge was abandoned as impracticable and uneconomical. In substitution for this, equipment was installed for the preparation and use of ferric chloride—sulfate solution—for sludge conditioning.

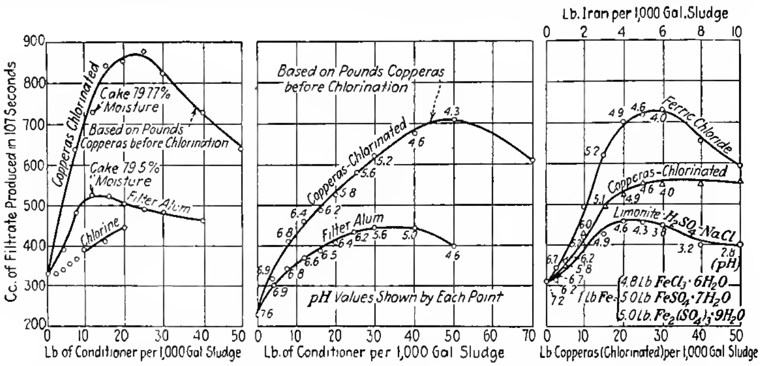
Recently many experiments have been conducted with commercial ferric chloride. Results have indicated filter-cake yields of 50 per cent in excess of any obtained with the prepared ferric chloride—ferric sulfate solution. Experiments have been entirely confined to the laboratory experimental filters, which are miniatures of the large filters; however, enough investigations have been made to insure the success of its use as a satisfactory sludge conditioner. It is hoped that in the near future commercial ferric chloride can be manufactured at such a price as to make it possible to use it in the Milwaukee plant. . . .

Yields from the filters vary considerably throughout the several seasons and vary for each of the conditioners employed. Using the several conditioners described in the foregoing, the yield has been maintained at a minimum of five tons per filter per day, based on weight of dry material, under the worst conditions of filtration. As the surface of each filter contains 500 sq. ft., the minimum production amounts to .014 lb. of dry material per minute per square foot of filter cloth.

It has always been the practice in the Milwaukee laboratory to record the filter yield on the basis of the weight of the material produced after passing the drier. This overcomes recording the different moistures in the cake and comparing on that basis.

Conditioner used	Yield in dry tons per filter per day	
	Maximum	Minimum
Sulfuric acid, alone	8	5
Aluminum sulfate, alone	10	5
Ferric chloride-sulfate	15 plus	5
Ferric chloride—commercial: experimental filter results	20	5

The fact that ferric salts, particularly ferric chloride, greatly facilitate filtration appears to have been discovered by Palmer. From comparative filtration tests in the laboratory, using equal weights of conditioning chemicals, 10 lb. per 1000 gal. of sludge in each case, Mohlman and Palmer (4) concluded that the order of effectiveness of the different substances employed was as follows: ferric chloride, ferric nitrate, ferric sulfate, aluminum chloride, aluminum nitrate and aluminum sulfate. Ferric chloride yielded the best results. Later tests indicated that with an equivalent weight of iron or aluminum the chlorides filtered most rapidly, while the nitrates came next and the sulfates were slowest in filtration. In all instances the ferric salts were superior to the corresponding aluminum salts, while the best results obtained with any of the aluminum salts, namely, chloride, were but slightly better than the poorest of the ferric salts, ferric sulfate. Further tests with a small filter leaf, covered with the same material as used on an Oliver filter, yielded the results shown in Fig. 208. Plant-scale tests at the Calumet works with an Oliver filter showed an increased yield of 60 to 100 per cent for iron salts over aluminum salts. With either alum or iron salts the



(a) (b) (c)

FIG. 208.—Results with various coagulants in conditioning activated sludge for filtration.[1] (a) Calumet plant 2/14/26. Tank 26. Moisture, 97.75 per cent. Using copperas, filter alum and chlorine. (b) Des Plaines plant, 2/16/26. Tank 3. Moisture, 97.82 per cent. Chlorinated copperas and filter alum. (c) Calumet plant, 7/16/26. Tank 26. Moisture, 96.80 per cent. Ferric chloride, chlorinated copperas and ferric sulfate plus salt.

moisture in the cake was practically the same. The average yield during 1926 for all iron runs was 0.032 lb. of cake per square foot a minute, corrected to 10 per cent moisture, as compared with a corresponding yield of 0.020 lb. with alum.

The value of ferric chloride as a conditioning agent was subsequently substantiated at Milwaukee, where it is used to condition winter sludge for filtering (5). The effect of various chemicals upon the rate of filtration of activated sludge as determined at Milwaukee is shown in Fig. 209. According to Cramer and Wilson, "where it is necessary to multiply the relative filtering efficiency of the sludge by 5 or less, sulfuric acid is the cheapest conditioning agent; for greater values ferric chloride is cheapest."

At Pasadena, Cal., alum has been employed together with a physical adsorbent in the form of diatomaceous earth in conditioning activated

[1] Tests made with filter leaf, ⅛ sq. ft. in area, using a vacuum of 25 in. of mercury.

sludge for filtration (6). The dosage reported was about 11 lb. per 1000 gal. of sludge, or 15 per cent by weight, of diatomaceous silica on the basis of the dry finished product, and 8 to 12 lb. of alum per 1000 gal. of sludge. Other so-called "filter aids," such as oil-gas carbon, spent oil shale and coke breeze, were recorded as yielding results inferior to those obtained with diatomaceous earth. In the absence of diatomaceous earth the filter cake produced by alum treatment alone contained 5 per cent more moisture.

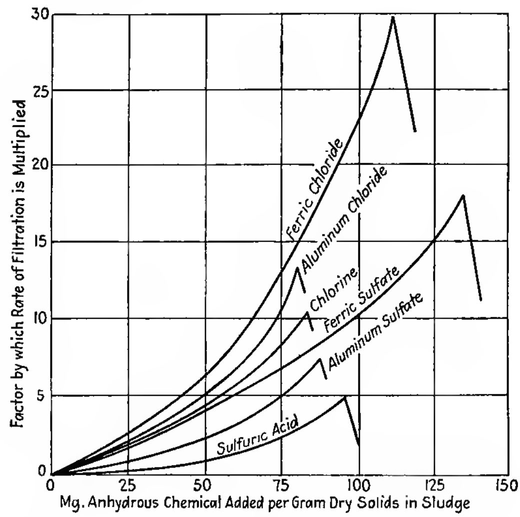


FIG. 209.—Effect of different chemicals upon the rate of filtration of activated sludge.

Extensive experiments on the conditioning and dewatering of activated sludge have been conducted at Houston, Tex. These are summarized by Fugate and Stanley (7) as follows:

The selection of the conditioner seems to be one of the most important factors of the whole process of sludge dewatering. Sulfur dioxide gas and sulfuric acid gave indifferent results at Houston, while aluminum sulfate and chlorinated copperas were efficient and the choice between the two is only a question of price. It was consequently decided to install suitable equipment for chlorinating copperas, as well as storage facilities for handling aluminum sulfate.

The pH at which Houston activated sludge filtered best was as follows:

1. For sulfur dioxide, pH between 3.2–3.6.
2. For sulfuric acid, pH 3.4.
3. For aluminum sulfate, between pH 4.4 and 4.8.
4. For chlorinated copperas, pH 5.4 was the optimum point.

It was found that at a high pH "mudding" of the cloth occurred. This was caused by wet sludge smearing over the surface of the cloth and stopping filtration. On the other hand, low pH caused white spotting. This was a peculiar condition of cloth when the conditioner used acted as a mordant and fixed certain colloidal substances in the pores of the cloth, thus checking the rate of filtration. Both conditions were quite detrimental to the successful operation of the filter.

Acidifying activated sludge with phosphoric acid, which is of fertilizing value, proved to be uneconomical at Milwaukee because most of the acid passed off in the filtrate (8).

Lime was first employed many years ago in connection with the filter-pressing of sludge from chemical-precipitation and septic tanks. It was observed that well-limed sludge pressed more readily than unlimed or poorly limed sludge. At Providence, R. I., and Worcester, Mass., 20 to 30 lb. of lime per 1000 gal. of chemical-precipitation sludge were added for many years and as much as 100 lb. has been required for sludge from septic tanks.

During the early experiments in dewatering activated sludge in Worthington and Simplex plate presses at Milwaukee, Copeland (9) found that lime accelerated the process of pressing and, therefore, reduced the time consumed. However, the lime added mineral matter and decreased the value of the sludge by driving off nitrogen.

In July and August, 1932, Keefer and Cromwell (10) conducted tests on the dewatering of digested sludge with an Oliver filter at Baltimore. One series of tests was made to determine the capacity of the filter when using different quantities of ferric chloride as a coagulant. As a result of these tests, it was estimated that, for the type of sludge handled, 8.0 lb. of ferric chloride would be needed for each 100 lb. of dry solids, with a production of 14 lb. of dry solids an hour per square foot of filter area. Another group of tests was made in order to find out to what extent the quantity of ferric chloride could be reduced if lime also were used as a coagulant. In these tests, when using about 5¾ lb. of ferric chloride and 14 lb. of lime to 100 lb. of dry solids, a production of about 13 or 14 lb. of dry sludge solids an hour per square foot of filter area was obtained. The estimated cost of dewatering sludge similar to that filtered during the tests was $5.00 per ton of dry solids, when using ferric chloride alone as a coagulant, in comparison with $5.50, when using both ferric chloride and lime.

In the past few years efforts have been made to find some augmenting or cheapening agent to use with ferric chloride in conditioning sludge. From laboratory experiments made during 1931 and 1932 at Chicago, Mohlman and Edwards (11) have drawn the following conclusions:

Substantial reduction was obtained in the amount of ferric chloride required by fresh sludge, by addition of a small amount, 1 per cent or less, of sodium dichromate. . . . A large saving in ferric chloride on digested sludge was indicated by the use of lime, following ferric chloride treatment, in amounts sufficient to raise the pH to 9.0 or more. Paper pulp seemed to have some value as a filter aid if used in large amounts, equal to 50 per cent or more of the weight of dry sludge solids. Moderate heating, from 72 to 90°F., before addition of ferric chloride improved filtration of fresh solids quite noticeably.

With reference to conditioning sludge for filtration Edwards (12) states that,

. . . after a great deal of experimentation, most plants have adopted the use of ferric chloride as the best of the conditioners, especially since the price has been materially reduced. Although in 1926 the cost of ferric chloride, dry basis, was $180.00 to $200.00 per ton, Chicago and Milwaukee are now paying about $40.00, delivered. Pasadena is using ferric chloride with diatomaceous earth and Charlotte, ferric sulfate. . . . The York Township (Ontario) plant has been using paper pulp as a filter aid both with and without lime and the plant at Hagerstown, Maryland, paper pulp and lime for conditioning a mixture of activated sludge, fresh solids and fine screenings.

Elutriation of Sludge.—Experiments have recently been conducted by Genter (13) at Baltimore on the elutriation of sludge preparatory to mechanical dewatering. The elutriation process is based on Genter's conclusions that soluble amino and ammonia compounds in sewage sludge are detrimental to the economical coagulation of the colloids by the chemicals used for dewatering. The object of the process is to reduce the proportion of these "ammoniacal compounds" in the sludge before treating it with coagulants preparatory to dewatering. Elutriation consists in mixing the sludge with sewage, effluent or water, containing a smaller proportion of the amino and ammonia compounds than is present in the sludge, allowing the sludge to settle, and decanting the elutriating water, thus reducing the concentration of the objectionable compounds. Water containing active chemicals or exchange adsorbents may also be used as the elutriating agent. The elutriate is discharged into the incoming sewage. Genter has applied for letters patent to cover this process.

The results of extensive, large-scale sludge-filtration tests conducted at Baltimore from Aug. 29, 1933, to May 31, 1934, have been reported by Keefer and Kratz (14). These results indicate that elutriation of the sludge will reduce by more than 50 per cent the quantity of ferric chloride required for conditioning digested primary sludge.

A study was made of the effect of the volume of elutriating water on the quantity of digested sludge filtered, when using various quantities of coagulant. When dewatering thickened unelutriated sludge, containing 6.5 to 6.7 per cent solids, which was coagulated with 6 lb. of anhydrous ferric chloride to 100 lb. of dry solids, the filter capacity was about 10.2 lb. of dry solids an hour per square foot of filter area. When using this same quantity of chemical to coagulate sludge which had been elutriated with an equal volume of water, the capacity was increased to 26.6 lb. an hour. The use of more elutriating water resulted in greater production of sludge cake with less coagulant. When three volumes of elutriating water to one volume of sludge were used, 4 lb. of anhydrous ferric chloride to 100 lb. of dry solids gave a production of 28.8 lb. of dry solids an hour per square foot of filter area. On the other hand, when this same sludge was filtered without elutriation, 4.7 lb. of ferric chloride yielded only 5.9 lb. of cake. In this instance, the filter capacity was increased almost fivefold with a reduction of about 15 per cent in coagulant, by the use of the elutriation process.

Keefer and Kratz have concluded that the filtering of elutriated sludge is a practicable method of dewatering the digested sludge at the Baltimore sewage works. They state that the removal of the amino-ammonia compounds will permit the use of much less coagulant, with a saving of several thousand dollars annually.

Early in 1935, plain-sedimentation sewage-treatment plants are being designed for the District of Columbia and for the Metropolitan District of Hartford, Conn. In both cases the plans provide for elutriation of digested sludge prior to conditioning and dewatering on vacuum filters.

Sludge Filters.—The technology of mechanical sludge-dewatering has come from the chemical industry, in which the filtration of numerous products has led to the development of a variety of different processes and machines (15). Aside from largely unsuccessful attempts at the filtration of sludge by gravity, through fine wire mesh, Filtros plates and absorbent material, such as garbage tankage, three types of filtering devices may be mentioned, in all of which changes in pressure, either above or below atmospheric, are employed to accelerate filtration.

These devices may be classified as follows: chamber or leaf filter presses, in which the sludge is put under pressure; bag-press filters, in which the sludge is contained in bags during squeezing; and vacuum filters, in which the sludge is dewatered by suction. The three types of sludge filters are shown in Figs. 210, 211 and 212.

The chamber or leaf press usually consists of cast-iron plates covered with cloth, such as 11-oz. army duck. The covered plates are hung in a frame commonly equipped with a fixed and a movable head. The plates are forced together, a space being left between the cloth surfaces on the plate faces and sludge being pumped through a central opening in the plates. The sludge covers the cloth and the liquid is forced through the cloth to the plate surface, whence it enters drainage holes which carry it away through openings in the bottom of the plates.

After the presses are filled and drainage is completed, the sludge valve is closed; the movable head is then pulled back and the cake drops out. There are several different makes of presses. Pressures of 80 to 120 lb. per square inch are employed. Breakage of cloth was observed at Chicago when the higher pressure was applied. This type of filter has

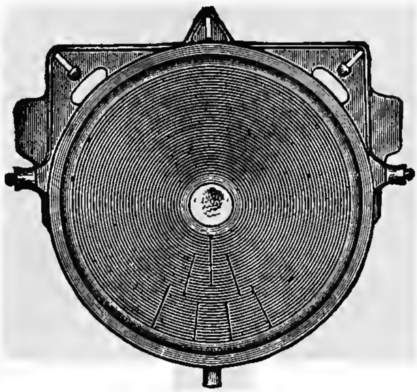


Plate used in filter press

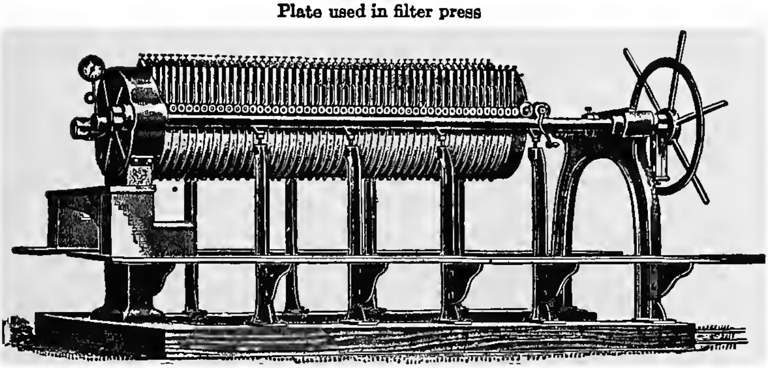


Fig. 210.—Chamber or leaf filter press.

been employed chiefly in older installations, of which the chemical-precipitation plants at Providence, R. I., and Worcester, Mass., now discontinued, are examples.

Burlap is generally employed as the filter cloth in bag filters, an 8- to 12-oz. grade being used. The bags are filled from the top or bottom with sludge. They are suspended between drainage sheets and squeezed by large platens, worked either by a toggle joint or by direct pressure. The use of Berrigan and Worthington presses for dewatering activated sludge has been studied at Chicago. Hydraulic pressure is employed.

The Worthington and Berrigan presses at the Des Plaines River plant of Chicago hold 18 bags on platens 5 by 8 ft. in size and 80 to 120 bags on

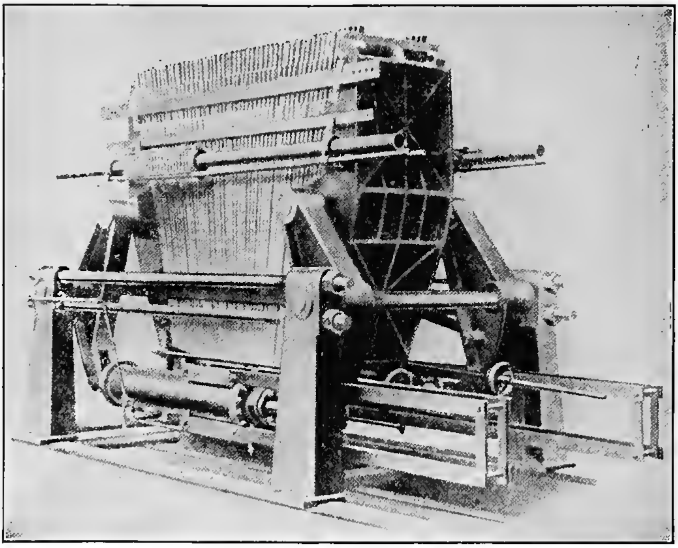


Fig. 211.—Bag press. (Worthington).

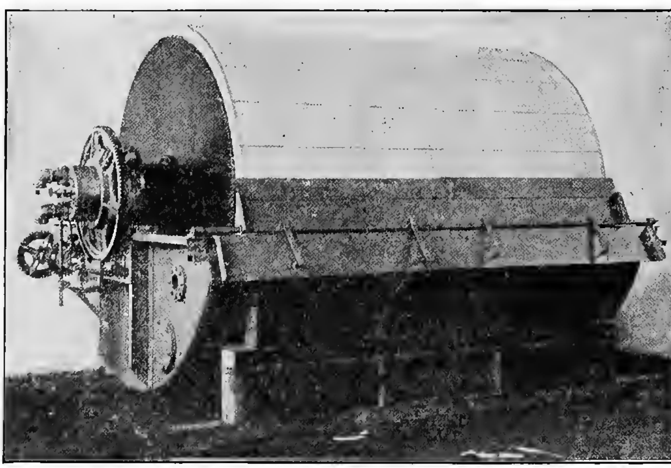


Fig. 212.—Rotary-drum vacuum filter. (Oliver).

platens 5 by 6 ft. in size, respectively. The Worthington press employs built-up steel drainage sheets and the bags are filled from the top and squeezed by large platens, worked by a toggle joint. The Berrigan press employs built-up drainage sheets of hard-wood slats and the bags are filled from the bottom, pressure being applied directly on the heads of the press. The cake drops out of the bags at the bottom in both cases. According to Pearse (16), it is possible to control the thickness of the press cake in the Berrigan press and the direct squeezing makes a simple apparatus.

Pearse states: "The comparison of press types is dependent on the sludge to some extent. A comparison of the presses in use at Chicago may be of interest as typical."

TABLE 141.—OPERATING RESULTS OF FILTERS DEWATERING ACTIVATED SLUDGE

Press or filter	Filter area, sq. ft.	Cake produced		Cycle, hr.	Yield of cake, lb. dry solids per sq. ft. per min.
		Lb.	Per cent moisture		
Simplex	1815	4,400	78	7	0.0013
Worthington	1440	3,500	76	5	0.0019
Berrigan	3520	10,000	78	8	0.0013
Oliver	495	1,400[1]	80	Continuous	0.0094

[1] Per hour.

The experience with mechanical dewatering equipment at Houston has been described by Fugate and Stanley (7) as follows:

On account of the high cost of labor for operation of the filter presses and the short life of filter cloth the use of the plate and frame presses was abandoned.

A standard wet machine used in paper pulp industry was then installed. Sulfur dioxide gas was used for conditioning. While this machine gave very high filtration rates and looked very promising, certain mechanical troubles developed which brought about the installation of the "Decker"-type machine, a modification of the standard wet machine.

The greatest disadvantage of the pulp machines was the loss of solids passing through the screen. The cost of replacement of the wire screen and felt blanket was quite high, and in general nonconsistent results as to capacity were obtained.

More recent experiments have been conducted with a 4-ft. American continuous vacuum filter using either aluminum sulphate or ferric salts as conditioning reagents. For control of the conditioning the hydrogen ion concentration was used as a guide in place of titration. Positive and delicate means of synchronizing the flow of sludge and conditioning solution were perfected. This enabled absolute control of the plant operation. In consequence, the results obtained were consistent and justified a large installation (installed in 1928).

There are two principal types of vacuum filters, the Oliver or drum filter and the American or disk filter, which are described below. These differ largely in the method of supporting the filtering surface. Experiments have been carried on with a few other types of vacuum filters.

Construction and operation features of vacuum filters used in American municipal plants are given in Table 142.

TABLE 142.—CONSTRUCTION AND OPERATION FEATURES OF VACUUM FILTERS

	Charlotte, N. C.	Houston, Tex.	Milwaukee, Wis.	Pasadena, Cal.
Type of filter	Oliver	American	Oliver	Oliver
Number	2	4	24	3
Size:				
Diameter, ft	8	8	11.5	11.5
Length, ft	10	...	14	14
Number of disks	..	8		
Effective filtering surface per filter, sq. ft	..	800	495	495
Vacuum, in. of mercury:				
Pick-up	20	...	22	21–25
Drying	..	...	11	11
Yield of cake, lb. dry solids per sq. ft. per min	..	...	0.014–0.056	0.016
Thickness of cake, in	..	...	$\frac{1}{16}$–$\frac{1}{4}$	
Initial moisture, per cent	99	...	98–99	99
Final moisture, per cent	85	...	82–85	78–80

Vacuum filters have been successfully employed in dewatering activated sludge at many plants. At Chicago a study was recently made of the possibility of filtering mixtures of fresh solids and activated sludge, thus eliminating the digestion process. According to Mohlman (17), fresh sludge filtered on an Oliver filter at higher rates and with less expense for ferric chloride, used as a coagulant, than digested sludge. Results of tests made by the authors at Toronto in 1932 corroborated this statement. Mohlman asserts that filtration of mixed fresh and activated sludges "appears to be preferable to filtration of activated sludge alone. The moisture content is lower, varying from 95 to 97 per cent; vacuum filters can be operated at higher speeds than with the dilute activated sludge."

In a series of experiments conducted at Baltimore in 1931 and 1932, Keefer and Cromwell (10) found vacuum filters capable of satisfactorily dewatering digested sludge, the average moisture content of the digested sludge cake from an Oliver filter being about 74.9 per cent, when ferric chloride was used as a coagulant. They believe that the merits of the process are sufficiently attractive to warrant the purchase of filtering equipment for Baltimore as soon as funds are available. For many years at this plant digested sludge has been dried on sand beds.

The Oliver Filter.—The Oliver filter, shown in Fig. 212, consists of a revolving drum or cylinder with a horizontal axis. The shell, or cylindrical part of the drum, is hollow and is made up of an outer filtering surface of cloth, supported on wire mesh, and an inner supporting surface of wood or metal, impervious to liquids and air. The intermediate space is divided into shallow compartments or sections, usually 12 or 24, by division strips laid parallel to the drum axis. Within each of these compartments is a grating of wood or metal, which gives additional support to the filtering surface. The filter cloth is wired on to the drum, commonly with No. 14 copper wires spaced about 1 in. apart.

Each compartment is connected by a pipe to an automatic valve on the drum axis at one end of the filter. This valve controls the application of vacuum to the compartment or the admission of compressed air or steam for lifting the sludge cake from the cloth or for cleansing the filtering medium. The valve consists of a revolving seat and a stationary chamber. A separate port is provided in the valve seat for each compartment of the drum shell and the valve chamber contains separate annular ports, which correspond to the different stages in the cycle of cake formation, drying and discharge.

The drum may be rotated either by an individual motor or by means of line shafting from a suitable source of power. The speed of rotation depends upon the characteristics of the sludge and commonly varies from 3 to 6 r.p.m.

The drum revolves, partly submerged, through a sludge chamber, in which the sludge is agitated by horizontal rakes, mounted on arcs, which oscillate slowly with the drum center as an axis, so that the travel of one rake overlaps that of the rest. The area of submergence is adjustable between 15 and 40 per cent.

As each compartment travels through the sludge chamber, a vacuum is applied and a layer of sludge, varying in thickness with the degree of suction, the character of the sludge and the speed of the drum, is picked up. A thickness of 1/16 to 1/4 in. is usual. As the compartment passes out of the sludge, the vacuum is commonly increased and held at the desired value until the compartment is about to reenter the sludge. Thus the cake is dewatered by suction, the filtrate passing through the vacuum pipes to a receiver, whence it is discharged. Just prior to resubmersion in the sludge, the vacuum is cut off and compressed air is admitted to the compartment. The dewatered cake is lifted from the cloth and removed on the descending side of the drum by steel scrapers resting on the wire winding. The loose filter cake is deflected by the scrapers on to a conveying mechanism, usually a belt, which carries it away.

There are numerous modifications of equipment depending upon the character of the sludge. The operating cycle, too, is subject to appreciable variations. Oliver filters are manufactured in sizes varying from 1 to 14 ft. in diameter and from 1 to 18 ft. in length. The effective area varies from 3 to 790 sq. ft. per filter.

There has been much experimentation with different cloths for vacuum filters, cotton duck, Canton flannel and woolen cloth having been used

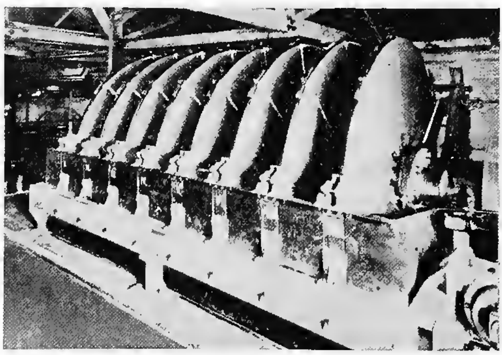


FIG. 213.—Disk vacuum filter at Houston, Tex.

on a large scale. The life of filter cloth seems to vary between 1 and 2 months. The sludge liquor is generally offensive and is usually treated with the sewage. The cloth may be cleaned from time to time by steaming, brushing and applying a caustic solution.

The American Filter.—The American filter, illustrated in Fig. 213, consists of a series of disks, made up of a number of sectors, mounted on a horizontal axis and revolving through a sludge chamber in much the same way as the Oliver filter. Each sector is constructed of wood which is corrugated on both sides similar to a wash-board. These sectors are covered with cloth bags. Depending upon the size of the filter, 8 to 10 sectors are assembled around a hollow-center shaft to form a complete disk. The shaft carries the filtrate channels, which run parallel to the axis and connect with openings in an automatic filter valve, which controls the vacuum and pressure applied to the sectors.

The operation and general arrangement of the American filter are similar to those of the Oliver filter. The disks are manufactured in sizes varying from 4 to 12.5 ft. in diameter and are mounted either singly or in groups up to 12 in number. The effective area varies from 22 to 2400 sq. ft. per filter.

Sludge Centrifuges.—In this country centrifugal separation of sludge liquor and solids has been attempted only on an experimental basis. A number of German plants, notably Frankfort-on-the-Main and Hannover, have had centrifuges in operation for a number of years. In the United States the most extensive experiments have been made at Milwaukee and Chicago on activated sludge and at Baltimore on semidigested sludge. The former were largely discouraging, because it was impossible to obtain a clear effluent when operating the centrifuge at economical rates.

In the Baltimore experiments an average removal of 65.1 per cent of total solids from the sludge liquor was obtained, the moisture of the cake being 71.1 per cent (18). The machine revolved continuously at 1200 r.p.m., sludge being admitted, cut off and the cake discharged. The dewatering process went on during the first two stages of operation, the first being known as the "inlet period" and the second as the "postcentrifuge period." The inlet period lasted 7.6 min. and the postcentrifuge period 3.7 min. The wet sludge averaged 93.9 per cent moisture and 68.9 per cent volatile matter. An average inlet rate of 40.1 gal. a minute was maintained, 11,800 gal. being handled every 8 hr. with a total current consumption of 295 kw-hr. The average solids content of the sludge liquor was 2.7 per cent and its B.O.D. 9100 p.p.m. The latter could be reduced to 2815 p.p.m. by adding 1.75 lb. of alum per 100 gal. of effluent. The offensive character of the sludge liquor is considered one of the disadvantages of the process. At Baltimore it was calculated that addition of the sludge liquor to the untreated sewage would increase the B.O.D. of the latter by 4.3 per cent. Adding sludge liquor to trickling-filter effluent in the ratio of 1 to 1250 reduced the relative stability of the effluent from 96 to 84 per cent.

Recently tests have been conducted to determine the operating characteristics of a centrifuge when dewatering primary sludge, activated sludge and digested sludge (19). The most efficient separation of solids from water was obtained when the machine was used for dewatering sludge from preliminary-sedimentation tanks. In a series of tests with this kind of sludge, the sludge cake from the centrifuge contained little more than 50 per cent of the total solids fed to the machine, the rest escaping with the effluent from the centrifuge.

Flotation.—Among the methods of dewatering sludge which have been studied is flotation, employing acid, heat or both. Pearse (16) comments as follows on this process:

Flotation is an attempt with or without heat to coagulate and float the sludge to the surface for ready removal. At Fort Worth on packing-house sludge, small amounts of acid (sulfuric) proved very effective on cold sludge, producing remarkable coagulation, with an underlying clear liquid. Such results have not been duplicated elsewhere. However, acidulation with heat was tried experimentally by the Dorr Company at Urbana and New Britain with some success. Enough acid was used to produce an acidity of 25 parts per million. Heat was then applied to raise the temperature to 50°C. The process requires accurate constant control and to date has not met favor, largely because the moisture content has not been reduced below a range from 85 to 90 per cent (New Britain), or 90 to 93 per cent (Milwaukee). At New Britain and Milwaukee large losses of solids were reported.

Downes (20) reports that a flotation process has been installed at Plainfield, N. J., to treat digested sludge preliminary to spray drying. He describes the process as follows:

In order to insure as dense a sludge as possible we concentrate the digested sludge by a scheme of flotation. This is accomplished by treating the digested sludge with chemicals which not only coagulate it but fill it with gas to give it buoyancy. Clear liquid is drawn off continuously at the bottom of the flotation tower and the thickened sludge is continuously removed from the surface and fed to the atomizer by means of a skimming rake.

Spray Drying.—The Industrial Associates, Inc., have developed a process for the drying and eventual incineration of wet sludge (21). The sludge to be dried is fed into a centrifugal-spray machine and atomized in a horizontal zone near the top of the spray chamber. The dried product falls to the floor of the chamber and is collected by a revolving rake, which delivers into a central chute, discharging into a car or conveyor. The exit gases, laden with moisture and a certain amount of the dust of the dried product, are carried out of the drying chamber through a duct into a cyclone dust collector. Rudolfs and Cleary report that "at a demonstration of this device at Poughkeepsie, N. Y., in 1931, the apparatus dried 3 gal. of sludge, moisture content 88 per cent, in one minute." So far as is known, results of tests of this process on a large scale have not been reported.

In the spray-drying installation at Plainfield, hot air is introduced into the drying chamber above the plane of the "atomizer" (20). This air instantly gives up its heat in evaporating the fog created by the atomizer. Then it is drawn off through ducts in the lower part of the drying chamber. An automatic stoker may be set to deliver air to the drying chamber at any temperature up to 900°F. The dried sludge may be delivered either to the cyclone dust collector or to the furnace, where it burns as pulverized fuel. As fuel it supplies one third of the heat required in the drying process.

Heat Dryers.—When a commercial fertilizer or fertilizer base is to be prepared from sewage sludge, the latter is generally dried to a moisture content of less than 5 per cent and sized to pass through an 8-mesh screen. Rotary heat dryers, such as are used in many industrial operations, are generally employed to drive off the excess moisture from sludge that has been subjected to primary dewatering.

At Milwaukee the activated-sludge cake from Oliver filters is dropped on to a rubber belt and conveyed to a battery of six Atlas, direct-indirect-heat, continuous, rotary dryers, each 7 ft. in diameter by 60 ft. long, enclosed in a brick setting (5). Since the sludge cake tends to ball up and dry irregularly, an equal quantity of previously dried sludge is mixed with it before it enters the dryers. According to Cramer and Wilson,

> This lowers the moisture content sufficiently to prevent balling. . . . As the drum revolves, the hot gases from the furnace, mixed with large volumes of outside air, circulate around and into the drum through numerous air valves attached to the drum shell. About 45 min. are allowed for the sludge to pass through the driers, during which time the moisture content is reduced from 82 per cent, as it leaves the filters, to about 3 per cent, as it emerges from the lower end of the driers. The temperature of the driers at the inletting end is kept at 1000°C. and at the outlet, 500°C.

Each dryer produces about 10 to 15 tons of dry sludge daily. The dried material is screened and the coarser portions are passed through a pulverator. The finest portions are mixed with the sludge cake entering the dryers.

At Pasadena, Cal., activated-sludge cake from Oliver filters is transported on a belt conveyor to a Ruggles-Coles dryer (6). Steel disks on this belt are set so as to be rotated by it, cutting the sludge cake into small pieces that can be handled more effectively in the dryer. Orbison is quoted as follows:

> Reduction of the moisture content to about 5 per cent is accomplished in a drier which is primarily a horizontal cylinder that rotates as heated gases pass through it. The hot gases are provided by a gas-burning combustion chamber whose fuel consumption is about 100,000 cu. ft. per day, rated at 1000 B.t.u. per cubic foot and costing 20 cents per 1000 cu. ft. Hot gases pass into the dryer at a temperature of about 1800°F. and are drawn through by induced draft. As originally installed, the dryer was 42 ft. long, but to make better use of the gases the length has been increased to 60 ft. The outer shell of the dryer is 70 in. in diameter and a concentric inner shell has a diameter of 30 in. Hot gases pass first through the inner shell for the full length and then return between the inner and the outer shells to the exhaust fan.
>
> Sludge cake passes through the length of the dryer only once, remaining between the inner and the outer shells; as the cylinder rotates, longitudinal

metal vanes on the inside of the outer and the outside of the inner shell keep it moving and dropping continually through the heated gases. The dryer slopes downward from the combustion chamber on a grade of 0.2 in. per foot, and the sludge cake fed in at the combustion chamber end is moved along to the lower end by the rotation. A 30-hp. variable-speed motor turns the dryer at 7½ r.p.m. (normal speed), this combination of grade and speed making the time for passing through the dryer about one hour. At this speed, about 700 lb. of sludge cake with moisture content reduced to 5 per cent can be delivered per hour.

From a pit at the discharge end of the dryer a bucket conveyor lifts the dried product to a 15- by 18-in. swing-hammer pulverizer that grinds it to a fineness such that it will all pass a ⅛-in. screen.

The exhaust gases from heat dryers generally carry away appreciable quantities of dust and may be quite odorous. Dust chambers or dust arresters, also called cyclones, and washers may be employed to remove the dust. To avoid causing offense the gases may also be deodorized by washing, chlorination or incineration. Experience at Pasadena, as related by Orbison, (6) is interesting in this connection:

After unsuccessful treatment of gases with chlorine, ozone, by washing and in coke towers, the gases were blown into the partly filled outfall sewer, which is 4 miles long. For a while this plan seemed to be satisfactory, absorption being aided by burlap baffles at the outlet. Odors did escape, however, and the outfall disposal of gas was finally abandoned.

The next step was the incineration of the gases—a method that seems to have solved the odor problem. At least it gives promise of being a wholly successful plan. The incinerator consists of a simple combustion chamber surrounded by a preheating duct through which the gases from the dryer are passed. The natural gas fuel is fed into the combustion chamber through a burner around which the dryer gases are also blown in, thus bringing them in direct contact with the flame. This plan has effected complete deodorization and does not produce smoke, so the products of combustion are passed through a 20-ft. stack directly into the atmosphere. The fan that draws the hot gases through the dryer and blows them into the incinerator is driven by a 20-hp. variable-speed motor and has a capacity of 7500 to 10,000 min.-ft. of gas under the present static pressure. In addition to this main exhaust blower a separate motor-driven blower of small size delivers through a 6-in. air line into the combustion chamber of the incinerator. This blower was installed to aid combustion.

Rotary heat dryers were operated for 6½ years at Baltimore, Md., for converting air-dried, semidigested sludge into fertilizer. The plant was shut down in 1923. Reviewing the experiences at Baltimore, Keefer (22) concluded as follows:

The six and one-half years of experience which Baltimore has had with the heat-drying and marketing of the sludge has indicated that it is an expensive method to use in disposing of the sludge under conditions as they now exist at the Baltimore plant. The high cost was due in part to the excessive overhead expenses, and the cost of hauling the material to its point of destination. Furthermore, the air-dried sludge was not in the best of condition to dry as it often contained sand and gravel, which adhered to it when the sludge was removed from the sludge drying beds. Moreover, this inert material decreased the percentage of nitrogen in the final product. The quantity of nitrogen in the sludge is not sufficiently large to keep from having a net operating loss. The accounts indicate, however, that if the Baltimore sludge had contained about 4 per cent of nitrogen as available ammonia, the plant could possibly have been operated at a slight profit.

Complete dewatering of sludge necessitates the use of much conveying machinery. The wet sludge is readily pumped, but sludge paste from vacuum filters balls together unless suitably handled. Since the dried sludge to be sold for fertilizing purposes is generally ground and bagged, suitable machinery is required to handle, grind and bag the material. Storage space is also needed and in some cases the sludge may require classification according to its nitrogen content.

The gases from the dryer plant at Milwaukee have at times produced odors, causing serious complaints. The odor is increased by occasional scorching in the dryer. Chlorine has been added to the gases in advance of their entering the cyclone chamber, when the wind is toward the city, but this has not entirely eliminated complaints. The construction of a washing and condensing plant at a cost of $85,000 was authorized in 1931. This plant is intended to remove the vapor from the gases which escape from the dryers. In a large-size test unit the suspended solids in the dryer gases were reduced about 98 per cent and it was

TABLE 143.—ANNUAL COSTS OF SLUDGE DEWATERING, DRYING AND DISPOSAL AT MILWAUKEE, BASED ON 1930 CONDITIONS

	Per ton of Milorganite	Per mil. gal. of sewage treated, at 1.28 tons per mil. gal.
Fixed charges, including interest and depreciation on $3,333,736.38	$ 7.17	$ 9.18
Operating charges	15.87	20.31
Total cost	$23.04	$29.49
Net income from sales	16.17	20.70
Net total cost of sludge dewatering, drying and disposal	$ 6.87	$ 8.79

reported to the authors that the gases discharged from the washers were practically odorless.

Costs of Construction and Operation of Plants for Dewatering and Drying Sludge.—Mechanical processes of sludge dewatering and drying are highly specialized for the individual plant.

The cost of the filtering equipment at the Irwin Creek plant in Charlotte was about $31,000. This included two Oliver vacuum filters, each 8 ft. in diameter and 10 ft. long. The Oliver vacuum filters used at Milwaukee and Pasadena, which are 11 ft. 6 in. in diameter and 14 ft. long, cost about $20,000 per unit, including piping and erection.

TABLE 144.—OPERATING COSTS OF SLUDGE DEWATERING AND DRYING AT PASADENA

	Cost per mil. gal. of sewage		Cost per ton of fertilizer, at 13.2 tons a day
	July 1, 1928, to Sept. 30, 1928 (6.7 m.g.d.)	Based on plant capacity (11 m.g.d.)	
Sludge conditioning and filtration:			
Labor, 26 days a mo.; 5 men at $5.95 a day	$ 3.88	$ 2.37	$ 1.98
Power, at 0.88¢ a kwh	1.81	1.81	1.51
Alum, 368 lb. per mil. gal., at 1.6¢ a lb.	5.90	5.90	4.92
Celite, 431 lb. per mil. gal., at 1.47¢ a lb.	6.35	6.35	5.30
Oil, grease, tools	0.12	0.12	0.10
Filter cloths, $150 every 4 months	0.18	0.18	0.15
Maintenance and renewals	0.49	0.49	0.41
Portion of general expense	1.80	1.10	0.91
Subtotal, sludge conditioning and filtration	$20.53	$18.32	$15.28
Sludge drying and storage and exhaust-gas burning:			
Labor, part time, 8 men at $5.53 a day	$ 4.55	$ 2.76	$ 2.30
Power, at 0.88¢ a kwh	0.93	0.93	0.77
Sacks, at 8¢ each	1.66	1.66	1.38
Gas for dryer, at 17½¢ per 1000 cu. ft.	2.94	2.94	2.45
Gas for burning exhaust gases from dryer	2.03	2.03	1.69
Miscellaneous, oil and grease	0.04	0.04	0.03
Maintenance and renewals	1.04	1.04	0.87
Portion of general expense	1.27	0.77	0.64
Subtotal, sludge drying and storage	$14.46	$12.17	$10.13
Total for sludge dewatering and drying	$34.99	$30.49	$25.41
Revenue from sale of sludge, 1.2 tons per mil. gal., at contract price of $25.25 a ton	$30.30	$30.30	$25.25
Net operating cost of sludge dewatering and drying	$ 4.69	$ 0.19	$ 0.16

The total cost of the Milwaukee sludge-dewatering and sludge-drying plant to Jan. 1, 1931, was about \$3,334,000. The average capacity was estimated to be adequate to treat the sludge produced from 85 m.g.d. or 589,000 persons. The cost is equivalent to about \$40,000 per mil. gal. daily of sewage treated and \$5.65 per capita served. The annual charges for dewatering, drying, storage and sale of the sludge at Milwaukee, based on 1930 conditions, may be summarized as in Table 143.

The cost of chemicals for conditioning at Milwaukee is about \$5 per ton of Milorganite produced.

The operating costs of the Pasadena dewatering and drying plant as operated in 1928, without allowance for interest and depreciation, may be summarized as in Table 144, from data furnished by Goudey (23):

Bibliography

1. Wilson, J. A., W. R. Copeland and H. M. Heisig: *Jour. Ind. Eng. Chem.*, 1922; **14**, 128.
2. Rept. on Industrial Wastes from Stockyards and Packingtown in Chicago, vol. 2, 1921.
3. Hatton, T. C., V. H. Kadish and H. M. Heisig; *Jour.*, Inst. Sanitary Eng., 1927; **31**, 121.
4. Mohlman, F. W., and J. R. Palmer: *Eng. News-Rec.*, 1928; **100**, 147.
5. Cramer, Robert, and J. A. Wilson: *Jour. Ind. Eng. Chem.*, 1928; **20**, 4.
6. *Eng. News-Rec.*, 1928; **101**, 469.
7. Fugate, G. L., and W. S. Stanley: *Sewage Works Jour.*, 1928; **1**, 70.
8. Kadish, V. H.: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 134.
9. Ann. Rept., Milwaukee Sewerage Comm., 1918.
10. Keefer, C. E., and E. C. Cromwell: *Sewage Works Jour.*, 1932; **4**, 929.
11. Mohlman, F. W., and G. P. Edwards: *Jour. Ind. Eng. Chem.*, 1934; **26**, 226.
12. Edwards, G. P.: *Sewage Works Jour.*, 1934; **6**, 444.
13. Genter, A. L.: *Sewage Works Jour.*, 1934; **6**, 689.
14. Keefer, C. E., and Herman Kratz, Jr.: *Sewage Works Jour.*, 1934; **6**, 845.
15. Wright, A. W.: "Industrial Filtration," 1923.
16. Pearse, Langdon: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 126.
17. Mohlman, F. W.: *Water Works and Sewerage*, 1932, **79**, 33.
18. Keefer, C. E., and Herman Kratz, Jr.: *Sewage Works Jour.*, 1929; **1**, 120.
19. Pecker, J. S.: Paper presented before N. Y. State Sewage Works Assoc., Jan. 15, 1935.
20. Downes, J. R.: *Water Works and Sewerage*, 1934; **81**, 210.
21. Rudolfs, Willem, and E. J. Cleary: *Sewage Works Jour.*, 1933; **5**, 409.
22. Keefer, C. E.: *Eng. News-Rec.*, 1926; **96**, 238.
23. Wyman, A. W.: *Sewage Works Jour.*, 1929; **1**, 339.

CHAPTER XXX

CHLORINATION AND DISINFECTION OF SEWAGE AND EFFLUENTS

Chlorination of sewage or effluents consists of treatment with chlorine or bleaching powder, for the purposes of disinfection for the protection of water supplies, bathing waters or shellfish layings, retardation of decomposition, reduction of biochemical-oxygen demand, control of odors or control of *Psychoda*, cleaning of the distribution system and destruction of surface films on trickling filters. *Disinfection* is the destruction, ordinarily by the use of some chemical, of the microorganisms likely to cause infection and disease. Although chlorine may be applied for one or more of the purposes mentioned, all of them may be accomplished to some extent when sewage is chlorinated. Thus, if chlorine is applied primarily for disinfection purposes, incidentally the treatment may deodorize the liquid more or less, reduce or retard putrefaction and reduce the biochemical-oxygen demand. By *sterilization* is meant the destruction of all organisms, both pathogenic and innocuous. In the methods of treating sewage previously considered, the removal of suspended matter or 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. Most of the treatment processes may show some reduction of bacteria, however, high bacterial removal being associated particularly with intermittent sand filtration and the activated-sludge process.

Disinfection may be accomplished in three general ways, by heat, chemicals, or actinic rays. As far as municipal-sewage treatment is concerned, chemicals are the only successful disinfecting agents and among these chlorine and its compounds offer at the present time the only practicable means of securing economical and adequate results. Discussion of the theory of disinfection, therefore, hinges upon chlorination.

Theory of Chlorination.—In modern sewage works chlorine is commonly added to sewage as chlorine gas. Use of chlorine in this form was preceded by that of hypochlorites, particularly of calcium hypochlorite or bleaching powder. In either case hypochlorous acid, a strong oxidizing agent, is formed, as shown by the following reactions:

$$\underset{\text{Chlorine gas}}{Cl_2} + \underset{\text{Water}}{H_2O} \leftrightarrows \underset{\text{Hypochlorous acid}}{HClO} + \underset{\text{Hydrochloric acid}}{HCl}$$

$$\underset{\text{Bleaching powder}}{2CaClOCl} + \underset{\text{Carbonic acid}}{2H_2CO_3} \leftrightarrows \underset{\text{Calcium bicarbonate}}{Ca(HCO_3)_2} + \underset{\text{Hypochlorous acid}}{2HClO}$$

$$\underset{\text{Hypochlorous acid}}{2HClO} \leftrightarrows \underset{\text{Hydrochloric acid}}{2HCl} + \underset{\text{Oxygen}}{O_2}$$

Consideration of the oxidizing properties of hypochlorous acid led to the theory that destruction of bacteria by chlorine was due to the nascent, or atomic, oxygen liberated by this unstable compound. Opposed to this theory, however, were, among other things, the observation that other oxidizing agents, such as hydrogen peroxide, H_2O_2, and ozone, O_3, having an oxidizing value equal to or greater than chlorine, do not possess the same disinfecting power, and the fact that chloramine, NH_2Cl,[1] which possesses no oxidizing power, may in certain instances exceed chlorine and hypochlorites in disinfecting efficiency.

While the exact manner of accomplishing cell destruction by chlorine remains uncertain, death is probably due to a combination of chlorine with the cell contents.

Like all disinfecting processes the destruction of bacteria by chlorination is a time-concentration phenomenon.[2] This means that disinfection is not instantaneous, but that it depends upon the length of time during which the bacteria are exposed to the disinfectant and upon the concentration of the bacteria and the disinfecting chemicals. In the chlorination of sewage, furthermore, the unstable organic matter exerts a marked and rapid chlorine demand which must be satisfied before disinfection may be expected. The chlorine added in excess of this initial demand is called "residual chlorine." The time required for disinfection is relatively short but must be taken into account to secure effective chlorination of sewage.

Disinfection of raw sewage is somewhat unreliable. As the grosser solids are not penetrated by chlorine, chlorination of sewage can be given a definite efficiency rating only after such matters have been removed by sedimentation or other means.

[1] $Cl_2 + NH_3 \leftrightarrows NH_2Cl + HCl$.

[2] In pure water the mortality of bacteria due to disinfecting agents is a logarithmic function of time, the number remaining after each time interval being proportionate to the number present at the beginning. Formulation of this type of relationship has been discussed in Chap. III.

The following formula holds:

$$N_t = \frac{N}{K^t} \text{ or } K = \frac{1}{t} \log \frac{N}{N_t}$$

where N_t = number of bacteria living after time t
N = initial number of bacteria
t = time
K = disinfection constant.

Chlorine and Its Compounds.—The disinfection or deodorizing of sewage is generally accomplished by chlorine in one of the following forms: liquid chlorine, bleach or electrolytic chlorine. All of these have their origin in the electrolysis of salt brine. The reactions may be indicated as follows:

$$\underset{\substack{\text{Sodium}\\\text{chloride}\\\text{(salt)}}}{\overset{2 \times 58}{2NaCl}} + \underset{\text{Water}}{\overset{2 \times 18}{2H_2O}} \leftrightarrows \underset{\substack{\text{Chlorine}\\\text{(gas)}}}{\overset{71}{Cl_2}} + \underset{\substack{\text{Sodium}\\\text{hydrox-}\\\text{ide}\\\text{(caustic}\\\text{soda)}}}{\overset{2 \times 40}{2NaOH}} + \underset{\substack{\text{Hydro-}\\\text{gen (gas)}}}{\overset{2}{H_2}}$$

The chlorine gas may be removed as produced, dried and liquefied by pressure to form "liquid chlorine"; it may be passed over slaked lime to form "chlorine of lime" or "bleaching powder"; or it may be utilized as "electrolytic chlorine" at the point of generation, in the form of chlorine gas when the products of electrolysis are separated, or as sodium hypochlorite when they are permitted to react upon one another.

In this country liquid chlorine and bleaching powder are by-products of the manufacture of caustic soda, while sodium hypochlorite is usually generated in chlorine cells at the point of application.

Liquid chlorine is commercially available in steel cylinders holding 100 or 150 lb. of chlorine, in drums of 1 ton net and in single- or multi-unit tank cars holding 15 tons of gas. The chlorine is under a pressure varying from about 40 lb. per square inch at the freezing point of water to 60, 100 and 140 lb. per square inch at 50, 75 and 100°F., respectively. The solubility of the gas in water is high, decreasing from 14,600 p.p.m. at 32°F. to 10,000, 6,600 and 4,200 p.p.m. at 50, 75 and 100°F., respectively, when pure water is exposed to an atmosphere of pure chlorine and the combined pressure of the gas and water vapor is one atmosphere. Bleaching powder comes on the market in iron drums holding 10, 50, 100, 300 or 700 lb. The chief constituent of bleaching powder is calcium chloro-hypochlorite, $CaClOCl$, the active substance in chlorination by bleach. Good bleach contains about 65 per cent of $CaClOCl$, which forms in water calcium hypochlorite, $Ca(OCl)_2$, and calcium chloride, $CaCl_2$. Through established commercial usage, the strength of bleaching powder is reported in terms of "available chlorine," which is the quantity of chlorine liberated on decomposing the hypochlorite with acid. It is commonly determined by finding the quantity of iodine liberated from potassium iodide, KI.[1] A study of the reactions of chlorine and hypochlorites in water will demonstrate that in whatever form chlorine is applied, two atoms of chlorine are required to liberate one atom of

[1] $2CaClOCl$ in water $\rightarrow Ca(OCl)_2 + CaCl_2$
$Ca(OCl)_2 + 4KI + 4HCl \rightarrow CaCl_2 + 4KCl + 2I_2 + 2H_2O$, or 4 atoms of iodine for 4 atoms of chlorine in the bleach.

oxygen and that the "available chlorine" is therefore a measure of the oxidizing power of chlorine or hypochlorite. Since the ratio of the chlorine content to the total weight of CaClOCl is $^{71}/_{127}$ = 0.56, the available chlorine is given by multiplying the percentage of CaClOCl in the bleaching powder by 0.56. With 65 per cent pure bleach, for example, the available chlorine is $0.56 \times 65 = 36.5$ per cent. A value of $33\frac{1}{3}$ per cent is common. Since chlorine gas is practically pure, its available chlorine is nearly 100 per cent. Bleaching powder absorbs moisture and loses chlorine on exposure to the air, with a concurrent loss of its disinfecting power. The use of liquid chlorine has superseded in large measure the formerly wide use of chloride of lime.

Several different cells have been constructed to produce chlorine or sodium hypochlorite, commonly the latter, at or near the point of chlorination. In the electrolysis of salt water, sodium is liberated at one pole and chlorine at the other. The sodium combines with the water to form sodium hydroxide and hydrogen. The chlorine, unless removed as gas, combines with the former and produces sodium hypochlorite while the hydrogen escapes. Side reactions reduce the quantity of chlorine or hypochlorite produced below the theoretical value and increase the power consumption. Theoretically the power needed to produce 1 lb. of available chlorine is 1.23 kw.-hr. and the quantity of salt is $^{58}/_{35}$ = 1.65 lb. Actual power and salt requirements, however, are respectively about two and three times the theoretical quantities. Electrolytic chlorine as yet has not found favor in sewage-treatment works and there are no important installations in which chlorine cells are used.

Methods of Chlorination.—Liquid-chlorine apparatus is now generally employed in large or permanent installations, while bleaching powder may be employed where sewage is to be chlorinated on a small scale or as a temporary expedient.

Liquid Chlorine.—Liquid chlorine is applied to sewage in two ways, designated as direct feed and solution feed. In the direct-feed method, measured quantities of chlorine gas are diffused through the sewage and are quickly taken into solution. In the solution-feed method, the gas is first dissolved in water and the chlorine solution then is added to the sewage. The quantity of chlorine commonly is controlled by either regulating the pressure drop of the gas across a fixed or variable orifice or adjusting the intermittent pulsating displacement of a definite volume of water by chlorine gas or vice versa, or else by regulating the rate of bubbling the chlorine through water. The first method is generally employed for measuring large quantities of chlorine, while the others are used for small ones. The rate of dosage is generally checked by recording the loss in weight of the chlorine container in a definite period of time. Recording scales have been developed, as has apparatus which will proportion the rate of feed to the sewage flow. Since the quantity of chlorine required depends upon the character as well as the volume of sewage, as will appear later in this chapter, manual operation is generally more satisfactory. Where sewage is pumped, arrangements

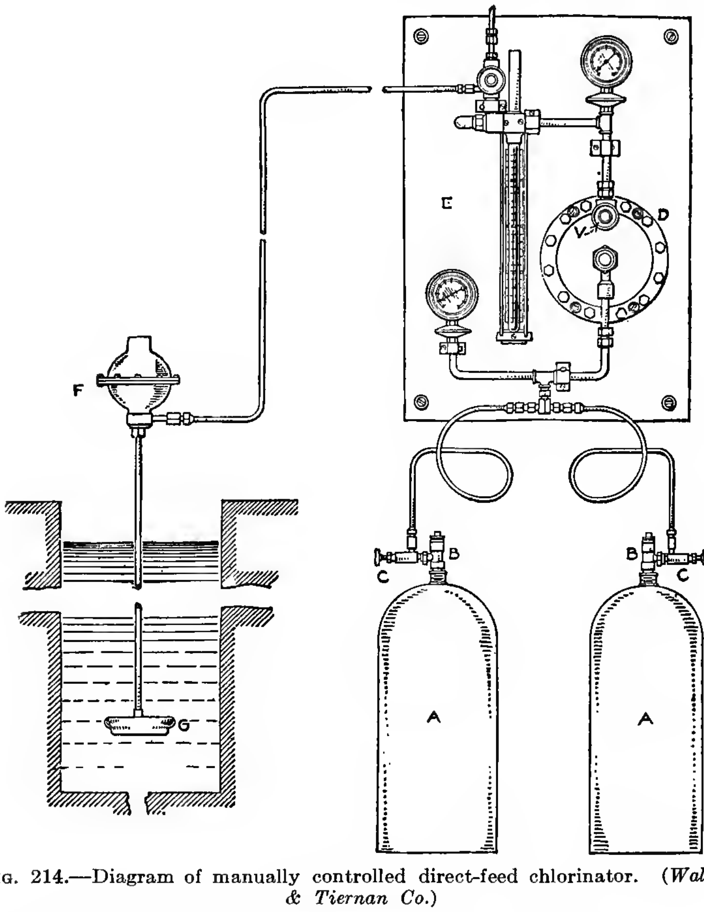


Fig. 214.—Diagram of manually controlled direct-feed chlorinator. (*Wallace & Tiernan Co.*)

can be made to start and stop the chlorinator and the pumps at the same time.

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 diagrammatically in Fig. 214. The tanks of liquid chlorine, *A*, *A*, generally are placed on scales, so that the variation in their weight as their contents are removed can be measured and in this way a check can be 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 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, as 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 from 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 noncorrodible 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 thoroughly mingles with the water and is discharged into the sewage.

Use of solution-feed chlorination, while generally more satisfactory in operation, is dependent upon a supply of water under adequate pressure to operate the auxiliary apparatus. Diffusers used on direct-feed installations must be submerged to a sufficient depth to prevent escape of chlorine.

Either automatically or manually controlled machines may be secured with capacities up to 2000 lb. a day.

As chlorine gas is extremely corrosive in the presence of moisture, gas lines, valves and pressure apparatus need to be kept free from moisture, unless they are constructed of resistant materials, such as silver, hard rubber or glass. At low temperatures chlorine hydrate is sometimes formed in the measuring apparatus. To prevent this, heat is supplied to the apparatus or the chlorinating room during cold weather.

Chlorinating apparatus is generally placed in a well-ventilated room, so that if any gas escapes, it can be removed immediately. This room preferably opens out of doors. The treatment of a person affected by the gas is to give fresh air and ether, the latter being administered carefully and only in sufficient quantity to relieve pain.

Bleaching Powder.[1]—Where bleaching powder was employed instead of liquid chlorine, provisions were made to dissolve the powder and to measure the volume of a solution of known strength which was added to the sewage. One or more mixing tanks, solution tanks and feed tanks were provided. The materials employed for the construction of these

[1] Hypochlorites containing much larger percentages of available chlorine and more soluble than bleaching powder are more available.

tanks may be wood, tile, concrete or cast iron. The bleach was first dissolved in the mixing tanks, which commonly held 4 or more gallons of water for each pound of bleaching powder. Mixing may be done by hand or by a paddle stirring-mechanism. A period of 12 to 24 hr. is required to ensure complete dissolving and clarification of the solution. The inert constituents of the bleach are drawn off through a drain pipe, the clear solution being led to the solution tank, where it generally is diluted to a strength of 1 or 2 per cent and stirred to maintain uniform strength. The hypochlorite solution may be fed to the sewage from an

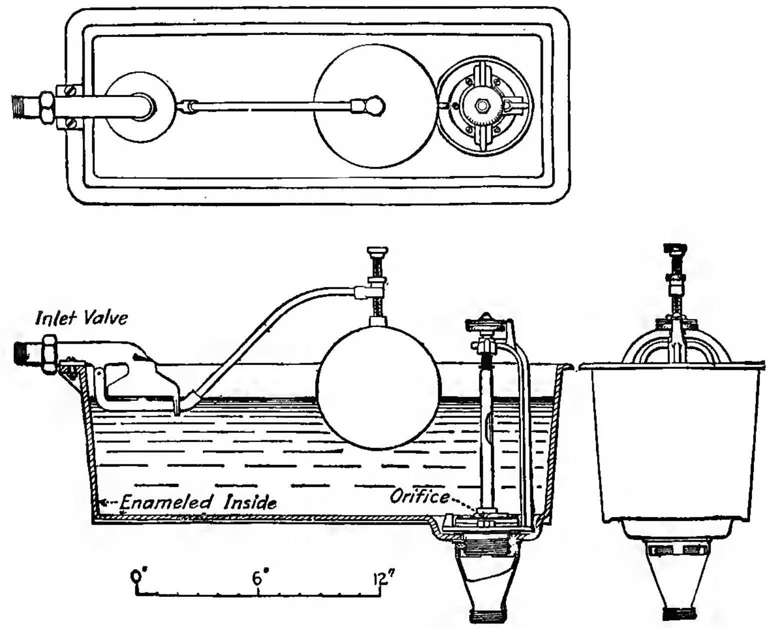


Fig. 215.—Orifice-feed tank (Pittsburgh).

orifice-feed tank, illustrated in Fig. 215, in which a constant head is maintained on an adjustable orifice. Such an orifice requires frequent inspection, as it is likely to become clogged with suspended matter carried over from the mixing tank.

An example of a simple apparatus for temporary, small-scale use is shown in Fig. 216 (1). It was constructed by the Indiana State Board of Health from three barrels, a commercial constant-level regulating box, a small quantity of piping and a geared mixing contrivance much like that used on some types of ice-cream freezers.

In practice, the preparation of a disinfecting solution from bleaching powder is disagreeable work, on account of the dust and fumes, and it is more or less unsatisfactory, because of the irregularity in the composition of the powder and consequent care needed to maintain a solution of suitable strength and because of the tendency of the measuring orifice to become clogged.

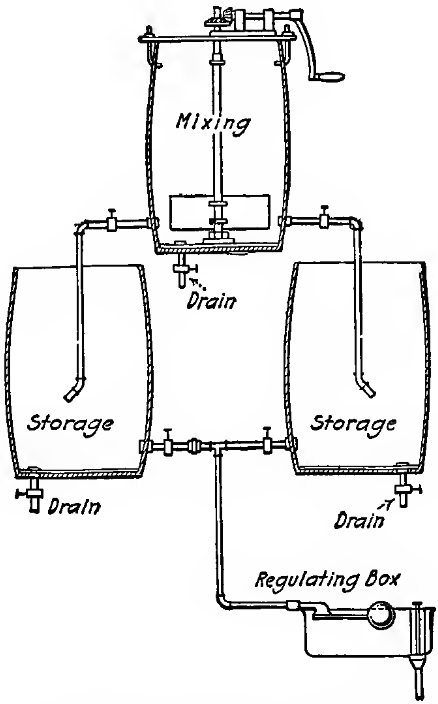


FIG. 216.—Emergency disinfecting apparatus, Indiana State Board of Health.

Quantity of Chlorine Required.—The quantity of chlorine required in the treatment of sewage or sewage effluents depends upon the chlorine demand of the sewage or effluents and the purpose for which chlorination is practiced. As previously pointed out, much of the chlorine added to sewage is used up by unstable sewage matters, before it can destroy the living organisms contained in the sewage. Chlorine demand may be exerted by certain kinds of mineral matter and by organic substances. During the treatment of sewage, the quantity of chlorine required for disinfection or for the control of odors becomes less as the material causing a chlorine demand is settled out or oxidized.

The oxidizable mineral substances are chiefly hydrogen sulfide, sulfites, nitrite nitrogen and ferrous iron. The important deodorizing effect of chlorine in sewage treatment is associated chiefly with its great affinity for H_2S, a foul-smelling gas. The quantity of chlorine used up by these mineral substances can be readily calculated from their combining weights. Thus 1 p.p.m. of H_2S will combine with $^{71}/_{34} = 2.1$ p.p.m. of Cl_2.

Since the constitution of the organic matter cannot be determined, its chlorine demand can be found only by test. Generally speaking, the chlorine demand of sewage parallels its oxygen demand. With the increase in staleness of sewage, the soluble organic substances which exert an immediate chlorine demand are added to by bacterial action.

The immediate chlorine demand of sewage at a number of American plants is given below (2):

City	Average sewage flow, gal. per capita daily	Immediate chlorine demand, p.p.m.
Akron, Ohio	156	25.5
Columbus, Ohio	97	6.7
Dayton, Ohio	92	11.0
Decatur, Ill	306	9.5
Flint, Mich	75	12.0
Indianapolis, Ind	140	7.9

For odor control a chlorine dose somewhat in excess of the quantity theoretically required to combine with the hydrogen sulfide is required, because of the interfering demand of other sewage matters. For disinfection enough chlorine is needed to provide what is known as a "residual chlorine" content. State authorities have specified a disinfecting dosage varying from 20 to 25 p.p.m. for raw sewage, 15 to 20 p.p.m. for settled sewage and 5 to 15 p.p.m. for completely treated sewage. Since sewages and sewage effluents vary greatly in character, it seems better practice to base the dosage on the residual chlorine to be secured. Values of 0.2 to 0.5 p.p.m. of residual chlorine after 10- to 15-min. contact appear to be adequate.

The chlorine demand of sewage varies markedly during the year. The monthly variations in chlorine dosage, as found by Tiedeman (3) in experiments on Imhoff-tank effluent at Huntington, N. Y., to be required at times of maximum hourly demand to maintain a residual of 0.5 p.p.m., were as follows:

Jan.	9.6
Feb.	6.4
Mar.	7.1
Apr.	8.7

May	9.7
June	11.7
July	12.7
Aug.	12.3
Sept	10.3
Oct	11.7
Nov	9.8
Dec	7.1
Ave	9.6

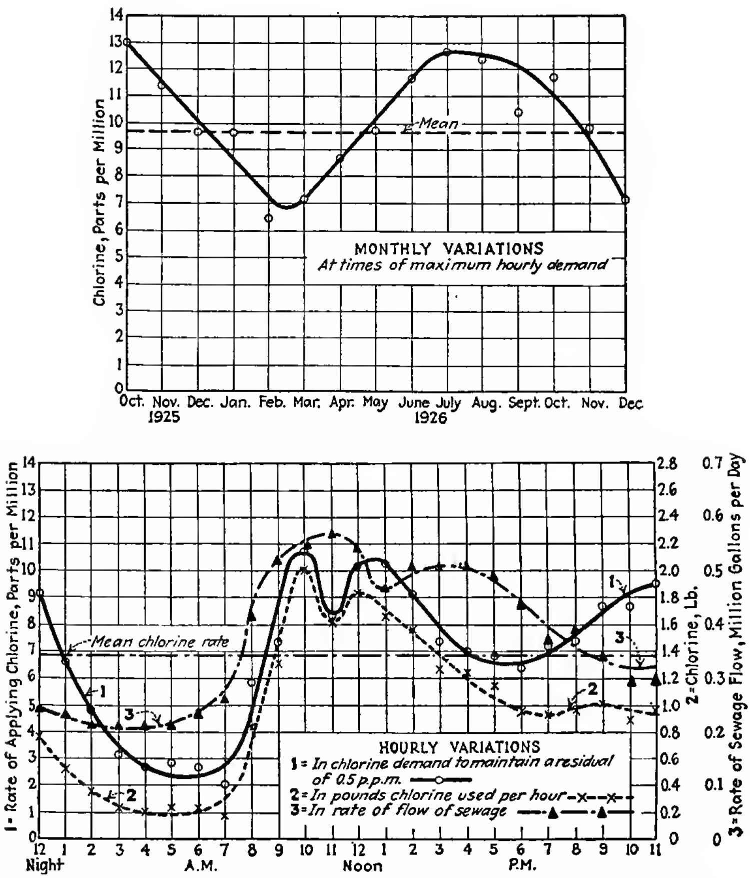


Fig. 217.—Monthly and hourly variations in chlorine demand of Imhoff-tank effluent at Huntington, N. Y.

The hourly variations during a single day were shown to vary from 2 to 10.6 p.p.m., with an average of 6.8 p.p.m. These variations are presented graphically in Fig. 217. This experience indicates how markedly the chlorine requirements vary and how economy of operation in a large plant calls for fairly close regulation of chlorine dosage in order to maintain residual chlorine in the ever changing sewage.

The chlorine dosage required to effect a residual of 0.3 p.p.m. after 10 min. varied in the case of the Bridgeport, Conn., screened sewage between 10 and 15 p.p.m. during the period from October, 1926, to April, 1927.

The doses of chlorine used by some of the New Jersey sewage-treatment plants as given by Daniels (4) are as follows:

Location	Other treatment	Available chlorine	
		Parts per million	Lb. per million gallons
Atlantic City	Coarse screens	25	208
Ventnor	Imhoff tank	18	150
Ocean City	Single-story tank	14	117
Wildwood	Fine screens	11–12	100
Millville	Double sedimentation	9–12	100
Morristown	Sand filter	3–7	59
Haddon Heights	Sand filter	1–2	17

The 1932 requirements of the Division of Sanitation of the New York State Department of Health call for maximum dosing capacities, based on average flows, as follows:

Nature of sewage	Available chlorine	
	Parts per million	Lb. per million gallons
Raw sewage	24	200
Settled sewage	18	150
Effluent from contact bed or trickling filter	12	100
Effluent from sand filter	6	50

Enslow (5) has summarized as follows the quantity of chlorine required to disinfect sewages and effluents of various kinds:

Nature of sewage	Available chlorine required, p.p.m.
Raw or settled:	
Fresh to stale	5–15
Septic	10–25
Trickling-filter effluent	2– 5
Activated-sludge effluent	1– 3.5

Tests for Free Chlorine.—The methods commonly employed for the determination of residual chlorine are the starch-iodide test and the orthotolidin test. They yield somewhat different results and generally the orthotolidin test is used. The latter consists simply of adding a small quantity of orthotolidin solution to the sample to be tested and comparing the resulting color with that of standards prepared from solutions of copper sulfate and potassium dichromate (6). Small quantities of free chlorine give a yellow color with orthotolidin and larger quantities give an orange color.

Point of Application of Chlorine.—The chlorine demand of sewage is affected by its concentration, composition and condition. In connection with the latter it is found that the chlorine demand of septic sewage is much greater than that of fresh sewage. It may be advantageous, if the sewage is septic before arriving at the plant or point of disposal, to apply the chlorine either in part or as a whole at some point nearer the origin. By destroying the sulfate-splitting organisms before hydrogen sulfide is produced, a considerable saving in chlorine may be secured. Thus, from tests made upon the Orange County, Cal., outfall sewer, it was estimated that, whereas it would have required approximately 75 p.p.m. of chlorine to neutralize the hydrogen sulfide at the end of a 2-mile force main, only 17 p.p.m. actually were required by applying it at the influent end (7).

At Independence, Kan., it was found that, when chlorine was applied to the effluent from Imhoff tanks, more chlorine was consumed, more hydrogen sulfide was found in the chlorinated sewage and odor reduction was less satisfactory than when prechlorination was employed (8). At Dallas, Tex., the crude sewage entering Imhoff tanks required 30 to 75 per cent less chlorine to produce residual chlorine than did the tank effluent (9). At West Haven, Conn., where the flow chambers were in bad condition at the time of the test, the chlorine demand of the effluent was 66 per cent greater than that of the crude influent. On the other hand, at the Fort Worth, Tex., plant, where the crude sewage is known to contain only a minor quantity of sulfate in solution, one or two tests indicated that practically the same dosage of chlorine was required for the crude influent as for the Imhoff effluent. Studies at Schenectady, N. Y., however, revealed a smaller chlorine demand by crude sewage than by Imhoff-tank effluent in the early spring, when the tanks were filled with sludge and the effluent had a tendency to become slightly septic (10). Under normal conditions, the same chlorine demand was exerted by the crude influent as by the settled effluent.

At Plainfield, N. J., in tests to secure elimination of odors, it was found by O'Connell that chlorination of sewage before it entered Imhoff tanks proved more efficient than chlorination of the tank effluent (11). However, he stated that at larger plants some economy should result from the combination of chlorination of tank influent with chlorination of tank effluent. O'Connell also stated that the addition of chlorine to sewage in advance of all treatment, including screening, except in rare cases cannot be considered as a feasible method of odor control. Re-inoculation and after-growth of bacteria, with subsequent formation of hydrogen sulfide, are said to appear inevitable. In discussion, Enslow reported that at Canton, O., chlorination at the plant is more effective than application of chlorine to the trunk sewer at a point more than 5 miles above the plant.

The application of chlorine to tank influents appears to favor tank operation. The flow chamber is maintained in a fresher condition than without chlorine and, as stated in Chap. XVI, digestion of the sludge solids has been found not to be affected adversely.

Concerning "split" chlorination Enslow (9) reports as follows:

> It appears from preliminary studies made at the Fort Worth plant that a minor dosage of chlorine (about one-third that necessary to satisfy the chlorine demand) is all that is necessary to stay hydrolysis and septic action in a highly organic and putrefiable substance, such as packing-house wastes.
>
> It is highly probable that a relatively small dosage of chlorine, if applied to the sewers direct, at a point some distance ahead of the disposal plant, may serve to retard materially the progress of biological activity in the sewers. If so, the sewage arriving at the plant in a partially preserved condition will have more of the qualities of fresh sewage. The solids will be less in solution and more of them will remain in a settleable condition. The oxygen demand should be less and the quantity of chlorine required materially less. Thus it appears probable that chlorination "split" between the sewer proper and the influent to the disposal plant will effect an over-all economy.

Efficiency of Chlorination.—The efficiency of disinfection with chlorine depends upon the quantity of chlorine added to the sewage and the period of contact. Tiedeman (3) found that at Huntington, N. Y., settled sewage could be disinfected satisfactorily when a residual of 0.2 p.p.m., as indicated by the orthotolidin test, was maintained. The results obtained at Huntington are shown in Fig. 218.

In order to exemplify the relative destruction of bacteria by chlorination and their removal from sewage by other sewage-treatment

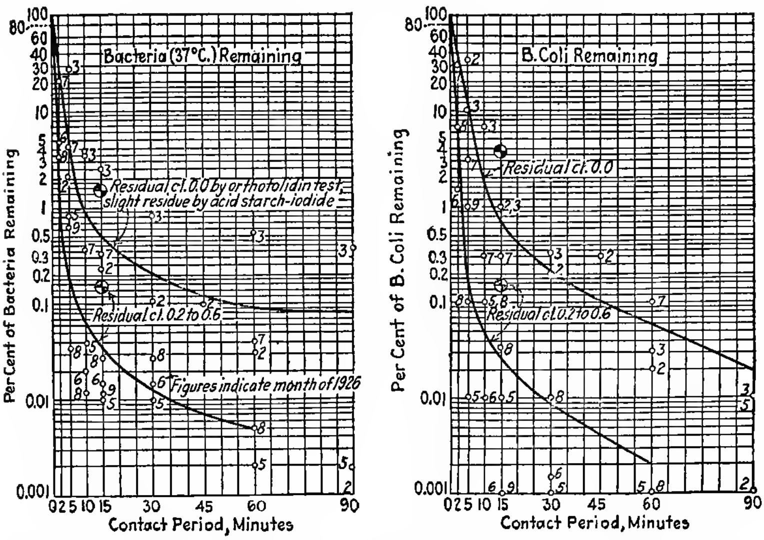


FIG. 218.—Efficiency of chlorination of Imhoff-tank effluent, Huntington, N. Y.

methods, Table 145 is presented. The figures given, however, are only approximations.

TABLE 145.—REMOVAL OR DESTRUCTION OF BACTERIA BY DIFFERENT TREATMENT PROCESSES

Process	Removal of bacteria, per cent	Process	Removal of bacteria, per cent
Coarse screening	0–5	Activated-sludge process	90–98
Fine screening	0–10	Intermittent sand filtration	95–99
Grit-chamber treatment	0–5	Sedimentation and chlorination	90–95
Plain sedimentation	25–75	Oxidation and chlorination	98–99
Septic-tank treatment	25–75		
Chemical precipitation	40–80		
Contact-bed treatment	40–80		
Trickling-filter treatment	70–85		

Chlorination at Cleveland, Ohio, in 1926, during the months of June to September, inclusive, when the crude sewage at the Easterly plant and the Imhoff-tank effluent at the Westerly plant were chlorinated, is described by Enslow (9) as follows:

> The easterly sewage is relatively fresh, whereas the westerly tank effluent is impregnated with packing wastes, sulphate of iron, and other industrial wastes. It is also septic and odoriferous during warm weather.
>
> The dosages of chlorine applied during the summer season averaged 8.9 p.p.m. to the westerly (Imhoff) effluent and 8.2 p.p.m. to the crude easterly sewage. In the Imhoff effluent the chlorine demand during the daytime was always in excess of that applied and no residual chlorine could be found upon test. In the crude easterly sewage residual chlorine to the extent of 0.2 p.p.m. or more could be maintained.
>
> In the case of the chlorinated crude sewage the total bacteria and *B.coli* reduction . . . averaged 96.33 and 94.5 per cent, respectively, as compared with an average reduction of only 79.4 per cent total bacteria and 84.6 per cent *B.coli* in the Imhoff effluent. This is but another indication of the difficulty of chlorinating septic sewage and also of the great importance attached to the maintenance of residual chlorine in a treated sewage.

Operating results at Pasadena, Cal., during the year ending June 30, 1929, show that after chlorination the effluent from the activated-sludge plant contained 20 *B.coli* per cubic centimeter. An average of 20 lb. of chlorine was applied per m.g., equivalent to 2.4 p.p.m., giving a residual of 0.22 p.p.m. of free chlorine.

In tests on chlorination of the activated-sludge effluent at the Des Plaines treatment plant in Chicago during September, 1927, *B.coli* in the effluent were reduced from 3900 per cubic centimeter to 115 per cubic centimeter by the application of 25 lb. of chlorine per million gallons, equivalent to a dosing rate of 3 p.p.m.

At Boonton, N. J., sewage is treated in sedimentation tanks, contact beds and sand filters and the effluent is chlorinated at a rate of 2 p.p.m. In 1930 a bacterial reduction of 99.99 per cent was obtained. At no time in the 27 months of operation reported upon were *B.coli* present in 10-cc. samples (12).

Reduction of Biochemical-oxygen Demand by Chlorination.—The fact that much of the chlorine applied to sewage is used up by unstable sewage matters is reflected in a reduction of the biochemical-oxygen demand of the sewage. This reduction may be of importance in preventing nuisances below treatment works which discharge their effluents into small streams that provide adequate dilution only by accession of other waters or by emptying into larger bodies of water.

At Columbus, Ohio, during 1927 some investigations were made to determine the effect of chlorine treatment on the B.O.D. of the trickling-filter effluent (13). In making these tests, the immediate chlorine demand was determined and the quantity of chlorine added was 0.2 p.p.m. less than the demand. The purpose of this underdosage was to prevent sterilization of the sample. The mean values from 45 determinations are shown in Table 146, the quantity of chlorine added averaging 2.11 p.p.m.

TABLE 146.—REDUCTION IN B.O.D. OF TRICKLING-FILTER EFFLUENT BY CHLORINATION AT COLUMBUS, OHIO

	Biochemical-oxygen demand			
	1 day	2 days	3 days	5 days
Untreated effluent, p.p.m.	14.2	20.7	25.9	33.0
Treated effluent, p.p.m.	4.9	10.2	15.0	19.3
Reduction, per cent	65.5	50.8	42.1	41.5

Experiments conducted at the Des Plaines sewage-treatment plant in Chicago during 1928 showed an average reduction in oxygen demand of 24.7 p.p.m., with 10.9 p.p.m. of chlorine applied to settled sewage. The residual chlorine was 0.2 p.p.m. Apparently the reduction was equivalent to 2.3 p.p.m. of oxygen demand for each part per million of chlorine absorbed.

At Allentown, Pa., catch samples of Imhoff-tank effluent for the period July, 1930, to May, 1931, inclusive, showed 95 p.p.m. of 5-day B.O.D. prior to chlorination and 83 p.p.m. after chlorination. The average rate of application was 4.86 p.p.m. of chlorine and the apparent reduction in B.O.D. was 2.5 p.p.m. for each part per million of chlorine applied. The samples were neither neutralized nor seeded (14).

The results of large-scale experiments by Baity and Bell (15) upon chlorination of Imhoff-tank effluent at Chapel Hill, N. C., in 1928 are given in Table 147.

TABLE 147.—REDUCTION IN B.O.D. OF IMHOFF-TANK EFFLUENT BY CHLORINATION AT CHAPEL HILL, N. C.

Chlorine dosage p.p.m.	Residual chlorine after 10 min., p.p.m.	B.O.D. of unchlorinated samples, p.p.m.	B.O.D. of chlorinated samples, p.p.m.	Reduction, per cent
5– 7.5	0	77.0	71.1	7.4
8– 9	Trace	94.3	79.6	15.5
10–15	0.2–0.5	85.2	48.9	42.7
Average of all samples	....	85.8	58.6	31.7

Further work by Baity (16) at Chapel Hill in 1930 indicated that an apparent reduction of 2.3 p.p.m. in 5-day B.O.D. was accomplished for each part per million of chlorine applied to the Imhoff-tank effluent.

As a result of their experiments, Baity and Bell concluded "that chlorination of sewage for B.O.D. reduction can be advantageously and economically applied in many cases, particularly where plants are overloaded, or slightly inadequate to protect stream quality, or to tide over dry periods of the year." They pointed out, however, that "chlorination is limited in efficiency, and cannot be employed alone where a high degree of removal of organic solids is required."

At Huntington, N. Y., Tiedeman found that with 0.2 p.p.m. of residual chlorine in the tank effluent the oxygen demand was one third less than that of the unchlorinated effluent.

From studies of sewage chlorination at Schenectady, N. Y., Cohn (10) concluded that chlorination followed by sedimentation in Imhoff tanks produced a reduction in B.O.D. 27 per cent greater than that brought about by the tanks without chlorination. Residual-chlorine determinations failed to disclose excess chlorine at any point in the tanks or in the effluent at any time.

After a review of the available data on the reduction of oxygen demand by chlorination, the Committee on Sewage Disposal, American Public Health Association, reached the following conclusions (17):

1. Chlorination, to give residuals of 0.2 to 0.5 p.p.m. after 10 min., produces a reduction in oxygen demand (5-day) of 15 to 35 per cent; the reduction expressed in p.p.m. being roughly proportional to the quantity of chlorine absorbed.
2. The evidence indicates a reduction of at least 2 p.p.m. of oxygen demand for each part per million of chlorine absorbed, the reduction expressed in parts per million being fairly constant, irrespective of the number of days in the incubation period.
3. The disturbance of the normal flora and fauna in sewage effluents by the addition of chlorine leads to difficulties in the use of the B.O.D. test which will require further research before the proper interpretation can be made of some of the reported results.
4. Chlorination sufficient to produce free chlorine in a stream receiving the chlorinated sewage or effluent will retard the normal decomposition processes, thereby tending to prevent anaerobic conditions during the normally critical period of rapid oxygen depletion. Such retardation of decomposition permits opportunity for reaeration and additional dilution from tributary streams to maintain aerobic conditions.
5. In the case of a stream containing extensive sludge deposits, the effect of excess chlorine carried in a sewage effluent may be negligible in so far as the maintenance of aerobic conditions is concerned.

Chlorine as a Deodorant of Sewage.—Chlorine has been used effectively for preventing the dissemination of disagreeable odors from sewage channels, tanks and trickling filters. As heretofore stated, the important deodorizing effort of chlorine in sewage treatment is associated chiefly with its great affinity for H_2S, a foul-smelling gas. The chlorine demand of other sewage matters, as well as of the H_2S, must first be satisfied in order to make odor control by chlorine wholly effective.

Chlorine has been utilized for the control of odors in sewers on a large scale in California, particularly in the Los Angeles County Sanitation Districts. Griffin has asserted that, in the control of odors in the trunk sewer at Elizabeth, N. J., the chlorine demand varied from 10 to 20 p.p.m. but that 8 p.p.m. proved an effective dose. Donaldson (18) has stated that, for an operating period of 5 months, the average quantity of chlorine used was 5 p.p.m.

Scott (19) has recommended, as a result of experience at Macon, Mo., that chlorine be applied to reduce the hydrogen sulfide in the Imhoff-tank effluent to 1.5 p.p.m., in order to avoid odors from the trickling filter. Cohn (20) reports that the application of 4 to 6 p.p.m. of chlorine to the influent of the Imhoff tanks was effective in eliminating odors from Imhoff tanks, dosing tanks and trickling filters at Schenectady, N. Y. Hackmaster (8) has found that application of 3.32 p.p.m. of chlorine to crude sewage at Independence, Kan., eliminates odors from Imhoff tanks and trickling filters.

A discussion of chlorination of trickling filters for the control of odors or for the destruction of surface clogging is given in Chap. XXI.

Gascoigne (21) states that more than 50 plants in this country are using chlorine for the control of odors, including Alliance, Ohio, Flint, Mich., Independence, Kan., Plainfield, N. J., San Bernardino, Cal., and Schenectady, N. Y.

Cost of Chlorination.—The cost of chlorination, of course, varies widely with different local conditions and requirements. Gascoigne (21) has prepared the following summary of costs of chlorination for disinfection, odor control, reduction of pooling on trickling filters and reduction of B.O.D.:

Chlorine.—Chlorine is now available for purchase in 100- and 150-lb. cylinders, ton containers, and tank cars, the price depending upon the type of container and upon the distance of shipment or freight cost. With cylinders purchased as needed, the price will range from 6 to 7 cents per pound of chlorine; with cylinders in carload lots from 4 to 5 cents per pound, while ton containers range from 2½ to 3 cents per pound, to which figures it is necessary to add the cost of freight in both directions.

Equipment.—Machines with a 10-lb. capacity in 24 hr. cost about $750, while machines with a capacity of 700 lb. per 24 hr. cost about $3,500. The variation in cost is more or less uniform between these limits.

Labor.—The cost of labor or attendance and maintenance of chlorinating equipment is relatively small. Very seldom, and even where small quantities of chlorine are used, does this figure exceed ½ cent per pound of chlorine used.

Cost of Disinfection.—Requirements may vary from 50 to 90 lb. per mil. gal. for crude, fine screened and settled sewage; from 15 to 25 lb. for activated sludge or trickling-filter effluent. The size of plant will affect the cost. For disinfecting the former class of sewage the cost is estimated at from $4.50 to $7 (per mil. gal.) for plants averaging under 1½ m.g.d.; from $3.25 to $4.50 for those averaging between 1½ and 2½ m.g.d.; and from $1.75 to $3.25 for those with more than 2½. For the latter class of sewage, the cost would range from $0.50 to $2.50 per mil. gal. daily.

Cost for Odor Control.—With a range in quantity of chlorine required from 75 to 100 lbs. per mil. gal., and applying it 3 to 10 P.M. during 150 days of the year, the cost will be $0.50 to $2.50 per mil. gal. This figure would be substantially reduced if spread over the flow for the entire year. The cost at Plainfield last summer, averaging 98 lb. of chlorine per day, was $1.42 per mil. gal. of sewage passing through the plant, which averaged 3.3 m.g.d. About 10,000 lb. of chlorine was used during the year 1929 to deodorize stack gases at the Milwaukee sewage plant at a cost of $3,000.

Cost of Relieving Pooling.—Assuming dosing during 12 hr. of the night for six consecutive nights, at rates of 20 to 30 p.p.m., and sewage application of 1 to 2 m.g. per acre of filter, figures about 1,000 lbs. of chlorine per acre, which might be increased 50% for extreme cases. On this basis, the cost for filters of less than fifteen acres is from $65 to $100 (per m.g.); for 15 to 25 acres, $45 to $65; and for 25 acres or over, $25 to $45.

Cost for Reducing B.O.D.—On the basis of applying 70 to 100 lb. per mil. gal. during 120 days of the year, the cost for plants of less than 1½ m.g.d. would be $5.50 to $8.50 (per mil. gal.); those of 1½ to 3 m.g.d., $3.75 to $5.50; and those more than 3 m.g.d. from $2.25 to $3.75.

Bibliography

1. *Eng. and Contracting*, 1912, **38**, 72.
2. Rept. of Comm. on Sludge Digestion, Sanitary Eng. Div., Am. Soc. C. E. (1931).
3. Tiedeman, W. V. D.: *Eng. News-Rec.*, 1927; **98**, 944.
4. Fuller, G. W., and J. R. McClintock: "Solving Sewage Problems," 1926.
5. Enslow, L. H.: *Eng. Jour.*, 1929; **12**, 136.
6. "Standard Methods for the Examination of Water and Sewage." Am. Pub. Health Assoc. and Am. Water Works Assoc., 7th ed., 1933.
7. Goudey, R. F.: *Sewage Works Jour.*, 1929; **1**, 196.
8. Hackmaster, G. H.: *Eng. News-Rec.*, 1927; **99**, 1031.
9. Enslow, L. H.: *Pub. Health Repts.*, 1927; **42**, 1623.
10. Cohn, M. M.: *Eng. News-Rec.*, 1927; **99**, 229.
11. O'Connell, W. J., Jr.: *Proc.*, N. J. Sewage Works Assoc., 1928, p. 20.
12. *Sewage Works Jour.*, 1932; **4**, 581.
13. *Public Works*, 1928; **59**, 309.
14. Krum, H. J.: *Sewage Works Jour.*, 1931, **3**, 647.

15. Baity, H. G., and F. M. Bell: *Jour.*, N. C. Water and Sewage Works Assoc., 1928; **6**, 151.
16. Baity, H. G., Fred Merryfield and A. B. Uzzle: *Sewage Works Jour.*, 1933; **5**, 429.
17. Rept. of Comm. on Sewage Disposal, Public Health Eng. Sec., Am. Pub. Health Assoc., 1934.
18. *Water Works and Sewerage*, 1933; **80**, 133.
19. *Public Works*, 1929; **60**, 400.
20. Cohn, M. M.: *Water Works and Sewerage*, 1931, **78**, 166.
21. *Public Works*, 1931; **62**, no. 2, p. 28.

CHAPTER XXXI

TREATMENT AND DISPOSAL OF RESIDENTIAL AND INSTITUTIONAL SEWAGE

The principles which govern the satisfactory disposal of the sewage from residences, estates and small institutions are the same as those underlying successful sewage disposal on a large scale. In applying the principles, however, allowance is made for certain factors which influence the design of small plants. In the first place, the sewage is discharged at rates which fluctuate widely, not only from hour to hour of the same day but also from day to day during a week. Where the desired degree of purification of the sewage is high and the treatment involves methods of filtration which should proceed at fairly regular rates, it is evident that the storage of sewage, so as to permit fairly uniform delivery to the filters and some uniformity in the composition of the applied liquid by mixing the laundry wastes, kitchen wastes and domestic sewage together, becomes particularly important. In the second place, the small size of the plants makes it desirable to have them as nearly fool-proof and automatic as possible. Even if the owner's means render economy in management unnecessary, the importance of automatic operation is great, because experience shows that regular attendance is rarely given to these small plants and they are likely to receive no care until something goes wrong and their existence is indicated in some unpleasant way.

There are two general methods of disposing of sewage from residences and small institutions: namely, by the dry-pail system and by the water-carriage system.

DISPOSAL OF FECAL MATTER NOT CARRIED BY WATER

Box-and-can Privies.—The box-and-can type of privy consists essentially of a flyproof box containing a removable water-proof receptacle or can for excreta. This type of privy has been used for many years in Europe and its general principles have been modified and adapted for use to a considerable extent in the unsewered areas of cities and towns in several of the southern states. To a smaller extent and in a somewhat different way, the box-and-can type of privy is used at camps and rural homes in the northern states. As used in the southern states, a scavenging system is essential to successful and sanitary operation. The filled cans are removed periodically. Either they are placed on a platform cart and washed cans are left in their places, or the contents of the cans are transferred to a tank truck and the cans are partially washed and returned. In either case, the collected material is disposed of by dumping it into a manhole or special hopper provided on a sewer, by applying it to land or by dumping it directly into a large

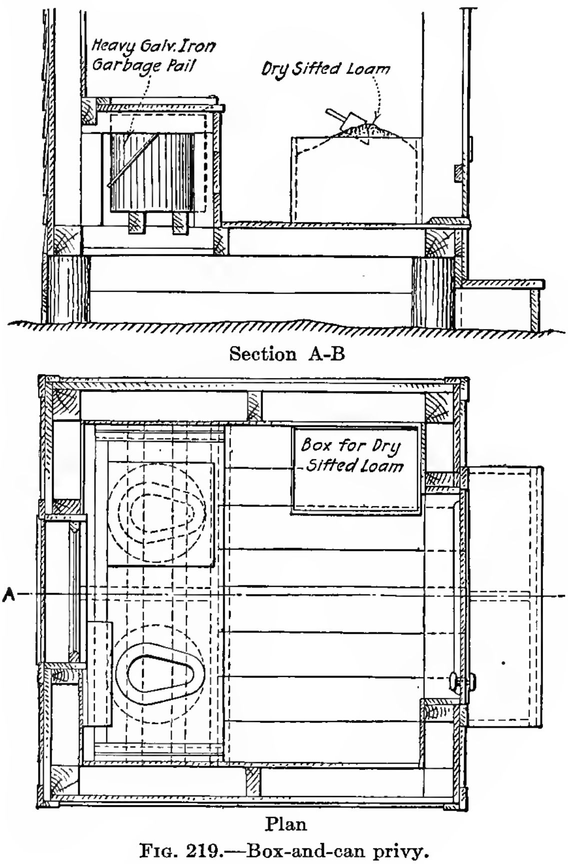


FIG. 219.—Box-and-can privy.

stream. The scavenging is usually done by city employees, but sometimes it is done by a contractor. The householder is assessed by the city for this service and payment is collected by some city official.

For rural use, where municipal collection is not practicable, the cans must be removed, emptied, washed and replaced frequently. The contents should be buried where they can do no harm. It is desirable to have duplicate cans, so that each may dry thoroughly before being replaced. The use of small cans which must be emptied before their contents become offensive is advantageous. The required addition of dry loam or dry powdered peat to the contents of the can minimizes odors. Figure 219 illustrates such a privy, in which exclusion of flies is accomplished by having the seat fit tightly upon the top of the can. Box-and-can privies have been used on certain watersheds as a means of protecting the water supply, the night soil being transported away from the watershed. The difficulties of caring for such privies during freezing weather are obvious.

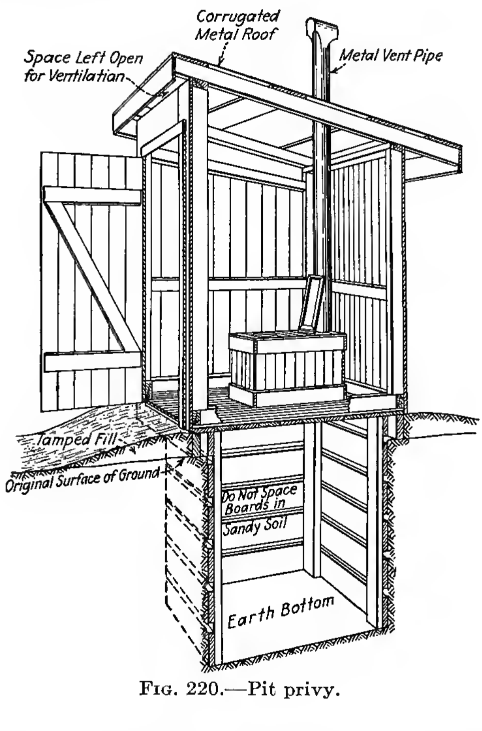


Fig. 220.—Pit privy.

Pit Privies.—The pit privy is the cheapest and simplest form of privy available. It consists of an excavation in the ground which, when suitably constructed, is flytight and curbed to prevent entrance of storm water. Over the excavation is placed a privy house containing a seat. Cleaning of this type of privy is not needed, for when the pit is nearly full, another can be dug near by. Then the house can be moved to a new location and the old pit filled in. Figure 220 represents such a privy, as recommended by the Tennessee State Department of Health. If such a structure were to be flytight it would be necessary to screen all ventilating openings and provide a spring which would ensure closing of the door. They recommend also a pit privy in a somewhat improved form, having a concrete slab for the floor with seat riser cast integrally or made of cast iron. The concrete slab carrying the house and seat can be moved to a new location when the pit becomes nearly full. In Texas a corrosion-resisting metal slab, with turned-down edges and metal-riser, is in use (1).

Vault Privies.—When space does not permit moving the privy to a new location each time that the pit is filled, a permanent location can be utilized by constructing the privy over a concrete vault, which is cleaned out periodically, the contents being disposed of by burial. An improved type consists of a two-compartment vault, each compartment of which has a capacity of 75 to 100 gal., or theoretically sufficient capacity to provide three to six months' storage for the wastes of an average family. A clean-out opening is provided at the rear of the privy house. One compartment is used until filled. The seat cover is then fastened down and the contents are allowed to remain undisturbed until the second compartment is filled. The first compartment is then scavenged and put into service, while the second stands idle in turn. When first developed, it was expected that this procedure would produce a somewhat dried, inodorous material, free from pathogenic bacteria, but in practice liquefaction tends to take place, making scavenging difficult. The contents are usually far from inodorous. The importance of fly-tight construction is obvious.

Many concrete-vault privies were constructed at the cantonments build during the World War. They lessened soil pollution and aided in preventing the spread of disease. More recently, however, these vaults have presented difficult problems of scavenging and maintenance. The actual disposal of the vault contents by burial often nullifies the value of a vault in preventing soil pollution.

Septic Privies.—A type of privy developed by Lumsden, Roberts and Stiles of the U. S. Public Health Service, sometimes called the "L.R.S. privy," consists of a two-compartment or baffled watertight tank, provided with an overflow to drainage tile. The seats and house are mounted directly over the tank. Water is added regularly and the scum is broken up frequently. Although at first widely advertised as a type of privy requiring little or no care, it actually needs regular attention in order to give satisfactory service. The capacity required depends upon conditions of use. The following capacities, recommended by Hardenbergh (2) for average installations, allow a margin for safety:

For 7 persons or less	200 gal. working capacity
For 8 to 10 persons	250 gal. working capacity
For more than 10 persons	Add 20 gal. per capita

Anaerobic bacterial action takes place within the septic privy, as in a septic tank, changing and reducing the solid matter. Addition of horse manure or sludge from another tank hastens bacterial action when starting a new tank. Provision is made, by means of a manhole or removable top, for removing sludge when necessary. The frequency with which cleaning is required depends largely upon the capacity of the tank and the care given it. The interval between cleanings has been found to vary from six months to five years. When the interval between cleanings is long, there is danger of flushing accumulated sludge solids into the tile drainage system, which may clog it.

Chemical Closets.—There are two types of toilets utilizing chemicals to deodorize and sterilize the excreta deposited within them. The commode type consists of an outer metal container or box, equipped with a water-closet seat and a ventilator pipe, which may be extended through the roof or wall or else connected to a chimney flue. Inside the outer container is a watertight can or pail, in which is placed a small quantity of chemical solution. This can is emptied, as in the case of the box-and-can privy, at intervals of a week or oftener, depending upon the number of persons using it.

The storage type consists of a tank, usually with an approximate capacity of 125 gal., directly over one end of which is placed a vitreous-enamel closet bowl, much resembling the ordinary flush bowl. The bowl is connected to the tank by a straight pipe. The tank may be so located that one end with a manhole is outside the building, while the other end, with the bowl connected, is inside. One manufacturer provides an overflow for the effluent, to be connected to a tile drainage field. Tanks produced by other manufacturers have a drain valve to empty the tank when nearly filled. In this case, the contents are discharged to a drainage sump. The tanks are provided with agitators to break up the solids and keep the contents well mixed. It is essential to satisfactory operation that the agitators be used daily or oftener.

Caustic soda is the principal ingredient of the chemical commonly used in the tanks and commodes. For the latter, phenol compounds and pine-tar disinfectants are sometimes supplied. Probably complete sterilization is seldom obtained with other than the caustic soda.

The tank type of chemical closet has been provided for use by construction crews of water-supply reservoirs, where it was necessary to prevent the pollution of a stream already being used as a water supply or to prevent contamination of the reservoir bottom. On one job a tank cart was utilized to transport the chemical-tank contents outside the drainage area.

DISPOSAL OF WATER-CARRIED WASTES

Quantity of Sewage from Residences and Institutions.—As pointed out at the beginning of this chapter, the sewage discharged from residences and small institutions is subject to uncertain and fluctuating flow. Hence it is often difficult to estimate the quantity of ewage to be handled in a small treatment plant.

A study of sewage flow from farm homes has led Lehmann, Kelleher and Buswell (3) to suggest that in designing septic tanks the following allowances be made for average sewage flow from homes of different sizes:

7 persons, 25 gal. per capita daily
9 persons, 23 gal. per capita daily
12 persons, 20 gal. per capita daily
15 persons, 18 gal. per capita daily

The quantity of sewage from institutional plants varies greatly with the extent of the laundry and bathing activities, the admission of roof water to the sewers and the ground-water infiltration. In the case of the treatment plant of the Boys' Industrial School at Lancaster, Ohio, the State Board of Health reported in 1908 that the rate of flow between 10 P.M. and 5 A.M. was uniformly 37 gal. per capita, while the average

TABLE 148.—QUANTITIES OF SEWAGE FROM INSTITUTIONS

Institution	Population	Gal. per capita daily
Alliance, Ohio, Children's Home	175	80 measured
Bedford Station, N. Y., Montefiore Home	300	100 estimated
Collingwood, Ohio, Lake Shore R. R. shops	1,525	131 measured
Dayton, Ohio, Montgomery County Infirmary	350	21– 29 measured
Gallipolis, Ohio, County Infirmary	40	40 estimated
Gallipolis, Ohio, State Hospital, main buildings	1,450	122 measured
Gallipolis, Ohio, State Hospital, Cottage I	225	127–211 measured
Lake Kushaqua, N. Y., Stony Wold Sanatorium	125	120 estimated
Lancaster, Ohio, Boys' Industrial Home	1,150	50– 55 measured
Mansfield, Ohio, State Reformatory	820	67– 83 measured
Marietta, Ohio, Washington County Infirmary	80	31 measured
Massillon, Ohio, State Hospital	1,600	137 measured
Morgan's, Ohio, Institute for Feeble Minded	265	53 measured
Oakdale, Mass., County Truant School	50	60 measured
Pleasantville, N. Y., Hebrew Children's Home	1,100	40–100 measured
Sandusky, Ohio, Soldiers' and Sailors' Home	1,125	151 measured
Southborough, Mass., St. Mark's School	216	140 measured
Toledo, Ohio, State Hospital	2,000	150 measured

for 24 hr. was only 50. In general, the quantity of sewage varies from 15 to 175 gal. per capita daily, depending upon the type of institution served.

The rates of flow given in Table 148, obtained by measurement except in three cases, indicate that institutional sewage is likely to have a considerable volume per capita.

The following figures give the range of quantities to be expected from various sources, as estimated by the authors in the design of small disposal plants:

	Gal. per Capita Daily
Industrial plants	15–50
Institutions other than hospitals	22–50
Summer residences	25–75
All-year residences	50–100
Hospitals	100–160

Cesspools.—Cesspools have been used for many years for the disposal of liquid wastes and are still quite common in this country. Their use has been strongly condemned by health officers, because of the danger of polluting nearby wells. In the absence of such wells and in a porous soil, however, they have a limited application.

The *leaching cesspool* is merely a dry-laid masonry well, usually without any masonry on the bottom. The sewage flows into it and leaches into the soil. Eventually the solids in the sewage fill the well and clog the soil, necessitating the digging of another cesspool. It seldom pays to remove the solids because of the clogging of the soil. In constructing a cesspool it is common practice to draw in the masonry of the top of the well and provide some sort of cover for the top, like the top of a manhole, and the upper 5 ft. of the masonry is often laid in mortar to prevent injury by frost. In North Carolina, where the soil is largely impervious clay, the health authorities do not allow the installation of leaching cesspools.

London and Paris at one time had tight cesspools, as described in the introduction to Vol. I, the contents being removed by contractors.

Two cesspools have sometimes been used in tandem on country estates, the first one being tight and acting to retain the solids so that there is little clogging of the soil surrounding the second or leaching cesspool. Septic action takes place in the first cesspool and the solid matter, or night soil, retained in it is removed occasionally.

According to Ryon (4), the size and form of leaching cesspools are of little importance, as long as sufficient absorptive area is provided. It is generally found convenient and economical, however, to make leaching cesspools circular in form, 6 or 8 ft. in diameter and 6 to 10 ft.

deep, constructing a sufficient number to provide the necessary absorptive area.

Table 149 shows the necessary absorptive area for different seepages, as given by Ryon. This table assumes that the cesspools are 6 to 10 ft. deep.

TABLE 149.—NECESSARY ABSORPTIVE AREAS FOR LEACHING CESSPOOLS IN SOILS OF VARIOUS ABSORPTIVE QUALITIES

Time for water to fall 1 in.	Square feet of absorptive area required per capita	
	Dwellings, etc.	Day schools and factories
10 sec. or less	10	2.5
15 sec	11	2.8
30 sec	12	2.9
45 sec	13	3.1
1 min	14	3.5
2 min	17	4.2
3 min	20	5.0
4 min	23	5.8
5 min	25	6.3
10 min	33	8.3
15 min	40	10.0
30 min	63	16.0
45 min	87	22.0
1 hr	110	28.0

In Table 149 the column headed "time for water to fall 1 in." affords a measure of the absorptive quality of the soil. The figures are determined in the following manner:

A pit is dug to approximately one half the depth of the proposed cesspool. A hole about 1 ft. square and 18 in. deep is then dug in the bottom of the pit. This hole, Ryon states, is filled to a depth of about 6 in. with water, care being taken to wet the soil before pouring the water in for the test, if it appears dry. The time that it takes for the surface of the water in the hole to fall 1 in. is then observed.

The temperature has some effect on this test, but for the ordinary temperatures of soil and water, about 50°F., it is sufficiently accurate for practical purposes. The test should not be made, however, in a hole that has been exposed for some time to a hot summer sun and hot water should not be used. In either of these cases, the time for the water to fall 1 in. would be too short to be a measure of the true absorptive quality. On the other hand, if the test were made in frozen soil or with ice water, the time would be too long.

Data concerning typical installations of cesspools are given in Table 150.

TABLE 150.—CHARACTERISTICS OF SEWAGE-TREATMENT PLANTS EMPLOYING SETTLING TANKS AND CESSPOOLS

Location	Barnstable Co. Infirmary, Pocasset, Mass.	Residence, Tyngsborough, Mass.
Number of persons served	60	6
Per capita allowance, gal. daily	50	50
Total sewage flow, gal. daily	3000	300
Settling tanks:		
Number	1	1
Total capacity, gal	3000	
Detention time, hr	24	
Leaching cesspools:		
Number	2	1
Depth below water surface, ft	3.8 and 6.3	3
Diameter, ft	6	4
Absorptive area per capita, sq. ft	3.2	6.3
Volume per capita, gal	35.6	47.1
Sludge-drying beds:		
Number	1	None
Total area, sq. ft	400	
Capacity, persons per acre	6550	
Depth of sand below distributor, in	12	
Character of subsoil at site	Sand	Sand and boulders

Single-story Septic Tanks.—Septic tanks, which have been discussed in Chap. XV, find their widest application in treatment of sewage from residences and institutions, where sewerage systems are not available. The primary object of the septic tank is the removal of suspended solids by sedimentation. A sufficiently large tank is provided to allow a suitable period for the heavier organic material to settle out and for the lighter constituents to rise to the surface as scum. Such tanks, when designed for residences and small institutions, commonly provide a detention period, including sludge space, of 14 to 72 hr., the usual allowance being about 24 hr.

Based upon a study of the factors affecting the efficiency and design of farm septic tanks, Lehmann, Kelleher and Buswell (3) make the following recommendations for the design of such tanks:

For a single-chamber tank provide an effective retention period of 48 hr., with an allowance of 50 per cent additional capacity for sludge storage, or a total retention period of 72 hr. of sewage flow.

For a more efficient plant use a two-chamber tank. Provide a retention period of 72 hr. in the first chamber (effective retention period of 48 hr., with 50 per cent additional capacity for sludge storage) and an additional retention period of 36 hr. in the second chamber, or a total retention period of 108 hr.

Make the minimum-sized tank large enough for 7 people: in order to maintain ample tank dimensions for proper settlement of solids; to allow for additional people in the house; because the reduction in cost is small for tanks under this suggested minimum; with less than 7 people, the additional capacity insures more efficient operation and less frequent cleaning.

The recommendations of these investigators as to capacity and dimensions of septic tanks are summarized in Table 151 (3).

TABLE 151.—SUGGESTED CAPACITY AND DIMENSIONS OF SEPTIC TANKS TO ACCOMMODATE DIFFERENT NUMBERS OF PERSONS

Number of persons	Sewage flow per capita daily, gal.	Capacity required in first chamber for 72-hr. retention		Effective cross-section, ft.	Length for 1-chamber tank or for first chamber of 2-chamber tank, ft.	Length for second chamber of 2-chamber tank, ft.
		Gal.	Cu. ft.			
7	25	525	70	3 × 4	6	3
9	23	620	83	3 × 4	7	3½
12	20	720	96	3 × 4	8	4
15	18	810	108	3 × 4	9	4½

Septic tanks are usually constructed of concrete, plain or reinforced, although brick, stone, tile and concrete block might serve fairly satisfactorily for the walls of small tanks. Iron tanks coated with asphaltic compounds seem also to give satisfactory results. These metal tanks, as sold by commercial houses, are equipped with baffles and pipe stubs, so that they are ready to install and connect to a filtration system.

Some provision is usually made at the inlet of a septic tank to prevent the entering sewage from disturbing the solids settling in the tank and also to prevent short-circuiting. This may be accomplished either by using a horizontal tee at the end of the inlet pipe or by placing a baffle in front of the inlet. The outlet of the tank commonly is equipped to prevent the escape of floating solids, by the use of a tee, bend or baffle. For convenience in examining the interior of the tank and for cleaning it, manholes are generally provided.

A tank design used by the authors for sewage-disposal plants serving 5 to 10 persons is shown in Fig. 221. The septic tank has a capacity of 500 gal. and the dosing tank a capacity of 115 gal.

The statements of some persons and commercial concerns that septic tanks dissolve all the solids, that they never need to be cleaned and that the liquid leaving them is pure and free from germs, are not true. Septic tanks generally require cleaning at intervals of six months to a year, if they are to perform their function of removing solids satisfactorily. The liquid leaving them may putrefy and give off odors, if not suitably disposed of, and it is never free from bacteria, often containing more than the raw sewage. If the tank is not cleaned

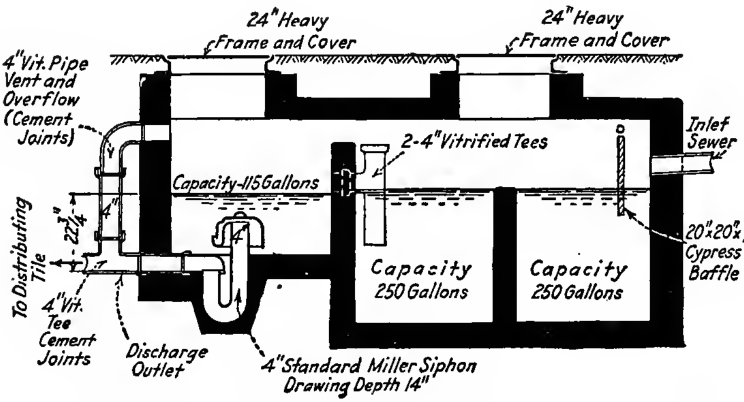


FIG. 221.—Septic tank and dosing tank for subsurface irrigation system.

sufficiently often, fine particles are carried out with the effluent and the secondary-treatment unit may become clogged with undue rapidity.

In designing septic tanks, when considerable fall is available it may be convenient to make the bottom of the tank in the form of a hopper, or several hoppers, with side slopes of about 45 deg. If pipes are led from the bottoms of these hoppers to the top of the ground at a point 4 or 5 ft. below the flow line of the tank, the sludge may be drawn off into containers or upon a sludge bed, generally without draining off all the liquid. This can also be accomplished by using a pump, if fall is not available for drawing off the sludge by gravity.

Where no provision is made for drawing off the sludge by gravity, the most satisfactory method of removing the solids from a septic tank is usually found to be pumping them into a tight tank with a diaphragm pump of a type made especially for pumping out cesspools. The contents of such tanks are emptied into the nearest manhole of a sewerage system, where such a system is available; otherwise, they are emptied into a body of water where there is no danger of polluting a water supply or shellfish beds, or else into shallow trenches dug in the ground.

Two-story Septic Tanks.—Two-story or Imhoff tanks have not usually been found satisfactory for small installations. They cannot be covered as can single-story tanks, for the entire surface of such a tank needs to be accessible, if it is to be operated efficiently, and an open tank or one covered by an easily removable roof or a small building is generally undesirable in a place where a small tank is required. Furthermore, two-story tanks need almost daily skimming and squeegeeing, if they are to operate in an entirely satisfactory manner, and such attention is seldom possible at a private dwelling or small institution.

Where conditions are suitable and the plant is of sufficient size, the advantages of the Imhoff tank over the septic tank are principally the possibility of producing an effluent containing less suspended matter and one that is less putrescible, as well as the possibility of producing an inoffensive sludge. In order to obtain these advantages, it is necessary that the tank be suitably designed to meet the conditions and be efficiently operated. Unless both these requirements are met, serious difficulties may be experienced. In numerous instances the effluent discharged from such a tank was of poorer quality than might have been obtained from a suitable septic tank installed to meet the same conditions.

It has been found particularly difficult to obtain satisfactory results from Imhoff tanks where the sewage is fresh and the solids are not broken up by pumping, by closely spaced racks or fine screens, or by travel through a long line of sewers. When sewage is quite fresh, it contains a considerable quantity of floating matter, which may form scum in the tank instead of settling. In a septic tank this is not nearly so serious as in an Imhoff tank.

Imhoff tanks for small plants commonly are designed with sedimentation compartments of such a size as to provide detention periods of 2.5 to 6 hr. Common allowances made for sludge are 1 to 2 cu. ft. per capita.

Sludge-drying Beds.—As a rule, the quantity of sludge to be disposed of from a residential or small institutional plant is too small to warrant the expense of constructing sludge-drying beds. In some instances, however, sludge beds have been provided in small treatment plants, especially to accompany the use of settling tanks and leaching cesspools.

It is particularly important to the satisfactory operation of cesspools that the preliminary treatment of the sewage in settling tanks be as efficient as possible, for if solids are allowed to enter a cesspool they tend to clog the soil and thus prevent the liquid material from leaching out. Furthermore, it is reasonable to assume that a settling tank which is provided with a sludge draw-off and a sludge-drying bed is more apt to be kept in condition for efficient operation than one which is not equipped thus. For this reason, it may be wise to build a sludge bed for a system involving settling tanks and leaching cesspools.

Sludge beds are commonly constructed of sand from 12 to 18 in. in depth. The area of sludge beds for small plants usually varies from 6 to 25 sq. ft. per capita. Table 150 contains some data on sludge beds used in connection with settling tanks and cesspools.

It is common practice to conduct the sludge on to the bed by means of an 8-in. pipe. Ordinarily underdrains are provided to collect the effluent after its passage through the sand, but if the soil beneath the sand is quite porous, it may be possible to omit underdrains.

Subsurface Irrigation.—A means of secondary treatment of septic-tank effluent of particular value for country houses is subsurface irrigation, which has been used extensively in the United States since about 1870. This system consists of a series of pipes, laid below the surface of the ground, with open joints so that the sewage may pass out into the surrounding soil. A dosing tank with a siphon is usually provided, so that the whole pipe system may be filled by the flush of sewage. In this manner all the flow is prevented from trickling out at a few joints, as it would if the sewage came in a slow, continuous flow. Intermittent discharge also permits all the earth about the laterals to be drained and aerated between doses. Experience shows that the best results are obtained when the ground where the pipes are located is not shaded. Subsurface irrigation systems are out of sight, do not generate odors and operate satisfactorily in the winter. On the other hand, the grass over the laterals grows rank, the effluent may not be so good as from a sand filter, owing to insufficient aeration, and if much suspended matter is allowed to pass from the septic tank into the pipe system, the latter has to be dug up and relaid after a few years. Systems of this kind with efficient septic tanks, however, remain in operation for years.

In 1920 a committee of the American Public Health Association made a report on rural sanitation, which includes a study of subsurface irrigation systems. The following outline, giving a theoretical basis for the design of tile distribution systems, follows the committee's report quite closely (5).

Area of Tile Field.—On the assumptions that the soil is highly compacted and that its grain size is that of very fine sand or silt with an effective size of 0.025 mm. and a uniformity coefficient of 2.5, Hazen's formula for the flow of water through sand, $v = cd^2\frac{h}{l}\left(\frac{T + 10}{60}\right)$, indicates that the daily downward percolation of water corresponds to a depth of sewage of 4 in. over the surface. In this computation c is taken at the extremely low value of 200 and the temperature is assumed to be 40°F.

If a factor of safety of 5, corresponding to 0.8 in. of water daily, be assumed, a limit of $0.8 \times 144 \div 231 = 0.5$ gal. per square foot is found, which is the amount used. One half gallon per square foot at 50 gal. per capita daily corresponds to 100 sq. ft. per capita. For a smaller per capita volume and correspondingly stronger sewage, increase the area per gallon by basing it upon 100 sq. ft. per capita. Thus is derived the first rule—*Area of Tile Field:* Not less than 100 sq. ft. per capita, nor less than 2 sq. ft. per gallon daily.

Length of Tile.—If a = area in sq. ft., p = population, l = length of tile in ft. and s = spacing of tiles in ft., then the area being fixed, length and spacing of tile are interdependent, for $a = pls$. It may be assumed that the liquid and colloidal organic matter permeates the entire area of the tile field for a greater or less depth, but solids which may be carried into the tile will move but a short distance away from the tile joints and be filtered out from the liquid which passes on by capillarity through the soil. It may be assumed that the suspended solids which are carried over into the tiles will not amount to more than 100 p.p.m. About one-third of these will be inorganic and about two-thirds will be organic and will be partially taken up by vegetation or dissipated in the soil. At a specific gravity of 1.5 this becomes for 50 gal.

$$\frac{50 \times 100 \times 231}{1{,}000{,}000 \times 1.5} = 0.77 \text{ cu. in. per capita daily.}$$

This material will be very fine and may, to some extent, be washed away from the joints into the earth by the flowing sewage; but if this action be neglected and it be assumed that it fills the voids in the soil adjacent to the tile for a radius of 6 in. and that the voids amount to 20 per cent, there will be a cylinder 12 in. in diameter and 12 in. long surrounding each joint, from which must be taken the volume of the tile. For a 2-in. tile ½ in. thick the volume of voids in the hypothetical cylinder of earth will be

$$12 \times \pi(6^2 - 1\tfrac{1}{2}^2) \times 0.20 = 255 \text{ cu. in.}$$
$$255 \div 0.77 = 331 \text{ days per capita per joint}$$

Applying a factor of safety of three gives 110.3 days per capita per joint. 6 years = 2192 days. $2{,}192 \div 110.3 = 19.8$. Hence use a length of tile of 20 ft. per capita and it may be expected that the plant will operate on the above basis for about 6 years.

If again it be assumed that the inorganic solids gradually collect in the tile and that the limit is reached when the tile is half filled, the following results: Volume of 2-in. tile, 20 ft. long, half full $= \pi \times 10 \times 12 = 377$ cu. in.

$$377 \div \frac{50 \times 231 \times 100}{3 \times 1.5 \times 1{,}000{,}000} = 1469 \text{ days or } 4.02 \text{ years}$$

Spacing of Tile.—The area and length of tile having been obtained, the spacing between lines of tile is determined as follows:

$$s = a \div pl$$
$$l = 20$$

whence

$$s = a \div 20p$$

By the first rule above mentioned, a must be not less than 100 sq. ft. per capita $= 100p$, nor less than 2 sq. ft. per gallon daily $= 2pG$ where $G =$ gal. per capita daily. If these values of a be used, the limiting minimum values are:

$$s = 100p \div 20p = 5$$
$$s = 2pG \div 20p = G \div 10$$

Slope of Tile.—An efficient system operates equally from every joint, covering the entire area. A small drizzling flow will pass out at the first joint and, at best, discharge from but few; therefore provide a flushing siphon and make each discharge equal to the capacity of the distributing tile. This is important and often, when overlooked, the result is failure.

The time of filling the drains is short compared with the time of emptying, hence the discharge from the tile joints is under hydrostatic conditions.

In order to make the flow from all joints equal, the head must be the same over the whole system, hence lay the tile level.[1]

Any collection of sediment in the tile would be apt to be carried on and piled up and the tile eventually plugged if operated under great velocity and high head.

For the longest system usually contemplated 20 lines of tile 100 ft. long will suffice and will readily fill from a common header. Five feet of 4-in. header will hold a volume equal to 20 ft. of 2-in. tile. Five-inch and 6-in. headers, respectively, will fill 31.25 ft. and 45 ft. of 2-in. tile.

Size of Tile and Number of Discharges Daily.—With the area, length and spacing determined, the size of tile is next to be found.

The area is based mainly on the amount of liquid handled, the length of tile upon the population contributing.

The amount of each discharge has been specified as equal to the capacity of all the tiles. Hence this capacity equals the total daily flow (Q) in gallons divided by the number of discharges per day (n), or algebraically as follows:

$$Q \div n = (\pi d^2 \times 12 \times 20p) \div (4 \times 231)$$

whence, if $d =$ the diameter of the tile, in.,

$$d^2 = (4 \times 231Q) \div (\pi \times 12 \times 20np) = G \div 0.816n,$$

hence $d = 1.11\sqrt{G \div n}$ and $n = G \div 0.816d^2$

For $d = 2$ in., $n = G \div 3.264$

For $d = 3$ in., $n = G \div 7.344$

For $d = 4$ in., $n = G \div 13.056$

If the values of n for various values of G in the last three equations be plotted, there result three lines for $d = 2$ in., 3 in. and 4 in., which are shown in Fig. 222.

[1] Slopes of 2 to 5 in. per 100 ft. are commonly employed in practice, because the joints nearest to the tank begin to discharge first and therefore will discharge more than the more distant joints, unless some slope is provided.

Note that 9.2 discharges daily will not be exceeded in 4-in. tile until the per capita flow is over 120 gal. daily, that 3-in. tile with 15 discharges will carry 110 gal. per capita daily and that 2-in. tile does not discharge more than 15.3 times for 50 gal. per capita daily. With a 2-in. tile the daily limit of ½ gal. per square foot will fill 3.06 ft. of the tile and at the prescribed spacing of 5 ft. there will be $5 \times 3.06 = 15.3$ discharges daily. Evidently the smaller diameter of tile is cheaper, as is also the smaller dosing chamber. If it be considered that about one-half the daily flow is concentrated in 8 busy hours and if the discharges during this period be limited to one per hour, we shall have 16 in 24 hr. This may be taken as the limit and will confine most rural plants to 2-in. tile. It will be noted

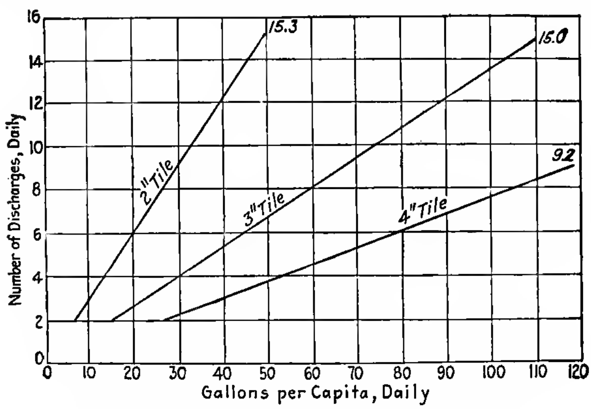


FIG. 222.—Capacity of subsurface irrigation tile.

from the diagram that 3-in. tile will not exceed 11 doses per day at 80 gal. per capita per day.

Thus the second rule is derived:

Tile Distributors—

(*a*) Length, 20 ft. per capita.

(*b*) Spacing, gal. per capita ÷ 10, but not less than 5 ft.

(*c*) Diameter, 2 in. up to 50 gal. per capita and 16 discharges daily; 3 in. above 50 gal. per capita daily.

From which follows the third rule:

Number of discharges per day = daily flow ÷ contents of tile, but not over 16.

It will be noted that this system depends upon the porosity of the soil and its drainability. If it is very open a smaller area might be used and the length of tile per capita reduced proportionally. The diameter and capacity, however, are dependent upon the number of discharges per day. A larger tile will hold a larger deposit of solids before it is choked, but efficient sedimentation should be provided in advance and it must be remembered that it may be necessary to relay the tile at intervals of 4 to 6 years, although there are cases where a system has operated successfully for 12 years.

The tile used in subsurface irrigation systems may be either unglazed drain tile or glazed tile, 2 to 6 in. in diameter. The authors recommend breaking away the upper two thirds of the bell of glazed pipe and then laying it with a 3/8-in. opening between the ends of the pipes. Crushed stone 2 to 3 in. in diameter is then placed by hand over the opening to prevent the entrance of small gravel and sand. In some cases the entire pipe is surrounded with graded gravel, extending two inches below and an equal distance above the pipe and for the full width of trench. The tile lines are commonly laid in trenches 12 to 18 in. wide and 1 to 3 ft. deep. A greater depth is not generally objectionable in part or even all of the field, if the elevations require it or if the soil at the greater depth is more porous. The lines of pipe usually are laid 3 to 6 ft. apart. The slope of the lateral pipes depends somewhat on the porosity of the soil. In porous material it is generally greater than in more impervious soil, so that some sewage may run all the way to the lower end of the system instead of all of it flowing out of the first few openings. In general, the slope is 2 to 5 in. per 100 ft., or 0.0017 to 0.0042. The length of lateral pipe necessary is generally estimated at 20 to 100 ft. per person connected, depending on the porosity of the soil.

Table 152 gives some characteristics of small sewage-treatment plants employing subsurface irrigation.

Subsurface Filters.—Where the soil is so dense and impervious as to make the use of filter trenches or leaching cesspools impracticable, subsurface filter beds are often useful, expecially if the best site for a filter is near a dwelling, so that an open filter might prove a source of annoyance.

The filter is usually constructed by excavating a suitable area to a depth of about 4 ft., with the bottom sloping to one or more depressions for underdrains. The underdrains of 4- to 6-in. tile pipe are laid 4 to 10 ft. on centers, with open joints surrounded by coarse gravel. Smaller gravel and a layer of sand are placed above the underdrains. This provides the filtering medium, which is preferably a clean sharp sand, free from dirt and not having too great a proportion of either fine material or gravel. Fine material may clog the filter or allow the sewage to pass very slowly, while coarse material may allow the sewage to pass through without oxidation. On the surface of the sand, 4- to 6-in. distribution pipes are laid parallel to, but not directly above, the underdrains. The distribution pipes are laid in a similar manner to the underdrains, 4 to 10 ft. on centers, with open joints, and are surrounded by graded gravel which sometimes extends completely over the surface of the sand, to aid in distribution. The authors provide a slope of 4 to 6 in. per 100 ft. for the underdrains but none for the distribution pipes.

TABLE 152.—CHARACTERISTICS OF SEWAGE-TREATMENT PLANTS EMPLOYING SETTLING TANKS AND SUBSURFACE IRRIGATION

Location	Residence, Hamilton, Mass.	Residence, Wenham, Mass.	Residence, Wrentham, Mass.	Stable, Middle-town, R. I.
Number served	10	14	10	22[1]
Per capita allowance, gal. daily	50	71	50	
Total sewage flow, gal. daily	500	1000	500	1100
Settling and dosing tanks:				
Settling compartments:				
Number	2	2	2	2
Total capacity, gal.	500	1000	500	2000
Detention time, hr.	24	24	24	43.6
Dosing compartment:				
Effective capacity, gal.	115	230	115	936
Size of siphon, in.	4	4	4	5
Drawing depth, in.	14	14	14	23
Number of daily doses	4.3	4.4	4.3	1.2
Average rate of discharge, c.f.s.	0.4	0.4	0.4	0.7
Subsurface irrigation:				
Total length of pipe, ft.	290	500	250	1100
Length per capita, ft.	29	36	25	
Gallons daily per ft. of length	1.7	2	2	1
Diameter of pipe, in.	6	4	4	4
Depth of invert below surface, ft.	3	3	2–3	2
Slope of pipe	0.0025	0.0025	0.0025	0.001
Character of subsoil at site.	Sandy	Sandy gravel	Loam and fine sand	Clayey hardpan
Assumed period of occupancy of building	Summer	All year	All year	All year

[1] 7 men and 15 horses.

The excavation above the gravel is filled to within a foot of the original ground surface with the excavated material and finished with loam to the original or desired ground level. The ground above the filter may be seeded with grass, but usually no trees, bushes or shrubbery are planted over the bed, because the roots are likely to clog the joints of the distribution pipes.

Subsurface filters commonly provide 0.8 to 2 sq. ft. of surface area for each gallon of sewage daily, depending upon the strength of the sewage and the fineness of the sand. For residences, areas between 25 and 90 sq. ft. per capita usually are employed, while for day schools the requirements are generally between 12 and 20 sq. ft. per capita.

Inverts of distribution pipes are commonly laid 20 to 30 in. below the surface of the ground. The depth of sand below the distributors generally ranges from 21 to 36 in. for residential plants, while for larger installations greater depths are sometimes employed.

Fig. 223 shows a plan and section of a typical subsurface filter.

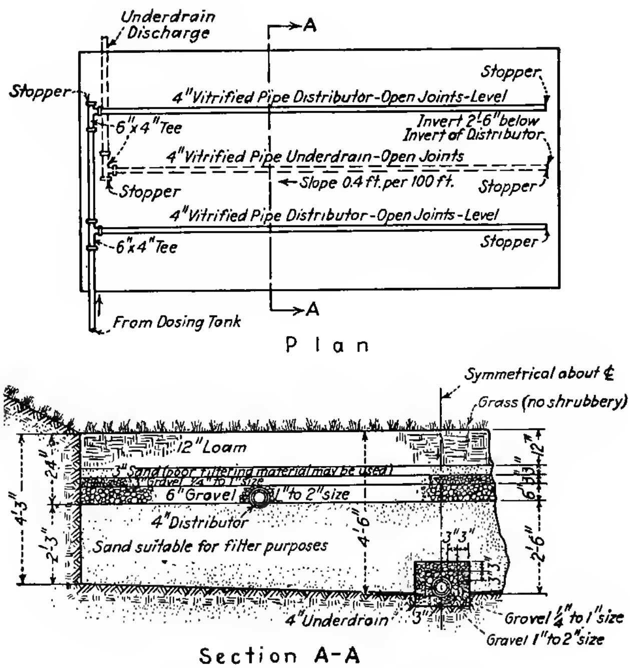


FIG. 223.—Typical subsurface filter.

Dosing Tanks.—When secondary treatment of sewage is provided in the form of subsurface irrigation or filtration, it is essential to apply the sewage in doses. While it is sometimes necessary, because of insufficient fall, to install a pump for this purpose, the dosing can usually be accomplished by means of a dosing tank equipped with an automatic siphon.

The purpose of a dosing tank is twofold: first, to secure intermittent discharge, thus allowing a considerable period of time for one dose to work off in the soil and for air to enter the pores before another flush is received, and second, to secure distribution over a larger area and in a more even manner than where the sewage is allowed to dribble and produce the objectionable condition of a small area of water-logged ground.

For subsurface irrigation, Ryon (4) states that the best results are obtained if the size of the dosing tank is such that the dose equals about 70 per cent of the cubic content of the tile in the section or sections of the field receiving the dose. If it is much smaller than this, the dose will not reach the extremities of the tile lines and if much larger, after the tile has clogged a little with use, the dose will be greater than the tile can hold and sewage will show on top of the ground. Dosing tanks for residential plants are commonly equipped with 4-in. siphons, while larger installations, such as those for hospitals, sometimes require siphons as large as 8 in.

The dosing tank is often constructed as an integral part of the settling-tank structure. The loss of head between inlet of settling tank and outlet of dosing tank varies, of course, with the depth of the various installations. The ordinary range of this loss of head is from 2 to 5½ ft. In fact, it is possible to place the outlet of the dosing tank at practically the same level as the inlet to the settling tank and allow the sewage to back up in the house sewer. Such a design is occasionally necessary, when the ground water is unusually high, allowing little fall between the plumbing fixtures and the point of final treatment of the sewage.

Grease Traps.—In order to prevent subsurface irrigation and filtration systems from rapidly becoming clogged with grease and soap from kitchen and laundry wastes, grease traps are often installed. The purpose of a grease trap is to provide a reservoir of sufficient capacity to reduce the temperature of such wastes, which in large estates, institutions and camps often contain much grease. Cooling of the hot wastes causes the grease to separate and float on the surface, from which it may be removed by skimming. If allowed to continue on in the sewer, the grease may coat the pipe and eventually clog it. Furthermore it tends to coat particles of organic matter, causing them to float instead of settle in sedimentation tanks. Grease digests with difficulty and often causes much trouble in Imhoff tanks.

Riker (6) states that at some of the army cantonments during 1917 and 1918 grease traps installed at sewage-treatment plants did not give satisfactory service. Where the grease did come to the top, the putrescible organic matter would settle to the bottom and become septic, later rising to the top and mixing with the grease. The army solved this problem by installing grease traps on the kitchen drain of each mess hall, so as to prevent the grease from mixing with the toilet wastes.

The most effective size of grease trap has not been determined. In the case of the army camps, opinion varied as to the advisability of building traps with total capacities of ⅔ gal. per capita. Data collected at Camp Merritt and Camp Meade indicated that grease traps providing 2 to 3 gal. per capita were more efficient than those with a capacity of ⅔ gal. per capita (7).

Many grease traps were constructed at the army camps, however, having capacities of approximately ⅔ gal. per capita. The special features of these traps were the relatively large area for the collection of grease, the steep-sided, pyramidal or conical bottom and the small diameter of the eduction pipe, which removed the settleable solids.

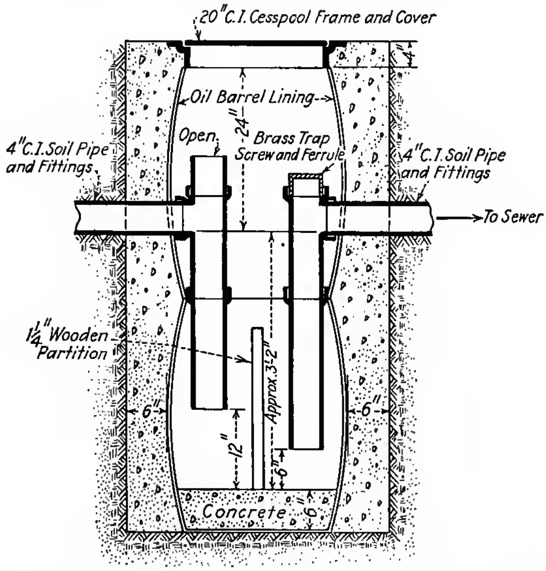


FIG. 224.—Grease trap for kitchen wastes in small installations.

The efficiency of these traps was determined by several series of carefully made tests, which indicated that the average efficiency under the usual operating conditions was at least 95 per cent (8). The practice at the army camps was to clean each trap once a week or every other week.

For small installations, such as might be required for the average country residence, a grease trap of the type shown in Fig. 224 can be provided at little expense.

Intermittent Sand Filtration.—Intermittent filtration is often used for treating the sewage of institutions and large residences, especially where sand can be obtained at low cost for the beds. It sometimes happens that sand suitable for intermittent filtration cannot be readily procured and in such cases coke, locomotive cinders or screenings may be used. This method of treatment is employed where a high degree of purification is desired and where it is practicable to locate the filter at a sufficient distance from habitations to avoid complaints concerning odors. A distinct drawback to the utilization of open intermittent filters for small residential installations is the fact that they require careful operating attention to insure satisfactory results.

In general, small installations of intermittent filters are built in accordance with the principles given in Chap. XVIII, dealing with the design of sand filters for municipal plants. Under certain conditions it may not be necessary to provide any underdrains to collect the effluent and conduct it away from the filters. The capacity of a residential filter is usually considered to be less per acre than that of a large municipal plant, because of the greater percentage of sediment carried on to the bed in the case of the former and the likelihood that it will receive less careful operating attention. Rates of filtration commonly allowed for in designs vary from 20,000 to 90,000 gal. per acre daily. The population served ranges from 400 to 1600 per acre for sand filters and may be as great as 3000 per acre for cinder filters.

At the Norfolk County Hospital in Braintree, Mass., a new disposal plant was built in 1928 to replace a system of leaching cesspools which had become so clogged that overflow pipes had to be installed to take care of the flow of sewage. The hospital accommodates 100 patients and 45 resident employees. Observations made on the flow of sewage for about three weeks showed the flow to vary from a minimum of 100 gal. an hour to a maximum of 700 gal. an hour on days when the laundry was in operation. Night flows in general were about 175 gal. an hour. The design of the new plant was based on a per capita allowance of 100 gal. daily for 200 persons.

Figure 225 shows the layout of both old and new plants. The existing sewer was intercepted just above the line of cesspools and the flow of sewage was diverted to the new treatment plant. A distribution manhole was built, to divide the flow equally between the two tanks, and gates were provided so that either tank might be put out of service for cleaning. The tanks were built below the surface of the ground with a dosing tank as part of the structure. Sand with an effective size ranging from 0.20 to 0.30 and a uniformity coefficient of about 2.65 was used in constructing the filter beds. There are two pairs of beds, each pair provided with manholes containing shear gates on the influent pipes, in order that any bed may be put out of service for cleaning. The effluent from the sewage filters is conducted through a tile pipe down the slope to a water-course, in which it flows to a swamp adjacent to a river. Sludge may be drawn periodically from the settling tanks by opening gate valves in the sludge line. A 10-in. vitrified tile drain conducts the sludge to one of the filter beds for drying. This bed is cut out of regular service during the short period when the sludge is drying.

An institutional sewage-treatment plant designed by Cleveland for a sanatorium at Loomis, N. Y., consists of screen chambers, two parallel Imhoff tanks, a dosing tank, four intermitent sand filters and a sludge-drying bed (9). The dosing tank has a capacity of 12,000 gal., equivalent to a dose of 3 in. over a single bed. The maximum time of discharge, with allowance for inflowing sewage, is about 32 min. The average rate of discharge is about 535 gal. a minute. There are four filter beds, each 6100 sq. ft. in area, providing an operating rate of 89,300 gal. per acre daily for a population of 500. The filters, which are

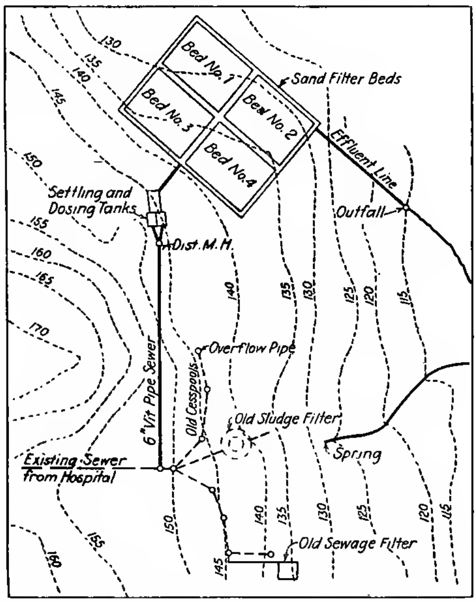


FIG. 225.—General plan of sewage-disposal plant at Norfolk County Hospital, Braintree, Mass.

36 in. in depth, are composed of sand with an effective size between 0.25 and 0.45 mm. and with a uniformity coefficient of not more than 3.0. Underdrainage is provided through a 6-in. vitrified pipe surrounded by graded gravel.

Contact Beds.—Contact beds have been extensively used in plants for the treatment of sewage from institutions, because they require little head, can be constructed in many places where filter sand can be obtained only at prohibitive prices, do not yield the odor which arises when sewage is sprayed over trickling filters, and can be filled from below in places where the appearance of the works requires special attention and it is desired to keep the sewage out of sight at all times.

In several cases they have been used as an intermediate step in a triple scheme of treatment, consisting of settling tanks, contact beds and sand filters. During recent years contact beds have not been so popular as formerly.

Trickling Filters.—Trickling filters have been used to some extent in small plants, but a serious obstacle to their use for residential and institutional plants is the head required for their operation. Other drawbacks which may become serious are the production of offensive odors and the presence of little flies. Where none of these obstacles is prohibitive, trickling filters afford a good means of treating tank effluent, if sand for intermittent filtration is unobtainable or the space available for a plant is restricted, but they do not furnish an effluent so stable and clear as that from a good intermittent filter. Trickling filters as well as contact beds have been used to treat sewage preliminary to sand filtration, where a high degree of purification was desired.

In some cases small trickling filters have been enclosed in buildings to mitigate fly and odor difficulties.

The filter medium in some of these small plants has consisted of laths instead of crushed stone. As a result of experimentation with both lath filters and ordinary trickling filters Frank and Rhynus (10) conclude as follows:

> Of the 2 kinds of filter material tried in these experiments, namely, limestone 1 to 2½ in. in size, and ordinary laths piled as described, the latter will give better and more uniform results.
>
> Laths used as trickling filter material should be piled so that adjacent layers are at right angles to each other and so that the laths of every second layer lie approximately over the centers of the vacant spaces below. They should be spaced with a 3-in. horizontal clearance. The quality of the effluent will not be appreciably improved by decreasing the clearance to 1½ in., and the larger clearance appears to afford complete immunity from internal clogging difficulties. In addition, the 3-in. spacing permits a considerable saving in laths, requiring only about 25 laths per cubic foot of filter.
>
> Eight cubic feet of lath filter material per person piled in this manner will be ample to produce a very highly oxidized effluent.
>
> A filter depth of 5 or 6 ft. will be ample for lath trickling filters providing the recommendation of the previous paragraph regarding volume per person is adopted. . . . It seems practically safe to recommend filters 3 ft. 6 in. deep for those cases where fall is not available and where moderate turbidity in the effluent will not be objectionable.

A special type of sewage distributor which will operate with little or no attention is generally used in small trickling-filter plants. The tipping tray and converging distributor boards shown in Fig. 226 have proved satisfactory, particularly with lath filters.

A rather complete treatment plant has been built for an institution at Annandale, N. J. (11). This plant consists of screen chamber, circular upward-flow preliminary-settling tank, two separate sludge-

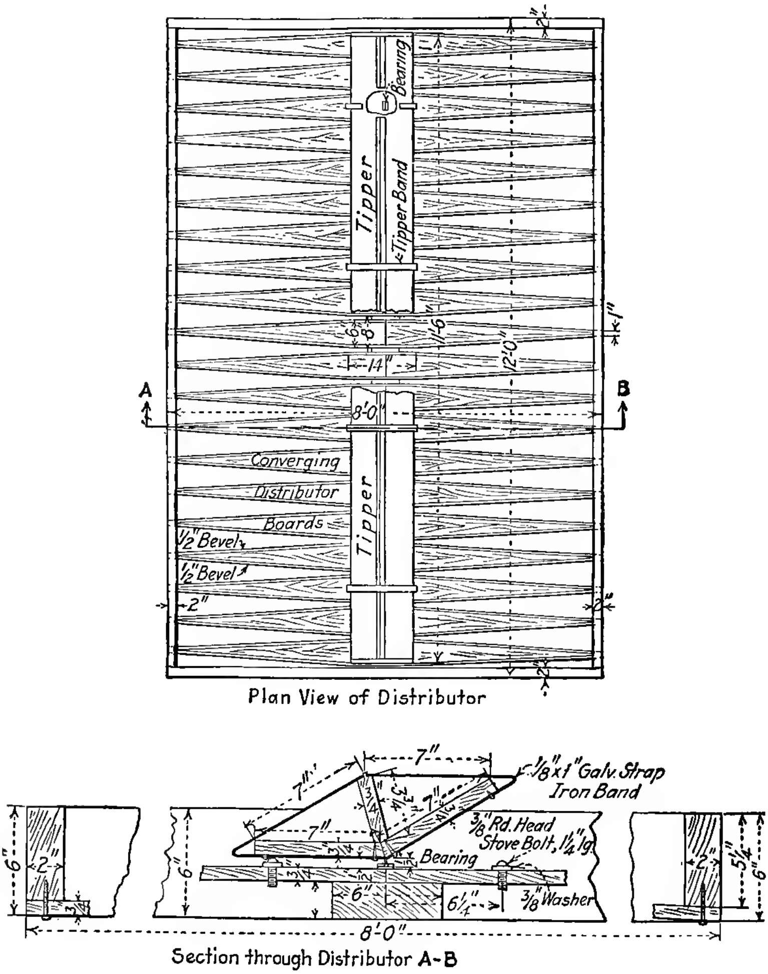


FIG. 226.—Tipping-tray distributor.

digestion tanks equipped with Downes floating covers, glass-covered sludge-drying beds, mechanical aeration tank preceding trickling filters, chlorination and final-settling tank. The completeness of the treatment and purification equipment provided in this case was considered desirable because of the small size of the receiving stream, which runs through an open, populated section of the country. The aeration in this case is not part of an activated-sludge process but was provided as an aid to the subsequent filtering process.

Activated-sludge Process.—An activated-sludge plant, consisting of screen chamber, preliminary-settling basin, two aeration tanks, final clarifier, sludge pump, sludge-digestion tank and sludge-drying bed, serves a community of 1000 persons at the Jackson County Farm near Kansas City, Mo. (12). The effluent from this plant had to be suitable to discharge into a stream with intermittent flow.

Disinfection.—Disinfection has been provided for in the treatment plants of a number of institutions, but it is questionable how well it has been carried out in some cases. Plants using bleach required careful and intelligent care and even the modern types of apparatus for applying liquid chlorine require some supervision to ensure the accomplishment of their purpose. That purpose may range from the disinfection of merely screened and settled sewage for preventing contamination of the water of bathing beaches and oyster layings to the production of an effluent meeting the standards for drinking water.

The institutional plant at Annandale, N. J., previously mentioned, employs chlorination in addition to thorough treatment.

Plant Operation.—Many small plants are sadly neglected after construction and thus fail to fulfill the purpose for which they were designed. Ridenour (13) comments as follows upon the situation in New Jersey:

> The supervision of operation usually falls upon the institutional engineer, who, in addition to this, generally has charge of all other tasks around the institution, varying from chasing birds out of lofts to supervising the power houses. Accordingly, he lacks time for other than casual and physical inspection of the plant performance. Most often he does not have the time, inclination or knowledge to give the plant the control work that it should have. A central control agency which has time, personnel, and facilities available for visiting the plant at regular intervals, running analyses, instructing operators and designing new units when required, is the most logical solution to this problem.

In 1928 an arrangement was made in New Jersey whereby the design of new plants, reconstruction and remodeling of old plants and supervision of operation were to be performed by the Department of Water Supplies and Sewage Disposal of the New Jersey Agricultural Experiment Station, in cooperation with the State Department of Institutions and Agencies and the State Board of Health.

The history of the first three years' service of a sewage-treatment plant for an orphanage at Pleasantville, N. Y., affords a good illustration of the need of intelligent care in operating an institutional plant.

These works were operated successfully for the first four months after their completion, under the direction of Lederle & Provost. For the next six months the orphanage staff operated them with unskilled labor. The sedimentation tanks were allowed to become highly septic. The ebullition of gas in the tanks carried so much suspended matter into the effluent that the trickling-filter nozzles became clogged, causing the sewage to back up in the pipes and the scum in the sedimentation tanks to overflow the outlet wall into the dosing tanks. When the nozzles were cleaned, the scum passed to the trickling filters and a large quantity of solid matter passed from the trickling filters to the intermittent filters, which became so clogged that the sewage pooled and froze on the surface. The operation of the plant was manifestly a failure.

At the end of this unsuccessful experience, the designing engineers were again retained to supervise the operation. An attendant was assigned to work under their direction and substantially all his time was devoted to it. He kept a daily record of the condition of each sedimentation tank, each half of each trickling filter, each sand filter and each sludge bed and a record of the bleach used in the morning and afternoon. The nature of the maintenance work done on each part of the plant was recorded at the same time and a record was kept of the volume of sewage treated. The expert supervision comprised visits to the plant every few days, methylene-blue tests at each visit and laboratory analyses of the effluent and brook water less frequently. The tanks remained in service 41 days between cleanings, on the average. Under this method of operation, the settling solids were reduced by 7 hours' detention in the sedimentation tanks from 5 cc. to a trace, when measured in an Imhoff cone.

Bibliography

1. Steel, E. W.: *Munic. Sanitation*, 1931; **2**, 15.
2. Hardenbergh, W. A.: "Home Sewage Disposal," 1924.
3. *Bull.* No. 27, Ill. State Water Survey, 1928.
4. Ryon, Henry: "Notes on the Design of Sewage Disposal Works with Special Reference to Small Installations," 1927.
5. *Am. Jour. Pub. Health*, 1921; **11**, 551.
6. Riker, Russell: *Public Works*, 1922; **52**, 39.
7. Johnson, G. A.: *Jour.*, Boston Soc. C. E., 1919; **6**, 181.
8. Doten, L S.: *Trans.*, A. S. C. E., 1919–1920; **83**, 337.
9. *Public Works*, 1930; **61**, no. 6, p. 22.
10. *Pub. Health Bull.* 101, U. S. Pub. Health Service, 1919.
11. Ridenour, G. M.: *Public Works*, 1931; **62**, no. 2, p. 19.
12. McDonnell, R. E.: *Munic. Sanitation*, 1931; **2**, 18.
13. Ridenour, G. M.: *Public Works*, 1930; **61**, no. 8, p. 38.

CHAPTER XXXII

SEPARATE TREATMENT AND DISPOSAL OF INDUSTRIAL WASTES

Tanneries, paper mills, wire-drawing and galvanizing works and many other industries use water in their manufacturing processes. After use, much of this process water carries substances which materially change its character.

Industrial wastes may be defined as the waste waters carrying suspended, colloidal and dissolved matters, produced by manufacturing processes and discharged therefrom as being of no commercial value. These wastes are also termed "manufacturing," "manufactural" and "trade" wastes.

Many such wastes from industries situated in cities are discharged into the public sewers and disposed of as a part of the municipal sewage. In some cases, however, the wastes are of such composition that they would damage the sewers, cause deposits and tend to clog the pipe lines, or interfere with the treatment of the sewage. In some such cases the wastes are treated separately from the sewage, to remove their harmful constituents before discharging the wastes into the sewers, and in other cases the wastes are disposed of separately from the sewage.

Other industries are situated in suburban or rural localities, where sewers are not available but where the streams into which the wastes are necessarily discharged are so small as to be polluted seriously by them. In such cases also the wastes require treatment and disposal by themselves and separate from any municipal sewage.

Coincident with the great increase in the number and size of the waste-producing industries in recent times and an accompanying increase in density of population, there has been a demand for higher standards of cleanness of natural waters than formerly, due to the marked growth in popularity of outdoor recreation. These several conditions have brought the treatment and disposal of wastes forcibly to the attention of industrialists and this problem has become one of importance.

Harmful Effects of Wastes.—Acid pickling liquors have caused disintegration of sewers in Pawtucket, R. I., and Milwaukee, Wis. The seriousness of disintegration in the latter case is illustrated by Fig. 227. In 1897 evidence was found that the concrete invert of the 13- by 18-ft. Mill Brook sewer in Worcester, Mass., had been attacked by the acid of the pickling liquors flowing through this sewer, notwithstanding the fact that it received also the entire sewage of the city, then having a population of 100,000, and the natural flow of a small stream equivalent in dry weather to perhaps 3 m.g.d. At Fostoria, Ohio, such wastes caused corrosion of pumps and disintegration of concrete at the sewage-treatment plant.

The Maryland State Board of Health, reporting upon its studies of the treatment of trade wastes, gives a pH of 4.0 as the minimum for plant discharge, to assure protection to concrete sewers, this being the

Fig. 227.—Disintegration of concrete sewer caused by acid pickling liquors in Milwaukee, Wis.

standard adopted by the city of Baltimore after consultation with the State Board of Health (1).

Certain tanneries situated within the area tributary to the Massachusets Metropolitan Sewerage District produce wastes containing such large quantities of settleable solids that, to prevent deposits, the industries are required to provide and operate plants for the removal of such solids before discharging the wastes into the sewers.

At Gloversville, N. Y., and Akron and Fostoria, Ohio, industrial wastes discharged into the sewers have contributed large quantities of suspended solids, which have placed a serious burden upon the processes of sedimentation and sludge disposal.

It has been estimated that in 1920 the wastes from the packing industry in Chicago, Ill., were equivalent in polluting matter to the sewage of half the population of the Sanitary District of Chicago. If and when these wastes are treated with the District sewage, they will exert a controlling influence upon the design of the treatment plant and upon its operation.

At Philadelphia, Pa., Webster and Stevenson found that wastes should not be admitted to sewers if they contain substances which would adhere to the walls or form deposits, if they contain inflammable substances, if they contain steam or extremely hot liquids, if they contain acids which would injure the material of the sewer, or if they contain substances which would seriously interfere with the operation of sewage-treatment works.

When discharged into natural waters, industrial wastes may affect these waters unfavorably for use as municipal water supply, boiler feed water or process water, for watering livestock or even for irrigating land. One of the common harmful effects is that upon fish life.

The Committee on Industrial Wastes in Relation to Water Supply, of the American Water Works Association, found that most industrial wastes are detrimental to water supplies and the injury depends upon the character and relative quantities of the substances discharged (2). The committee's classification of these substances and their effects is as follows:

1. Suspended or colloidal mineral matters which increase turbidity and add to the difficulty and expense of coagulation or of filtration, as, for example, coal and ore washing wastes.
2. Dissolved mineral matter impairing the quality of the water and increasing the difficulty and expense of water purification, as, for example, acid mine drainage and salt water from oil wells.
3. Vegetable and animal organic matters in solution or in suspension, which increase the color, turbidity, suspended matter and bacterial content, thereby increasing the difficulty and expense of water purification, as, for example, beet sugar refinery wastes and tannery wastes.
4. Taste- and odor-producing substances, either organic or mineral in nature, as, for example, phenols in the wastes from gas and coke manufacturing, and sulfite liquors from paper pulp mills.
5. Substances, either organic or mineral, tending to stimulate growths of organisms and thereby increase the difficulty and expense of treatment, as, for example, organic sulfur compounds in wool scouring wastes, and mineralized nitrogen from oxidation of organic wastes of various kinds.
6. Harmful bacteria, as, for example, the anthrax bacillus in tannery wastes.
7. Poisons, as, for example, cyanide from the cyanide process of gold extraction.

Frequently, wastes are discharged in such quantities or are of such a character as to make the water difficult to purify for domestic use, or actually to prohibit such use.

Pollution, such as acid pickling liquids and mine drainage, which causes the water to have a corrosive action, may be seriously detrimental to its use in steam boilers. Wastes which add certain salts to the water may cause priming or foaming in such boilers or result in the formation of either hard or soft scale, which interferes with the heat transfer and reduces efficiency.

Industrial wastes frequently increase hardness, color and turbidity and add impurities to such a degree as not only to render the natural water unsuitable for the dyeing and washing of texile goods, the manufacture of paper, the tanning of leather and other process purposes, but also to make its treatment for such uses expensive if not physically and financially impracticable.

Cattle are said to have been poisoned by cyanide wastes discharged into their drinking water (3) and it has been claimed that streams carrying tannery wastes have spread anthrax bacilli and actually caused the infection and death of livestock (4). Wastes from mines and oil refineries have rendered streams unsatisfactory for the irrigation of crops.

One of the most frequent complaints is that the presence of industrial wastes in streams is driving away and killing the natural fish life. Such effects are sometimes due to suffocation, caused either by depletion of the dissolved oxygen naturally present in the water or by the presence of substances which interfere with the action of the gills, to actual poisoning by toxic chemicals such as gas-house wastes, to the lack of food due to change in aquatic life, or to the prevention of propagation through the formation of sludge beds on the spawning grounds. It has been claimed that the discharge of paper-mill wastes at Monroe, Mich., into the Raisin River caused the killing of carp in artificial propagating ponds some two miles downstream.

Perhaps the most common harmful effects of the discharge of wastes into natural waters are the depletion of the dissolved oxygen and the rendering of the waters unsightly and offensive. The oxygen may be depleted or even exhausted chemically by many inorganic substances, such as sulfite liquors from pulp mills, as in the Fox River in Wisconsin, ferrous sulfate in pickling liquors, as in the Portage River in Ohio, and sodium sulfide from tanneries, as in the Neponset River in Massachusetts. Biological action also causes depletion and exhaustion of oxygen, due to the decomposition of the organic matter dissolved in the water and deposited on the bed of the stream. Such exhaustion of dissolved oxygen may result in the formation and escape of hydrogen sulfide and organic sulfides in such quantities as to be offensive to residents in the neighborhood and to cause lead paint on boats and on neighboring buildings to turn dark. There have been many illustrations of this condition, one of the most noteworthy being that of the Passaic

River in New Jersey. Furthermore, during decomposition large quantities of unsightly and malodorous, gas-lifted solids of industrial origin may be brought to the surface. This has occurred at times in the Neponset and other rivers receiving wastes.

Financial Return from Wastes Disposal.—Manufacturers often feel that there may be a way of recovering substances of commercial value, which will defray at least a part of the cost of any treatment of wastes which may be required. Although there is usually little possibility of this recovery, because all that is of value has been taken out or conserved in the manufacturing process, each study of an industrial-wastes-disposal problem should include an investigation to determine the feasibility of recovery of materials of value.

Occasionally a study of wastes will disclose losses which have gone on for years without being discovered. A notable example of this resulted from a study of the sewage of the city of Worcester, Mass., many years ago, when it was discovered that large quantities of copper sulfate were being discharged into the sewers. This was stopped and a substantial saving to the industry was effected.

The recovery of wool grease from scouring liquors has been practiced for many years. Under most conditions this is not attractive financially, although during the war when prices were high some profit was made. In some paper mills it has been possible to utilize paper fiber which escaped from the save-alls and previously had been allowed to go to waste; at some rubber-reclaiming plants provision has been made for a measure of wastes treatment, resulting in the saving of fine particles of rubber which otherwise would be lost.

Perhaps the greatest economy which has resulted from the necessity of treating wastes is in the refinement of manufacturing processes, by which valuable materials have been saved and process water has been conserved by the reduction of its prodigal use and by its re-use in one process after another.

Laws and Regulations Relative to Discharge of Wastes.—The legal aspects of sewage disposal have been discussed in Chap. II and in general are applicable to wastes disposal. Many cities have ordinances for the protection of sewerage works. Worcester, Mass., stipulates:

No live, exhaust or waste steam and no water of a temperature above 140°F. shall be discharged into the public sewers or into any drain connected with them. . .

This city also stipulates:

No person shall throw into any drain or sewer, inlet, manhole or catch-basin, any earth, dirt, stones, bricks, sawdust, ashes, cinders, shavings, hair, oyster, lobster or clam shells, or any other substance detrimental to the sewers or the use thereof.

Following serious trouble at the sewage-treatment plant Fostoria, Ohio, enacted a protective ordinance:

Section 1. That it is deemed necessary for the safe, economical and efficient management and protection of the sewerage system, and sewage pumping, treatment and disposal works, that the emptying . of any grease, fats, oils, acids, carbon, iron or mineral wastes, or the wastes from any mercantile, manufacturing or industrial enterprise, other than domestic sewage, which would cause clogging, or which is injurious to said sewers, sewage pumping, treatment or disposal works, or interferes with the proper treatment of domestic sewage, or the operation and maintenance of the sewage disposal works shall be and the said is hereby forbidden and prohibited.

Section 2. That the director of Public Service shall have authority . . . to require any and all . industrial establishments . . . to treat all . . wastes . . . prior to the discharge thereof into the sewers or sewage system of said city, when he deems it necessary for safe, economical and efficient management and protection of the sewage system, and sewage pumping, treatment or disposal works . . When such . . . waste plant or device is installed, the degree of treatment shall be satisfactory to the Director of Public Service, and he shall have access to these local treatment plants at all reasonable times for the purpose of inspection and tests. . . . Whenever such . . . industrial enterprise . shall fail to comply with the requirements of the Director of Public Service, they shall forfeit their right to the use of such sewer or sewers, and the Director of Public Service shall forbid and exclude the use of the sewers of said City for the emptying or discharge of such waste therein, until the same is treated according to the requirements and to the satisfaction of the Director of Public Service.

Section 3. The discharge into the sewers . . . from any industrial-waste-treatment plant . . . or from any industrial establishment . which contains a greater amount of wastes than that set forth in the following tentative standards . . . shall be prima facie evidence of the same being injurious to said sewers, sewage pumping, treatment, or disposal works, to wit:

1. No free acidity.
2. Iron to be limited according to the following schedule:

Total Flow of Wastes, Gal. Daily	Upper Limit for Iron as Fe, P.P.M.
0 to 50,000	400
50,000 to 100,000	200
100,000 to 150,000	150
150,000 to 200,000	100

3. Carbon, measured as suspended solids in effluents from settling tanks for wastes from wet grinding process, to be limited according to following schedule:

Total Flow of Wastes, Gal. Daily	Upper Limit for Suspended Solids, P.P.M.
0 to 20,000	200
20,000 to 30,000	150
30,000 to 40,000	125
40,000 to 50,000	100

4. Oils and greases to be intercepted by suitable traps and kept out of sewers.

Flows should be gaged at least four times per year. When average daily flow of wastes exceeds the maximum figures given in tables, the limits for iron and carbon should be adjusted in accordance with flow of sewage. Present limits based on average daily flow of sewage equivalent to 2 mil. gal. and sewage-treatment plant of that capacity; with larger flows of sewage, the limits may be less rigorous providing the sewage-treatment plant is increased in proportion.

Authority to regulate and control the pollution of streams by industrial wastes has been vested in the departments of health of some states. In Pennsylvania, Connecticut and Rhode Island water boards, boards of conservation or special commissions have been vested with such authority. An example of this is the act of 1925 of the General Assembly of Ohio:

Section 1240–1. No city . corporation or officer or employee thereof . . shall establish . . any industrial establishment, process, trade or business, in the operation of which an industrial waste is produced, or make a change in or enlargement whereby an industrial waste is produced or materially increased or changed in character, or install works for the treatment or disposal of any such waste until the plans for the disposal of such waste have been submitted to and approved by the state department of health .

Section 1240-2. The state department of health shall exercise general supervision of the disposal of sewage and industrial wastes and the operation and maintenance of works or means installed for the collection, treatment or disposal of sewage and industrial wastes.

Section 1240–3. The state department of health shall study and investigate the streams . . for the purpose of determining the uses of such waters, the causes contributing to their pollution and the effects of the same, and the practicability of preventing and correcting their pollution and of maintaining such streams in such condition as to prevent damage to public health and welfare. For the purpose of providing effective control . . . and for preventing the undue pollution thereof the state department of health may adopt and enforce such special or general regulations as it may deem necessary for the protection of the public health and welfare.

In some cases, special legislation has been enacted to cover specific streams, as illustrated by the Neponset River Act[1] of Massachusetts:

[1] Mass. Acts 1902, Chap. 541, as amended.

Section 1. The state department of health is hereby authorized and directed to prohibit the entrance or discharge of sewage into any part of the Neponset River or its tributaries, and to prevent the entrance or discharge therein of any other substance which may be injurious to the public health or may tend to cause a public nuisance or to obstruct the flow of water, including all waste or refuse from any factory or other establishment where persons are employed, unless the owner thereof shall use the best practicable and reasonably available means to render such waste or refuse harmless.

Section 3. The supreme judicial court or any justice thereof and the superior court or any justice thereof shall have jurisdiction in equity to enforce the provisions of this act and any order made by the state board of health in conformity therewith.

Section 4. Whoever permits the entrance or discharge into any part of the Neponset River or its tributaries of sewage or of any other substance injurious to public health or tending to create a public nuisance shall be punished by a fine not exceeding five hundred dollars for each offense.

Notwithstanding the fact that the discharge of industrial wastes into streams is regulated by legislative action, the present trend is for state or municipal authorities to cooperate in such a way as to secure voluntary action by the industries rather than to proceed under the law. Maryland, Michigan, Ohio, Pennsylvania, Wisconsin and other states have cooperated thus and have had considerable success.

Wastes Survey.—As the term implies, a wastes survey is a study of the volume and quality of wastes produced and their effect upon the problem of sewage disposal, if the industry is or can be made tributary to a municipal sewerage system, or upon the body of water into which they must be discharged, if they are to be disposed of separately. The wastes from a single industry may change materially the volume and quality of sewagefrom an entire city, as shown in Chap. V, or may exert a controlling influence upon the pollution of a river or a lake.

Whether the engineer is employed by the municipality or by one or more industries, his relations to the industries are intimate and resemble those of the lawyer to his client or the doctor to his patient. If the engineer is to deal intelligently with the problems involved, he must be furnished information regarding the industry. Sometimes this information is of a confidential nature, in which case it must be guarded scrupulously from communication to anyone.

Such a survey obviously deals with conditions found at the time. Suitable estimates can be made to cover operations at other proportions of plant capacity and future growth. Such a survey, however, cannot cover new industries, radical increase in existing industries, or new processes. Any one of these changes may have a controlling effect upon the general problem under consideration. This, however, is not a justification for omitting such a survey, for it is frequently found that unexpected conditions exist, which materially modify the design of works which otherwise would have been planned. In making a wastes survey for a municipality or one or more industrial plants, the following may be included among the general data to be obtained:

Name and address of company
Persons, with their official positions, who furnish information
Portion of information which should be treated as confidential
Number of employees
Hours of operation
Relation of current production to capacity output
Probability of expansion of industry
Source, volume and quality of water supply
Uses of water
Map of drainage system, showing topography and outlets
Kind, size and capacity of municipal sewage-treatment plant at which wastes might be treated
Kind, size, character and subsequent use of streams into which wastes may be discharged

The following may be included among the operating data to be obtained:

Kind and quantity of raw materials and finished products
Processes of manufacture
Kinds and quantities of chemicals used
Kinds of wastes produced
Quantities and composition of each kind of wastes
  Temperature of wastes
  Volume of wastes by weir or other measurements
  Collection and analysis of samples

With adequate data available, it then becomes possible to give consideration to the following matters relative to which conclusions and decisions are to be made:

Desirability of separation of sewage and wastes
Interpretation of results of analyses of wastes
  Effect on sewers
  Effect on treatment of municipal sewage
  Effect on streams
Practicable methods of treatment
Cost

Quantity of Wastes.—The quantity of industrial wastes produced depends upon the kind of industry and the size of plant. Industries manufacturing the same product often produce wastes of materially different quantities and chemical composition, the differences depending in large measure upon the quantity of process water available, differences in methods of manufacture and idiosyncrasies of individual operators in the dyeing, tanning and other departments. For example, the authors have found that the volumes of objectionable wastes produced by different silk-finishing plants ranged from 2.2 to 5.1 gal. per yard of goods finished. In small communities one industry may produce a volume of wastes several times as great as the volume of municipal sewage. In a town of 6700 population the sewage amounted to 350,000 gal. a day, whereas one local paper mill produced 3,000,000 gal. of wastes daily. In 1923 the volume of sewage produced in Akron, Ohio, was about 18,500,000 gal. a day and the aggregate daily quantity of rubber-reclaiming wastes was about 5,000,000 gal.

Composition of Wastes.—Because of the variety of the wastes produced in different industries and even in some individual plants, it is not practicable to set up a clearcut classification of wastes based upon the character of the industries. Neither is it practicable to establish a classification based upon the effects of the wastes on municipal sewerage systems or on the streams into which the wastes are discharged.

Sometimes wastes have been grouped according to the character of the predominating impurities, such as organic wastes, mineral wastes and organic and mineral wastes combined. Examples of organic wastes are those from canneries, dairies, packing houses and wool-scouring plants; of mineral wastes are those from wire mills, copper-working plants, plating industries, paint manufactories and mines; and of organic and mineral wastes combined are those from tanneries, textile mills and paper mills. Obviously this is a loose classification, subject to many exceptions and affording little aid in the consideration of the effects of the wastes and of suitable methods for their treatment.

There is wide diversity in the chemical composition of wastes, as would be expected from a knowledge of the different kinds of work done by the industries and of the processes employed. The results of a few analyses made by the authors of certain typical wastes are given in Table 153, simply for illustrative purposes. There also is given, for comparison, the average[1] of the results of analysis of five sewages, practically uninfluenced by wastes, from cities provided with combined sewers.

The suspended solids, which usually are one of the most troublesome constituents of wastes, range from 50 to 2530 p.p.m. as compared with 245 parts in the sewage. The total volatile solids, which represent organic matter, range from 192 to 4610 p.p.m., as compared with 236 parts in the sewage.

Settling Solids and Colloidal Matter in Industrial Wastes.—Because of the quantity and character of suspended and colloidal matter in indus-

[1] From Table 31.

TABLE 153.—ANALYSES OF TYPICAL SEWAGE AND OF INDUSTRIAL WASTES
Parts per Million

Kind of wastes	Sewage from combined sewers[1] witbnut wastes	Woolen mill	Wire mill	Mixture of wastes[3]	Pulp and paper mill	Tomato cannery	Gelatin plant	Creamery	Paper mill: Paper machine wastes	Paper mill: Stock washer wastes	Sheepskin tannery	Packing bouse	Leatber tannery	Rubber reclaiming plant[4]
Flow of wastes, gal. per day	......	4,044,000	307,000	7,280,000	10,000,000	1,817,000	1,200,000	1000	3,570,000	281,000	518,000	1,020,000	3,880,000	2,000,000
Ammonia nitrogen	12.2	1.2	......	2.2	......	5.0	2.7	4.5	0.5	1.8	16.7	90.0	27.5	5.4
Albuminoid nitrogen	4.8	1.6	......	3.2	......	......	7.8	......	3.6	16.2	27.2	......	34.0	
Organic nitrogen	9.8	......	......	......	......	33.6	......	90.8	......	......	......	173.3	......	22.9
Nitrite nitrogen	0.08	......	......	0.49	......	......	......	......	......	......	......	......	0.01	
Nitrate nitrogen	0.53	......	......	16.35	......	......	......	......	......	......	......	......	0.20	
Oxygen consumed, 30 min	91	112	......	126	426	520	37	1183	158	480	638	228	1105	2320
Chlorides as Cl	64	24	......	52	......	......	107	......	39	103	......	......	......	390
Alkalinity	......	35[2]	......	259[2]	154[2]	......	1510	1380	......	......	......	1030	......	1555
Acidity	......	......	182	......	......	49								
B.O.D., 5 days at 20°C	176	34	......	119	77	1000	......	......	......	......	......	1210	506	
Iron as Fe	......	......	500											
Fats by ether extraction	......	......	......	......	......	......	......	789	......	......	640	659	234	
Total solids:														
Total	838	322	2720	1400	1280	1340	2082	3122	1188	3908	3072	9036	6156	8740
Volatile	......	192	900	416	550	1045	263	2276	620	2212	1627	2142	2323	4610
Fixed	......	130	1820	984	730	295	1819	846	568	1696	1445	6894	3833	4130
Dissolved solids:														
Total	593	272	2508	1173	864	845	1569	2506	440	1564	2112	7556	4350	6210
Volatile	......	149	798	361	360	640	250	1766	136	828	1034	910	905	2480
Fixed	......	123	1710	812	504	205	1319	740	304	736	1078	6646	3445	3730
Suspended solids:														
Total	245	50	212	227	416	495	513	616	748	2344	960	1480	1806	2530
Vnlatile	145	43	102	55	190	405	13	510	484	1384	593	1232	1418	2130
Fixed	100	7	110	172	226	90	500	106	264	960	367	248	388	400

[1] Based nn analyses of sewages from five cities. [2] Bicarbonate alkalinity.
[3] From paper mill, baryta plant, gelatin plant, and cotton-nitrating and -washing plant.
[4] Combined wastes from three plants.

trial wastes, they often impose a heavy burden upon a treatment plant. The data in Table 154 were compiled from analyses of wastes from a

TABLE 154.—CHARACTER OF SETTLING AND DISSOLVING SOLIDS AND COLLOIDAL MATTER IN WASTES FROM A TANNERY AND WOOL-SCOURING PLANT

	Parts per million	Percentage of total solids
Total solids in original sample	5960	100.0
Total solids after settling 24 hr	4788	80.3
Settling solids by difference	1172	19.7
Solids after filtering through paper	4530	76.0
Additional solids removed by filter	258	4.3
Colloids after diffusion through parchment	478	8.0
Crystalloids or dissolved solids by difference	4052	68.0
Total suspended and colloidal matters	1908	32.0

large tannery and wool-scouring plant. A sample of paper-mill washer wastes, containing about 2300 p.p.m. of total suspended matter, allowed to stand quiescent, precipitated in 12 hr. 95 per cent and in 6 hr. 88 per cent of the suspended matter capable of settling in 24 hr. After these wastes had passed through settling tanks with a period of flow of about 10 hr., they were found to contain 564 p.p.m. of suspended matter, in addition to 665 p.p.m. of colloidal matter. Sedimentation tests of wastes from a woolen mill, not including wool-scouring wastes, gave the results stated in Table 155.

TABLE 155.—SETTLING SOLIDS IN CRUDE WOOLEN-MILL WASTES, EXCLUDING WOOL-SCOURING WASTES

Period of sedimentation, hr.	Total solids in supernatant liquid, p.p.m.	Settling solids, p.p.m.	Percentage of removal in 24 hr.	Percentage of total suspended matter removed[1]
0	807	0	0.0	0.0
1	742	65	54.6	32.3
2	728	79	66.4	39.3
4	716	91	76.5	45 3
8	705	102	85.7	50.7
24	688	119	100.0	59.2

[1] Total suspended matter 201 p.p.m.

Tannery Wastes.—The first process in tanning is soaking the hides, which removes some organic matter and any soluble chemical used in curing the skins. The hair is then removed in lime vats or otherwise. When lime is used, large quantities of it are carried off with the waste liquor. In tanning, large quantities of chemicals are used, in some cases nearly a pound of chemical for every pound of hides. In Gloversville, N. Y., it was found that fully one half of the total weight of hides and chemicals passed off with the wastes. The quantities of suspended matter in tannery wastes are so large that the latter should pass through settling tanks before entering the sewers. This prevents the deposition of large quantities of solids in the sewers and sedimentation tanks at the disposal works.

Tannery wastes contain small numbers of bacteria and in some cases may be actually sterile. When mixed with domestic sewage they have some disinfecting action, but unless the proportion of wastes to sewage is quite large, this action does not prevent successful treatment of the combined sewage by biological methods (5). Tannery wastes are usually cold and if they constitute a large proportion of the sewage of a community, they may affect unfavorably the process of purification, on account of reduced temperature.

As a result of 6 years' study, including investigations at a full-size experimental tannery-wastes disposal plant at Emporium, Pa., the Tannery Waste Disposal Committee of Pennsylvania, created by an agreement in 1924 between the Sanitary Water Board and the leather-tanning companies, reported to the Sanitary Water Board two plans of tannery-wastes treatment (6). The general plan, applicable in successive steps to nearly all tanneries, consists of mixing, plain sedimentation, primary filtration on coke or "hogwood,"[1] secondary sedimentation, secondary filtration on hogwood and final settling; coagulant to be added to the filter effluent when thought necessary. A special plan, adapted to tanneries where definite color reduction is desired, but only moderate reduction in oxygen demand, consists of mixing, settling with coagulation, followed by primary filtration, secondary sedimentation, secondary filtration and final settling as in the general plan. The sludge under either plan is to go either to sludge beds or to lagoons.

Attention was called to the interest with which other states had awaited the results of this investigation and to the fact that cooperation between states in making available such results avoids duplication of work and results in economy of the public funds.

Wastes from Scouring and Washing Wool.—Raw wool contains much dirt and grease, which are removed by scouring or washing in warm water, with alkali and soap. One to 5 gal. of water per pound of wool are required for washing by modern methods, and much more is required by methods less economical in the use of water; 100 gal. of water to 1 lb.

[1] Wood chips varying from matchstick to clothespin size.

of wool have been noted. The waste liquid is laden with fats and suspended mineral and organic matter. Shrinkage in the weight of the wool by this process may amount to 60 to 75 per cent for American wools and 50 to 60 per cent for English wools. The proportion of grease varies from 8 to 12 per cent for American and from 12 to 16 per cent for English wools.

It is sometimes important to remove the grease from wool washings before they are discharged into the sewage, because it is likely to interfere seriously with the process of purification. The grease discharged from a wool-scouring plant into municipal sewers has been known to seal the surface of sand filters and put the beds absolutely out of service. The removal of the grease requires the use of acids or other chemicals, which remain to a large extent in the liquors after the grease has been settled out or skimmed off. In case the wastes constitute a large proportion of the municipal sewage, if these chemicals are not neutralized before being discharged into the sewers, they may have enough disinfecting action upon the sewage to render biological treatment difficult.

Wastes from Washing Woolen Cloth.—Woolen-cloth washings consist of soapy water used in washing cloth before dyeing, spent dyes from vats and water used in rinsing the dyed cloth. These wastes are generally cold and contain few bacteria. They are not readily purified by biological methods unless mixed with sewage. If, however, the proportion of the wastes to domestic sewage is not large, they will not seriously interfere with the purification of the sewage by biological methods.

Paper-mill Wastes.—The basis of all papers is vegetable fiber, or cellulose, and a part of the paper-trade wastes comes from the treatment of raw materials to rid them of resins, gums, silica, fats, oils, dirt and, in fact, everything of which the raw material is composed except the cellulose.

In one process of manufacture the treatment consists of boiling the stock with alkali for several hours, after which the material is washed. With some stock, beating and pulping are necessary, after which the material is bleached and then made into paper. The wastes from the boilers are small in quantity, hot and strongly alkaline, containing the gum and dirt from the raw material. The washer wastes are similar to the boiler wastes but are much more dilute, the quantity of washer wastes being frequently ten times the quantity of boiler wastes. The washer wastes contain considerable fiber, which can be removed by screening and by settling in tanks. The details of the processes at any particular mill affect the nature and quantity of the wastes.

In another process, chips of wood are cooked in a digester with sulfite liquor, obtained by either drawing the fumes of burning sulfur through lime water by a vacuum process, or passing them through broken limestone. The digested pulp is blown into tanks and washed. The sulfite liquor from the blow tanks is a heavy, dark-brown liquid, containing the dissolved material from the wood, sulfite of lime, sulfurous acid and some pulp. The treatment of these wastes is extremely difficult and both the paper industry and sanitary authorities have devoted much attention to it.

The machine wastes, amounting to five to ten times the quantity of washer wastes, contain the fine fiber which escapes from the paper-making machines, together with small quantities of clay, if it is used with the pulp, and coloring and coating materials, where colored and coated papers are being manufactured. The fine fiber, the clay and usually the coloring matter are in suspension, but they are divided so finely that the wastes must be dosed with some chemical, such as alum, to produce rapid precipitation in tanks. This usually removes 90 to 95 per cent of the suspended matter, which may be utilized in the machines where the lower grades of paper are produced.

All machine wastes are cold and contain few bacteria. They are not readily purified by biological methods unless they are mixed with sewage. In many cases it is not practicable to do this, on account of the large quantities of wastes in comparison with the sewage from the communities in which the mills are situated. Often paper-mill wastes may be sufficiently purified without biological treatment, by chemical treatment, sedimentation, or combinations of these processes, to be discharged into streams.

Temperature of Wastes.—Some industries, such as certain canneries, produce hot wastes which, if discharged into municipal sewers or into streams, may cause rapid biological decomposition and the production of offensive odors. On the other hand, large quantities of cold wastes, such as those from paper mills in winter, may cool the municipal sewage sufficiently to retard the biological action required for effective treatment. In general, however, the effect of the temperature of industrial wastes is not great.

Under certain conditions, where relatively large quantities of clean, hot, condenser waters are discharged into small, polluted streams, the resulting increase in temperature of the streams may hasten decomposition processes so as to bring about offensive conditions, which otherwise would be prevented by normal processes of self-purification.

Disposal of Wastes by Dilution.—The disposal of organic wastes by dilution is subject to substantially the same natural laws and agencies as that of sewage. Mineral wastes, however, generally introduce some different problems, both when disposed of by themselves and when mixed with organic wastes. In several cases it has been possible to supplement, at times of low flow, the natural dilution afforded by streams, by drawing water from reservoirs, thus adjusting the degree of dilution to the biological requirements of the streams and preventing objectionable conditions.

Separation of Clean Waters.—The drainage systems of industrial establishments are commonly built so that they receive roof and surface water, relatively clean wastes and the foul and objectionable wastes. Frequently the roof and surface water may be separated from the wastes, thereby permitting a reduction in the volume of wastes to be treated.

In many industries the quantity of relatively clean wastes produced is large. For example, a certain wool-scouring plant produces approximately 280,000 gal. of wastes daily, of which only about 30,000 gal. are foul and need treatment under local conditions. Another example is afforded by a silk mill, where the volume of the several kinds of wastes aggregates 1,300,000 gal. daily, of which about 950,000 gal., or 75 per cent, are relatively clean waters which do not require treatment.

When local treatment is required, it is distinctly advantageous to separate the roof water and other clean water from the objectionable wastes and to treat the latter only. In some cases, however, where a process produces alternately foul wastes and clean wastes from the same machine and the separation is made by the regulation of valves and other devices by the operators, unsatisfactory results have been obtained, because of failure of employees to attend to the separation. In order that separation may be effective, it may be brought about in many cases by methods largely independent of the human element; in some instances, however, satisfactory results of manual operation have been secured by rigid supervision and more or less constant inspection.

Equalization of Flow and Composition of Wastes.—In many industries the wastes vary widely from time to time, both in rate of flow and in composition. As such irregularities seriously interfere with the efficiency of local treatment, some plants provide for more or less equalization by means of storage or otherwise. In cases where there is a continuous, 24-hr. stream flow and wastes are produced only during a working day of 8 to 10 hr., the wastes may be stored in tanks and discharged uniformly throughout the 24 hr.

Even where discharged into public sewers of large cities, the wastes may be discharged intermittently in such quantities as to influence materially the treatment of the sewage. For many years at the sewage-treatment plant in Worcester, Mass., certain tanks were reserved to receive the sewage at times when it contained large quantities of acid iron salts and the quantity thus stored was utilized throughout the remainder of the day, to supply the ferrous sulfate required for the treatment of the sewage.

Reactions between Different Kinds of Wastes.—At some plants wastes from different processes may be mixed in such a way and in such proportions as to aid materially in the treatment of the combined volume of wastes. In the case of one tannery where wool is scoured, the unfavorable effect of the discharge of large quantities of acid wool liquors after cracking is avoided by providing for their gradual discharge throughout the day. Thus they are neutralized by the alkalies in the wastes from the tannery. At a certain woolen mill the spent aluminum-chloride carbonizing liquor has been stored and utilized as a precipitant for the treatment of the other wastes. At a certain cotton bleachery neither the bleach wastes nor the sour wastes are suitable for discharge into the river, but when these wastes are brought together under favorable conditions for interaction, precipitation takes place which renders the combined wastes suitable for discharge after sedimentation. At one tannery the effective treatment of the wastes as produced and discharged from the several processes was found to be impracticable, but after equalizing them by detention and thorough mixing, their treatment is entirely feasible. This is the result of equalization in both volume and quality and of interaction of the constituent liquors.

Treatment Processes.—The methods used for the treatment of sewage are applicable to the treatment of many wastes; for others either special adaptations of existing methods are made or new processes are developed.

Racks commonly are used whether the wastes are discharged into municipal sewers or into water courses. They retain large objects, such as boxes and boards, which otherwise might enter the sewers and clog them or cause damage to pumps and other machinery. Such racks are often of some slight financial benefit in holding back articles of value which otherwise would be lost, such as skins and hides at tanneries and silks and other textiles at finishing plants.

Fine screens are used for the removal of some of the larger suspended particles from certain industrial wastes. Such screens have been employed for removing small quantities of wool from wool-scouring rinse waters and for treating the combined wastes from certain tanneries. Fine screens have been recommended for removing skins, stems, seeds and small tomatoes from wastes produced in the food industry. Save-all screens are employed in many paper mills, to retain for use much fiber which otherwise would pass away in the wastes and be lost.

Fine screens have also been used with success by various industries, to keep out of sewers solid matters which might form deposits or cause the production of large quantities of inert sludge at municipal treatment plants.

Sedimentation tanks are used at many industrial plants. Sedimentation goes much farther than fine screening in the removal of suspended solids. Efficient sedimentation usually removes the settleable solids capable of forming extensive mud banks or deposits in streams. At times, however, interaction of the dissolved or colloidal matters in the wastes and of the chemical constituents of the receiving stream or tidal waters may cause after-precipitation, even though the suspended solids be removed before the wastes are discharged. In many cases sedimentation furnishes a degree of purification which is sufficient to meet local needs.

Wastes from some processes in the paper industry are passed through sedimentation tanks, by means of which 95 to 98 per cent of the fiber is removed. The stock thus recovered usually is returned to the beaters or thickeners and utilized.

As previously pointed out, certain wastes contain large quantities of suspended solids, which may form objectionable deposits in sewers and add greatly to the sludge deposited in the sedimentation tanks of municipal treatment plants. In a number of cases such tanks have been seriously overloaded by solids of industrial origin. To prevent such difficulty, sedimentation tanks may be provided at industrial establishments, for the removal of the suspended matter from their wastes before discharge into the municipal sewers. Such tanks have been employed for many years in the Boston Metropolitan Sewerage District, at Gloversville, N. Y. and, more recently, at Fostoria, Ohio.

Chemical precipitation is a useful process for the partial purification of certain industrial wastes. By such treatment some coloring substances, turbidity, suspended solids and even colloidal solids, which would not be held back in the ordinary sedimentation process, can be removed effectively. Under certain conditions chemical treatment is adequate and under other conditions it is a valuable preparatory process, to be supplemented by other treatment in order to secure the necessary degree of purification.

Alum, copperas and lime, sodium aluminate and other chemicals, with or without control of the hydrogen ion concentration, have been used for the treatment of various kinds of wastes. In many cases the reactions between different kinds of wastes, whether subjected to artifical control or not, constitute the chemical treatment.

Chemical precipitation has been employed successfully for the treatment of cannery, textile, tannery and many other wastes. This process is rarely employed for the treatment of wastes before their discharge into municipal sewerage systems.

Oxidation processes have come into relatively frequent use for treating wastes, in order to meet the more rigid standards of cleanness of streams, demanded by the public in recent years. As in the case of sewage treatment, these processes provide a greater degree of purification of organic wastes than the simpler methods of screening, sedimentation and chemical precipitation.

All the oxidation processes, intermittent filtration, application to contact beds or trickling filters and activated-sludge treatment, have been employed in the treatment of industrial wastes. The selection of a particular process has depended upon the character of the industrial wastes to be treated and other local conditions. The quantity of wastes which can be successfully treated in such plants varies widely and depends upon the constituents, strength, temperature and condition of the wastes as well as upon the degree of purification required. Wastes high in organic matter are more susceptible to biological treatment than those in which mineral constituents predominate. Cannery, paper-mill, tannery, sugar-refinery and many other wastes are being successfully treated by oxidation processes. Such treatment of wastes is seldom required before their discharge into municipal sewers.

Chlorine treatment has been employed to a limited extent for the disinfection of wastes deemed to contain pathogenic organisms, the reduction of biochemical-oxygen demand, the mitigation of objectionable odors and the prevention of the growth of fungi in streams.

Milk wastes have been treated with chlorine to prevent septic action and resulting objectionable odors, where the wastes were applied to land and pooled for a time before disappearing in the soil. Tannery wastes have been disinfected as a safeguard against the spread of the anthrax bacillus upon pasture lands.

Miscellaneous Processes.—In the case of gas and by-product coke wastes special treatment processes have been developed. Wastes from by-product coke plants have been evaporated by using them to quench the glowing coke. Residual wastes from a water-gas plant have been treated with ferrous sulfate and filtered through coke breeze, the effluent being used for boiler feed water. Processes have also been developed for the recovery of phenols from the wastes.

These instances are typical of special treatment processes devised to meet particular needs.

Regulation of Treatment According to Climatic Conditions.—The effect of organic wastes discharged into rivers is influenced greatly by climatic conditions, particularly precipitation, with its resulting stream flow, and temperature, which vary markedly from season to season as well as from year to year. The cost of treating some wastes is so great that, where possible, intelligent regulation of the treatment process according to climatic conditions is highly desirable and much more important than in the case of municipal sewage.

Fales (7) has described in detail such regulation, as applied to the treatment of the wastes from a paper mill and a tannery in Massachusetts and to the Neponset River, into which the treated wastes are discharged.

The principles involved and the method of carrying out such regulation may be illustrated by the following quotation from one of the authors (8):

This plant provides for degreasing the wool scouring liquors and treating highly polluted wastes by coarse screening, plain sedimentation, chemical precipitation, intermittent filtration and dilution.

The operation of all features of this plant at maximum practicable efficiency during all seasons of the year, would require an exceedingly large expenditure for results unnecessarily refined. The plant is therefore so operated as to provide only such degree of purification and dilution as is necessary to prevent the formation of deposits, gross discoloration, high turbidity and putrefactive conditions along the river.

Treatment is subject to seasonal regulation. At times of *extreme high river and low temperature*, the tannery and scouring wastes are simply screened and settled. With an *ordinary high river and low temperature*, the scouring liquors are degreased prior to admixture with the tannery wastes. When the *river flow and temperature are moderate*, the wastes from the sedimentation tanks are diluted with an appropriate measured volume of water from a large storage reservoir. When the *river is low and the temperature is high*, about one half the settled wastes are diluted before discharge, and the remainder subjected to chemical precipitation followed by intermittent filtration—a stable effluent being produced. During a few short periods of *extreme drought in warm weather*, sodium nitrate has been added to the final effluent to provide a supply of oxygen. At *all times* a portion of the wastes which are only very slightly polluted, is discharged directly into the river without treatment.

The regulation of the treatment is under the supervision of a chemist who devotes his entire attention to this work and who is aided in the exercise of judgment by weekly inspections and analyses of river water for several miles below the tannery.

Sludge Disposal.—The disposal of the solids removed from industrial wastes presents a problem similar to that of municipal treatment plants, but often far more difficult and expensive to solve. Certain sludges have a small manurial value, tannery sludge being an example. The lime usually present in this sludge may be beneficial to some soils, but the large quantity of grease in some of these sludges is detrimental.

Certain paper-mill wastes produce large quantities of sludge which has no manurial value and which requires disposal at large expense. At wire mills and other plants discharging pickling liquors, the precipitation of the iron forms a voluminous, valueless sludge, which can be gotten rid of only at relatively large expense.

The methods of dewatering and drying sewage sludge are applicable to wastes sludges, but the size of plant required per million gallons to be treated often is far in excess of that necessary for municipal sewage.

Cost of Wastes Treatment.—The costs of wastes-treatment plants and their maintenance and operation vary greatly according to the volume and character of wastes and the degree of purification required. Such costs are seldom less than those for a corresponding degree of purification of municipal sewage. The construction cost of plants may be expected to vary from less than $9000 per m.g.d. of wastes for fine screens to more than $260,000 per m.g.d. for much more complete treatment, including oxidation by filtration or by the activated-sludge process.

The annual costs of maintenance and operation vary even more widely than the cost of construction and are dependent not only upon the volume and character of the wastes to be treated and the degree of purification for which the plants are designed, but also upon the saving which can be made by varying the treatment from time to time according to climatic conditions. Such costs may be expected to range from less than $3 to more than $200 per mil. gal. of wastes treated. Where the wastes are difficult to treat and the required degree of purification is high, an annual expenditure of upwards of $25,000 per mil. gal. treated daily may be expected.

Influence of Wastes-disposal Problem upon Selection of Industrial Site.—When selecting sites for the construction of industrial works, careful consideration is usually given to the volume and quality of the available water supply. It is also important to look forward to the disposal of the wastes produced. In some cases it has been found that wastes treatment probably could be postponed for a number of years and that for a considerable time thereafter only partial treatment would be required. The relative expense for wastes disposal may, therefore, become the deciding condition as between available sites.

Attitude of Industrialists toward Wastes Treatment.—It has been the experience of the authors that industrialists do not welcome law-enforcement measures in the matter of stream pollution by wastes. On the other hand, when the need for treatment has been explained fully and the manufacturers have been convinced that the plant for such treatment is as economical and in all respects as reasonable as possible, they have shown a marked willingness to cooperate in the maintenance of the rivers in reasonably good condition.

The magnitude of the problem of industrial wastes disposal is realized by few engineers, manufacturers and legislators. There are many single industries the polluting properties of whose wastes are equivalent to those of the domestic sewage from a city of 100,000 persons and some of the wastes are far more expensive to treat than the sewage from an equivalent population. It is therefore of the utmost importance that the laws be administered with due regard for the "rule of reason," that treatment be regulated according to the actual needs in each case, and that treatment plants be designed and operated with a keen appreciation of cost, on the one hand, and of the accomplishment of the requisite results, on the other.

Bibliography

1. Ann. Rept., Md. State Board of Health, 1930.
2. *Jour.*, Am. Water Works Assoc., 1923; **10,** 415.
3. Wieters, A. H.: *Water Works Eng.*, 1928; **31,** 204.
4. Ravenel, M. P.: *Public Health*, 1898; **24,** 302.
5. Rept. to Common Council of City of Gloversville, N. Y., on Sewage Purification Experiments and Sewage Disposal, 1909.
6. "Treatment of Tannery Wastes." Pa. Dept. of Health, 1930.
7. Fales, A. L.: *Jour. Ind. Eng. Chem.*, 1929; **21,** 216.
8. Eddy, H. P.: *Trans.*, Int. Conference on Sanitary Eng., 1924, p. 217.

CHAPTER XXXIII

OPERATION AND MAINTENANCE OF SEWAGE-TREATMENT PLANTS

The efficient operation and suitable maintenance of sewage-treatment plants are quite as important to their successful functioning as good design. There are far too many instances of costly treatment works which have been neglected grossly after construction. It is probably no exaggeration to state that millions of dollars of investment in sewage treatment have been wasted as a result of such neglect. Fortunately there is a steadily growing appreciation of the need of trained supervision of operation of treatment plants and an increasing number of plants are being operated by technically trained men. Furthermore, the numerous sewage-works operators' associations throughout the United States are doing much in the way of educating the nontechnical operators of the smaller plants.

Responsibility for Successful Operation.—The immediate responsibility for successful operation of a sewage-treatment plant rests, of course, with the man in direct charge. In a small plant the man in charge constitutes the entire staff, in which case the whole responsibility rests upon him. It is the duty of the man in charge of a larger plant to establish routine operation schedules, whereby each employee is assigned certain definite tasks to be performed regularly. It is also his duty to see that all tasks are performed efficiently and promptly.

In addition to supervising other employees, the chief operator, superintendent, or whatever his title may be, must determine when certain special work is required; for example, when to draw sludge, when to reverse the flow in Imhoff tanks, when to flood trickling filters for *Psychoda* control, and so forth. Certain details of operation require attention at practically every treatment plant. There may be other details, however, which are peculiar to any given plant, and the scope and character of these details must be determined by the man in charge. It is his responsibility to see that these operating details receive the requisite attention.

Results of Faulty Management.—If a sewage-treatment plant is not managed efficiently, many unsatisfactory conditions may result. One of the first results of poor operation is deterioration in the quality of the effluent. In fact, it is possible for a treatment plant to be operated so as to produce an effluent which is no better than the incoming sewage.

Inefficiently operated plants may cause annoyance to the public, due to obnoxious odors, the breeding of flies and rats and the production of other objectionable conditions.

In some instances, poor management may be responsible for unduly high operation costs. Such instances, however, are probably the exception rather than the rule, if the comparison of expenditures is based upon costs per million gallons of sewage treated, due to the fact that available funds for operation are generally none too lavish. On the other hand, if the comparison is made upon some standard of accomplishment, such as the cost of removing 100 lb. of suspended solids or the cost of satisfying 100 lb. of oxygen demand, it is probable that instances of poor economy in operation would be found to be comparatively numerous.

The failure of sewage-treatment plants to accomplish all the purposes for which they are designed often is due to combinations of poor management and inadequate funds for operation. Under such conditions the results may be even more unsatisfactory than when only one of the unfavorable factors is present.

Providing Funds for Operation and Maintenance.—The recognition of inadequate appropriations as the most serious obstacle to securing efficient operation has led state departments of health to foster and, in some cases, to obtain legislation whereby sufficient funds can be assured. Such legislation has generally taken the form of enabling acts which permit municipalities to make annual charges or rentals for the use of sewers, similar to charges for water service. Ohio was a pioneer state along these lines, passing a sewer-rental law in 1923. In Massachusetts and Texas sewer rentals have been in effect in several municipalities without a special act. Brockton, Mass., has used a sewer-rental plan since 1894.

Intelligent Operation Necessary.—Conscientious and faithful attention to duty, valuable as it may be, is not the sole criterion as to what constitute the characteristics of a successful sewage-plant operator. In addition to the attributes of conscientiousness and faithfulness, the operator should possess somewhat more than average intelligence and common sense and should have some understanding of the general principles involved in the treatment process under his care.

For example, a faithful but untrained operator may withdraw the entire accumulation of digested sludge from an Imhoff tank on the assumption that it is desirable to do a thorough job of cleaning. Such action, of course, results in serious difficulty in the reestablishment of good digestion. Instances of this character have been comparatively frequent. Another example of unintelligent operation occurs in the case of intermittent sand filters, when the operator plows or rakes into the filter beds the sludge which has accumulated on the surface of the beds. Although such a course of action temporarily increases the filtering capacity of the beds, serious clogging thereof eventually occurs.

Where the more complicated processes of treatment are involved, such as separate sludge digestion, activated-sludge treatment and the like, the opportunities for trouble to result from unintelligent operation are even greater.

Operation under Consulting Supervision.—It is becoming rather general practice for designing engineers to retain for one or two years consulting supervision of operation of new plants constructed after their designs. This practice is of benefit to the municipalities because the engineers know the details of the plant, for what purpose they were included in the design and how they are supposed to function. Usually the engineers have had previous operating experience which is of value in the "tuning up" of a new plant.

For small plants, where the funds available do not permit the engagement of a full-time, technically trained operator, the retention of consulting engineers to maintain general supervision of operation with occasional visits to the plant is distinctly advantageous. The value of such supervision lies in the fact that occasional visits and the receipt of regular reports permit the engineers to give operating instructions which, if carried out, will result in maintaining satisfactory performance on the part of the treatment plant. Furthermore, this general supervision has a tendency to maintain the interest and morale of the local operator. Another advantage of the maintenance of close contact between the engineer and the treatment plant is that difficulties may be anticipated and improvements or changes may be put into effect before such difficulties occur.

Records of Operation.—There are several reasons why regular records of operation of sewage-treatment plants should be kept. In the first place, such records may be of invaluable assistance to the operator in guiding his judgment in the details of operation. Furthermore, where consulting supervision is maintained, it is necessary for the consultants to have regular and complete reports, in order to permit the formation of sound judgments and the giving of correct advice. In some states regular monthly reports of sewage-works operation are required by the state departments of health. Another reason for the keeping of operation records is the possibility of their being required in event of litigation relative to the treatment plant. The keeping of regular records is also of material help in maintaining the interest and morale of operators.

The details of the records to be kept depend largely upon the type of plant, its size and other conditions which are more or less variable. Certain essential data should be kept at all plants, but there may be wide latitude in the details of operation of which records should be maintained. Space does not permit a discussion of the various records which may be kept. This matter has been reported upon by committees of the New England Sewage Works Association and the American Society of Civil Engineers (1) (2).

Generally speaking, it is better to use record forms of ordinary letter-paper size, namely, 8½ by 11 in., or else folded forms, not more than 11 in. high. Long record sheets are cumbersome and difficult to file. Although the use of the letter-size forms may require more than one form for any given period of time, it is possible to group the items in such a way as to have related data on the same sheet.

Analyses of Sewage and Effluent.—As is the case with the keeping of operation records, the analytical determinations which should be made at treatment plants depend upon the type of treatment in use, the size of the plant and the adequacy of laboratory facilities and personnel for performing analytical work.

The simplest determinations, which can be made by operators without technical training, are those of settleable solids in Imhoff cones and of relative stability by the methylene-blue test. The former test indicates roughly the efficiency of settling tanks and the latter affords information as to the degree of oxidation of the effluent from oxidation processes.

At large plants employing more or less complicated methods of treatment, it is desirable to carry on rather complete routine analyses. In general, the two determinations which are most significant are those of suspended solids and biochemical-oxygen demand. With these determinations made upon the sewage and effluent, it is possible, within limits, to know with a fair degree of accuracy the extent of improvement brought about by a treatment plant. Local conditions often control the significance of any particular determination. For example, the determination of iron may be of the greatest importance in the case of a sewage which receives acid iron wastes.

In some cases it may be fully as important to make analyses of the stream or other body of water into which the effluent is discharged as to make them of the sewage and effluent. The analyses may consist of determinations of dissolved oxygen and oxygen demand in samples collected at various places upstream and downstream from the point of effluent discharge. Other determinations also might be made, including plankton studies, but the two tests suggested are probably the most valuable ones which could be employed under ordinary conditions.

In routine plant operation it is generally desirable to make measurements and analyses of the sludge produced and requiring disposal. Among the essential determinations are specific gravity, total solids, loss on ignition and pH. In some cases complete fertilizer analyses may be of value, while in other cases determinations of ether-soluble matter and total nitrogen may assist in solving the sludge-disposal problem. Where gas is collected and utilized, measurements of its volume and occasional analyses to determine its methane and carbon dioxide content are needed.

Great care must be exercised in collecting samples which are representative, if reliable results are to be obtained.

State Control.—In many states general oversight of the operation of sewage-treatment plants is maintained by the engineering divisions of the state health departments. In certain states the health departments require monthly records of operation, while in other states they maintain regular inspections of plants. In Massachusetts samples of sewage and effluent from treatment plants are analyzed monthly. The thoroughness and efficiency of the oversight exercised are largely dependent upon the funds available for such service.

Maintenance.—At every sewage-treatment plant there is need of maintaining the physical condition of the plant and structures. Where machinery is used, such as pumps, mechanical screens and other equipment with moving parts, it is necessary to keep such equipment well lubricated and clean, as well as to paint or otherwise protect all exposed metal parts. Buildings of wood and the trim of masonry structures should be kept painted, brickwork should be pointed from time to time and necessary repairs should be made promptly. All parts of the plant should be kept clean and shipshape at all times.

The increasing tendency for the grounds of sewage works to be landscaped attractively involves more attention to the care of such grounds. Lawns require mowing and raking at frequent intervals, driveways, roads and walks need to be maintained and trees and shrubs should be pruned and sprayed.

Too much emphasis cannot be placed upon the suitable maintenance of sewage-treatment plants. Elaborate landscaping is not essential, but neatness and cleanliness are. Furthermore, maintenance of structures and equipment in good condition retards depreciation and minimizes cost of operation in the long run.

Costs of Operation and Maintenance.—The costs of operation and maintenance of sewage-treatment plants vary with the size of plant, type of treatment, degree of purification effected and local costs of power, labor and supplies. For comparative purposes, costs are commonly figured on the basis of either annual cost per capita served or cost per million gallons of sewage treated. To make such figures more readily comparable, the costs of operation and maintenance of sewage pumping units should be excluded from those of sewage-treatment plants.

Wisely and Ferguson (3) have studied the costs of operation of certain Illinois plants employing intermittent sand filters, trickling filters and the activated-sludge process, and have summarized the ranges of unit costs as given in Table 156.

TABLE 156.—SUMMARY OF UNIT OPERATING COSTS FOR VARIOUS TYPES OF TREATMENT AT CERTAIN SEWAGE-TREATMENT PLANTS IN ILLINOIS

Type of treatment	Number of plants	Unit	Range of cost		Adjusted average cost
			Minimum	Maximum	
Activated-sludge, mechanical aeration	3	Per capita yearly	$0.90	$0.98	$ 0.93
		Per m.g.	16.25	27.01	24.00
Trickling filters, total, excluding cost of power	17	Per capita yearly	0.17	3.05	0.50
		Per m.g.	3.83	20.21	9.75
Trickling filters, sewage not pumped	3	Per capita yearly	0.44	0.68	0.57
		Per m.g.	3.83	14.69	10.95
Trickling filters, sewage pumped	14	Per capita yearly	0.29	4.42	0.75
		Per m.g.	6.49	23.83	13.85
Intermittent sand filters, excluding cost of power	8	Per capita yearly	0.25	0.75	0.47
		Per m.g.	6.97	21.67	11.90

The cost of operation of the trickling-filter plant at Schenectady, N. Y., for 1932 was $7.47 per mil. gal. treated. The cost of operating a similar plant at Worcester, Mass., for 1931 was $8.20 per mil. gal., equivalent to $0.31 annually per capita served. The net cost of operating the plain-sedimentation, separate-sludge-digestion plant at Grand Rapids, Mich., for the 12 months ending March 31, 1934, excluding pumping, was $2.92 per mil. gal. of sewage treated, equivalent to $0.16 annually per capita served. Including pumping about 73 per cent of the sewage, the net cost of operation was $4.53 per mil. gal. treated, or $0.24 annually per capita served.

The operation and maintenance costs of the activated-sludge plant at Springfield, Ill., during 1932 were $13.11 per mil. gal. treated, or $0.53 annually per capita served. The cost of operating the activated-sludge plant at Peoria, Ill., for 1932 was $22.92 per mil. gal. of sewage treated, or $0.70 annually per capita served. The operation cost of the activated-sludge plant at Indianapolis, Ind., for 1931 was $13.27 per mil. gal. The operation cost of the activated-sludge plant at San Antonio, Tex., for 1933 was $8.76 per mil. gal. of sewage treated.

Bibliography

1. *Sewage Works Jour.*, 1931; **3,** 108.
2. *Civil Eng.*, 1930; **1,** 91.
3. WISELY, W. H., and H. F. FERGUSON: *Eng. News-Rec.*, 1933; **111,** 780.

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