WORKS OF

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SEWER DESIGN

BY

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PREFACE TO FIRST EDITION

THE following pages represent, except for some necessary minor changes, a course of lectures given in the College of Civil Engineering, Cornell University.

The course is an elective one, intended for those students whose intention to enter the field of Sanitary Engineering calls for more special and detailed work than is required of all Civil Engineering students. Supplementing as it does the regular course in Sanitary Engineering, it must preserve without duplication a continuity in the two courses which is obtained through the direct supervision of the Dean of the College, Professor E. A. Fuertes, who also gives the general course. These conditions may serve to explain some recognized peculiarities and omissions in the subject-matter of the following pages, tolerated only on account of the general work already done by students here specially studying that in which they wish to excel.

Another cause, leading to the omission of certain discussions which might properly be brought up under the title chosen, lies in the fact that the lectures here given represent but one-third of the year's work, the subjects of Sewage Disposal and Sewer Construction being taken up in the other two terms of the year.

Thus, merely to serve the divisions of the college year, all questions of constructive design and field construction are remanded to another course of lectures not conveniently included here.

It is believed that due acknowledgment has been made to the various books and periodicals and to the reports of the prominent engineers from which this monograph has been prepared, and it is hoped that the collection and unification of this scattered material may not only aid the students examining the question of Sewer Design for the first time, but may also be a convenient reference for older engineers who have hitherto been obliged to put together the data from many publications.

Special acknowledgment is made to the published papers of Emil Kuichling, C.E., of Rochester, N. Y., for the chapters on Storm-water Discharge and Mathematical Formulæ; to the report of Dexter Brackett, C.E., on the Future Water-supply of Boston, Mass.; to the thesis of Elon H. Hooker on Suspension of Solids in Flowing Water; to the Hering & Trautwine Translation of Kutter for the chapter on Kutter's Formula, and for that on the Development of the other and earlier Hydraulic Equations; to Hering's Report to the National Board of Health for the chapter on Lateral Location; and to Baumeister for the general arrangement of sewer systems.

PREFACE TO SECOND EDITION

After fifteen years of use in his classes, the author has been enabled to prepare a second edition of this little book, partly for the purpose of correcting certain minor errors and partly that certain additional material might be inserted.

In recent years a great deal of attention has been given to the careful analysis of the problem of rainfall and run-off; notably by Messrs. Grunsky, Gregory and Nordell, and only after much hesitation was it decided to omit any extended review of their thorough, detailed discussion. The relation between rainfall and run-off is dependent upon many very uncertain factors, and any close mathematical analysis of the way in which water, delivered to any area as rain, is collected into one channel at a single point of the area must include so many assumptions that the result may be far from the truth, although correct according to the theory followed. It was decided, therefore, to again follow the method of Kuichling, adopted in the first edition, as being reasonably safe in principle and at the same time holding clearly before the student the chief factor that enters into the relation, viz., the impervious character of the surface on which the rain falls. Reference is made, however, to the more mathematical treatment, and anyone interested may consult the original papers if he so desires.

The use of better jointing material for vitrified pipe has made it possible to eliminate leakage entirely, if sufficient care be taken in construction. Various asphaltic preparations and other patented articles are on the market and if properly used will ensure a perfectly water-tight pipe line. This fact modifies somewhat the earlier statements about the necessary amount of leakage to be considered.

The problems that have been added have been found very useful in affording an insight into some of the broader questions that enter into the design of sewers, as well as in giving the student a chance to apply his theory to a concrete example.

The book as a whole is not materially changed, and continued experience in large classes has confirmed the author in his belief that for the general course student in Civil Engineering it contains the essentials of sewer design so arranged as to be readily intelligible to the average mind.

H. N. O.

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SEWER DESIGN

CHAPTER I

GENERAL CONSIDERATIONS

In preparing the design and making the plans for a system of sewers for any given city, there are some preliminary questions to be settled before the location of the mains and the sizes of the pipes can be determined. Perhaps the first of these questions is whether the system shall be designed to carry house-sewage alone, or rain-water from the streets, roofs, and yards as well. Arguments for the one arrangement or the other have been carried on in the abstract for many years, chiefly from the sanitary standpoint, but the question is properly settled by the local conditions of the place under consideration.

The combined system, as the system intended for rain-water and sewage is called, is the result of growth and development and so has the prestige that comes from age. Not very long ago, the function of sewers was to carry off the storm-water falling on the streets and to keep the yards and basements dry; while the privies, which were then generally used, were cleaned and cared for without reference to the sewers. When water-closets came into use they were, after a time,—for it was at first forbidden by law,—allowed to discharge into the storm-sewers. In this way the channels, which may have been in the beginning brooks, afterwards walled in and arched over as the city grew, came to be the receptacles for the house-refuse and water-closet wastes. Naturally, the channels thus developed were not of the best section or design for this, their final use, and in England and in the older cities of this country, where examples of the process are yet to be seen, accumulations of filth and deposits of rubbish are the evidences of rough interiors, flat gradients, and shallow depths.

Small sewers for house-sewage were used in the United States before 1880, and sanitary engineers now prominent in this country prepared plans for sewerage systems, using small pipes and keeping out practically all the storm-water. But it is due to Col. Geo. E. Waring that the old prejudices have been so entirely removed and the manifest advantages of small pipe-sewers so strongly emphasized. It was in 1880, in a public address, that he said that the conditions of drainage had been changed, and that engineers must recognize that the number of water-closets now used made the construction of sewers for their exclusive care a necessity.

This use of small pipe-sewers with its accompanying details of construction was patented under the name "Waring's Separate System," and under this patent Col. Waring was paid large royalties by some of the cities for which he acted as consulting engineer.* The principal features of the "Waring System" as described under U. S. patents 236,740 and 278,339 are: first, absolute exclusion of the rain-water; second, ventilation of street-sewers through house-pipes not trapped against the sewer; third, automatic flush-tanks at the head of every lateral; and fourth, soil-drainage by pipes laid in the sewer-trenches.

Waring's prejudice against combined sewers was very strong, as indicated by the following quotation from a public address delivered in 1880: "In closing permit me to formulate my

* In a letter to the author dated January 30, 1899, the executor of Col. Waring wrote as follows in reference to these patents:

"The patents to which you refer (which are the property of the Drainage Construction Company of Boston) are still in force. The patents have been disputed, and suits are now in progress, with a view to establishing their validity. Pending decision, the owners are granting licenses upon a cash payment of half royalties—five cents per lineal foot of sewer—or an agreement to pay full royalties if the patents are sustained by the courts."

Both patents have now (1912) expired.

opinion on this subject by saying that the present manner of disposing of storm-water in sewered towns by removing it from the surface where it is needed to the sewer where it creates a nuisance is a relic of barbarism, and that its continuance indicates an overriding of reason by tradition." This he later qualified by saying: "I think that the necessary sanitary requirements may be met by the combined system if due attention is given to the details, and if enough money is spent." He aroused much controversy among engineers, and the sanitary advantages of both methods were discussed at length.

In the second annual report of the Massachusetts State Board of Health, 1881, p. 25, is a paper by E. C. Clarke, then engineer in charge of the Boston sewers, giving briefly all the arguments in favor of the combined system; and a paper by Benezette Williams, in the Journal of the Ass'n of Eng. Soc., Vol. IV, page 175,* gives additional discussion from the same point of view. In his book on Sewerage, Col. Waring devotes a chapter to the question, discussing the papers here referred to, and, while disavowing himself a hard-and-fast advocate of the separate system, practically says that storm-water sewers are incidental, and that for them only main outlets are in any case needed, while the sewers of his system are everywhere essential.

The arguments for the combined system are as follows:

1. Sewage from houses forms only an inconsiderable part of the noxious materials that constitute the wastes of a town; chemical analysis fails to detect any great difference between the sewage of a water-closet town and that of a town where earth privies and the pail system are used; that is, the waste water from sinks, baths, laundries, and the wash of paved streets contains enough organic matter to be nearly as foul, chemically, as the discharge of water-closets. This other material therefore requires as careful treatment as the water-closet matter.

To this it is answered that while this may be true so far as

* See also Jour. Ass'n. Eng. Soc., Vol. III, pp. 37, 67, 158, 183.

chemical examination goes, the real danger in sewage comes from definite disease-germs which are only found in sewage proper, and therefore the latter is the dangerous material. It is further said that the sewers should not in any case be made to take the place of street-sweeping carts, and that if the streets are kept clean, as they should be, the wash-water from them will not be foul, and will require no special care.

State Boards of Health have gradually acquired the right to regulate the disposal of sewage, and it is their usual practice to insist upon small pipe-sewers for house-wastes only and to disregard storm-sewage altogether. This indicates that the current scientific belief is that the real danger to health does not come from street wash, even if it does decompose and appear offensive.

2. In the matter of keeping the sewers clean, the large sewer, it is said, has the advantage over the small, both in that it can accumulate a larger amount of sewage for flushing purposes, which from the greater hydraulic radius will have a greater velocity and scouring power for the same grade, and in that, while ordinary obstructions will be cared for by flushing, there will be times when excessive deposits will occur which must be removed by hand, and then a sewer large enough for a man to enter can be cleaned at a much less expense than the small pipe which must be opened from the surface or cleaned by rods worked from the manholes.

To this it is answered that experience shows that flushing by automatic flush-tanks is sufficient to keep the smallest sewer constantly clean, and that stoppages in the pipes are of rare occurrence and easily removed. On the other hand, a large sewer, in which there is a variable flow, allows floating matter carried along in large volumes to be deposited later on the walls of the sewer, clinging in a slimy layer to the uneven brick surfaces; when the amount of sewage becomes less, this matter, in the warmth and darkness, generates noxious gases and fosters the development of bacteria. These micro-organisms when dried may float off in the air to escape through the traps into houses or through manholes into the streets.

3. On account of the larger air-space over the flowing liquid in the combined sewer, the gases of decomposition given off by the sewage are largely diluted, and there is nothing to fear from them; whereas, with the small sewers, the degree of concentration is greater, and there is consequently more danger of forced traps and greater annoyance from ventilating manholes.

The reply is that, owing to the greater amount of air to be moved, the ventilation is really less perfect in the large sewer, and that, from the slime which accumulates on the walls after flooding, there is more matter to decompose. The deposits which, it is admitted, occur in the mains of the combined system add considerably to the offensive gases during decomposition.

4. The combined system is the more economical; for if the use of the sewer is restricted to house-sewage, then there will be required for the rain-water another system of pipes of equal extent, and the cost of the two systems will be greater than that of one. This follows from the fact that the cost of engineering, superintendence, pumping, sheeting, etc., are practically the same for a large sewer as for a small one, and that the cost of excavation does not increase in a direct ratio with the size of the pipe used; and further, since the flow of sewage is insignificant compared with that of the storm-water, a sewer large enough for the latter will serve for the former purpose without additional expense.

The answer to this is that the system for rain-water need never be co-extensive with that for house-sewage, since the street-gutters will serve for the former purpose so long as the flow in them does not become a nuisance; consequently the length of the large main may be reduced nearly one-half. Further, that the rain-water drains when built seldom need be laid to the same depth or to the same outfall, as they may be discharged into any convenient watercourse at the nearest point. Also, if rain-water flowing on the streets does accumulate in excessive amounts, the result is nothing more serious than a temporary inconvenience, and no damage is done, as might be the case were sewers to be gorged with an excess of rain.

5. Finally, it is said that if the rain-water is kept out of the sewers, periodic flushing, which is of great value, will be lost, and in the case of the street serving as a storm-sewer there will be yards and alleys too low to be drained into it, whereas they could be drained into a storm-water sewer.

To this answer is made that for the irregular flushing by rain the regular use of flush-tanks can be substituted; and in case the sewage has to be pumped or treated, instead of being discharged directly into a river, the presence of the rain-water is not only undesirable but absolutely forbidden.

To sum up the reasons for selecting, for a city, sewers to carry storm-water and sewage, or sewage only, the arguments just cited may be reduced as follows: It is improbable that any house-refuse that would go into a combined system would be kept out of a separate system, so that the only contribution to the former not allowed in the latter is the rain-water from the roofs, yards, and streets and any large amount of manufacturing refuse which might be rejected from a separate system on account of the large proportion not requiring purification. If the streets are decently cleaned, there is no reason for expecting the rain to act as a scavenger, and it is better to dispose of the street-sweepings by means of sweeping-machines than to allow the rain to wash these accumulations into the sewer, to be cleaned out by hand or discharged into a river or harbor, there to be dredged out. There is, therefore, no sanitary reason why rain-water should not be separately disposed of.

As to the dangers from slime deposited on the walls of large sewers the case is suppositionary, and the evil effects entirely unproved. Notwithstanding numerous examinations of sewer-air, no pathogenic germs have ever been found—a negative argument, to be sure, but of some weight. Judged by chemical standards, the air in sewers is generally better than that in schools, halls, etc. General statistics of the health of sewer-laborers show no ill effects from their employment.

It is hard to see why it should not be possible to keep both systems clean, since the inverts of both may be made to the same radius, and so the velocity with the same grade kept equal. If the water for flushing has to be bought, the same quantity used in frequent flushes of small pipes will probably keep the sewer cleaner than single flushes, larger in amount but applied at such infrequent intervals to a large sewer that the deposits become hard and fast between times. However, modern practice seems to have the tendency to do away with flush-tanks and to keep the sewers clean by hand-flushing in such amounts and at such times and places as may be found necessary.

The alleged advantage of having the sewer large enough to enter is nothing, since with proper grades and velocities the cost of removing the few stoppages that occur is much less than the interest on the money required to build the larger sewer.

Only a few years ago, the author heard of a city engineer who still believed that large sewers only should be built and therefore, even for laterals of short blocks, constructed only brick sewers 24 inches in diameter. The excessive cost thus incurred was a heavy burden on the small village (near Pittsburgh) and no better sewer was provided than if a 6-inch pipe had been used. Indeed, on a low grade the large sewer would be distinctly inferior.

The question of ventilation is based on conditions which do not, or should not, occur. Both sewers are designed to carry all material to the outfall before decomposition has begun, so that, unless by some accident deposits take place, there is no decomposition in the sewer, and therefore no gases to be dispelled. Should deposits occur and gases arise, the ventilation through the manholes for the same sewage-flow should be as complete in the one case as in the other; any slime left on the sides of the large sewer after a rain would in decay be so diluted by the greater amount of air that the offence would probably not be any greater. The sewers which are cited to show the bad quality of the air contained are those of fifty years ago, when the laws governing the flow of sewage were not so carefully heeded, and when the street-washings were hurried into the sewer to form deposits. With equal care in the design there seems to be no reason why the small or the larger sewer should be the better ventilated.

While it is true that two systems, one for sewage and one for storm-water, over the same area, will cost about two-fifths more than a single combined one, yet the assumption that their lengths will be the same is not true. The need for storm-sewers and their necessary length to reach a watercourse are matters to be based on a study of the local conditions, but it is safe to say that in any city there are many blocks which would carry all the storm-water from the centre of the block to the cross street at the end without any nuisance or damage, and that therefore the construction of storm-water sewers in those blocks would be a municipal waste. In a printed discussion of some years ago,* Mr. Robert Moore of St. Louis, stated that on the steep streets of Kansas City, Mo., the storm-water wash in the gutters becomes a serious matter after it has run 500 to 600 feet, and that 1000 feet is the limit of endurance. Mr. Chanute, in replying, said that from actual experience in Kansas City he has yet to find the water at 1500 feet the unendurable nuisance mentioned. From his own experience the author believes that in small cities water from 2000 feet of paved street does not unduly gorge the gutter or cause any annoyance. Mr. Horner of the St. Louis Sewer Dept., says † that while there are in St. Louis sewer inlets which drain 1200 feet of street satisfactorily, he believes that this is a greater length than is generally warranted, since at the lower end the flow of water in the gutters is inconveniently deep.

On the other hand, it is more than likely that in a large part of the city there are streets where the two sewers would have to be carried at the same depth and in the same direction, and

* Jour. Ass'n. Eng. Soc., Vol. III. p. 69.
† *Engineering News*, Vol. LXIV, p. 326.

that therefore it would be economy to combine the two and build one sewer for the two purposes. In what streets this should be done, and how far it is economy to do it, must be determined by careful study and comparative computations. It is as grave a fault to design a separate sewer for a street that needs a storm-water sewer discharging into the same outfall as to build a storm-water sewer where none is needed.

It is often possible, in reconstruction or improvements, to use an old sewer for storm-water alone with entire success, building new sewers for the house-sewage. It is therefore necessary to know with accuracy the sizes and grades of all old sewers in order that if possible they may be incorporated into the design in hand.

The regulations of the different State Boards of Health are in many cases the real cause of determining whether a separate system or a combined system shall be built. In New York State, for example, no system of sewers in any community may be built unless, according to the laws, the places have received the approval of the State Department of Health. This approval is withheld as a matter of established policy for plans showing combined sewers discharging without the action of a disposal plant into inland waters. Since the cost of purifying or even treating storm-water is financially as impossible as it is unreasonable, it follows that in New York State all sewer systems are now built on the separate plan and the necessary storm drains are built according to a separate design.

Where combined sewers are already in operation and it is proposed to make extensions for the sake of house connections, the attitude of the Department of Health is made to agree with local conditions. If the extension is long compared with that part of the combined system affected, they may require new separate sewers laid to replace the old pipes. If short, they may require the entire sewer system to be replaced by pipes for domestic sewage, the system discharging into a disposal plant. The construction of this last, or at least its design and a promise to construct, is often made a condition precedent to granting the approval for the construction of the extensions asked for.

Sometimes extensions to combined systems are approved provided the dry-weather flow shall be treated in a disposal plant and that there shall also be treated dilutions of from 3- to 5-fold, depending on the ordinary concentration of the domestic sewage, the excess being discharged untreated into watercourses where it is assumed, because of its greater dilution, it will not be dangerous nor cause offence.

The desirability of the sewer acting as a ground-water drain has been strongly urged. Waring says that the primary object of sewerage is the removal of fouled waste water and of subsoil water. Therefore he makes a line of drain-tile laid in the sewer-trench an essential part of his system. The sanitary advantages of a dry subsoil are sufficiently evident and are not a question for discussion here. Whether the sewer-pipes shall serve for the purpose or not is of some interest. The disadvantages apparent are the uncertainty introduced in determining the proper size of the pipe-lines, since the amount of ground-water flow can only be determined when it is encountered, and then for that time only, the ground-water flow being quite as variable as any other stream flow. Again, since the height of ground-water seldom remains permanent, it is possible that openings left to admit ground-water may at times allow sewage to escape, thus polluting the soil and reducing the carrying power of the sewage. It seems better therefore either to provide a separate line of pipes for ground-water, discharging at near and convenient points, or else to arrange for the ground-water to enter without the opportunity for the sewage to escape. This may be done by providing special openings, as in a pipe with siphon attachment patented by S. E. Babcock of Little Falls, N. Y., and described in *Engineering News*, Vol. XXVII, p. 661, or by a special pipe, laid where the amount of ground-water makes it necessary and discharged into the sewer at a convenient manhole. It is the general practice to-day, however, to omit any such connection, and if drain-tile are laid, to carry them separately to one or more outfalls.

The question of drainage of cellars and low yards through the storm-water sewers is a serious one best settled by expediency. Like exceptionally low basement fixtures, they require for their individual accommodation a general lowering of the sewer-line or some special pipe or arrangement. The question resolves itself into one of the general *versus* the individual good: whether it is just to add to the general cost of the whole work for the peculiar benefit of one or two. When the whole section is low and relief can be given only by special means, then, as citizens, the householders are entitled to it, but it is probable that the individual case is more fairly neglected.

The combined system is not adapted to any case where the sewage has to be pumped, treated by chemicals, or disposed of on land; the rain-water must be kept out of the sewers, and no ground-water or other unpolluted water allowed to enter in all such cases.

PROBLEMS

1. Given the formula of hydraulics $Q = AC\sqrt{RS}$, define carefully each of the factors used, and notice particularly the units employed.

2. Given the formula of hydraulics $V = C\sqrt{RS}$, show algebraically that on the same grade a 6-inch pipe flowing half full has a less velocity than a 12-inch pipe half full. Express numerically if $C = 100$ and $S = 1$ foot in 100 feet.

3. Compute the velocities in a pipe if, in one case the pipe flows full, in another, half full, and in another one-quarter full. Assume a 12-inch pipe on a grade of 26 feet to the mile, and that $C = 100$.

4. Assume an area of one square mile on which there are 12,000 persons. If the rainfall is assumed to be at the rate of 2 inches per hour, how much water falls on the area in cubic feet per second? If the domestic sewage flow is at the rate of 100 gallons per head per day, what is the sewage flow? If one-third the rainfall reaches the sewer, what is the ratio of storm-water flow to house-sewage flow?

5. If a 12-inch pipe-sewer laid, costs 90 c. per running foot and brickwork for a 4-foot circular sewer costs $20 per thousand, and excavation in trench costs 60 c. per cubic yard, compare the cost of the 12-inch domestic sewer with the 4-foot storm-sewer, each 3000 feet long. Brick sewer requires 200 brick per lin. ft.

6. A pipe lateral system contains 18,000 feet of 8-inch pipe, 2000 feet of 10-inch pipe and 600 feet of 12-inch pipe. A small section containing 100 people is too low to drain into the system, and it is found that to serve this section the system must be lowered an average depth of 5 feet, the soil being a hard clay, in which excavation costs 75 cents per cubic yard. Compute the interest at 5 per cent on this additional amount and compare with the cost of pumping the sewage of 100 persons (80 gallons per head per day) 5 feet high, if the installation costs $650, and the running expenses are at the rate of 20 cents per million gallons lifted one foot.

CHAPTER II

PREPARATORY MAPS AND DATA

A PROPER treatment of the subject-matter of the last chapter, as well as of many other points to come up in the study of sewer design, will depend upon a thorough knowledge of the physical and topographical conditions affecting the work. It is therefore necessary to obtain certain information before a real consideration can be given to the design proper. The first requisite is a map on which to lay down the desired lines of sewers, locate the mains, and determine the possible position of the outfall or outfalls. This may be an old city map, provided it covers the ground required and is reasonably accurate, more inaccuracy in the map being tolerated if the grades are all good and the location of the outfall practically determined within a short distance. On the other hand, if the ground is generally level and the location of the outfall undetermined, the need for accuracy in the preliminary study of grades is correspondingly increased. This map should be drawn on a scale of from 200 to 400 feet to an inch; and the area covered should be that of the existing town or city, of the region where the outfall may be located, and of all territory which may ultimately be drained into the system; it should also include any high land which may furnish storm-water to drain into a system of storm-water sewers, should they be contemplated.* It is often easiest to collect these data by taking an old map which covers, perhaps, only a part of the territory desired and extending it where necessary by means of new surveys. The

* In the regulations of the New York State Department of Health, it is specified that the maps and the designs made, shall include all the territory within the boundary lines of the city, whether the proposed construction covers the entire area or not.

old map must always be regarded with suspicion, however, and its accuracy questioned until proved. Since it will in no case be possible to scale horizontal distances from such a map with sufficient exactness for the statement of the lengths of pipe required for the proposal of the bidders, it is better to chain the lengths of all the streets, and for the preliminary study of grades use the true distances. These distances can be recorded at the street intersections, as stations starting from the centre of some street assumed as zero, and then the profiles which are made will require no correction, and the first statement of quantities made will be true until the pipe is laid in the ground. This map will show (see Plate I) the location of the lines of pipe, their sizes and grades, and the location and character of the outfall. The sizes and grades are generally marked by figures, thus: $\frac{12}{.6}$,—indicating a 12-inch pipe on a .6 per cent grade, or a grade of .6 feet in a hundred feet. It has been suggested that the size of the pipes might be expressed by the thickness of broken lines, making the width of the lines in inches equal the diameter of the pipe in feet, and the length of the dash five times the diameter of the pipe, the spaces between the dashes being the length of the dash, all being drawn to the scale of the map. The advantage claimed is that on a map reduced by photography, or otherwise, the sizes can be read in this way when figures would be illegible. (Proc. Phila. Engrs., Vol. XII, p. 105).

If no plans are available, a survey must be made on which the preliminary studies can be based. A most economical and satisfactory method of collecting approximate data is described by J. H. Fuertes in the Transactions of the American Society of Civil Engineers * as practised by him in connection with investigations made of the New York water supplies and in a survey for a report on an improved sewerage system for Harrisburg, Pa. The method involves the use of two aneroid barometers with vertical scales in feet and adjusted

* Summarized in *Engineering Record*, Vol. XLIII, p. 349.

to read alike at the office at the beginning of each half day. One barometer remains in the office, where it is read at three-minute intervals with the times noted. The other is taken through the city by horse and buggy and readings taken at every street intersection, at intermediate points where the surface grade changes, at water levels, etc. The times of reading are also noted, while the distances are estimated from the number of revolutions of the buggy wheel. The office readings, when plotted, show graphically the variations due to changes in barometric pressure alone and give the corrections from the initial reading to be applied to the field readings. About 20 miles of street can be covered in a day and the elevations should check to within a few feet of the true elevations, the difference being too small to invalidate any preliminary plans or conclusions. Mr. Fuertes, referring to some topographical work done in this way outside of a certain city, gives the cost as $457 for field work and $136 for office mapping, a total of $593. The area covered was 9500 acres, so that the cost of the fielp work was at the rate of 4.8 cents per acre and the mapping 1.5 cents per acre, a total cost of 6.3 cents per acre. This served to provide a topographical map with contours at 20-foot intervals and on the scale used (one mile to the inch) made maps very similar to those of the United States Geological Survey.

It will be necessary to have another set of maps to show details of location not possible on the small-scale maps—details which are not needed until actual construction begins. This work is therefore usually carried along with the construction. These latter maps (see Plate II*), made on a scale of 40 to 60 feet to an inch, are plotted on separate sheets about 20×30 inches, as nearly as the size of paper at hand makes convenient. The paper should be of parchment or a similar thin paper from which blue-prints can be made; if this is not procurable, a medium weight of bond paper will serve the purpose. These large-scale sheets are of great use to the field-party, who, when engaged in staking out the line on the ground,

* Copied from *Engineering News*, Vol. XXXV. p. 2.

take from the office either blue-prints made from the parchment paper, or the bond maps or notes made from them. On these sheets, usually mapping two blocks, are plotted the street, curb, and gutter lines, trees, lamp-posts, hydrants, and catch-basins, the front and side lines of the houses and barns, the existing water- and gas-mains, and all old sewers. The profiles of the streets plotted just below on the same sheet show the street surface, with lines of intersecting streets, depth of rock, and position of all pipes and drains. Cellar-bottoms which might govern the depth of the sewer are plotted. These last are easily obtained by reading the level-rod on the house-sill outside and then adding the inside measurements of the cellar height.

The reasons for these requirements are self-evident. It is for the city's interest to have the sewer laid in another part of the street from the water or gas, first because thereby dangers of breakage during construction are lessened, and second because future repairs to any of the lines will be more easily accomplished in separate trenches. If the sewer has to be laid in a trench along the middle of which a water-main must be slung up, the work on the sewer is done at an additional cost to the contractor, who is likely to claim an " extra " for it, and with the chance of damage to both pipes. If the sewers are laid out regardless of the position of other pipe-lines, it still remains possible to move either line when they are found to lie in the same trench, but the cost of this re-trenching, once or twice repeated, more than covers the cost of a preliminary investigation.

Information concerning the location of the water- and gas-pipes can often be obtained from the companies' offices, but it is rarely accurate and is often only to be had through the good will of a foreman who has grown old on the works. The position of gas-drips and water-gates should always be located in the field and plotted as a check. The profiles showing the depth of the cross-pipes are of value in determining the depth of the sewer-pipes, and it is desirable to dig enough test-pits to make sure of critical points and of the main crossings. It rarely happens that the system of sewer-pipes will lie above the other pipes, and it is necessary, therefore, that they be designed to come enough below to allow at least 6 inches of dirt between the top of the sewer-pipe and the bottom of the water- or gas-pipe. It is awkward to find, just before joining in a lateral that is already laid, that the sewer-grade is of the same elevation as a 15-inch water-main, and that the sewer must go over or under and get back to the old grade within 50 feet. The old sewers and drains should be incorporated in the survey and mapped with care, since they may be made a part of the new system. They should be thoroughly examined and their condition, grade, and position personally noted and recorded. It may be that a small house-sewer can be laid inside of an old storm-sewer, saving the cost of re-excavation.

The data for these maps are generally only to be obtained by a survey, which in open and unimproved land may be made at the time of staking out, but under ordinary conditions should be done before, since the information is needed for the staking out. A convenient field-party for this survey is a transit man, two chainmen, and the chief of party, who acts as note-keeper. In one day such a party will, from actual experience, survey from 2000 to 6000 feet of street on both sides, taking plus distances of fence-lines and side lines of houses* (prolonging them by eye across the transit-line), measuring from the transit-line to curb- and street-lines, and pacing to the front lines of houses. The average distance run by such a party in the small city of Ithaca in the summer of 1895 was 4400 feet per day. This work was plotted by one man in six days, making cost of the work, mapped and plotted, about $18.50 per day, or $22.25 per mile.

Additional data as to the cost of surveys such as might be needed for work of this character are given as follows:

At St. Louis, where the entire cost of a careful survey of

* For detailed directions of approved surveying methods for locating buildings along a street, see *Engineering News*, Vol. LIV, pp. 173, 310, 380.

the city was $16,900, the different parts of the survey were divided up into triangulation, 11 per cent; precise levelling, 16 per cent; topography, 36 per cent; and office work, 37 per cent. The average cost *in toto* is given as $724.50 per square mile, or $1.13 per acre.*

In the Trans. Am. Soc. C.E., Vol. XXX, p. 611, are given a number of instances of the cost of topographical surveys in different parts of the world, most of them, however, covering larger areas and using other methods than those required for the survey of towns. A letter is quoted from Mr. J. C. Olmstead to the effect that for the purposes of landscape architecture the ordinary cost of suitable survey will range from $2.50 to $20 per acre, being generally about $5 per acre.

The following summary, Table I, is taken from an article† on the cost of survey of a 4000-acre tract near Chicago, and is as complete in all details as would be needed for any sewer-survey. The article gives an admirable description of the various elements entering the cost and their effect upon the accuracy and total cost of the survey.

In connection with the New York State Canals, extensive surveys were made in 1899, under the direction of the U. S. Deep Waterways Commission. Along the Mohawk Valley from Albany to Herkimer about 47,000 acres were surveyed and mapped to a scale of 200 feet to an inch, with 2-foot contours. The average cost was 86 cents per acre. A large amount of detail was included, especially at the cities and villages along the route.

Borings were made by driving a casing and washing out the material inside by forcing water down through a smaller interior pipe. The average cost of 55,521 lineal feet of penetration, about 2 per cent being in river bottom, and the average depth of hole being 29.5 feet, was 54 cents per vertical foot.‡

* Jour. Ass'n Eng. Soc., Vol. XII, p. 1.

† Trans. Ass'n Civ. Engrs. of Cornell University, 1898, p. 68.

‡ For detailed descriptions and costs of surveys in Croton Drainage Area, see *Engineering News*, Vol. LXII, p. 428.

In 1909, the New York State Water Supply Commission * made stadia surveys of three small areas of ground, the character of the work being similar to that required for preliminary design of sewers in outlying territory of large cities. The work included detailed topography, the location of all railroads, highways, buildings, fences, etc. The three separate areas were of 18.2, 56.1, and 21.8 square miles in extent, and the

TABLE I

SHOWING COSTS OF SURVEY NEAR CHICAGO

	Cost.		
	Total.	Per Acre.	Per Cent.
Superintendence	$200.24	0.051	8.2
Bases	385.02	.097	15.7
Bench-marks	62.52	.016	2.6
Transit-lines for locating contours	850.98	.216	34.8
Levels for contours	594.65	.151	24.3
Topography	139.31	.035	5.7
True meridian	6.90	.002	0.3
Soundings, etc	66.02	.017	2.7
Indexing notes	10.00	.002	0.4
Perpetuating survey	131.09	.033	5.3
	2446.73	0.620	100.0
Mapping 8 section maps	674.11	0.171	69.3
" 1 general map	180.07	.046	18.5
Incidental	118.86	.030	12.2
	937.04	0.247	100.0
Total	3419.77	0.867	

Or 87 cents per acre.

field work in each case cost almost exactly 75 per cent of the total cost. The work was done by stadia and was plotted in the field to a scale of 100 feet to the inch and then reduced in the office to the smaller scale of 400 feet to the inch.

The depth of rock and the character of the soil must be

* *Engineering-Contracting*, Vol. xxxiii, p. 419.

determined by borings or test-pits, the latter being preferable. Enough soundings should be taken to explore thoroughly the ground through which the sewer is to pass, since the location of the mains may depend on the character of the ground. If there are two possible locations for a deep main, one through rock and one through soil, the cheaper design will of course locate it in the soil as determined by the borings, and even a longer line may be cheaper to build on account of the character of the trenching. Contracts are now rarely let at a lump sum for the system, but rather at unit prices for the different kinds of work, so that rock found in unexpected places has to be paid for, and goes to make the work cost more than the engineer's estimate. Where water and quicksand are encountered, there has as yet been found no just way of paying for the extra work involved, and it must be covered by the percentage added by the contractor to cover such contingencies. It may be noted here that a cheaper and fairer method would be to pay the contractor directly, and just in proportion to the amount of this extra work. In laying out the best lines, the designing engineer should have the location of rock, quicksand, water-pockets, and soft clay in mind, to avoid them if possible, and get the maximum efficiency at the minimum cost. The proper attitude towards the contractor, also, is to give him all the information possible as to the nature of the work, in order to reduce the percentage added for unknown difficulties and to secure closer bids.

The examination for rock is most easily made by driving a bar or pipe, 1 to $1\frac{1}{2}$ inches in diameter, to refusal, although the method is open to the objection that a large boulder may be mistaken for the solid rock. Such a rod, driven by mauls and twisted by a wrench as it goes down, will easily penetrate 30 to 40 feet of soil or clay, and by the use of an open pipe a core may be brought up. A $\frac{3}{8}$-inch pipe will drive better (in 8-foot lengths the driving protected by a cap), but a 2-inch pipe will bring up the better core. If it is decided to thoroughly explore the ground, it is a simple and effective plan to rig up a small hand pile-driver, using a block of wood for the weight.

Fig. 1 (from *Engineering News*, Vol. XXIX, p. 242) shows a portable and economical pile-driver for such a purpose. The verticals are made of 2×4-inch stuff, and the hammer of a section of an oak or other hard-wood tree which may be growing

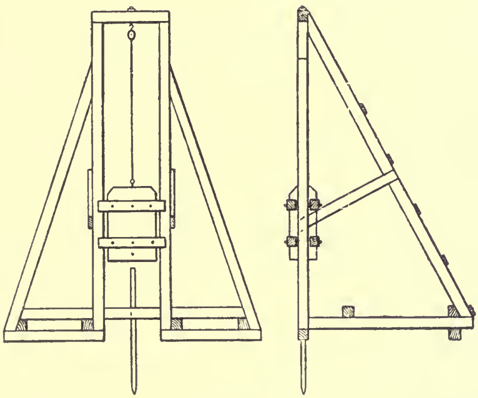


FIG. 1.

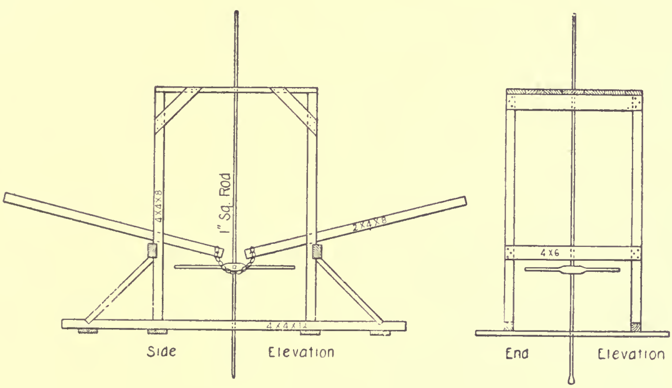


FIG. 2.

conveniently at hand. It may be run on wheels or slid on runners. To hold the uprights steady, snub-lines are provided. The hammer is worked by hand power, three or more men raising and lowering the weight.

Fig. 2 illustrates a test boring-machine described in *Engineer-*

ing News, Vol. XXI, p. 423. The cost is given as not more than \$25, and it is said to bore through earth of any kind to a depth of 28 to 30 feet. The drill-rod should be square, and the flare of the chisel-point about $\frac{3}{16}$ inch on each side. The iron cross-bar is made of bar iron, $1\frac{1}{4}$ inches square and about 4 feet long, with an eye for the drill-rod forged in. The cross-

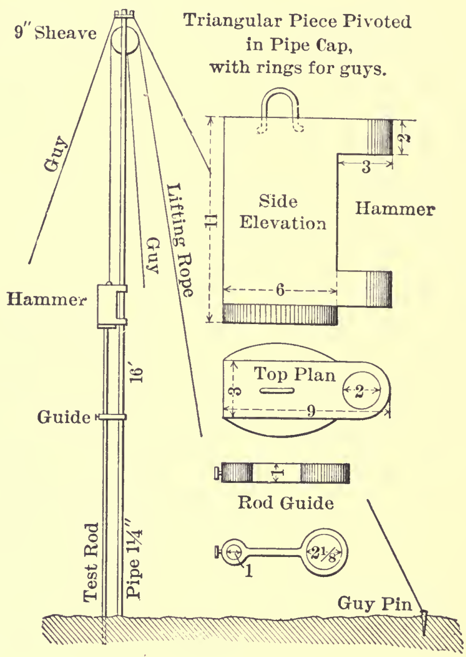


FIG. 3.

bar is held to the drill-rod by a set-screw $\frac{3}{4}$ inch diameter, and holes in the drill-rod allow the placing of a $\frac{3}{8}$-inch pin for the lifting-chain to bear against.

Fig. 3 shows in detail another form of driver (*Engineering News*, Vol. XXI, p. 484), the construction and arrangement being sufficiently well shown in the drawing.

With a common wood-auger $1\frac{1}{2}$ inches diameter, with extension-rods keyed on, and with levers 3 feet long, borings 50 to

100 feet deep can be very expeditiously made in common soil or clay. In addition, the auger will bring up samples of the material passed through in sufficient quantity to determine the nature of the soil. (*Baker.*) A post-hole auger in dry soils will reach depths of 10 to 12 feet and bring up the soil. A more satisfactory method in some respects is to follow the work of the engineers for the Rapid Transit Commission in New York City in sounding for rock on Broadway, which was as follows:

"Here two or three lengths of 2-inch pipe were driven first to serve as a casing. In order to drive this pipe a small portable pile-driver was used, the top of the pipe being covered with a protecting cap. The hammer, weighing 150 pounds, was directed between four light metal guides, and had a fall of about 6 feet, the whole arrangement being supported on a cast-iron stand. The hammer was raised by hand-power. After the casing had been put down, the protecting cap was removed and a tee screwed on in its place, and down the pipe was inserted a $\frac{3}{4}$-inch wash-pipe with a chisel-point, in the corners of which were two small holes. Water was forced into this wash-pipe while two men worked the pipe down by hand. The water thus discharged, washing the sand away from the foot of the wash-pipe, flowed upward between the wash-pipe and the casing, carrying the sand with it. This water and sand flowed out of the side opening on the tee at the top and was caught in a bucket and sampled by the inspector in charge."*

These borings were made at an average rate of 6 feet per hour, three laborers and an inspector being employed on each machine. The soil was sand and gravel, and about $\frac{2}{3}$ of each boring was cased.

Patton, in his treatise on Foundations, gives the following method as satisfactory: A 3- to 8-inch pipe of terra cotta or iron is pressed into the ground as far as possible; then a long narrow bucket with cutting-edge and a flap-valve a little distance above the cutting-edge, opening inwards, is lowered into

* Am. Soc. C. E., Vol. XXVIII, p. 13.

the pipe and is alternately raised and dropped. The material is collected in the bucket, and at intervals the bucket is lifted entirely out and emptied. This is repeated; the pipe gradually sinks, a man standing on the top if necessary. Other sections of the pipe are added from time to time. It becomes necessary sometimes to pour water into the pipe to aid in the cutting and flow of the material into the bucket. The bucket should be connected by a rope passing over a sheave connected with a frame or shears above. Great depths can be reached by this method with reasonable rapidity and at no great cost.

Levels should be run and frequent benches established and checked along all the streets. For the preliminary study on the large map the levels are best expressed as contours showing on flat ground differences of 1 foot. The profiles on the separate sheets will require a vertical scale different from the horizontal, depending on the grade of the street, and it is better to hold to one vertical scale through all, rather than change it for each sheet. Ten feet to the inch will generally serve, though 4 feet to the inch is not too large for flat country. Levels should be read at the bottom, and at the surfaces of all creeks or brooks crossed by the sewer. Such points may serve for outfalls or for flushing-gates, so that the high- and low-water elevations should be found if possible.

Of late years, particularly through the experience of the U. S. Geological Survey, much data has been collected on the value of various methods of running levels, particularly as those methods affect accuracy and cost. On the long aqueduct lines now under construction, as for Los Angeles and for New York City, similar valuable data has been acquired.*

The most common practice is to use two rodmen and to read more than one wire on the level rod. On the Los Angeles work the average number of miles run per day was 5.2 and the cost including that involved in establishing frequent benchmarks was $13.20 per mile. On the Catskill Aqueduct work, with the same force, but with the addition of a note-keeper,

* See *Engineering News*, Vol. LX, p. 311, and Vol. LIX, p. 186.

the rate was from two to four miles per day, with bench-marks established every quarter mile.

These sheet-maps may be indexed on the large map, numbering the sheets to correspond with the numbers on the map; or a separate index-map may be drawn on one of the sheets and bound up with the others, in sections if need be.

It is interesting to note that the directions offered above, which have been developed from the general practice of this country, agree in scales, etc., with the instructions for similar work issued by the Local Government Board of England (see Rawlinson's Suggestions, 1878).*

PROBLEMS

7. If 5-foot contours on a certain map drawn to a scale of 300 feet to the inch are $\frac{1}{4}$ inch apart, what is the surface slope in feet per mile? In per cent?

8. With two barometers, make a survey of an area of about 25 acres, plotting 2-foot contours. Select a part of the city that has marked differences in elevation and for which the street plan is available.

10. With chain or tape only, make a survey of a portion of some assigned street (about 600 lineal feet). Record the position of all street-lines, trees, posts, fences, including back lines of lots, houses and barns, with all surfaces indications of underground pipes. Plot to a scale of 40 feet to the inch. Compute the cost of the survey in units of acres per day.

11. Compare the cost of making a survey of the city of ......, using the data of this chapter and checking by the results of Prob. 10.

12. Compare the cost of digging (no sheeting) test-pits, 3 feet wide by 4 feet long to average depths of 12 feet, with the cost of wash-borings as given in this chapter.

13. With a given city map, estimate the cost of running levels and establishing bench-marks for future work, assuming necessary salaries for different members of the party.

* See also *Municipal Engineering,* Vol. XXI p. 161.

RULES AND REGULATIONS FOR THE PREPARATION AND SUBMISSION OF PLANS FOR SEWERAGE SYSTEMS AND SEWAGE-DISPOSAL WORKS

As Enacted by the State Dept. of Health of New York State, 1912

1. General Plans.—General plans on a scale of not less than 300 feet to one inch, and preferably not greater than 100 feet to one inch, covering the entire area of the municipality, must accompany every application in the case of a new sewer system, or any extension or modification of any existing sewer system, unless such a general plan of the entire area of the municipality has already been submitted. These plans must have shown upon them all existing and proposed streets, the surface elevation at all street intersections and at all points where changes of grade occur, and contour lines for intervals of not less than 5 nor more than 10 feet. The plans must also show sewers upon *all streets* in the municipality or the sewerage district, even if the construction of some of the sewers may be deferred. Should there be areas, which on account of the topography, or for other reasons, cannot drain into the proposed system, a definite statement to this effect must be made in the engineer's report and the probable future drainage of this disregarded territory discussed. The plans must also show clearly the location of all existing "sanitary" and "combined" sewers, but not of drains used *exclusively* for sub-soils or for surface-water; the location and general arrangement of existing and proposed sewage-disposal works, and the location of all existing and proposed outlets. The magnetic meridian, title and date, and the direction of flow and mean-water elevation of the principal streams must also be clearly shown.

2. Lettering, Figures and Symbols.—The lettering and figures must be of appropriate size, and of distinct outline. Surface elevations should preferably be placed just outside street lines opposite their respective positions, and at street corners, preferably in the angle outside street line, in the upper right angle if but one, and in the other angles if mote than one. The elevations of all sewer inverts must be shown at street intersections, at the ends of all lines, and at all points where changes in alignment and grade occur. These elevations must be clearly and distinctly written close to the manhole or flush-tank, parallel with sewer-line, and expressed at least to the nearest $1\frac{1}{10}$ foot. All manholes, flush-tanks, catch-basins, lampholes and other appurtenances must be shown upon the plans in suitable symbols appropriately "referenced" in the title. The sizes and gradients of all proposed sewers, and of existing sewers, must be marked appropriately along sewer-lines between all consecutive manholes or flush-tanks, with arrows showing the direction

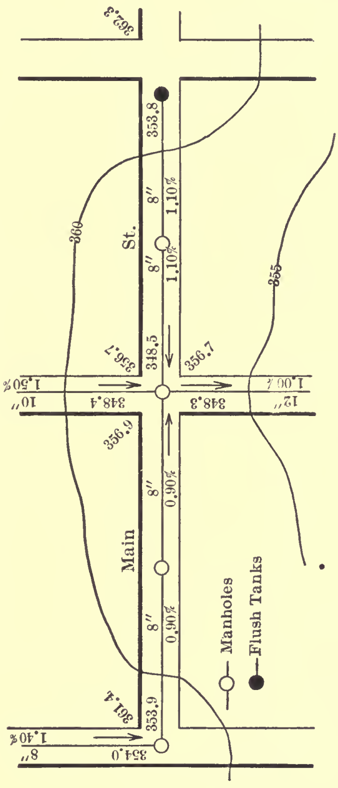


FIG. 4.

of flow. All sewers and other appurtenances must be shown by *black* lines and conventional signs, and not by *colors*. As an example of the lettering to be used at a street intersection, the following sketch is offered.*

3. Profiles.—Profiles upon separate sheets, showing all sewers 12 inches or more in diameter, and all main intercepting and outfall sewers

* Plate loaned by Theodore Horton, Chief Engineer, N. Y. State Dept. of Health.

when less than 12 inches in diameter, and all other available profiles, must be submitted with the plans. Upon these profiles must be shown all manholes, flush-tanks, inverted siphons and other appurtenances. The horizontal scale of these profiles must be at least as great as the scale of the corresponding plans, and the vertical scales not less than 10 feet to 1 inch. Both scales must be clearly shown upon the profiles. Figures showing the sizes and gradients of sewers, the surface elevations and sewer inverts, must be shown upon the profiles with the same frequency, or at the same points, as shown upon the plans. All stream crossings and sewer outlets must be shown upon profiles with elevations of stream-bed, and the normal, high, and low water levels if these data are available.

4. Detail Plans.—(*a*) Detailed plans of sewer sections and of all ordinary sewer appurtenances, such as manholes, flush-tanks, inspection-chambers, inverted siphons, as well as of any special appurtenance or structure, must accompany general sewer plans. These detail plans must be drawn to such a scale as to show suitably and clearly the nature of the design and all details, such as manhole-frames and covers, iron pipes and valves, flushing-gates, siphons, etc. They should have marked upon them all dimensions, grades and explanatory notes necessary to make them readily intelligible and a complete guide for construction.

(*b*) Complete detailed plans for sewage-disposal works must be submitted in all cases where it is proposed to construct works for complete purification at the time of the construction of the sewarage system. If, however, it is proposed to construct only a portion of the complete works, at this time, detailed plans of such portions only need by submitted. In the latter case future provision must always be made for complete purification works, and a *reserve area* must be shown upon the general plans for these works, and, if possible, a statement of the general type or method which it is proposed to adopt when complete purification works may be required.

5. Specifications, Estimates of Cost.—Specifications for the construction of the system of sewers and sewage-disposal works, including estimates of cost of the same, where these have been made, must accompany all plans for original or new systems. With plans for extensions of existing systems, specifications may be omitted, provided these extensions are to be constructed in accordance with specifications filed previously with original plans.

6. Engineer's Report.—A report, which would usually be written by the designing engineer, must be presented with all plans for original systems, giving full information upon which the design is based. This report must include a description of the extent of area which it is proposed to include within the system at the present time and in the future; the estimated present and future population to be served; the estimated per capita rates or volume of sewage to be provided for; the allowance, if any, for storm- or roof-water, and the full reasons for the inclusion of such water in the system. The report should include a description of all conditions peculiar to the locality, affecting in any way the design of the system; a description of all special devices used in the design and of any special points to be observed in the maintenance and operation of the system. The report must contain a full description of the general arrangement and all special features of the proposed sewage-disposal works, the reasons for the choice of the method or type proposed and a full description of the proposed operation of the plant. A full statement must be given of the capacities of the various parts of the works, the population which works are designed to serve and the reasons for any unusual capacities adopted.

7. General Requirements.—(*a*) All plans, specifications and reports must be submitted in duplicate, and the application for approval must be submitted on blanks issued by the Department for the purpose and signed by the proper municipal authorities or their properly authorized agent (in the latter case a letter of authorization must accompany the application). If approved, the original set will in general be returned and the duplicate set, if clear and distinct prints, on suitable paper or cloth, will be filed with the Department according to law.

(*b*) In general, and except in certain restricted districts and for very cogent reasons, the Department will approve of plans for new systems only when designed upon the "separate" plan, in which all rain-water from roofs, streets and other areas, and all ground-water other than unavoidable leakage and a very restricted allowance for cellar drainage, shall be excluded. When plans for the extension of "combined" sewers, already built and in operation in full accordance with the provisions of the Health Law, are submitted, approval will in general be given only in cases where the district tributary to the sewer extension is of limited area and cannot be included in a new and distinct sewer district construction upon the" separate" plan.

(*c*) The Department will in general approve only of plans which include disposal works for such complete purification as will produce a clear and "stable" effluent. If it is proposed to omit certain portions of these works for the complete purification of the sewage, there must be shown upon the plan a reserve area and the general arrangement of the type, methods and devices which it is proposed to instal in the future when such complete works are required; and included in the engineer's report the full reasons why any of such portions of the complete works are temtemporarily omitted.

(*d*) The approval of the Department is not required by law for drains designed and used *exclusively* for storm-water, subsoil-water and for other purposes of drainage where no sewage or wastes are allowed to enter them. If any sewage or other waste matter or products are discharged into such drains, they come under the definition and classification of sewers, for which the approval of the Department, and a permit therefrom, must be secured in accordance with the Public Health Law and the above regulations.

CHAPTER III

EXCESSIVE RAINS

If storm-water drains are to be constructed, it becomes necessary to determine, as closely as possible, the amount of storm-water likely to enter the sewer. Evidently it will be due to two influencing conditions: the actual amount of rain falling in a given time, and the proportion of that amount reaching and carried off by the sewer. Only recently has careful observation been brought to bear on these points, and even now only an approximate estimate is possible, as the conditions are continually changing.

It should here be pointed out that the run-off for sewers differs in this respect from the run-off for storage and for power purposes. As long-time stream-gagings become more available, there is a growing tendency to disregard altogether studies of the rainfall over the watershed, as being valueless if not misleading. Thus Clemens Herschel has declared * that in forming a judgment as to the discharge of a river, a knowledge of rainfall is of no importance and that therefore rainfall records should cease to have the attention given them as in the illogical reports on stream flow of the past.

Mr. J. C. Hoyt, of the U. S. Geological Survey, also says † that it is difficult to understand how engineers can continue to estimate run-off from rainfall data.

For maximum rates of run-off, however, a knowledge of maximum rates of rainfall is essential and the local conditions known before any indication of the probable run-off is obtained.

It may also be asserted that, while the annual rainfall is an interesting meteorological study and while engineers

* Trans. Am. Soc. C. E., Vol. LVIII, p. 30.
† Trans. Am. Soc. C. E., Vol. LVIII, p. 34.

interested in water-power and water-storage will continue to refer to the amount of water falling per year as a basis for their studies, such facts have no bearing on the needed capacity of sewers. The fact that in New England the annual rainfall is about 40 vertical inches and in Kansas only 20, does not mean that the sewers in the former area must all be twice the capacity of those in the latter. Nor is it necessary, because the rainfall in Washington is 120 inches per year to build sewers there three times as large as in New England. There is then a difference between the amount of rain falling per year and some other characteristic that determines the necessary size of a sewer.

In this country the first extended study of the subject was made by Col. J. W. Adams in designing the early sewers for Brooklyn. He noted the fact, since emphasized by A. J. Henry in a special report of the Weather Bureau, that excessive rains, or those that do damage, are naturally divided into two broad classes: (*a*) rains of great intensity and short duration, and (*b*) rains of light intensity and long duration; and that of the two classes, the first are far more damaging and destructive. Col. Adams, after consulting all the meagre rainfall records available, chiefly those of 1849 to 1856, and noting that there were but 19 days in which the rainfall in 4 hours was an inch or over, and but 15 days in which the rainfall for the entire 24 hours was as much as 2 inches, that the heaviest storms reported were two of $2\frac{1}{2}$ inches in 4 hours, and that there was no reported occurrence of as much as 1 inch within the hour, concluded that if he made provision in his Brooklyn sewers to carry off a rainfall of 1 inch per hour it would be sufficient.

The two reports on the sewerage of Providence, one by J. H. Shedd, published in 1874, and one by S. M. Gray, published in 1884, represent the next advance in the study of the question of rainfall. Mr. Shedd noted that of 185 storms recorded for the 26 years before 1860 only 20 were at a rate of over $\frac{1}{2}$ inch per hour, while 165 were of less, and that of 139 storms recorded in the 14 years, 1861–1875, 20 were over

½ inch, and 119 were less. Mr. Gray pointed out that great care must be taken to determine the exact duration of the storm, and also of the heavy showers that may fall during a long rain, and that meteorological records are to be used only with great caution. He explained that the records, as generally made, can seldom be depended on for the rates of fall, since as a rule they give only the total amount of rain falling at certain times, paying little heed to the exact time when the storms begin or end; that is, the records fail to distinguish between a fall of 1 inch within the hour, however short the actual duration of the storm, and another which continues at a constant rate for the whole hour. Mr. Gray, however, gave no precise data as to the proper amount to be considered in the case of the Providence sewers.

With the demand for more knowledge, aroused in great part by the work of a few engineers, came more data from different parts of the country to which the engineers of the Boston Water Board contributed largely. It was soon found that the early records were not entirely trustworthy, that the location of the gage had not been well considered, and that the rate of fall could not be derived with any exactness from published records either public or private.

In 1888 the U. S. Weather Bureau began reporting excessive rains, i.e., rains of 2.50 inches or more in twenty-four hours and of 1 inch or more in one hour, but from the nature of the observations it is rarely known, in the case of rains of an inch or more in an hour, whether the rain was of an even intensity for the whole period, or whether most of it fell in a small fraction of the time. These records, with such value as they possess, are now available, as noted at the Weather Bureau stations through the United States (see *Monthly Weather Review*).

By a study of these figures it is seen that rainfalls of the rate of an inch per hour, assumed by Col. Adams and Mr. Shedd to be rare, are by no means infrequent. It is now proved that such storms occur several times a year, instead of once in several years as was thought to be the case; also, that rains of a much heavier rate occur, lasting from ten to forty minutes. For example, in 1890 there were reported in New York State eleven storms contributing over an inch in an hour, and in Massachusetts six.*

In the spring of 1889 five self-registering rain-gages were stationed throughout the country. This number has since been increased to over two hundred, and there are now published in the *Monthly Weather Review* tables of maximum rainfall in five-, ten-, and sixty-minute intervals, giving valuable data for all parts of the country.

The special bulletin " D " of the Weather Bureau for 1897 deals largely with this question of excessive rains. This bulletin, issued in direct response to the request of a number of civil and municipal engineers, gives the maximum intensities at Weather Bureau stations equipped with self-registering rain-gages. Its accompanying text is, in part, as follows:

" Excessive rains of high intensity are not prevalent on the Western coast, although there the total annual rainfall is greater than in any other portion of the United States. In the Western States are found the most violent rains of this class, that is, the cloudbursts of the mountainous and arid regions. The rain seems to pour down rather than to fall in drops. The amount of water falling has never been ascertained. In August, 1890, a storm passed over Palmetto, Nev., and contributed to a rain-gage, not exposed to the full intensity of the storm, 8.8 inches in an hour. In August, 1891, two storms passed Campo, Col., within a few moments of each other, and the gage, before being carried away by the storm, showed a fall of 11.5 inches during the hour. But these downpours are found only between the Sierras and the foothills of the Rockies; while the common heavy rainfalls are found east of the 105th meridian, and principally during the summer months. They are most frequent in connection with summer-afternoon thunder-storms, but occasionally occur in the track of the West Indian

* *Weather Review* for 1890.

Table II

HIGHEST RATE PER HOUR OF RAINSTORMS OCCURRING AT WASHINGTON, D. C., DURING THE PAST 16 YEARS

For any	Inches.	Date.
5 consecutive minutes	7.50	Sept. 3, 1882
10 " "	5.10	Sept. 16, 1888
15 " "	4.50	June 27, 1881
20 " "	3.90	June 27, 1881
25 " "	3.60	June 27, 1881
30 " "	3.15	June 27, 1881
40 " "	2.75	June 10, 1876
50 " "	2.30	June 10, 1876
60 " "	1.98	June 10, 1876
120 " "	1.23	July 26, 1886

Table III

MAXIMUM INTENSITY OF RAINFALL FOR PERIODS OF 5, 10, AND 60 MINUTES AT WEATHER BUREAU STATIONS EQUIPPED WITH SELF-REGISTERING GAGES, COMPILED FROM ALL AVAILABLE RECORDS.

	5 Minutes	10 Minutes.	60 Minutes.
Bismarck	9.00	6.00	2.00
St. Paul	8.40	6.00	1.30
New Orleans	8.16	4.86	2.18
Milwaukee	7.80	4.20	1.25
Kansas City	7.80	6.60	2.40
Washington	7.50	5.10	1.78
Jacksonville	7.44	7.08	2.20
Detroit	7.20	6.00	2.15
New York City	7.20	4.92	1.60
Boston	6.72	4.98	1.68
Savannah	6.60	6.00	2.21
Indianapolis	6.60	3.90	1.60
Memphis	6.60	4.80	1.86
Chicago	6.60	5.92	1.60
Galveston	6.48	5.58	2.55
Omaha	6.00	4.80	1.55
Dodge City	6.00	4.20	1.34
Norfolk	5.76	5.46	1.55
Cleveland	5.64	3.66	1.12
Atlanta	5.46	5.46	1.50
Key West	5.40	4.80	2.25
Philadelphia	5.40	4.02	1.50
St. Louis	4.80	3.84	2.25
Cincinnati	4.56	4.20	1.70
Denver	3.60	3.30	1.18
Duluth	3.60	2.40	1.35

hurricanes. They are more abundant on the Gulf and South Atlantic coasts than at inland points."

This report shows for Washington, D. C., 73 storms raining at the rate of 1 inch per hour or over in fifteen years before January 1, 1897. For Savannah, 62 in eight years; for St. Louis, 36 in the same time. To show that the intensity becomes a maximum as the time of the storm becomes less, the table opposite is given.

Table III from the same Bulletin shows, in another form, the same thing. It gives the maximum intensity of rainfall for periods of five, ten, and sixty minutes at Weather Bureau stations equipped with self-registering gages, and is compiled from all available sources.

Table IV, abridged from a paper by C. W. Sherman * on "Maximum Rates of Rainfall at Boston," shows again the relation between the intensity and length of the storm.

TABLE IV

MAXIMUM RATES OF RAINFALL, AUTHENTICALLY REPORTED FOR THE EASTERN UNITED STATES

Duration in minutes	Rate, inches per hour	Place	Date	Reported by
4	8.40	Philadelphia....	Aug. 3, 1898	Webster
5	8.16	"	Aug. 3, 1898	Webster
5	8.50	"	Sept. 14, 1904	Webster
5	8.40	Boston.........	July 18, 1884	Sherman
15	9.20	Embarrass, Wis.	May , 1881	Allen
15	9.00	Sandusky, O....	July, , 1879	Allen
20	6.78	Brattleboro, Vt..	July 7, 1897	Allen
25	6.00	Kansas City, Mo.	May 12, 1886	Allen
25	5.76	Indianapolis, Ind.	July, 1876	Allen
30	5.60	....	....	Hoxie
37	5.80	....	....	Talbot
60	4.50	....	....	Talbot
70	3.78	....	....	Talbot
75	4.02	....	....	Nipher
110	2.95	Philadelphia....	Aug. 3, 1898	Henry
120	3.00	....	....	Hoxie

Heavy rains at the Chestnut Hill Reservoir in Boston

* Trans. Am. Soc. C. E., Vol. LIV, p. 212.

have been reported as follows, the figures given showing the intensity in inches per hour:*

Date.	Duration.		
	20 Min.	30 Min.	60 Min.
August, 1899	4.80	3.40	2.10
August, 1892	2.55	2.54	1.60
July, 1880	3.06	2.34	1.33
July, 1894	3.24	2.20	1.13

From St. Louis, Mo., the following data † are reported by W. W. Horner, Asst. Engr., St. Louis Sewer Department, the figures again expressing intensity in inches per hour:

Date.	Duration.		
	10 Min.	30 Min.	60 Min.
1897	5.58		
1898	6.00		
1900	5.40	....	2.85
1900	5.22	2.56	1.49
1902	....	2.81	
1903	....	2.02	1.40
1907	5.04	2.80	1.59
1908	....	2.18	
1909	....	....	1.37

A. J. Mitchell, of the U. S. Weather Bureau Station at Jacksonville, Fla., reports ‡ the following high rates in the several cities mentioned, in addition to those already quoted by Mr. Sherman in Table IV:

Location.	Date.	Amt. of Rain.	Duration.	Intensity in Inches per Hour.
Washington, D. C.	June, 1881	2.34 inches	37 min.	3.78
Biscayne Bay, Fla.	March, 1874	4.1 "	30 "	8.2
Newton, Pa.	Aug., 1843	5.5 "	40 "	8.25
Galveston, Tex.	June, 1871	3.95 "	14 "	16.8
Concordia, Pa.	Aug., 1843	16.0 "	3 "	32.0

* *Engineering Record*, Vol. XLII, p. 29.
† *Engineering News*, Vol. 64, p. 329.
‡ *Engineering Record*, Vol. XLVII, p. 539.

Mr. Mitchell very properly points out that while the occurrence of very heavy rains lasting a few moments only is interesting, such rains have but little practical significance, both because they occur at such infrequent intervals and because they affect so limited an area. He also recalls that such extraordinary occurrences are considered by the courts before whom suits for damage have come * as beyond the need for consideration in engineering work, being thus classed with earthquakes, fires, strikes and other calamities impossible to foresee.

Mr. J. N. Hazlehurst, in studying the question of excessive rains for the purpose of designing sewers for Mobile, Ala., found † that the maximum intensity for a storm lasting for thirty minutes was 3.1 inches, for sixty minutes, 2.7 inches and for 120 minutes 1.5 inches. He concluded, however, that such storms were so infrequent as to be not worth considering and he assumed an intensity of 1.5 inches per hour as a practical maximum to be cared for in designing the sewers.

The first extended and detailed study of the excessive storms for a single locality was made in 1889 by Emil Kuichling, C. E., who included in his elaborate report to the city of Rochester on the East Side Sewer a discussion of the probable rainfall, and the amount of storm-water to be expected. His work, based on the records of the Weather Bureau at Rochester, Oswego, and Buffalo, and on other records kept at Cornell University, Mt. Hope Reservoir, Hemlock Lake, and by two special employes of the city, emphasizes the fact, as just given, that the rate of rainfall varies with the unit of time chosen for the rain-measurement, and that for the greatest intensities a shorter period than an hour must be chosen for a unit. He also points out that the area covered by a storm is of limited extent, and that the heavier the rate of fall, the less the area affected. From his observations, however, he finds that generally the clouds which furnish the rainfalls of large rate extend

* 133 N. W. Rep. p. 835; 32 N. Y. p. 489.

† *Engineering News*, Vol. XLVI, p. 26.

farther than any single drainage-area within ordinary municipal limits.

The relation between the intensity of a rain and the duration of that intensity, shown by Table III above, was brought out by Kuichling very clearly, by means of which he finds a method of determining the duration of any rain of a given assumed intensity. A similar method is generally applicable. The exact relation is *unreliable*, as it varies in different localities, and, the data being uncertain, it is probable that for some time to come conclusions will be only approximate. The method outlined is, however, the best available for gaining this first step in determining the amount of rain to be considered in the sewer design.

The method may be reduced to the following: First collect all the rainfall statistics that are available for the city in question and for any other places that are in the same locality and under the same meteorological conditions. Unfortunately such data are usually defective in accuracy and in the time covered, but no other method will ever give as good results as a study of past records. With all the available data at hand, compute the intensities of all rainfalls whose rate of fall is greater than $\frac{1}{2}$ inch per hour, regardless of the duration of the storm, and for every recorded storm, plot a point on cross-section paper with the intensity as ordinate and the duration of the storm as abscissa. A number of points, each corresponding to a storm, are thus obtained. The rainfalls of low intensities are, of course, most frequent, so that that part of the diagram will be well studded with points; but the isolated points representing the heavier rains will usually be sufficient in number to show that the shorter rains and heavier intensities correspond, and that there is some proportionate relation between the two. By joining the points by a series of broken lines, selecting those points which represent the greatest recognized intensities for that time, an irregular envelope is found, the ordinates of which give the probable maximum intensities for that locality for the corresponding period of time. This envelope is only located with judgment, and it may be necessary to omit two or three uncommon and rarely severe storms.

Several years ago Prof. A. N. Talbot of the University of Illinois made use of the U. S. Weather Bureau records for the group of States considered, in making a study of the maximum

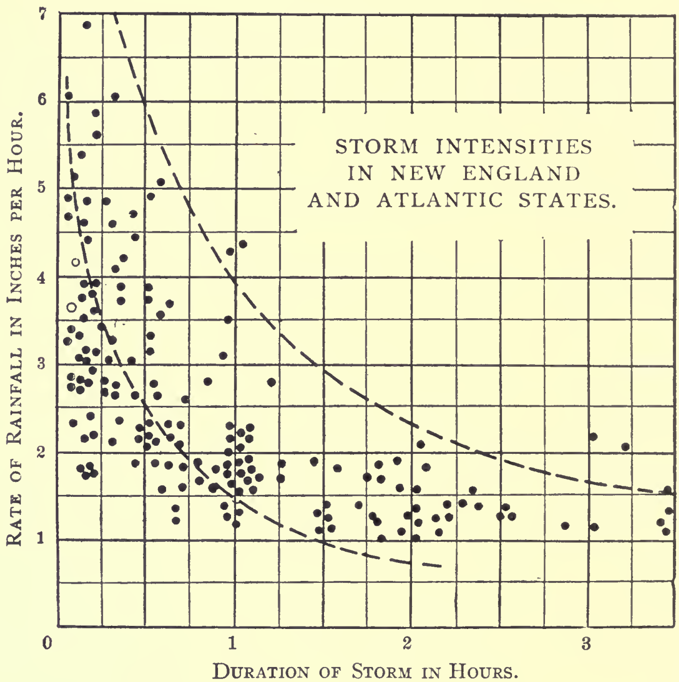


FIG. 5.

intensity of storms, and published his results in the *Technograph.** The records of these stations range from 1 to 50 years and include those from 499 stations. After the storms were plotted as indicated above (see Fig. 5), two enveloping curves were drawn, one giving what might be called the very rare rainfalls, and the other the ordinary maximum. The curves drawn were in

* *Technograph*, 1891–1892, pp. 103–117.

both cases rectangular hyperbolas. After drawing the two curves their equations were determined to be

$$y=\frac{360}{x+30}$$

for the curve of rare occurrence, and

$$y=\frac{105}{x+15}$$

for the rains of frequent occurrence, where y is the rate of rainfall in inches per hour, and x is the duration of the storm in minutes. The two curves give the following comparisons, which Prof. Talbot says are found to hold pretty generally throughout the country in spite of great differences in the total annual rainfall.

10	minutes'	duration,	9.0 or 4.2	inches rate
20	"	"	7.2 or 3.0	"
30	"	"	6.0 or 2.3	"
45	"	"	4.8 or 1.7	"
60	"	"	4.0 or 1.4	"

In checking his two curves it was noted that they were drawn so that the rainfall shown by the upper curve of maximum rain would be exceeded once in 83, 107, 100, and 91 years for the North Atlantic, South Atlantic, Gulf, and North Central States, respectively; and that the other curve would be exceeded once in 3.6, 3.1, 3.8, and 3.7 years for the same group of States, respectively.

In commenting on the individual city records, Prof. Talbot says: " In summarizing the data of seventy-one years of rainfall of self-recording gages shown on these diagrams, it may be noted that the curve of rare rainfall has not been reached in a single instance, and that the curve of ordinary maximum rate of rainfall for periods of less than forty minutes has not, with one exception, been exceeded in any marked degree. It is further probable that in each of these cities storms giving rates of rainfall for any length of storm up to forty minutes will reach the values given by the curve $y=\frac{105}{x+15}$ at least as frequently as two or three times in ten years."

In St. Louis Prof. Nipher has used the same method to

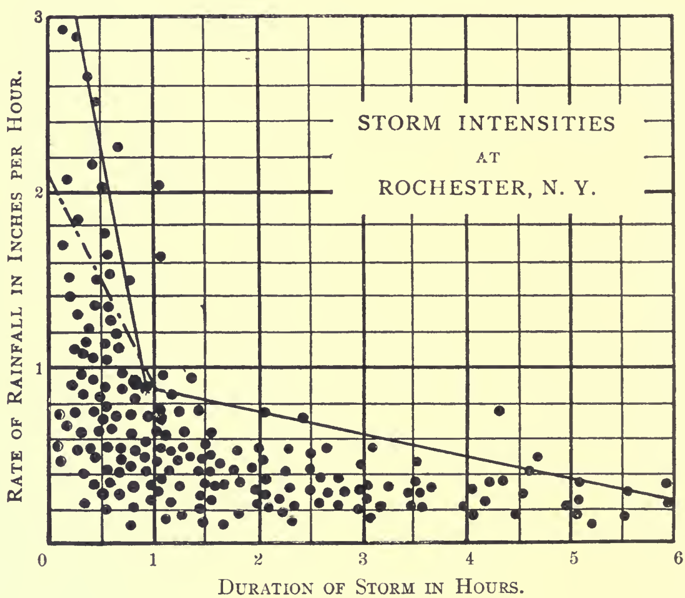


FIG. 6.

determine the probable intensity for a given duration, and has plotted the storms of that city in the manner indicated above. He also assumed an equilateral hyperbola as the enveloping curve, and determined its equation to be $yx=360$. This was made up from the rainfall records of St. Louis extending over a period of forty-seven years.

Kuichling plotted (see Fig. 6) the local data of Rochester combined with that of two neighboring stations, and used two straight lines as the envelope of the points instead of the hyperbola used formerly.* These two lines meet at a point of the diagram representing a duration of one hour and an intensity of .87 inch. The line to the left of the point of intersection is quite steep, while that on the right is more nearly horizontal. The combination shows very clearly that the maximum intensity of the rainfall diminishes rapidly as the duration increases from a few minutes to an hour, and that for rains of uniform intensity lasting more than one hour the rate of diminution is quite

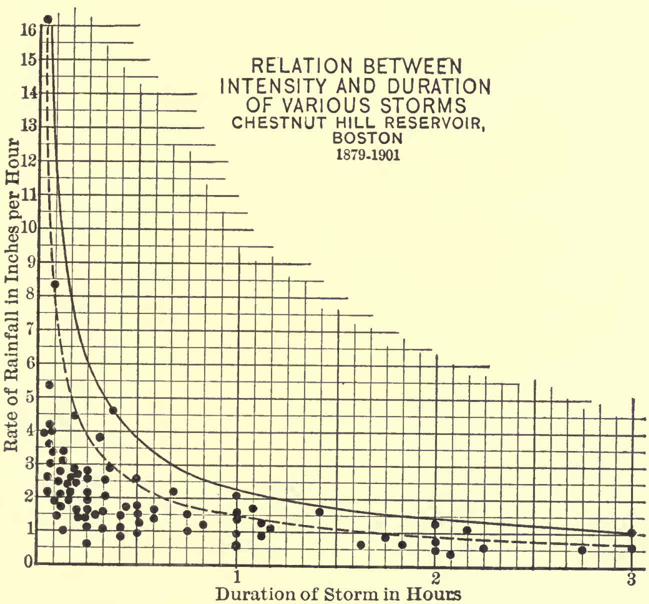


FIG. 7.

slow. By getting the equations of the two enveloping lines, he has for storms of less than one hour $y=3.73-0.0506x$, and for those over one and less than five hours the relation $y=0.99-0.002x$. Repeating the work for Rochester alone, he gets $y=2.10-0.0205x$. Kuichling distinctly states that no great accuracy can be claimed for this formula, nor can he recommend it for general use, despite its great value in the

* See Trans. Am. Soc. C. E., Vol. XX, p. 1.

connection for which it was made. "It is," he says, "merely an attempt to utilize the available data as to the local rainfall

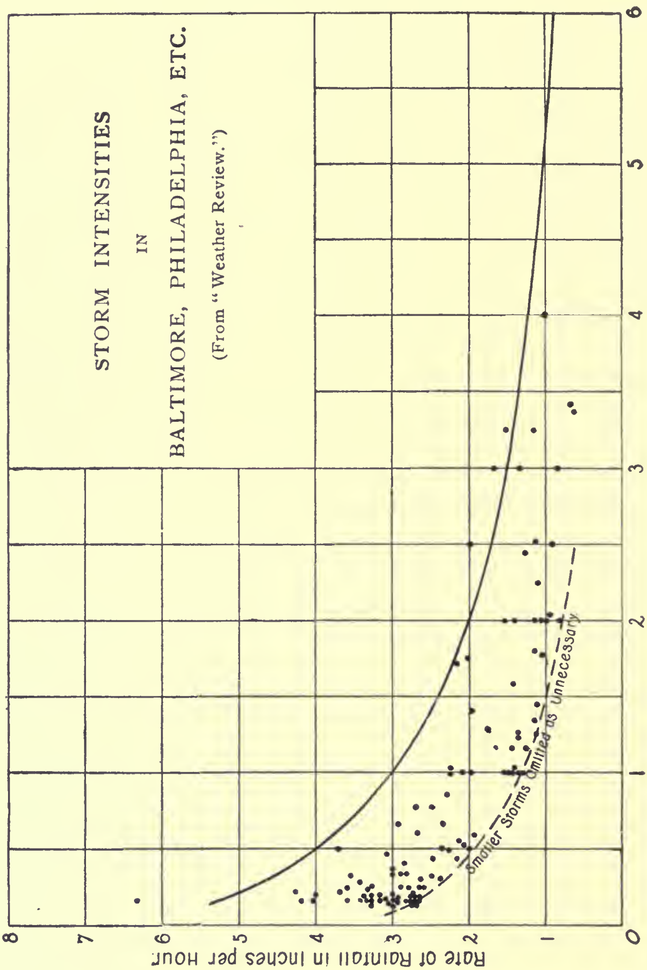


Fig. 8.

in a rational manner, and to remove the subject of urban sewerage from the realm of vague conjecture."

Fig. 7 shows a diagram from the paper * by C. W. Sherman

* Trans. Am. Soc. C. E., Vol. LIV, p. 176.

already referred to, based on rainfall records at Chestnut Hill Reservoir at Boston. His equations, in logarithmic form, are $y=\frac{38.64}{x^{0.687}}$, for rains of maximum intensity for any period, and $y=\frac{25.12}{x^{0.687}}$ for rains giving the greatest intensity of pre-

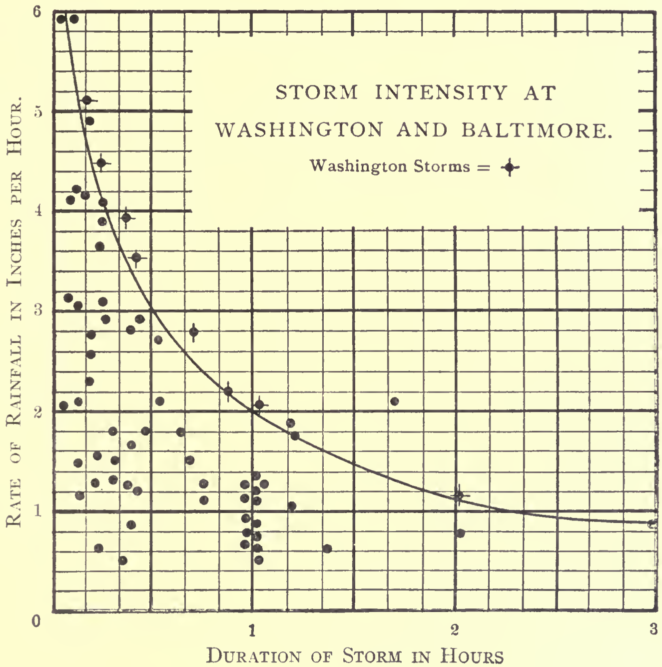


FIG. 9.

cipitation which it would ordinarily be necessary to consider in engineering design.

Figs. 8 and 9 show similar diagrams from the report of the Sewerage Commission of the City of Baltimore, 1897.

Fig. 10, prepared * by Mr. George S. Webster, Chief Engineer,

* Trans. Am. Soc. C. E., Vol. LIV, p. 210.

Bureau of Surveys, Philadelphia, shows three curves, the upper one for the storms of maximum intensity observed for the nine years for which self-registering rain-gages made

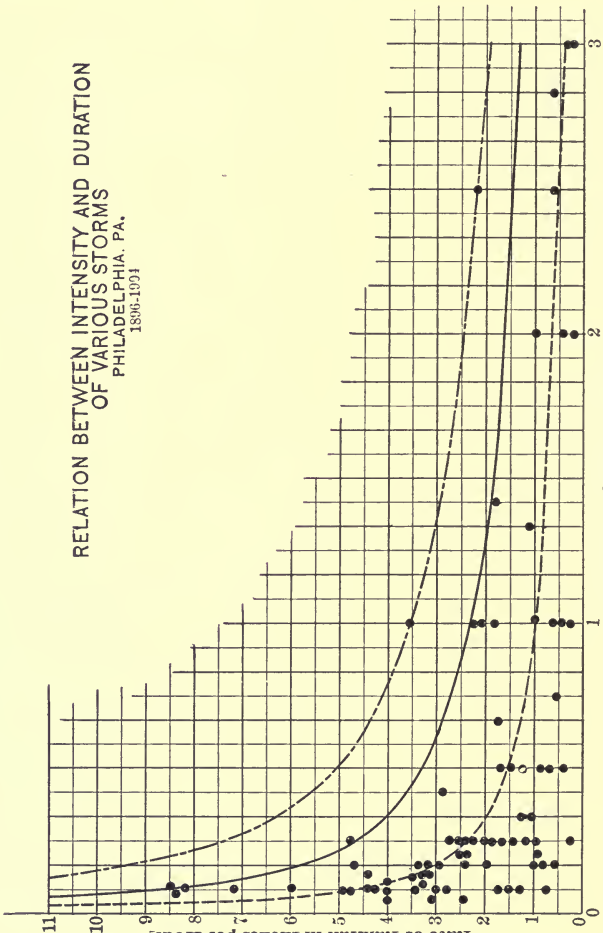


FIG. 10.

exact records possible. The middle curve represents storms of extraordinary rainfall, occurring about once a year. The lower curve represents the maximum intensity of ordinary storms. The equations for the three curves are $y=\frac{30.585}{x^{0.5253}}$, $y=\frac{18}{x^{0.5}}$, and $y=\frac{12}{x^{0.6}}$, respectively.

Fig. 11, from an article * by J. de Bruyn-Kops, of Savannah, Ga., shows even more markedly the variation in the position of the curve, depending on the number of infrequent, isolated

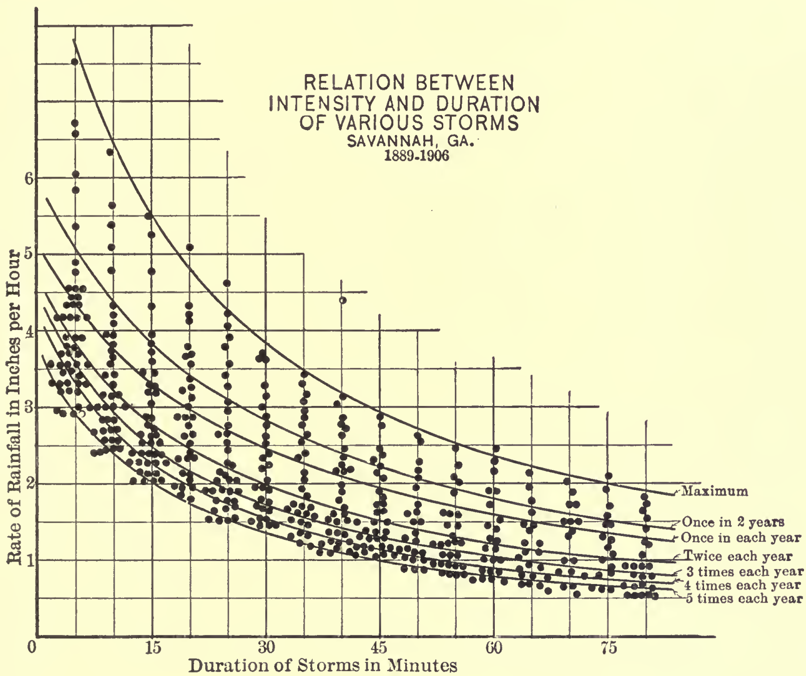


FIG. 11.

storms of maximum intensity, which are chosen to locate the position of the curve on the diagram. In Fig. 11 there are seven curves, each one so drawn as to indicate definitely the intensities of storms of designated frequency, varying from storms of such high intensity as to give absolute maxima for the seven years considered through those storms occurring once

* Trans. Am. Soc. C. E., Vol. LX, p. 250.

in two years and once a year to storms as frequent as five times a year. The equations of Mr. Bruyn-Kops are shown in the following table:

Table V

TABLE SHOWING EQUATIONS FOR RAINFALL CURVES FOR STORMS OF VARIOUS FREQUENCY—SAVANNAH, GA.

Maximum storms	$y=\dfrac{191}{x+19}$
Storms occurring once in two years	$y=\dfrac{163}{x+27}$
Storms occurring once a year	$y=\dfrac{141}{x+27}$
Storms occurring twice a year	$y=\dfrac{104}{x+22}$
Storms occurring three times a year	$y=\dfrac{86}{x+19}$
Storms occurring four times a year	$y=\dfrac{73}{x+17}$
Storms occurring five times a year	$y=\dfrac{63}{x+16}$

Fig. 12, from an article * by C. E. Gregory of the firm of Hering & Gregory, on "Rainfall and Run-off in Storm Water Sewers," shows some of the curves above referred to together with one by Edwin Duryea,† and three prepared by himself from various records of high intensity storms. His equations are $y=\frac{12}{x^{\frac{1}{2}}}$, $y=\frac{32}{x^{\frac{4}{5}}}$ and $y=\frac{6}{x^{\frac{1}{2}}}$ for the curves numbered 4, 9, and 8 respectively. The last is particularly selected as applicable to winter storms, the intensity of which is shown to be less than storms at other seasons of the year. By this diagram, it is seen that if the curves numbered 2, 10, and 3 are eliminated as giving excessive values for ordinary engineering practice; the others are similar both as to form and position on the diagram and the fact is thus emphasized that the curve must indicate to the best judgment of the engineer, not only the existing relation between the duration of the storm and its intensity, but also the frequency of the storms considered.

* Trans. Am. Soc. C. E., Vol. LVIII, p. 458.

† West. Soc. C. E., Vol. IV, p. 73.

The judgment of the engineer in placing the curve on the diagram is aided chiefly by a study of the possible damage to be done by a storm so great as to overflow the sewers and fill the basements, injuring whatever may be housed there. Evidently, if a sewer capable of taking care of the largest storms known should cost $25,000 more than one sufficient only for

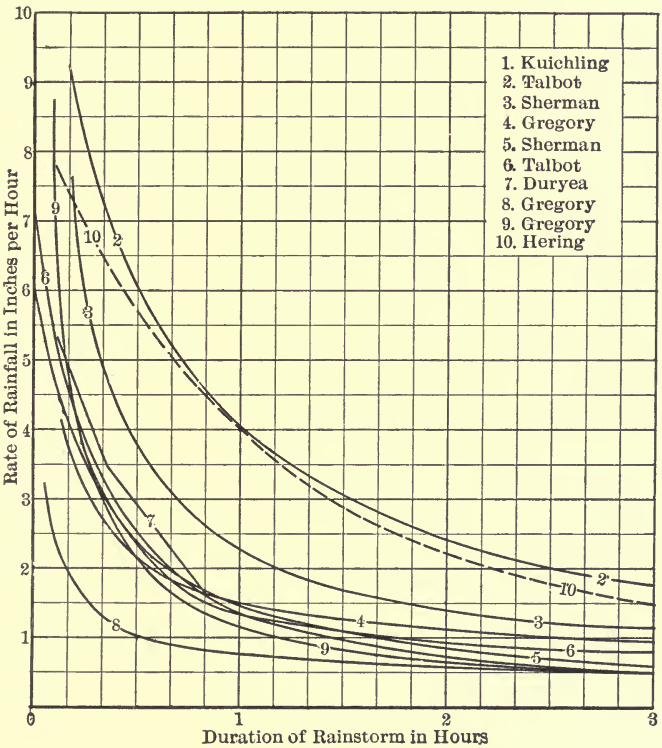


FIG. 12.

the maximum storms coming once a year, the cost of the occasional damage due to infrequent rains must not be less than the accumulated interest on $25,000 or the larger sewer is not worth what it costs. Mr. E. W. Clarke of the Board of Public Water Supply of New York City says * that the extent to

* *Engineering News*, Vol. XLVIII, p. 388.

which the saving in the size of sewers occasioned by assuming storms of less than the absolute maximum is carried, must be offset by occasional damage suits, including costs, and an important factor in the latter is the use made of the buildings occupying the area drained. If the watershed were covered with business blocks, where extra stock was stored in the basements, and were subject to water damage, it might easily happen that one suit decided against the city would cost more than would justify the construction of the smaller sewer. In residential districts, on the other hand, an occasional flooding of the cellars, that is, once in three or four years, might cause comparatively little damage or even inconvenience. In Kansas City, Mo., the practice according to Mr. J. B. Balcomb * is to design the main sewers with the expectation of flooding every ten years, branch sewers every five years, and laterals every two years. The possible future development of a district must, however, not be lost sight of, nor the fact that sewers known to be inadequate and subject to overflowing may seriously affect real estate values and manufacturing enterprises and so indirectly be a source of loss to the city.

When the curve has been finally fixed on the diagram showing rates of fall varying directly with the duration of the storm, what rate is to be taken as that by which the sewers are to be designed? Following Kuichling, the time by which the intensity is made determinate should be equal to that required for water, starting from the point on the line of the sewer farthest from the outfall to reach that outfall. It is plain that, considering the outlet-pipe, a maximum flow will occur when all the laterals are discharging their maximum at the same time. But as some laterals are near and some far away, it is possible for one set to have discharged its volume before the water from the more distant pipes has reached the outfall; so that a rain must continue at a definite rate for a definite time in order that the outfall discharge may represent the maximum discharge due to that rain-intensity. The time required must be that

* Jour. West. Soc. C. E., Vol. XV. p. 706.

necessary for water to flow from the farthest laterals. This time with its corresponding intensity will give the greatest probable discharge at the outfall. A secondary maximum may occur as follows: If a part of the contributing territory should be steep and near the outfall, it may be that the higher rain-intensity corresponding to the shorter time for that section will give more storm-water at the outfall than the less intensity over the whole section. It can be worked out by trial in a few sections and the rain conditions for the real maximum determined. The time required for the passage of the water from the farthest point to the outfall is a matter of trial and judgment. From five to eight minutes is allowed for the rain to pass along the ground from where it fell to the nearest inlet to the sewer or to the gutter if inlets are not located at each block corner. Two feet per second may be taken as a minimum rate of flow in the sewer, and 15 feet per second as the maximum, but between the two the rate of flow will depend on the water surface grade and on the size of the pipe. Therefore the size and grade of the imaginary sewer must be assumed for a preliminary trial. From the surface grade and intensity thus established the sizes can be roughly worked out, and if very different from those assumed at first, the new intensity must be found and the sizes redetermined. It must be remembered that not all of the rainfall is carried off by the sewers, and that only a certain proportion is to be considered, a subject taken up in the next chapter.

It has been pointed out by some writers * that, since no rain can be assumed to flow uniformly for any length of time, but that rather it increases in intensity to some maximum and then decreases, it is an error to require the theory of the duration chosen to depend on the assumption that the entire area is affected equally by a storm of any certain intensity. It is argued that if a certain time is required for a drop of water to reach the outlet from the most distant point, then those distant points

* *Engineering News*, Vol. XLVIII, p. 387, Jour. Assn. Eng. Soc., Vol. XX, p. 204.

were affected by rain of a different intensity from nearer points, which can discharge their share of the run-off only after the crest of the rainstorm has passed. By assuming a number of zones around the outfall, an additive method has been developed * by which each zone has its approximate duration and intensity. The resulting average time of concentration is thus shown to be only about one-half that if the rain were assumed to fall uniformly for the entire period. It is believed, however, that such a refinement is not justified. There are rains falling with a maximum intensity almost uniformly for the short period involved in these studies. The rate of rainfall is only one factor in determining the run-off and the other factors, such as slope, form of the tributary area, and character of the surface introduce such uncertainties that any attempt to subdivide the time of concentration into the time by zones is more of theoretical mathematical interest than of real practical advantage. Further, if another rate of rainfall than that shown by the observed time elapsing from the beginning of the storm to the time of the maximum flow in the sewer, checked in the case of Rochester at least by the actual time required for water to reach the outfall from the most distant point, is taken, then other values than those found in Rochester for the percentage of the rainfall reaching the sewer must be taken.

For an example of the use of the diagrams described above see Chapter XIV.

PROBLEMS

14. Using the data of Table II plot a curve similar to that shown in Fig. 4 and derive its equation in the form $y=\frac{a}{x+b}$.

15. Plot the data of Table III, using logarithmic paper, and derive the equation of the curve passing approximately through those points of the form $y=\frac{a}{x^b}$.

* *Engineering News*, Vol. LXI, p. 265; *Engineering Record*, Vol. LIX, p. 265; Trans. Am. Soc. C. E., Vol. LVIII, p. 458; Vol. LXV, p. 321; Jour. Ass'n Eng. Soc., Vol. XX, p. 204.

16. Refer to the judicial decisions of p. 38 and summarize the opinion of the court in those two cases.

17. From the publications of the U. S. Weather Bureau, secure data on maximum storms covering a period of 10 years for the city of .... Construct an intensity diagram and draw two curves, one for maximum storms, and one excluding storms more infrequent than once in 3 years.

18. Change the form of the equations of Table IV to that of the equation for the Philadelphia curves, and judge of the frequency of "storms of extraordinary rainfall" by the comparison.

19. If an extraordinary storm, occurring on the average once in 12 years, might cause an estimated damage of $25,000, how much money might reasonably be spent on enlarging a sewer designed to safely carry storm water from maximum storms occurring once in 3 years?

20. Using data from *Monthly Weather Review* plot curves for three typical storms in order to show the relation between duration and intensity for any one storm.

CHAPTER IV

PROPORTION REACHING THE SEWERS

The maximum intensity of the rainfall to be cared for by the sewer having been determined, either by carefully examining the tabulated records or by making a diagram of the storms, as indicated in the last chapter, the other part of the problem, already stated, needs to be solved, viz., what proportion of the amount of rain fallen reaches and is carried off by the sewer, and at what rate of flow does the discharge take place? Evidently these are variable quantities, depending on many unknown conditions. The general slope of the surface, its geological character, its physical condition, whether paved or unpaved, the amount of roof and yard surface compared with lawn and garden surface, the grade of the lateral sewers, and the temperature of the air as affecting evaporation, will all influence that proportion. Perhaps more than any other condition, the previous state of the atmosphere will affect this amount. If there has been for some time before the excessive rain a steady drizzle, so that the ground has been well soaked and made partially impervious, the amount afterward absorbed by the soil is very small and the sewer receives a correspondingly larger amount of water. It is therefore impossible to say, even with a surface of known slope or known physical conditions, that 50 or 70 or 90 per cent of a rainfall will enter the sewer, because no account can be taken of the soil permeability.* The only absolute conditions occur when there is no exposed surface, that is, when the district is entirely covered with roofs; then, of course, all the rain is discharged at once into the sewers.

One method suggested for determining the rate of discharge is to compare the time required for discharge with that required

* Jour. West. Soc. C. E., Vol. IV, p. 152.

for the rain to fall; but this relation, depending as it does on the conditions already mentioned, is uncertain, and therefore the method cannot be regarded as reliable. It has, however, been stated that, judging from the limited number of observations accessible, in none of which was the time for discharge from the sewers as short as twice the duration of the storm, but rather exceeding this three, four, and five times, it is always possible to divide the rate of rainfall by at least two to get the rate of discharge. But this must be the result of imperfect observations and inattention to details. Col. Adams reports using a series of gagings made in London by Mr. Wm. Hayward, Engineer to the Metropolitan Board of Works, London, and designing the Brooklyn sewers to carry off one-half the rainfall, believing from his study of those gagings that his sewers would have twice as long to discharge the rain as it takes to fall. Therefore, having decided that a rainfall of 1 inch per hour was to be expected with sufficient frequency to make a provision for it desirable, he made the sewer of such size as to take care of half an inch per hour over all the territory draining to that sewer. Other English experiments, which are given by Baldwin Latham, and on which most of the work done in this country has apparently been based, were made in London in 1857. Here the Savoy Street sewer, draining an entirely built-up part of London, discharged from a rainfall of 1 inch in one and a quarter hours 0.34 cubic foot per second, or 34 per cent of the rainfall. Later Sir Jos. Bazelgette, in the Savoy Street and Ratcliff Street sewers, determined that from rainfalls of 2.9 inches in thirty-six and twenty-five hours there was discharged an average amount equal to 64.5 and 52 per cent respectively. From these gagings and a few others the engineers of the London Main Drainage Works concluded that a rainfall of 0.25 inch would discharge 0.125 inch, while one of 0.40 inch might discharge 0.25 inch. In 1865 Col. Wm. Hayward published a gaging of another London sewer, showing that of a rain of 2.75 inches in thirty-six hours 53 per cent was discharged, and in 1858 of a rain of 0.24

inch the same sewer discharged 74 per cent; and in the same year the Irongate sewer, from a district entirely paved and built up, discharged 94 per cent of a rainfall of 0.54 inch in five hours, and in August the same sewer discharged 78 per cent of a rain of 0.48 inch in 1.67 hours. Kuichling, in citing these records, notes the absence of details as to the character of the rain, manner of observation, location of gages; and suggests possible inaccuracies in the recorded percentages. He quotes another gaging by John Rae, C.E., engineer of the Holborn and Finsbury sewers, who states that during the continuance of a rain of 1 inch per hour 41 to 54 per cent of the precipitation will reach the sewer, according to the amount of garden or lawn surface upon the drainage area. Kuichling adds:

"Upon the foregoing indefinite data, which may be found quoted more or less extensively in nearly every treatise on sewerage, and in most of the elaborate reports, engineers have hitherto been content to rely, and thus it has come to be in some measure traditional that about 50 per cent of the rainfall will run off from urban surfaces during the progress of the storm, while the remainder may follow at leisure." Until the recent (1889) work of Mr. Kuichling, this has been undoubtedly true, and in Providence, Brooklyn, St. Louis, and other cities the old sewers, often gorged and overflowing, have proved that the old assumptions in regard to rainfall are not accurate, but require modification. Of late a German formula has been much used, in which the coefficients may be modified for different kinds of surface, and the amount of run-off considered in the design has thereby been much increased. The discussion of this formula is reserved for the next chapter.

Kuichling proved by his experiments at Rochester that these inconsistencies and failures were due to the unit period of time used both for the rainfall and for the gagings. He observed that the volume of water discharged at different stages at the mouth of an outfall sewer increased and diminished directly with the intensity of the rain, and that a certain time was required in each case before a change in the rate of rain was manifested in the outlet. In preparing the design for the East Side Sewer an extensive series of observations were carried out, containing valuable data and contributing largely to our knowledge of the subject. The four rain-gages, already alluded to, gave him as accurate a knowledge of the rainfall as was possible without automatic gages. Simple self-recording gages were placed in the principal outlet sewers of the East Side, and the cross-sections, dimensions, and slopes of those sewers were all carefully determined. It was noted even without the gage-reading that slight variations in the rate of precipitation were quickly felt in the sewers, and the flood-heights therefore were due to the maximum intensity of the rain, usually lasting but for a few moments, and not to the average intensity for the whole period of the storm. Moreover, the periods of maximum intensity of rainfall corresponded closely with the period of maximum discharge, and in a rain of varying intensity the volume of sewer-discharge followed the rain in parallel waves.

The drainage-areas were carefully determined, so that the actual volume of the rain falling was obtained, and the amount discharged was calculated by Kutter's formula from the height of flow in the sewer, as shown by the gage, and from the hydraulic slope of the sewage. During 1888, 17 storms were gaged, their intensities varying from 0.24 inch to 3.20 inches per hour in the different sewers. A summary of the results is given in the following table, for which Kuichling claims no great accuracy, since the amounts of the intermediate showers were not always well known, though the totals are reliable. They are well worth regarding, however, as being the only careful records of the relation of rainfall to sewer-discharge that are available.

It can be seen on inspection that the discharge from District X is invariably the largest, accounted for by the fact that it has the largest proportion of roof-surface and other impervious ground-covering. The effect of a light rain immediately pre-

Table VI *

SHOWING THE COMPUTED PERCENTAGES OF THE HEAVIEST RAINFALL DISCHARGED FROM FIVE DIFFERENT CITY DISTRICTS BY THE RESPECTIVE OUTLET SEWERS DURING THE PERIOD OF MAXIMUM FLOW, ALSO THE AVERAGE VALUES OF SUCH PERCENTAGES.

Arranged with reference to duration of heaviest rainfall.

Date.	Maximum Intensity of Rainfall. Inches per Hour.	Duration of Rain at Maximum Intensity. Minutes.	Percentage of Rainfall Discharged.				
			District I. Gage 2. Trib. Area 356.94 Acres.	District IV. Gage 8. Trib. Area 128.67 Acres.	District X. Gage 18. Trib. Area 25.12 Acres.	District IX. Gage 18. Trib. Area 132.96 Acres.	District XVII. Gage 30. Trib. Area 92.27 Acres.
Dec. 10, 1887....	0.31†	60	13.8	24.1	58.2	41.6	26.0
Sept. 16, 1888....	0.47‡	50	19.8	38.2	....	....	37.2
	Averages	55	16.8	31.1	58.2	41.6	31.6
May 9, 1888.....	1.315‡ to 0.75‖	35	16.4	26.2	52.1	29.0	26.0
April 5, 1888.....	0.24†	30	10.4	15.5	....	38.2	20.8
May 12, 1888....	0.30†	30	11.0	15.8	35.3	29.6	17.0
	Averages	30	10.7	15.7	35.3	34.9	18.9
June 24, 1888....	2.62‡	20	6.3	21.1	32.0¶	13.2¶	11.8¶
June 28, 1888....	0.80†	20	14.3	28.7	35.2	35.2	37.4
	Averages	20	10.3	24.9	33.6	24.2	24.6
June 2, 1888.....	0.40‡	15	5.5	9.0	....	37.5	8.7**
July 11, 1888....	0.76§	15	7.4	15.8	41.2	21.8	19.4
Aug. 16, 1888....	1.616‖	15	4.7	12.5	24.7	18.0	19.1¶
	Averages	15	5.9	12.4	32.9	25.8	19.2
May 4, 1888.....	0.30†	13	6.8	14.4	64.8**	36.1**	28.2**
May 26, 1888....	1.00†	13	8.6	25.9**	31.8	18.7	11.7
Aug. 4, 1888.....	1.00‡	12	4.6	10.0	....	15.0	13.8
Aug. 26, 1888....	2.50‖	14	4.0	12.2	33.5¶	13.8¶	12.3¶
	Averages	13	6.0	12.2	32.6	15.8	12.6
July 18, 1888....	0.75†	10	7.6	12.2	25.0	14.8	10.3
Aug. 17, 1888....	1.33‡	10	5.5	8.7	18.4	11.9	8.9
	Averages	10	6.5	10.4	21.7	13.3	9.6
Probable time required for concentration of flow at gages. Minutes................			44	26	16	23	24

* From Kuichling's Report, p. 165. † Preceded and followed by lighter rain.
‡ Sudden shower followed by lighter rain. § Heavy shower preceded by lighter rain.
‖ Intensity roughly estimated.
¶ Sewer here ran under head; percentage is computed from maximum discharge without head previous to surcharge.
** Figures obviously too high or low, and rejected in deriving averages.

ceding is clearly seen, and the variation in the percentages discharged from the same district. From the most urban district the maximum discharge was 58.2 per cent of the rainfall, and from the most rural it was as low as 4.0 per cent.

The following gives the general characteristics of the several drainage districts (Kuichling's Report, Table XIX).

District I. About one-half of this area has a dense population, averaging about 35 per acre, and is well developed, while the remainder is thinly settled, with much agricultural or vacant land. Nearly all of the existing streets are sewered or graded, but only a small proportion of the aggregate length is improved with macadam, the rest having earthen roadways. Soil-surface is generally clayey loam, interspersed with some gravel. Surface slightly undulating, the average slope of the sewered streets being about 1:150. Sewer-grades range from 1:47 to 1:910. Outlet sewer is of good rubble masonry with flat segmental invert of brick. Length of main and tributary sewers at Gage No. 2, is 10.35 miles.

District IV. Area is generally well developed, beginning in the central portion of the city and extending northerly to Gage No. 8, in the form of a comparatively narrow strip about 4800 feet long by 1200 feet wide on the average. All of the streets are sewered and graded, and about one-third the aggregate length is improved with stone block, asphalt, macadam, and gravel pavement, the macadam, however, predominating; the remainder of the streets have common earthen roadways. Along the principal street (North Avenue) many large business blocks have been built, but the rest of the territory is occupied chiefly by residences. The population may be taken at about 32 per acre. The houses are generally large, and lots of medium size. Below Gage No. 8 few of the streets are improved, and there is considerable vacant land. The soil is mainly a clayey loam, with muck in the lower portions. The surface slopes gently to the north as far as the N. Y. C. & H. R. R. R., and then becomes very flat. The average grade of the streets is about 1:130, and the sewer-grades range from 1:50 to 1:630. At the gages the outlet sewer is of good rubble masonry with flat and somewhat irregular rocky bottom. Length of main and tributary sewers at Gage No. 8 is 4.37 miles.

Districts IX and X. Discharge measured by Gages Nos. 18 and 19, in East Main and Alexander Street sewers respectively. The former serves a small but densely populated area traversed by the principal street, while the latter serves a large and well-developed residential district. In District IX the sewer grades range from 1:54 to 1:400, and the average surface-slopes of the streets is about 1:151; and in District X the sewer-grades range from 1:70 to 1:330, the average surface-slope being 1:172. From its general character this latter district should give the greatest percentage of rainfall-discharge, as the amount of roof-surface is here proportionally the greatest. The length of main and tributary sewers at Gage No. 18 is 0.76 mile.

District XVII. Discharge measured by Gages No. 30 and 31 in the Griffith Street sewer. The tributary area is well sewered and developed, and the average density of population may be estimated at about 35 per acre. Every street has an improved roadway, about one-fifth of the total street-surface being asphalt, one-fourth stone block, and the remainder macadam and gravel pavement. Numerous large business blocks and apartment-houses are found on the territory, but the greater portion of it is occupied by residences, standing generally on lots of medium size, although in about twenty-five acres of the area the lots are very deep and afford opportunity for additional streets. The surface-grades in about one-half of the area are of good inclination, while in the remainder they are rather flat, the average being about 1:240 for Gage No. 30 and 1:175 for Gage No. 31. Sewer-grades vary from 1:100 to 1:350. The soil is generally a clayey loam, and much of the rainfall is as yet absorbed into the ground. Length of main and tributary sewers at Gage No. 30 is 2.56 miles.

In an article on flood-waves by Alvah Grover in the Trans. Am. Soc. C. E., Vol. XXVIII, an apparatus is described for automatically measuring the height of waves in sewers.* The heights thus obtained, plotted on the same sheet and to the same scale as the depths of rainfall, give at a glance the relation between the two. The article in question is largely devoted to a description of the apparatus, but the relation between five storms and the resulting sewage-flow is given. The largest percentage found is as follows:

Date.	Duration by U. S. Weather Bureau.	Amount of Rainfall by Writer's Gage.	Duration of Disturbance in Sewer in Seconds.	Per Cent of Total Rainfall Discharged Reg. by Gage.
Sept. 27, '92...	2 hr. 55 min.	0.32 inches	22,853	60.6

Fig. 40 shows the daily record of sewage-flow as recorded by the apparatus.

It is interesting in this connection, although the percentages have no bearing on the present question, to compare the results of the gaging of the Sudbury River watershed, as given in the Geol. Report of N. J., Vol. II, p. 6, with like data of many other streams (see Table VII).

The tables following illustrate a relation between the rainfall and the discharge of a watershed of 78 square miles, very similar to that at first thought to exist between the same quantities in the case of sewers, and show that while the annual average holds not far from 50 per cent, the monthly relation is much more variable. In the case of sewers, in order to reach the true relation between the rain and the discharge, the unit time must be reduced from the month not only to the day and hour, but to the five-minute or minute interval.

Baumeister in considering this subject says: "In England from 0 to 70 per cent of the rainfall reaches the drains, averaging about 50 per cent. In different districts of London from 53 to 94 per cent has been registered. It required from three to four times the duration of the storm to carry off the water, and the maximum flow per second in the sewers rose as high as

* At Omaha, Neb.

2.4 times the average, obtained by dividing the total effluent due to the storm by the number of seconds of flowing. Hence it will be seen that the necessary capacity will be $0.5\times\frac{2.4}{3.5}=\frac{1}{3}$ of the rainfall per second.

Table VII

TABLE SHOWING THE RAINFALL AND STREAM-FLOW ON THE SUDBURY RIVER

Month.	1880—1881.		1881—1882.		1882—1883.		1883—1884.		1884—1885.	
December.	2.83	0.31	3.96	1.38	2.30	0.56	3.55	0.35	5.17	1.65
January...	5.56	0.74	5.95	2.21	2.81	0.60	5.09	1.76	4.71	2.20
February.	4.65	2.49	4.55	3.87	3.87	1.66	6.54	4.74	3.87	2.18
March....	5.73	7.14	2.65	5.06	1.78	2.87	4.72	6.75	1.07	2.81
April......	2.00	2.67	1.82	1.50	1.85	2.33	4.41	4.93	3.61	3.13
May......	3.51	1.72	5.07	2.30	4.19	1.67	3.47	1.84	3.49	2.38
June......	5.40	2.31	1.66	0.91	2.40	0.52	3.45	0.72	2.87	0.74
July......	2.35	0.49	1.77	0.15	2.68	0.21	3.65	0.40	1.43	0.11
August....	1.36	0.26	1.67	0.10	0.74	0.14	4.65	0.46	7.19	0.43
September.	2.62	0.34	8.74	0.53	1.52	0.16	0.86	0.08	1.43	0.21
October...	2.96	0.33	2.07	0.53	5.60	0.33	2.48	0. 5	5.10	0.60
November.	4.09	0.68	1.15	0.36	1.81	0.35	2.65	0.30	6.10	2.03
	43.06	19.48	41.06	18.90	31.55	11.40	45.52	22.48	46.04	18.47

Month.	1885—1886.		1886—1887.		1887—1888.		1888—1889.		1889—1890.	
December..	2.72	2.09	4.98	1.82	3.88	1.15	5.40	5.43	3.14	4.00
January...	6.37	2.61	5.20	4.62	4.15	1.88	5.37	4.96	2.53	2.24
February..	6.28	7.73	4.78	4.56	3.69	3.26	1.66	1.93	3.51	2.46
March....	3.61	3.67	4.90	5.12	6.02	5.76	2.37	2.39	7.74	6.50
April......	2.23	3.36	4.27	4.52	2.43	4.57	3.41	2.43	2.65	3.24
May......	3.00	1.29	1.17	1.80	4.83	2.91	2.95	1.57	5.21	2.44
June......	1.47	0.35	2.65	0.71	2.54	0.73	2.80	1.13	2.03	0.98
July......	3.27	0.21	3.76	0.20	1.41	0.21	8.94	1.13	2.46	0.19
August....	4.10	0.17	5.28	0.38	6.22	0.68	4.18	2.55	3.87	0.24
September.	2.91	0.20	1.32	0.19	8.59	1.99	4.61	1.42	6.00	0.79
October...	3.24	0.26	2.84	0.34	4.99	3.57	4.26	2.19	10.51	4.05
November.	4.65	1.16	2.67	0.64	7.23	4.76	6.29	3.35	1.20	2.10
	43.85	23.104	3.82	24.90	55.98	31.47	52.24	30.48	50.85	29.23

In a table for European cities the percentage of the rainfall provided for varies from $\frac{1}{6}$ of a rainfall of 0.9 inch for suburban territory in Berlin to $\frac{1}{4}$ of a rainfall of 2.9 inches in Köningsberg

After a careful study of his records Kuichling formulated the following conclusions:

"1. The percentage of rainfall discharged from any given drainage-area is nearly constant for rains of all considerable intensities and lasting equal periods of time. This can be attributed only to the fact that the amount of impervious surface on a definite drainage-area is also practically constant during the time occupied by the experiments.

"2. The said percentage varies directly with the degree of urban development of a district, or, in other words, with the amount of impervious surface thereon. This fact is clearly shown by the large percentage derived from the relatively most developed district, X, in contrast with the smaller percentages from the relatively less developed districts, IX, IV, and XVII, and the least improved district, I.

"3. The said percentage increases directly or uniformly with the duration of the maximum intensity of the rainfall until a point is reached which is equal to the time recorded for the concentration of the drainage-waters from the entire tributary area at the point of observation; but if the rainfall continues at the same intensity for a longer period, the said percentage will continue to increase for the additional period of time, but at a much smaller rate than previously. In other words, the proportion of impervious surface slowly increases with the duration of the rainfall.

"4. The said percentage becomes larger if a moderate rain has immediately preceded a heavy shower, thereby partially saturating the permeable territory and correspondingly increasing the impervious surface.

"5. The sewer-discharge varies immediately in all appreciable fluctuations in the intensity of the rainfall, and thus constitutes an exceedingly sensitive index of the rate and its variations of intensity.

"6. The diagrams also show that the time when the rate of increase in the said percentages of discharge changes abruptly from a high to a low figure, agrees closely with the computed lengths of time required for the concentration of the storm-waters from the whole tributary area; and hence the said percentages at such times may be taken as the proportion of impervious surface upon the respective areas."

Since the early work of Kuichling in Rochester no similar work on a generous scale has been reported to the engineering world. In 1908, a committee of the Boston Society of Civil Engineers was appointed * to compile available data and to direct experiments to be made by members of the society, on this subject. Some work has been done, but no report published. Similarly, the Sanitary and Municipal Section of the Western Society of Engineers has undertaken to collect data from their members. The engineers in charge of sewer work in Philadelphia, in St. Louis, in Cleveland, and doubtless in many other cities, have on hand experimental data giving valuable comparisons for those particular cities and is much to be hoped that they will some day allow them all to be published. The time may come when the uncertainties of the percentage of run-off to rainfall will be recognized as so great as to make values based on it useless. Then if accumulated records will allow, storm-water flow can be referred directly to area and surface conditions. Until such data are available, at all events, a percentage of the rainfall is the only known method of approximating to the truth.

Three large districts of Chicago have been studied, the sewage measured and the rainfall gaged. L. K. Sherman presented the results † to the Western Society of Engineers, Jan. 15, 1912. The unit of time used however was the day so that the effect of short storms of high intensity is not shown. It was demonstrated that while the maximum rainfall was such as to deliver water to the area at rates of 95, 47, and 48 cubic feet per second per square mile, the run-off was 50, 15,

* *Engineering News*, Vol. LIX, p. 219.
† Jour. West. Soc. Engrs., Vol. XVII, p. 361.

and 38 cubic feet per second, or 52, 32, and 78 per cent. These districts are 22 square miles, 8.3 square miles and 0.15 square mile in area, and the densities of population are given as 19, 5.3, and 35 per acre, the last being in the centre of the business district of South Chicago. This verification of the work of the engineers of 1860 is a tribute to the work of those pioneers.

Mr. Parmley, after much experience in Cleveland, believes* that, for safety in business districts, 100 per cent of the rainfall should be expected to reach the sewers. For residence districts, he suggests 20 to 50 per cent, although he says that where the lots are not large and the district is well built up, the percentage of the rain entering the sewer may be 70 per cent.

In Kansas City, approaching the problem from the other side, Mr. Balcomb † has assigned values to the amount of rain in inches per hour that may be expected to be absorbed by surfaces of various soils. Thus he assumes that paved streets absorb 0.50 inch per hour at the beginning of a storm, decreasing to 0.25 inch at the end of fifteen minutes, and that garden soils absorb 1.00 inch at the beginning, decreasing to 0 at the end of 120 minutes. Most engineers, however, prefer the direct percentage, although absorption is undoubtedly one of its important factors.

Some experiments were carried on by students ‡ of the College of Civil Engineering (Cornell) in 1910, in which it was found that from a residential area of 42 acres on a steep side-hill, the maximum percentage of rainfall intensity shown in the sewage flow was 34.8. The resident population, exclusive of students, was 22 per acre, and was not different from other suburban property where there is one house and a lawn to every 100-foot lot with occasionally one not built on. In the thesis based on these experiments, it was pointed out that the needs of growing vegetation should be considered, since the amount

* Jour. Assn. Eng. Soc., Vol. XX, p. 212.

† Jour. West. Soc. Engrs., Vol. XV, p. 707.

‡ Thesis by P. Z. Horton and R. Taylor, 1910.

of rain absorbed by grasses and vegetation generally was an important factor quite distinct from the amount of percolation into the ground.

Mr. Alvord, who has had a long practical experience in the application of the theory of run-off to actual construction, lays great stress on the absorption capacity of the soil.* He believes that the earth becomes a great storage reservior after a dry season and may entirely absorb the water from a short storm. He shows that a certain district of Chicago, of an average residential character, would require by theory a sewer $8\frac{1}{2}$ feet diameter, but by reason of the soil conditions, the existing sewer 4 feet diameter, $\frac{1}{4}$ the capacity, is found to be large enough.

Mr. J. H. Fuertes † has reported that by actual measurements of rainfall and run-off, he found the percentage on an open field, of hard clay soil, covered with grass, on a 5 per cent slope to be 29 per cent, a high ratio for grass land.

Perhaps no better summary can be had than that given by Professor Marston, who says: ‡ "The most important part of this paper (by C. E. Gregory) seems to the writer to be the general presentation of the following principles:

"First. The water falling on the so-called impervious area of an ordinary sewer watershed does not all run off as fast as it falls, but part of it accumulates in increasing quantities on the surface during downpours of moderate length, such as cause the maximum discharges from sewer districts of ordinary size.

"Second. As the storm continues at the same rate, the ratio of run-off from the impervious area to the rate of rainfall increases, owing to the increased depth and velocity of the surface flow toward the sewer, until, finally, if the storm lasts long enough, the rate of run-off from the impervious areas becomes equal to 100 per cent of the rate of rainfall.

"Third. The pervious area becomes more and more

* Jour. West. Soc. Engrs., Vol. IV, p. 154.

† Jour. West. Soc. Engrs., Vol. IV, p. 170.

‡ Trans. Am. Soc. C. E., Vol. LVIII, p. 498.

saturated with water as the storm continues, and there will be a percentage of run-off from pervious areas increasing from 0 for short storms to quite a large percentage for storms lasting several hours.

"There can be no question as to the general truth of these three principles, but when the author (Mr. Gregory) attempts to go further and present definite curves and formulas, purporting to show the exact laws of change in the percentage of run-off with relation to the time elapsed since the beginning of the storm, the writer believes he is going beyond what the present meagre data from gagings of storm-sewers warrant.

"These data are not very extensive, and the author himself points out their many defects. For many years engineers have been criticising the old run-off formulas, and new formulas, based on insufficient data, should be avoided.

"All that engineers are at present warranted in doing is to make some deduction from 100 per cent run-off from the impervious areas for short storms in favorable cases, and some increase above 0 per cent (say varying up to 20 per cent for one-hour storms with average soil and slopes) in the run-off from the pervious areas for long storms, both the deduction and the addition being at present left to the judgment of the engineer, in view of his general knowledge and his familiarity with local conditions."

To still further emphasize the fact that there is no infallible rule for predicting the percentage of rainfall for which sewers are designed, Mr. C. B. Burdick has prepared * the diagram shown in Fig. 13, which gives the practice of the various cities named in the matter of sewer capacity. Since the difference between the rainfall curves of various cities is rather a question of position on the diagram than of actual difference in rate of rainfall, the figure shows for a district of 100 acres (about 20 blocks) a minimum run-off of 0.18 cubic foot per second at Gary and a maximum of 1.9 at Baltimore. If the time of concentration be taken at forty minutes for such a district (distance of flow would be about 4000 feet) and, from Fig. 11,

* Trans. Am. Soc. C. E., Vol. LVIII, p. 507.

the rate of rainfall be taken at 2 inches, then the percentage for Gary is only $9\frac{1}{2}$, while in Baltimore it is 95 and in St. Louis it is 70. With such variation in values, it seems hopeless to

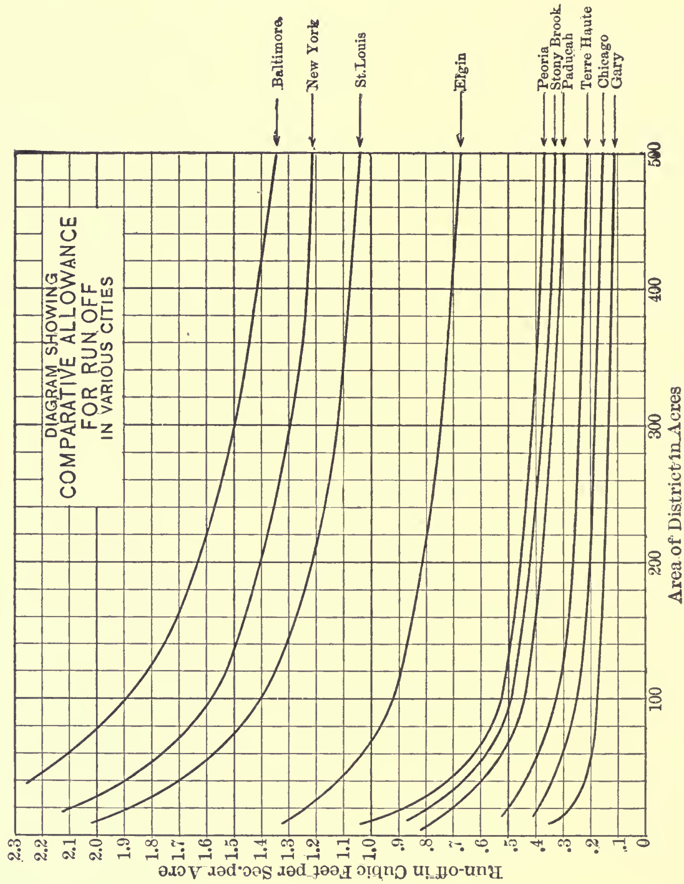


FIG. 13.

extend efforts on perfecting details of any one factor while the combined factors give such widely different results.

A proper solution of the problem calls for a careful study of the district to be drained, with careful attention to the soil, the shape, the slope, the relative amount of impervious area, all uncertain and not susceptible of mathematical expression.

With the effect of the size of the district, i.e., the time of concentration, upon the rate of rainfall in mind, a proper value for the latter can readily be selected, but the proper percentage of this to be used in determining the size of the sewer is far more uncertain. The methods developed in the next chapter will be of service for such a purpose.

PROBLEMS

21. Show that for a rainfall of $2\frac{1}{2}$ inches per hour on an impervious area, the run-off is approximately $2\frac{1}{2}$ cubic feet per second.

22. From any city plan available, scale not less than 200 feet to the inch, determine the percentage of some district, of about 100 acres, that is devoted to street surface.

23. In a residential district of a city, inspect three selected blocks, pacing distances between street centre lines and estimating as closely as possible the areas of roofs, determine the percentage of roof area and street pavement area to total areas of the blocks.

24. Assume an area of 100 acres to be of three different shapes, wide and shallow, square, narrow and deep. Assume further that the path of the sewer may be either around the edge of the area or along a diagonal. Show how such changes would affect the time of concentration if the velocity remained the same.

25. Assume a square area of sides equal $\frac{1}{2}$ mile and assume that this area has varying slopes from 0 to 10 per cent (take 1 per cent, 2 per cent and 5 per cent as intermediate values). Assuming a rainfall to follow Fig. 6 (lower curve) with 35 per cent of the area impervious and the rest not contributing. Show the effect of the slope on the time of concentration and on the sewage flow. Establish a relation between Q and S.

CHAPTER V

RELATION OF DENSITY TO PERCENTAGE

WHILE it is evident that more rain will be discharged into a sewer from a closely built-up territory than from an open and agricultural district, yet so far as the data of the last chapter go, no light has been thrown on the proper variation of the percentage as determined by the relative amounts of pervious and impervious surface. Our knowledge on this subject is due to Mr. Kuichling.

If it is assumed, as indeed seems reasonable, that the density of population bears a direct ratio to the percentage of impervious area in a given district, and if that ratio is once determined under general conditions, the determination for other places of their population-densities will serve approximately, at least, by means of the same ratio, to determine the percentage of impervious surface also, and so the percentage of the rainfall discharged through the sewers. The relation between the population and the impervious surface was found by a laborious compilation of the amount and character of street-surface, roofs, lawns, gardens, etc., and of the population, all in typical districts, and by a reduction of all the areas of semi-impervious nature to the areas of impervious surface, equivalent in discharging power. It was assumed that the duration of the storms was such that even from impervious pavements not all the rain was discharged—an assumption only justifiable in dealing with storms of great magnitude, whose duration is expressed in minutes. For long rains even garden or lawn surfaces may reduce the losses due to evaporation and surface inequalities, so that if the duration of the storm is sufficient, the surface becomes practically impervious; but in general such soils will absorb nearly all the rain falling.

In some German practice it is customary to deduct such surfaces from the contributing area.

The various kinds of relatively impervious surface found on urban territory were classified by Kuichling as follows:

1. The different varieties of roofs from which nearly all water runs off.

2. The first-class sidewalks and pavements, such as asphalt, and cut-stone blocks or brick with asphalted joints.

3. The second-class sidewalks and pavements, such as the common Medina blocks with large open joints.

4. The third-class sidewalks and macadam or gravel pavements.

5. Ordinary graded roadways and similar surfaces.

From the best pavements and sidewalks a considerably less proportion of water is discharged than from roofs because of the irregularities of surface and because of the absorption by the dust and dirt, even if the surface itself is practically non-absorbent. The other classes, of course, retain a still larger percentage, owing to deeper depressions and ruts and to the greater absorptive power of the material itself.

By an analysis of the conditions in cities like Buffalo, Syracuse, and Rochester it was found that in well-developed city districts there are on an average 32 persons per acre.* With an assumption of 5.6 persons per dwelling, there should be therefore about six dwellings per acre in such territory. In the cities investigated it was further found that about 27 per cent of the entire area was occupied by public streets and alleys, of which 43 per cent, or one-tenth of the entire surface, was provided with some kind of pavement varying in quality with the character of the district. In the growth of cities this proportion is likely to increase, it was observed, until all of the 27 per cent has some more or less impervious pavement. A certain roof-area was assumed for the six dwellings, and that for an assumed business block or tenement added, with something more for possible barns or sheds for each acre, the result being

* In Ithaca, N. Y., by actual count there are 26.2 in the residential district.

that 18 per cent of the acre would probably be roof-surface. To this was added the impervious surface of the streets, which, with due allowance for the future, was taken as 16 per cent of the acre, making in all 34 per cent impervious, the rest being well-compacted earthen surfaces of back yards and courts which are specially drained. These last were taken to be of such a character and amount as to discharge rain-water from an area equal to 25 per cent of the whole. For a density of 50 persons per acre it was assumed that there were no vacant lots, that both the dwellings and the business and apartment buildings were more crowded together, since the land is more valuable, and that therefore the roof-surface amounts to 28 per cent of the acre. The amount of street-surface will not differ materially from the amount previously estimated, but nine-tenths of it, or 25 per cent of the whole, may be regarded as impervious. Since the yards are more likely to be paved, they may be considered to discharge an amount equal to 28 per cent of the whole. Similar analyses were made for other densities, and the final relations determined on are as follows:

TABLE VIII

Average Number of Persons per Acre.	Percentage of Roof-surface per Acre.	Percentage of Improved Street-surface per Acre.	Percentage of Hard-earth Streets and Yards per Acre.	Total Percentage of Relatively Impervious Surface per Acre.
15	8.4	7.4	15.0	30.8
25	14.0	12.5	21.5	48.0
32	18.0	16.0	25.0	59.0
40	22.5	20.0	27.5	70.0
50	28.0	25.0	28.0	81.0

If the population-densities increase beyond 50 persons per acre, the roof and impervious street-surface will also increase up to a maximum, while the hard-earth surface will increase up to a certain point and then rapidly decrease, being replaced by a larger value of the other two factors, the open-earth space being taken up entirely with paved yards. The amount of roof-surface and improved street-surface and paved yards seldom reaches 100 per cent, as there are always a few open spaces, gardens, small parks, etc., so that for areas of any magnitude, even in the largest cities, the limit may be set at 90 per cent. The street-area cannot exceed 27 per cent of the entire area unless yards be included, when it may amount to 40 per cent; and the roof-area will reach 60 per cent, as a maximum, for cities like Rochester.

But the paved streets are not absolutely impervious, only relatively so, and the hard-earth yards, while allowing some rain to run off, also retain some, so that the percentages given above are only of the areas to be considered. It remains to determine what proportion runs off from the four classes. The loss of water by absorption and evaporation from roofs is generally so small in heavy rains that it may be neglected, so that the roof-surface may be taken as truly impervious. As to the percentages furnished from pavements and sidewalks the amount varies with the quality of the pavement; and while no record of exact experiments was available, it was estimated that from a well-paved stone or asphalt pavement 80 per cent of the rain ran off. From well-kept macadam or gravel roads from 30 to 50 per cent of the rain was obtained, and, interpolating for other pavements, for second-class sidewalks and stone pavements the discharge would be 60 per cent; for the best macadam, 50 per cent; and for inferior macadam and gravel roads not more than 40 per cent would reach the sewers during a hard storm. The proportion to be expected from the hard-earth surfaces of streets and yards is evidently subject to great variation, but it was assumed that it would be 20 per cent of the rain falling.

Correcting Table VIII by these percentages of discharge, and assuming, as indicated by the Rochester studies, that the quantities of the different classes of pavement were divided as given, in proportion to the different densities, we obtain the following table:

TABLE IX

Average Number of Persons per Acre.	Total Percentage of Improved Street Surface.	Subdivided into Pavements of			Proportion Considered as Fully Impervious			Equivalent Per Cent of Fully Impervious Surface.
		1st Class.	2d Class.	3d Class.	1st Class.	2d Class.	3d Class.	
15	7.4	.....	1.5	5.9	.....	0.60	0.40	3.3
25	12.5	4.0	2.0	6.5	0.80	0.60	0.40	7.0
32	16.0	8.0	3.0	5.0	0.80	0.60	0.40	10.2
40	20.0	13.3	6.7	.....	0.80	0.60	.....	14.7
50	25.0	20.0	5.0	.....	0.80.	0.60	.....	19.0

Then assuming that the hard earth yields 20 per cent of the rain, and reducing from Table VIII, we get finally the amount of water discharged from a given rain in terms of the varying densities, as follows:

TABLE X

Average Number of Persons per Acre.	Percentage of Fully Impervious Surface.			Total Percentage of Fully Impervious Surface per Acre
	Roofs.	Improved Streets.	Unimproved Streets and Yards.	
15	8.4	3.3	3.0	14.7
25	14.0	7.0	4.3	25.3
32	18.0	10.2	5.0	33.2
40	22.5	14.7	5.4	42.6
50	28.0	19.0	5.6	52.6

By plotting the final percentages as ordinates with the corresponding densities as abscissæ, a curve may be drawn which will express the relation between these two variables (see Fig. 14). The equation of this curve may also be found if desired.

A small amount of additional data may be added as corroboration to the results of Mr. Kuichling. In Ithaca, 1910, the two seniors already referred to carefully measured the various surfaces in a district of 42 acres that had a normal population of 22 per acre, with a house on every lot but one, with results given in the following table:

Table XI

TABLE SHOWING PERCENTAGES OF DIFFERENT KINDS OF SURFACES IN A TYPICAL DISTRICT OF ITHACA, N. Y.

No. Persons per Acre.	Percentage of Roof Surface per Acre.	Percentage of Impervious Road Surface per Acre.	Percentage of Hard Earth and Macadam Road per Acre.	Total Percentage of Relatively Impervious Surface.	Percentage of Lawns and Flower-beds per Acre.
22	17	8	15	40	60

If this be reduced to the total amount of impervious surface, by assuming that 80 per cent of the rain was delivered from the improved streets and 40 per cent from the hard-earth and macadam roads, and nothing from the lawns and flower-beds, the amount becomes 29.4, while the maximum percentage

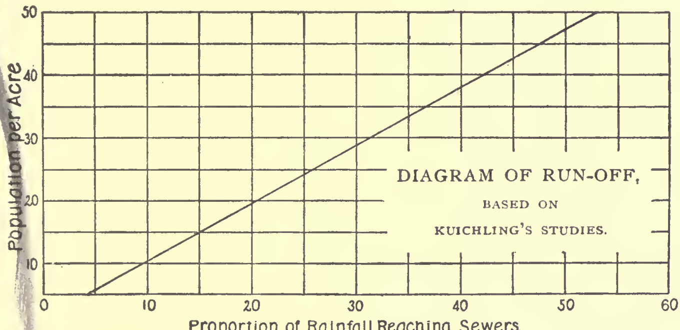


FIG. 14

found in the experimental work was 34.8, an earlier rain probably adding a small run-off from the grass.

In another thesis by Mr. H. E. Green, '06, a portion of the campus was studied and the run-off measured by a weir across the small creek that furnished drainage for this particular district. The percentages of relatively impervious areas were as follows:

Roof Surfaces.	Sidewalks and Macadam.	Gravel Roads and Walks.	Open Ground.
5.5	3.5	4.2	86.8

By reference to Table VIII it is plainly seen that if it were residential area, the roof surface and road area would accord with a population of less than 15 per acre, and that therefore by Table X there should be somewhat less than 15 per cent the intensity of rain found in the rate of run-off. The maximum storm measured had a flow of thirty-five minutes at the rate of 0.78 inch per hour, the duration being that for concentration. The rate of rainfall was 46.0 cubic feet per second for the entire area and the maximum run-off was 2.5 cubic feet or 5.4 per cent. The entire storm lasted $3\frac{1}{2}$ hours, and the ratio of the entire volume of run-off to the entire rainfall was 19.3, the run-off, however, lasting sixteen hours.

In New York City in 1888, gagings * by Rudolph Hering were made of the run-off from a district in the lower part of the city containing 221 acres. The time of concentration was probably about forty-five minutes. The percentages of relatively impervious area were as follows:

Roof.	Paved Area.	Grass Area.
43.5	46.5	10

That is, assuming 70 per cent of the paved area and 20 per cent of the grass area to be relatively impervious, 78 per cent of the rainfall should reach the sewer. The actual percentages reported range from less than 10 to 75 per cent. A percentage of 65 was common, and one rain following a snow storm appeared to discharge even more than 75 per cent. It is, however, worth noting that the greatest flow per second in the sewer was due to a very heavy thunder shower, lasting thirteen minutes, 38 per cent of which gave the maximum sewage flow recorded.

In applying the relation to cases where greater densities of population occur than are given in Table X, it must be remembered that the rate of increase of the reduced imper-

* Trans. Am. Soc. C. E., Vol. LVIII, p. 464.

vious surface diminishes until that surface reaches a limit of 80 or 90 per cent, corresponding to a density of about 75 persons per acre. Beyond this density there can be no material increase of such surface, since then the whole available area becomes covered with pavements and buildings, and any additional population is accommodated by crowding more persons into the houses. It is also proper to remark that the figures given refer only to certain average urban conditions and are therefore subject to such modifications as may be appropriate under different conditions. For example, in a rapidly growing suburban village the amount of water delivered from the surface twenty years hence may be very different from what the present indications would show. The measured amounts of water, in the case of Rochester, served to check the assumptions made, and have shown that they are very near the truth, so that there can be no doubt that the method as given will furnish, except under very exceptional conditions of building or surface, results nearer the truth than can be obtained in any other way.

PROBLEMS

26. Assuming 5 persons per house, determine from a count of the number of houses in a given district the number of persons per acre. Pace the distances needed to determine the area.

27. By noting where additional houses might be built and where business blocks might be put, estimate the possible future density of population of this same district.

28. On a given area of 240 acres, 50 acres are under roof, 36 acres are brick pavement, 15 acres are stone block, 12 acres are bituminous macadam, and 60 acres are hard earth streets and yards. What is the percentage of reduced fully impervious surface in percentage and what population per acre does it correspond to?

29. In a part of Ithaca, population 26.2 per acre, the lots are 66′ × 132′, and the blocks are 10 lots long and two deep (660′ × 264′). Estimate the size of a house and the amount of street surface in a block to compare with Table VIII.

CHAPTER VI

MATHEMATICAL FORMULÆ

WE have seen how the amount of rainfall to be provided for in a sewer depends on the rate of rainfall and on the duration of the storm; that this amount is an uncertain quantity, and that its value is generally made to depend more on a long-established custom than on any experimental certainty. We have seen that the rate of rainfall may vary from the least dampness through rates of an inch per hour, which is the rate usually given in the text-books, up to 4, 5, or even 6 inches per hour. We have further seen that the maximum rate is a function of the length of the storm, and that it is not possible to make a determination of a rain rate unless the length of the storm considered is also known. It has been pointed out that while high rates are usually only for short periods, they may nevertheless be more troublesome than a more moderate rain lasting a longer time and yielding a larger volume. We have seen that the period of time adopted as a unit is of importance for calculating the intensity of the storm, and that the size of the district from which the run-off is to be determined governs the choice of this period. The fact that the condition of the ground, its slope, porosity, degree of saturation, all have an influence on the proportion of rainfall furnished to the sewers has also been pointed out. And the evident conclusion is that there is a wide latitude for judgment, that it is not possible to make a design with the precision used in other engineering constructions, but that the size of the storm-sewer can only be properly designed by carefully considering the district to be served, and by basing the judgment, which must be used, on as thorough an acquaintance with the district as possible.

In spite of all the uncertainty as to the data of the problem, various attempts have been made at different times to express in mathematical terms the relation existing between the rainfall, the general slope of the surface, the drainage-area, and the storm-water discharge, but experience has proved them all more or less unsatisfactory. Could the coefficients of these formulæ be well known by experiment, and then could they be used by the same investigator on similar territory, doubtless the results would be sufficiently accurate; but the coefficients are made by one engineer and their values used by another, whose knowledge of the original conditions can be at best very limited. As to the density or character of the district, often nothing more is known than that the territory is "urban."

The best-known formulæ are those of Hawksley, Bürkli-Ziegler, Adams, and McMath. The following analytical comparison is taken from a lecture by Emil Kuichling delivered before the Association of Civil Engineers of Cornell University in 1893.*

Hawksley's formula was probably established some time between the years 1853 and 1856, and was the result of an endeavor to find the relation existing between the diameter of a circular sewer and the other factors above named, on the assumptions of a rainfall of 1 inch per hour, one-half reaching the sewer, with the sewer-grade parallel to that of the street. This formula expressed analytically the relations brought out in a table prepared by John Roe, showing the measured discharges from a number of sewers in the city of London, during and after rain-storms of different intensities and under other different conditions. An intensity of 1 inch per hour was regarded as the maximum for which provision should be made, as rains yielding more than that are exceedingly rare in London. Hawksley considered that this rate was general, and concluded, therefore, that a formula based on the measurements made would serve for any other sewer to be constructed in that vicinity—a fair conclusion, except that it omits any consideration of the character of the soil or of the relative amount

* See also Trans. Am. Soc. C. E., Vol. LVIII, p. 458.

of impervious surface. The formula was first published in this country in the report of James P. Kirkwood on the Water-works of Brooklyn. It was used by Sir Joseph Bazelgette and Mr. William Haywood in preparing the plans for the main drainage-works of London, and has been much used elsewhere both in this country and in England.

In its original form it was

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

or, divested of its logarithmic form,

$$d^{10} = A^3 N 6309574;$$

where d = diameter of sewer in inches;

A = number of acres drained;

N = length in which the main falls one foot, which equals $1/s$, where s is sine of slope,

If $1/s$ be substituted for N, and D in feet for d in inches

$$(12D)^{10} = A^3 \frac{6309574}{s},$$

or

$$D^{10} = \frac{A^3}{9813s} = 0.0001019 \frac{A^3}{s}.$$

This formula is still used by the Borough of Brooklyn except that the term 6.8 in the formula is changed to 8, making

$$D^{10} = .001615 \frac{A^3}{s}.$$

Since the rainfall is assumed to be 1 inch per hour, and since half of it is assumed to enter the sewers, these two factors are really understood, so that if r = the rainfall in inches per hour reaching the sewers, which is equal to the actual rainfall multiplied by some constant, depending mainly on the character of the surface, the substitution of this gives

$$D^{10}=0.0001019r^3\frac{A^3}{s},$$

with $c=\frac{1}{2}$, and $r=1$; but

$$Q=Av=\frac{\pi D^2 v}{4},$$

and

$$v=100\sqrt{Rs},$$

assuming the constant 100: R is the hydraulic radius and s is the slope; but R for a circular pipe flowing full $=D/4$. Therefore

$$Q=\frac{\pi D^2}{4}50\sqrt{D.s}=39.27\sqrt{D^5 s},$$

whence

$$D^5=\left(\frac{Q}{39.27}\right)^2\cdot\frac{1}{s},$$

or

$$D^{10}=\left(\frac{Q}{39.27}\right)^4\cdot\frac{1}{s^2}.$$

Equating this value of D^{10} with that from the formula given above,

$$\left(\frac{Q}{39.27}\right)^4\cdot\frac{1}{s^2}=0.0001019\frac{A^3r^3}{s},$$

or

$$Q=3.946Ar\sqrt[4]{\frac{s}{Ar}},$$

which is a modified form of the Hawksley formula.

Adams, on the ground that experience showed that, while this formula was sufficiently satisfactory for small districts, it gave sewers of inadequate dimensions in the case of larger areas, proposed a modification of the ordinary formula for flow in pipes in order to secure a satisfactory capacity for all sizes.

Taking the formula as deduced above,

$$D^5 = \left(\frac{Q}{39.27}\right)^2 \cdot \frac{1}{s} = \frac{Q^2}{1542.s},$$

he changed the exponent of D from 5 to 6 in order to get a larger value for the amount of run-off.

Then substituting $\frac{A}{2}$ for Q, on the assumption that $\frac{1}{2}$ of a precipitation of r, $=1$ inch per hour, will reach the sewer during this period of time, he has

$$D^6 = \frac{A^2}{6168.s}, \text{ or } D = \sqrt[6]{\frac{A^2}{6168.s}}.$$

For any other value of r than unity, $\frac{Ar}{2}$ would have to be substituted for Q, giving

$$D = \sqrt[6]{\frac{A^2r^2}{6168.s}}.$$

But for the flow in the conduit, as above,

$$D = \sqrt[5]{\frac{Q^2}{1542.s}};$$

and equating the two values of D,

$$Q = 1.035Ar\sqrt[12]{\frac{s}{A^2r^2}}.$$

Bürkli-Ziegler published in 1880 a paper on the discharge of sewers,* and in it proposed a variation of Hawksley's for-

* Introduced into this country in 1881 by Rudolph Hering in his classic report to the National Board of Health.

mula, to allow its use under other conditions than those of the London districts. In French units his formula was

$$q=cr\sqrt[4]{\frac{S}{A}};$$

where q = volume of storm-water (in liters) reaching the sewer per second from each hectare of surface drained;
c = constant varying with the character of the surface;
r = average rainfall in liters per hectare per second during the heaviest fall;
S = general fall of the surface per thousand;
A = area drained in hectares.

Bürkli-Ziegler recommended that for ordinary conditions c be made 0.60 for thickly populated urban districts and 0.25 for suburban ones, with an average value of 0.50, and that the maximum rainfall assumed be taken at 125 to 200 liters per hectare per second.

One liter per hectare per second equals 0.0143 cubic foot per second, so that the rainfall corresponding to 125 to 200 liters per hectare = 1.79 to 2.86 cubic feet per acre per second, or rainfalls of 1.79 and 2.86 inches per hour.

Transforming the whole formula into English units, reading Q in cubic feet per second per acre, r in inches per hour, A in acres, s for S, and making, by definition, $s=S/1000$, we have,

$$Q=c\ 7.05r\ .0143\sqrt[4]{\frac{s}{A}};$$

the values of c corresponding to 0.25 and 0.60 will be in English measure 1.76 and 4.23, and for the mean 3.52, so that the formula in English, if Q = the total discharge, is

$$Q=cA\cdot r\cdot\sqrt[4]{\frac{s}{A}},$$

where c has the values just given, and r is taken at values of 1.79 to 2.86 inches per hour.

In 1887 Robert E. McMath of St. Louis published in the Transactions of the Am. Soc. C.E.* a paper on the necessary size of sewers to discharge the run-off from the excessive rains of St. Louis, and deduced a formula which by actual experience was so framed as to answer every purpose for that city. It was derived by observing, during periods of excessive rains, the sewers which were overcharged, and plotting them as points on a diagram whose abscissæ were the areas drained in acres, and whose ordinates were the calculated capacities of the sewers, computed by Kutter's formula. By drawing a curve that should pass above these points of surcharge and below or among the other plotted points taken from sewers of known capacity, the constants and coefficients for the curve were used as those to represent the run-off to be expected. The equation of the curve taken was.

$$Q=0.75\times 2.75\sqrt[5]{15A^4},$$

Q being the quantity of water reaching the sewer in cubic feet per second, and A the area drained. In symbols it would be

$$Q=c'\,r\cdot\sqrt[5]{SA^4},$$

where c' is the proportion of the rainfall reaching the sewers, after making the proper allowance for evaporation, absorption, and retention. The value taken at St. Louis, probably for the built-up part of the city, was 0.75. The symbol r stands for the number of cubic feet of water falling on an acre per second, or practically the rainfall in inches per hour. It was assumed by Mr. McMath to be 2.75 inches per hour. s is taken as the mean surface-slope in feet per thousand, and in the diagram is made 15. The form of the Bürkli-Ziegler formula was taken, and if S be changed to s, whence S equals $1000s$, so that $c=c'/1000$, it will be comparable with the others. Mr. McMath

* Vol. XVI, p. 179.

adds that the improvement over the Bürkli-Zeigler formula lies in the fact that the latter, based as it is on observations of small areas, is inapplicable to districts containing 1000 acres or more, while the St. Louis coefficients make the formula good to 10,000 acres.

In a report to the city of Baltimore by the Sewerage Commission (1897) is a report by Rudolph Hering and Samuel M. Gray, Consulting Engineers. The four formulæ given above are discussed therein, together with a fifth deduced from diagrams prepared for the Department of Public Works of New York in 1889. This discussion is as follows (the formulæ are here repeated for convenience):

Hawksley: $Q=c\cdot A^{\frac{3}{4}}r^{\frac{3}{4}}s^{\frac{1}{4}}$; for $r=1$, $cr=3.95$.
Adams: $Q=c\cdot A^{\frac{5}{6}}r^{\frac{5}{6}}s^{\frac{1}{12}}$; for $r=1$, $cr=1.03$.
Bürkli-Zeigler: $Q=c\cdot rA^{\frac{3}{4}}s^{\frac{1}{4}}$.
for $r=2.75$, $cr=11.61$ for built-up areas;
$cr=9.59$ for average city areas;
$cr=4.79$ for rural areas.
McMath: $Q=cr\cdot s^{\frac{1}{5}}A^{\frac{4}{5}}$;
for $r=2.75$, $cr=8.21$ for built-up areas;
$cr=3.39$ for suburban areas.
N. Y. diagrams: $Q=cr\,A^{.85}s^{.27}$;
$cr=10.59$ for completely built-up areas;
$cr=8.97$ for well-built-up areas;
$cr=6.59$ for suburban areas.

Rainfall.—Assuming all the factors except the run-off and the rainfall to remain constant, the formulæ become:

Hawskley: $Q=\text{const.}\times r^{.75}$.
Adams: $Q=\text{const.}\times r^{.83}$.
Bürkli-Ziegler: $Q=\text{const.}\times r$.
McMath: $Q=\text{const.}\times r$
N. Y. diagrams: $Q=\text{const.}\times r$.

Hering and Gray say: "There is hardly a question that, all other factors being equal, the run-off from such small areas as are considered for city drainage should vary directly with the rainfall in all cases of heavy storms, and also for short periods if absorption and evaporation can be neglected. Therefore, as these assumptions can generally be made for city work, the three latter formulæ, which have a direct variation with the rainfall, are preferred.

Slope.—" When the maximum rate of fall does not cease before the run-off from the entire area has reached its lowest point, then for this area the run-off will be independent of the slope. But when the maximum rate ceases before this takes place, the slope will have a decided influence upon the amount of water accumulated. The greater the slope of the surface, that is, the steeper the territory, the more rapidly will the water run off and accumulate along the lowest lines. It is not practicable at this time to state how large the area must be before the variation of the slope should be considered. It depends upon the maximum rate of rainfall, upon the steepness of the area, and upon other local conditions. Assuming that the run-off increases with the slope, what is the ratio between these two quantities? If all factors except these two are assumed to be constant, then the ratio in the different formulæ is shown as follows:

Hawksley:	$Q = \text{const.} \times s^{.25}$.
Adams:	$Q = \text{const.} \times s^{.083}$.
Bürkli-Ziegler:	$Q = \text{const.} \times s^{.25}$.
McMath:	$Q = \text{const.} \times s^{.20}$.
N. Y. diagrams:	$Q = \text{const.} \times s^{.27}$.

" The exponent showing little variation indicates that there is but slight difference in the formulæ as to the weight attached to the slope, but that the N. Y. diagrams with the largest exponent give it the most importance.

Area.—" The larger the area the greater is the total run-off. But the larger the area the smaller is the run-off per unit of area. This variation is important and demonstrates that a drain taking the water from a large area, say 100 acres, does not require to have ten times the capacity of one taking the water from only 10 acres.

"If it is assumed that all the factors are constant excepting the run-off and drainage-area, then the above formulæ give the following values:

Hawksley:	$Q=\text{const.} \times A^{.75}$.
Adams:	$Q=\text{const.} \times A^{.833}$.
Bürkli-Ziegler:	$Q=\text{const.} \times A^{.75}$.
McMath:	$Q=\text{const.} \times A^{.80}$.
N. Y. diagrams:	$Q=\text{const.} \times A^{.85}$.

"From this it is seen that the coefficients fail to show any great difference in the formulæ.

"All the formulæ have the form

$$Q=c\cdot r^x A^x S^x.$$

"From what was said above, the only formulæ giving any other exponent than unity to r are those of Hawksley and Adams, and it is as well to ignore such variation. Therefore the preferred formulæ have the form

$$Q=c\cdot r\cdot A^x S^x.$$

As they are practically derived independently of a knowledge of the exact maximum rainfall, we may substitute for $c\cdot r$ the one value C and write

$$Q=CA^x S^x."$$

In the Bürkli-Ziegler formula we may therefore write, for the greatest storms, values for $c\cdot r$ or for C, modifying the numerical values to correspond with the slope in feet per thousand:

$C=11.61$ for built-up areas;
$C=\ 9.59$ for average city areas;
$C=\ 4.79$ for rural or suburban areas.

McMath's formula for St. Louis gives:

$C=8.21$ for built-up areas;
$C=3.39$ for rural and suburban areas.

On the N. Y. diagrams the values are:

$C=10.59$ for built-up areas;
$C=\ 8.97$ for average areas;
$C=\ 6.59$ for rural areas.

In the design of the Walworth Run Sewer in Cleveland, use was made of an original formula:

$$Q=ACR\sqrt[6]{\frac{S^{\frac{2}{3}}}{A}},$$

or in the form of p. 85

$$Q=c\cdot r\cdot A^{.83}s^{.11},$$

a somewhat lower value of the exponent of s, but otherwise very similar to the others. The values of $c\cdot r$ chosen range from 3 to 7, the latter being for the most densely built-up part of the city. Fig. 13 already referred to shows that the form of the equation is not the important factor but rather the value assigned to the constant C. This has already been shown to vary not only with the amount of impervious surface, but with other factors, so that an engineer in one city might properly assume 75 per cent of the rainfall to enter the sewers while in another city, with the same rain intensity and impervious area, 50 per cent might be ample. It is impossible to reduce a question of judgment and experience, such as the assignment of a proper value to C becomes, to the fixity of a mathematical table. The opinion of an experienced engineer on this point should always be sought by a young engineer with no previous experience who is designing storm-sewers.

As a further example of work done in this direction, Fig.

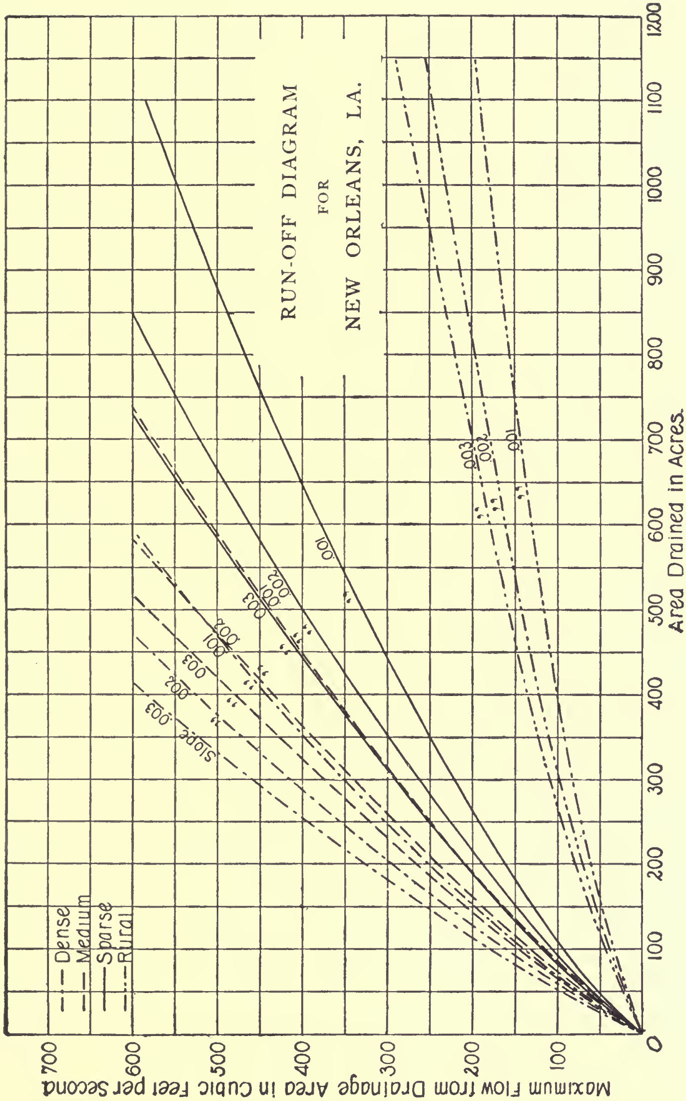

RUN-OFF DIAGRAM
FOR
NEW ORLEANS, LA.
Dense
Medium
Sparse
Rural
Slope .003
Area Drained in Acres.
Maximum Flow from Drainage Area in Cubic Feet per Second

FIG. 15.

15 is given, taken from the report on the drainage of the city of New Orleans, 1895. These curves are based upon the results of the surveys, gagings, and observations made by the city engineer's department under the advice of the advisory board during the years 1893 and 1894, and upon a comparison of these results with those of similar observations in other cities presenting like conditions.

Fig. 16 shows an ingenious arrangement which converts the solution of the McMath formula into a mechanical process. The logarithms of the quantities involved are taken and plotted to form the runner and scale of a slide-rule. The device is the invention of Mr. A. S. Crane, formerly of the Department of Sewers, Brooklyn, N. Y., and has recently been largely used in determining the sizes of storm-water sewers for that city.

By either of the two ways just outlined, viz., by estimating the probable future population of each district of the city and, by Table IX of Chapter V, noting the percentage of rainfall that may be expected to run off, the rainfall having been determined by the diagrams explained in Chapter III; or else, more quickly but less intelligently, by using one of the formulæ or diagrams of this chapter, the amount of storm-water to be cared for by the sewer can be found. In the report already alluded to, Mr. McMath shows that, according to the experience at St. Louis, the Bürkli-Ziegler formula gives, except in the case of small areas, insufficient amounts. Comparisons might be made in a similar way for all the formulæ and diagrams extant, but as each has been made to accord with some special data, a discrepancy only shows that the amount of run-off varies in different cities and localities. From the method of construction, the formula of Mr. McMath must give the best possible results for St. Louis, and similar formulæ might be built up for other cities having an equally long sewer experience. Excepting only the use of a formula made up in the manner of that for St. Louis, no method can give as intelligent and reliable results as that detailed in Chapter V.

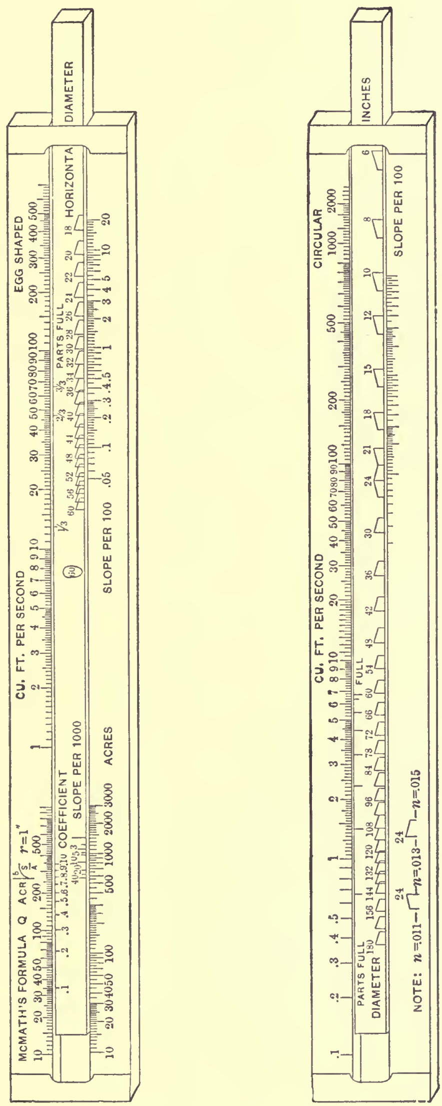

DIAMETER
MCMATH'S FORMULA
EGG SHAPED
CU. FT. PER SECOND
PARTS FULL
HORIZONTAL
COEFFICIENT
SLOPE PER 1000
SLOPE PER 100
ACRES
INCHES
CIRCULAR
CU. FT. PER SECOND
PARTS FULL
DIAMETER
SLOPE PER 100
NOTE: n=.011 n=.013 n=.015

FIG. 16.

Before considering the relation between the amount of water finally determined on and the resulting size of the sewer, other sources of sewage are to be considered.

PROBLEMS

30. Show that by changing 6.8 to 8 in the Hawksley formula, the value of D^{10} becomes $.001615\frac{A^3}{S}$.

31. Show that the numerical value of the constant in the Bürkli-Ziegler formula is changed from 0.60 to 4.23 by substituting English units of measure.

32. Taking the five formulæ of p. 85, plot curves for each equation, taking areas up to 1000 acres for abscissæ and run-off in cubic feet per second per acre for ordinates.

33. Determine the probable run-off from some definite district or area. Use all the methods and compare results and make a final decision in the light of all the evidence.

34. Show by numerical explanation how the constant of a formula representing the method of Chapter V, viz., $Q=C\cdot v\cdot A$ can be reconciled with the constants of Chapter VI.

35. Show by a diagram how much effect the term $S^{.25}$ has in the mathematical formula, that is, plot two curves with run-off per second per acre for ordinates, and values of S for abscissæ, with $A=500$, and $=50$ acres, and with assumed values of C and r.

36. Plot curves from the 5 formulæ of p. 85, with run-offs as ordinates and areas as abscissæ in order to show the effect of the exponent of A. Take a fixed value for $C\cdot r$, and the values of S, i.e., $S=.001$, $S=.005$ and $S=.05$.

CHAPTER VII

ESTIMATING FUTURE POPULATION

THE amount of storm-water reaching a sewer, and the consequent size of the sewer, bear only an indirect relation to the population on the area drained, but the number of people in a given district is a direct function of the amount of domestic sewage to be cared for. In order to determine, therefore, the amount of house-sewage which a system of sewers must carry, it is primarily essential to determine the population on the area to be sewered.

The number of persons on a given area may be approximately determined at any time in several ways. The U. S. Census reports, published every ten years, furnish a basis for an estimate of the population for intermediate years, but as a sewer system has always to be designed for use during an indefinite number of years in the future, some method of predicting the population for that future time must be devised. It is usual to base the prediction on two things. First, after noting the past growth of the city in question, it is assumed that it will continue to increase regularly according to the law of its past. Thus in Chicago, at the time of the first report of the Sanitary Commission, the future population of the Sanitary District was estimated in this way, as shown in Fig. 17. Curves were drawn for other large cities and used as a guide, but they proved of little value. Messrs. Hering and Gray used the same method in their Baltimore report as shown in Fig. 18. They had recourse to other sources of information besides the U. S. Census reports; the police estimates of population, made every year, and obtained by multiplying the voting population by a constant, were made the basis of the prediction quite as much as the more authenticated U. S. reports.

The values of these predictions can now be tested and shown to be reasonably accurate.

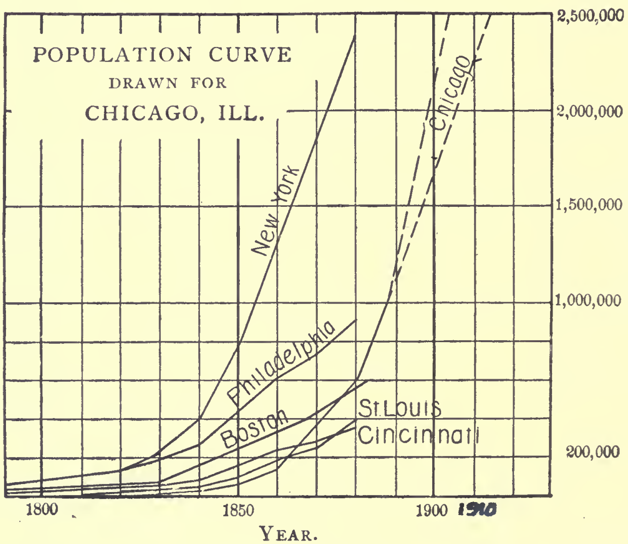


FIG. 17.

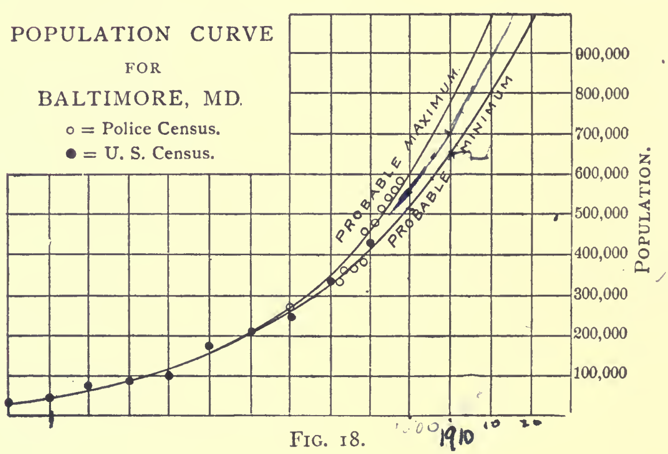


FIG. 18.

The census population for 1900 and for 1910 for Chicago was 1,698,575 and 2,185,283, while the lower branch of the curve predicts for those years populations of 1,690,000 and 2,250,000. For Baltimore the agreement between prediction and census are equally good. The census figures for 1900 and

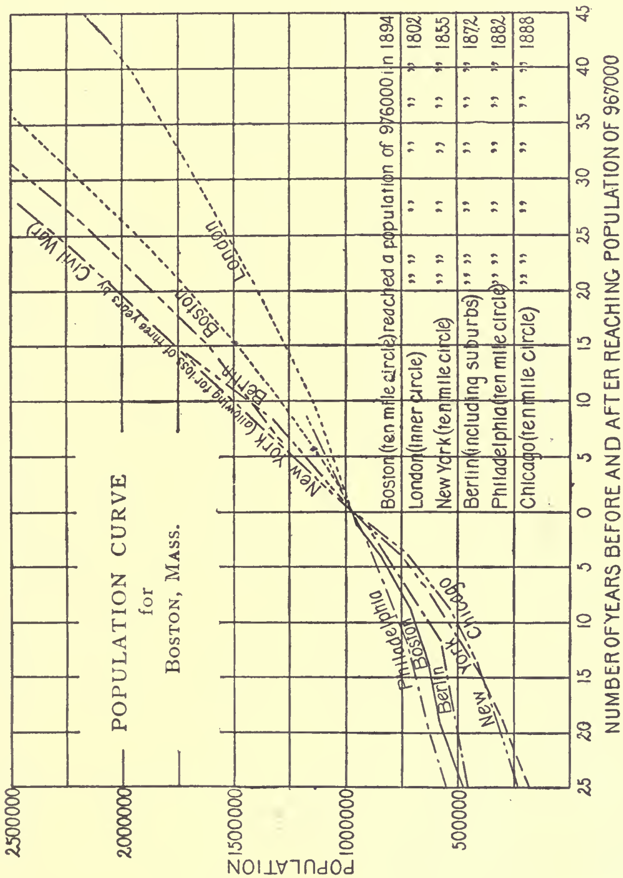


Fig. 19.

1910 are 508,957 and 558,485 ~~670,585~~ and the predicted values, scaled from the diagram, are 520,000 and 650,000.

The other method assumes that the city in question is like other cities of the same size as regards its rate of increase, and that it will follow, approximately at least, the same law. This method was followed in Appendix No. 1 to the report of the

Chief Engineer of the Metropolitan Water-supply made to the State Board of Health of Massachusetts in 1895, and is described as follows (see Fig. 19). First, ignoring the city limits and taking the metropolitan area within 10 to 15 miles radius from the centre of business, the U. S. Census for Boston was found to give 269,754 population in 1850, gradually increasing to 844,814 in 1890. Then, by using partial and incomplete censuses, such as assessed polls, names in directory, enumeration of school-children, and making a compilation of other statistics which indicate, to some extent, the growth of communities, such as the number of buildings erected and the number of water services added, and comparing these quantities with the known population in census years, it was possible to obtain the population of the district for the years 1891–1894 with much greater accuracy than could have been done by projecting ahead the previous rate of growth. In this way the probable population for 1894 was found to be 967,000. On the diagram given, which is taken from the report mentioned, are plotted population curves of Boston and five other cities, with five-year spaces for abscissæ and population for ordinates, and all the curves are so placed as to coincide at a point corresponding to a population of 967,000 on each. Philadelphia and Chicago are of little value in showing the tendency of the curve, but London, Berlin, and New York show the rate of growth of those cities beyond the point where they had Boston's population, and by assuming that Boston's future growth would be influenced by no tremendous shock of pestilence, war, or business disaster its population line was drawn to follow approximately these other cities.

The check which the 1910 census has given to this work shows it to have been surprisingly accurate. Thus in 1910, sixteen years after the diagram was made, the census showed a population in this district of 1,520,470, while the diagram gives 1,500,000.

The same method was followed in the smaller city of Brockton, as found in the Report of the Sewerage Commission of 1893 prepared by the engineer, Mr. H. F. Snow (see Fig. 20). Here all cities in the United States reaching a population of 27,000 between 1851 and 1870 were plotted, together with the past records of Brockton, whose population in 1890 was 27,294.

The growth of all these other cities being plotted (some extending for 40 years), and making due allowance for natural advantages possessed by some cities and not by Brockton, and giving due weight to the municipalities existing under the same

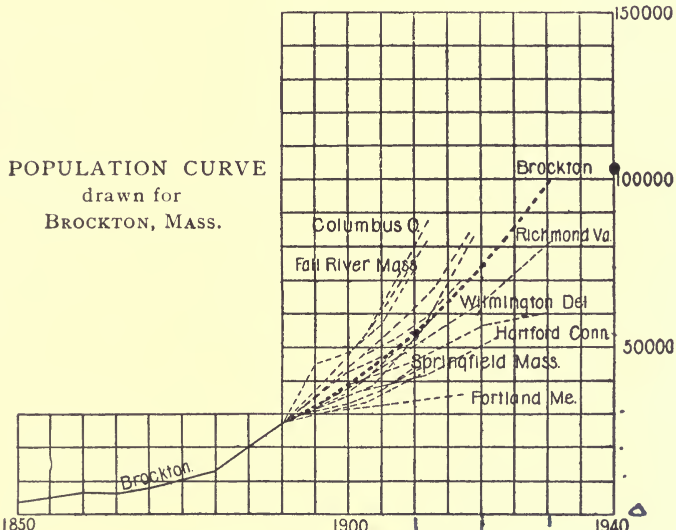


FIG. 20.

conditions as nearly as could be, the probable future population of Brockton was obtained.

The actual growth of this city has been more than was expected, so that the discrepancy is greater in this case between the prediction and the reality than in any of the other cities noted. Thus the diagram indicates in 1900 and 1910, populations of 39,000 and ~~47,000~~ 57,000, while the census gives values of 40,063 and 56,878.

Rafter and Baker in a discussion of this subject give some tables taken from Census Bulletin No. 52, showing the increase in population during the years 1881–1890 for cities of 8000 to 50,000 inhabitants, and also for cities of over 50,000 inhabitants; and while the increase for the first series varies from 4 to 267 per cent, and for the second from 7 to 360 per cent, they conclude as a rapid generalization, first, that in American towns having a population less than 50,000 the present rate of increase may be taken at about 100 per cent in from 15 to 20 years; and second, that in the larger towns the increase will be about 50 per cent in the same time. They further say that

Table XII

TABLE SHOWING RATES OF INCREASE IN POPULATION FOR CITIES OF DIFFERENT SIZES

When the Population is	Number of Cities Considered in Deriving Average Rate of Increase.	Range of Rates of Annual Increase.		Average Annual Rate of Increase in Per Cent.	
		Maximum Per Cent.	Minimum Per Cent.	Average of all the Different Values.	Probable Average Value.
10000	9	30.50	6.50	14.82	....
20000	15	24.20	4.10	11.17	....
30000	19	18.00	2.40	8.34	....
40000	20	15.50	2.60	6.45	....
50000	20	13.00	2.35	6.05	6.05
60000	15	10.40	1.40	5.50	5.60
70000	13	9.10	3.00	5.57	5.30
80000	12	8.30	2.10	4.95	5.03
90000	11	7.95	1.10	4.80	4.85
100000	10	7.30	2.35	4.93	4.66
110000	9	8.25	2.80	5.21	4.52
120000	7	6.40	3.10	4.38	4.40
130000	5	6.05	3.10	4.37	4.26
140000	5	5.75	3.07	4.30	4.15
150000	4	5.65	3.43	4.62	4.04
160000	4	6.00	3.40	4.51	3.93

analyses of 400 towns given in the Census bulletin referred to above show that about 25 per cent have doubled in the decade 1880–1890, and that the towns showing this large increase are situated in all parts of the country, many of them in the older settled States where fixed conditions may be supposed to have been reached. In the case of towns of over

50,000, the number increasing from 50 to 100 per cent is smaller, only 14 per cent of 56 towns given increasing more than 100 per cent.

Mr. Kuichling extends this study farther, and suggests that the rate of increase is so well fixed to correspond with the size of town that the relation once established may in most cases be used to predict the future growth of any town of known size. He bases the relation which he believes to exist on a

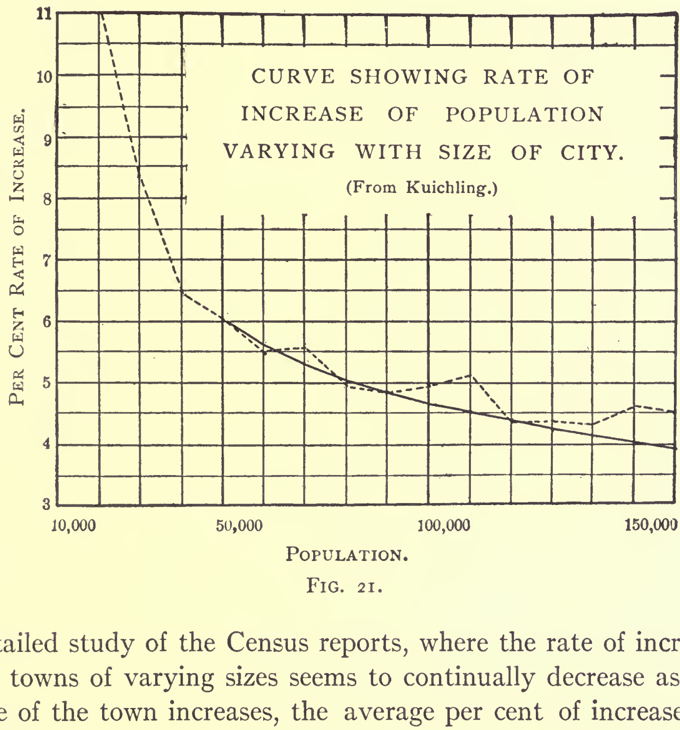


FIG. 21.

detailed study of the Census reports, where the rate of increase for towns of varying sizes seems to continually decrease as the size of the town increases, the average per cent of increase for towns of the same size agreeing very closely. The preceding table is taken from his report as compiled from the U. S. Census of 1880, and shows average rates of annual increase for cities of the United States.

The table shows very plainly (see Fig. 21 for the graphical representation) a law of decrease in the annual rates as the size of the city increases, which law, as shown in the report, holds if cities of from 160,000 to 900,000 be included in the comparison. As a check on the law and for comparison, the author has taken the Census report for 1910 and computed the rates of increase in a similar manner for cities of between 10,000 and 200,000, with results as shown in Table XIII and in Fig. 22.

TABLE XI

TABLE SHOWING RATES OF INCREASE IN POPULATION FOR CITIES OF DIFFERENT SIZES

When the Population is	Number of Cities Considered in Deriving Average Rate of Increase.	Range of Rates of Annual Increase.		Average Annual Rate of Increase in Per Cent of All the Different Values.
		Maximum Per Cent.	Minimum Per Cent.	
10000	44	19.139	0.288	5.715
20000	53	10.691	0.000	2.714
30000	37	23.223	0.092	4.814
40000	25	20.599	0.405	3.858
50000	16	11.260	1.680	5.652
60000	10	9.785	1.803	3.873
70000	11	7.655	1.846	4.483
80000	5	6.012	1.425	3.741
90000	7	8.119	2.167	4.449
100000	4	4.375	0.648	1.906
110000	4	3.661	1.376	2.885
120000	4	5.006	1.942	2.994
130000	3	2.813	2.368	2.615
140000	1	2.327	2.327	2.327
150000	2	12.427	7.229	9.828
160000	1	2.782	2.782	2.782
180000	1	4.456	4.456	4.456
200000	1	3.416	3.416	3.416

In this latter comparison the law seems to be lost, the increase in the annual rate depending apparently not so much on the size of the city as on other unknown factors. In both diagrams, populations are plotted as abscissæ, and the average annual rate of increase in per cent as ordinates. The broken irregular line shown is obtained by joining the points thus plotted, and if a continuous regular curve be drawn between the points, as nearly as may be, it will represent the probable general law of growth of American cities of the class under consideration, and will give the general percentages found in the sixth column of Kuichling's table. No such curve was drawn for the growth in 1900–1910, as the points plotted were so irregular as to show rather the lack of any general law than the evidence of the law itself. Kuichling makes the diagram give a method of estimating a future population as follows: Take from the diagram

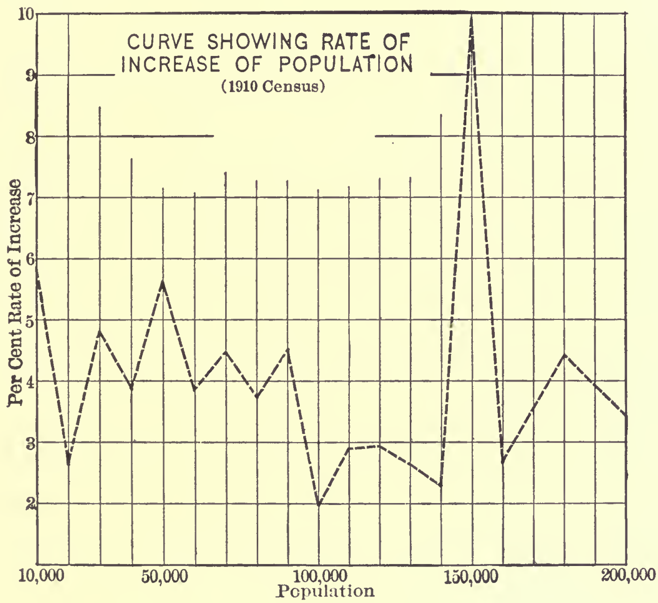


FIG. 22.

or table the rate of increase corresponding to the present popula tion, interpolating if necessary; add that increase to the present population; take the rate for that sum; find the increase corresponding; add the latter to the former sum, and continue this for as many years as desired. The method is rather tedious and gives only the general and probable law, with a result which must be modified by such conditions as the previous rate of growth, locality, facilities for manufacture, and trade would suggest.

This law and the discussion must be used with great caution in the case of any particular city, the general law being often very wide from the truth. From Table XIII it is seen that cities of 10,000 inhabitants increased in population in 1900–1910 from 0.0 to 10.7 per cent per year, while the general law would indicate about 11 per cent. Cities of 30,000, however, apparently may, as a maximum, increase at the rate of 23 per cent. Should the city in question not be an average city, a large error would evidently result from trying to apply the general law.

No law or estimate can be found for new cities such as spring up in the western part of the United States. There may be cited as an example San Diego, which, in January, 1887, when the plans were made for its sewer system, had a population of 5000. In February, 1888, there was a population of 33,000, and by the Census of 1890 the town had a population of 16,129. In 1910 the population was 39,578.

To further illustrate the method of securing an idea of the future population, Fig. 23 is given from a thesis on the sewerage of Ithaca, by Mr. W. E. Truesdell, C.E., Cornell University, 1896. The city population of Ithaca was given by U. S. Census for 1880 and 1890, and there was also available an unofficial census in 1892 which did not, however, check with the other two. The following additional records were consulted and plotted on the same diagram as the Census figures: the maximum vote in city elections for every five years from 1855 to 1897; the yearly public-school registration from 1879 to 1897; the school population from 1871 to 1891. The rate of increas- of the population of the city was taken as the mean of the rates of increase in votes, in school registration, in school population, and in the Census reports, weighting the different records as the peculiar condition seemed to justify. In the figure, the long broken line shows the apparent increase as indicated by the local censuses, while the long heavy line shows the adopted line, modified by the two government censuses of 1880–1890. By Kuichling's method the population in 1920 will be 113,000, while by Mr. Truesdell's it will be only 18,500.

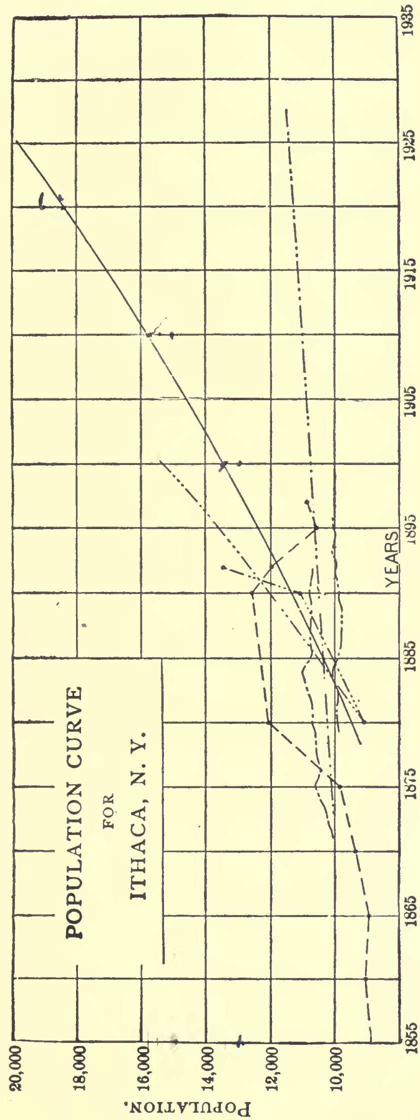

POPULATION CURVE
FOR
ITHACA, N. Y.
YEARS
1855
1865
1875
1885
1895
1905
1915
1925
1935
20,000
18,000
16,000
14,000
12,000
10,000
POPULATION.

FIG. 23.

In the design of sewers for the place, Mr. Hering assumes the future population as 30,000, not stating, however, when this number is to be expected. The actual population by the 1910 Census was 14,802.

The result of this study into methods of forecasting the population of any city at some definite future time is that it is a matter for the judgment of the engineer. That while he may make use of certain auxiliaries, such as census reports for past growth and for the growth of other cities, while he may consult the local reports of growth in various municipal directions, while he may construct diagrams and tables, these are all only aids. The actual determination of the future population must be made by the individual judgment, based and guided by such methods as have been outlined, but modified by an intimate knowledge of the local conditions of situation and enterprise, and of the other often unknown factors which govern the growth of a modern city.

PROBLEMS

37. By reference to the publications of the U. S. Census Bureau, find the average numbers of persons per house in five cities of New York State. Choose cities differing in location, size, and kind of industry.

38. Collect records of population as given by U. S. Census Bureau for years 1860–1910 inclusive for City of Rochester, N. Y., and by plotting the curve of its growth, estimate the probable population for 1950.

39. Take a ...... city directory for 1900, and by comparing the number of names with the census population, get the ratio of names to total population. Multiply the number of names in the 1910 directory by this ratio and compare the result with the census population.

40. Using census populations for 1900 and 1910 for the city of ......, compute the annual ratios, assuming both an arithmetical and geometrical law of increase. Using both ratios determine the populations for 1905 and for 1915.

41. Find the probable population of Elmira in 1950, by comparing with population of Syracuse, Rochester, Cambridge, Lowell, and Newark, after method of Fig. 20.

CHAPTER VIII

AMOUNT OF SEWAGE PER CAPITA

THE probable future population of the city for whose use the sewers are designed being determined, it remains to assume a daily sewage-flow per capita, with such variations from hour to hour from the average flow as may be found to be usual. The amount of sewage contributed per head per day is a quantity variable in different parts of the country and in different cities, depending on the variation in the water-supply, and it has been customary in this country to assume that the daily water-supply of a place is all converted into sewage, and that a determination of the amount of sewage is made when the amount of water-supply is found. This undoubtedly approaches the truth, although it is more in accordance with sewer-gagings to say that the hourly and daily variation in flow of sewage corresponds closely to that of the water-supply, while the actual amount of sewage is something less. That the records show the volume of sewage always less than that of the water used is partly due to the fact that the houses supplied by city water-works generally exceed in number those connected with the sewers. And, further, since the water-connections precede the sewer-connections, there can never be, as long as connections with either water- or sewer-pipes are being made, an equal flow of water and sewage. Nor can there be any fixed relation between the two volumes until the final number of houses and buildings in a city are supplied with both connections.

Fig. 24 * shows the pumping records of the water-supply, and the relation between the two volumes, at Atlantic City, N. J., for the several months of the year 1892, with the water-

* From *Engineering News*, Vol. XXIX, p. 124.

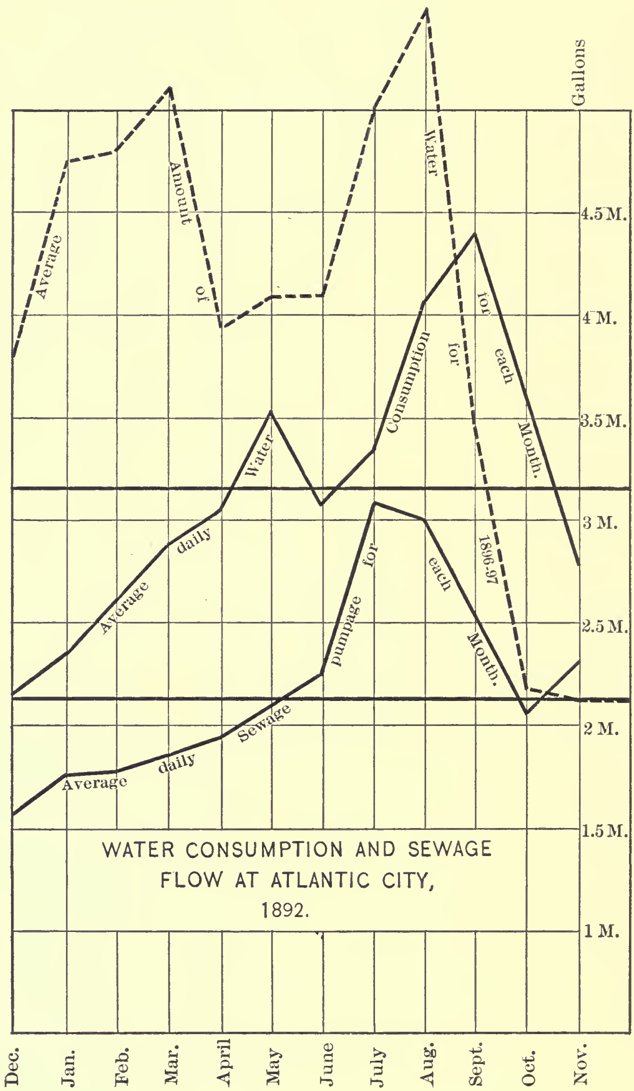

Gallons
4.5 M.
4 M.
3.5 M.
3 M.
2.5 M.
2 M.
1.5 M.
1 M.
Average Amount of Water for 1896-97 for each Month.
Average daily Water Consumption for each Month.
Average daily Sewage pumpage for each Month.
WATER CONSUMPTION AND SEWAGE
FLOW AT ATLANTIC CITY,
1892.
Dec.
Jan.
Feb.
Mar.
April
May
June
July
Aug.
Sept.
Oct.
Nov.

Fig. 24.

consumption for 1896. Fig. 25 shows the two curves for Des Moines, Iowa, for 1895.* Both diagrams show the sewage-flow to be about 35 per cent less than the water-consumption.

The variation in the water-supply of a city is almost incredible, cities of the same size and character often having a difference in daily consumption of as much as 150 gallons per head. To what cause this is due it is hard to say, as there seems to be no law as to the relation between the consumption and the size of the city. Nor does any one cause seem responsible. Probably the largest factor is leakage, caused by poor construction of the main line, and in the house-fixtures, and by carelessness on the part of the house-holder and by neglect on the part of the water-works superintendent in making proper repairs. Any discussion (notably such as have taken place in the New England Water-works and in the American Water-works Associations) on the question of leakage brings out its importance very plainly, and the reports on the various devices for detecting water-waste make their efficiency unmistakable. For example, in his annual report for 1892, Mr. Trautwine mentions that in Philadelphia, out of 782 appliances in 142 houses inspected for waste, 22 were leaking slightly and 32 running continually. The daily consumption per capita for these houses was found to be 222 gallons, of which 192 were wasted, 30 only being used. It is generally in the smaller cities that municipal oversight is most lax, the increased consumption in the larger cities making a total volume of waste so large as to demand investigation; yet this has so many exceptions as to be of little value.

The following tables are given to show what amounts of water are actually used per head per day in typical cities of the United States. Most of the figures given are derived from the records of pumping-plants, because actual measurements of the amount of water used in gravity supplies are very few. From the meager data available there seems to be no reason to believe that the fact and cost of pumping offers

* The data supplied through the kindness of Professor Marston, Ames, Iowa.

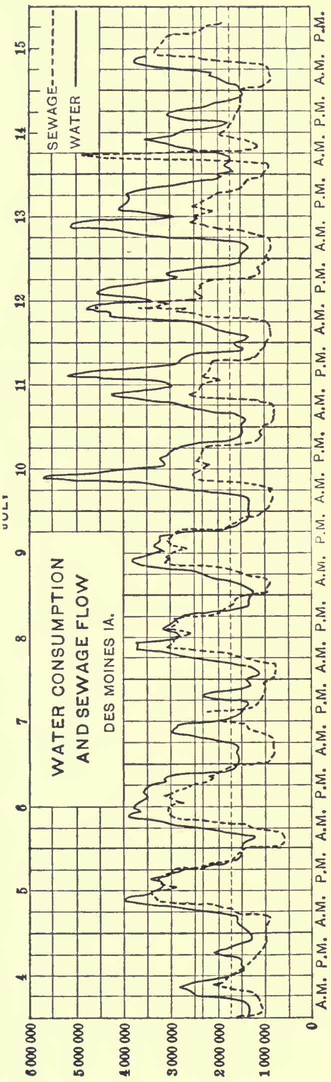

WATER CONSUMPTION
AND SEWAGE FLOW
DES MOINES IA.
SEWAGE
WATER
6 000 000
5 000 000
4 000 000
3 000 000
2 000 000
1 000 000
0
4
5
6
7
8
9
10
11
12
13
14
15

Fig. 25.

any restriction on the unlimited use of water, even though the use of each additional gallon of water means additional expense. The first table (Table XIV), is based on a report by the *Municipal Engineering Magazine* (Vol. XXXVII, pp. 258, 330), with populations from the 1910 Census. Table XV is from a report of Mr. E. C. Bailey, then superintendent of the Albany filter-plant, and represents conditions in 1905. It seems plain from a study of these two tables that there is no relation between the amount of water used and either the size of the city, or its location, but that rather there must be a special inquiry for each city whose water-supply is to be determined. It should also be noted that in many cities, particularly where the water flows by gravity, no method of measuring the flow is provided and the quantity named in the city reports or by the water-works superintendent is little more than a wild guess. If, in addition, the proportion of the population using the municipal supply is uncertain, the per capita consumption may be sadly in error.

As an example of the method of analyzing the probable amount of water to be provided in a given city, the following extract is made from Appendix II of the report by Dexter Brackett on the Metropolitan Water-supply, Massachusetts State Board of Health:

"The water used in any city or town may be subdivided under four heads:

"1. Quantity used for domestic purposes.

"2. Quantity used for trade and manufacturing purposes.

"3. Quantity used for public purposes.

"4. Quantity wasted."

Under the first head should be included not only the amount used for household purposes, but also the quantity required for stores, stables, laundries, and all requirements of a purely residential community.

The following table (XVI), from the report, shows by actual measurement the per capita consumption for purely domestic use by different classes of people in a number of cities.

Table XIV

SHOWING CONSUMPTION OF WATER IN VARIOUS SMALL CITIES OF THE UNITED STATES—1910

Name of City.	Source of Supply.	Population.	Daily Consumption.	Daily Consumption Per Capita.
Petersburg, Va.	Creek, pump, filter	24,127	454,080	19
Pensacola, Fla.	Wells, pumps	22,982	500,000	22
Charlotte, N. C.	Creek, pump	34,014	1,324,567	40
Wilmington, N. C.	River, pump, filter	25,748	1,022,000	40
Tampa, Fla.	Spring, wells, pump	37,782	1,700,000	45
Malden, Mass.	Boston Supply, gravity	44,404	2,019,500	45
Battle Creek, Mich.	Lake, stream, pump	25,267	1,226,000	49
Lincoln, Neb.	Wells, pump	43,973	2,192,000	50
Jacksonville, Fla.	Wells, pump	57,699	2,956,200	51
Gloucester, Mass.	Streams, pump	24,398	1,246,149	51
Superior, Wis.	Wells, pump	40,384	2,226,700	55
Taunton, Mass.	Ponds, pump	34,259	1,903,935	56
Newport, Ky.	River, pump	30,309	1,843,000	60
Madison, Wis.	Wells, pump	25,531	1,564,085	61
Salem, Mass.	Lake, river, pump	43,697	2,987,000	68
Waltham, Mass.	Wells, pump	27,834	1,929,726	70
South Bend, Ind.	Wells, pump	53,684	4,064,529	75
Everett, Mass.	Boston Supply, gravity	33,484	2,592,400	77
Rockford, Ill.	Wells, pump	45,401	3,555,000	78
Fitchburg, Mass.	Boston Supply, gravity	37,826	2,900,000	80
Ft. Wayne, Ind.	Wells, pump	63,933	5,131,833	80
Macon, Ga.	River, pump, filter	40,665	3,373,000	83
Columbia, S. C.	River, pump, filter	26,319	2,369,720	90
Springfield, Ill.	River, pump	51,678	4,738,000	91
Quincy, Mass.	Surface water, reservoir	32,642	3,050,000	93
McKeesport, Pa.	Wells, pump	42,694	4,031,281	94
Peoria, Ill.	Wells, pump	66,950	6,806,000	101
Norfolk, Va.	Creeks, pump, filter	67,452	6,952,756	103
Haverhill, Mass.	Ponds, pump, and gravity	44,115	4,651,779	106
Roanoke, Va.	Spring, pump	34,874	4,076,830	117
Chelsea, Mass.	Boston Supply, gravity	32,452	4,091,200	126
Atlantic City, N. J.	Wells, pump	46,150	5,874,000	128
Binghamton, N. Y.	River, filter, pump	48,443	6,313,000	130
Houston, Texas	Wells, pump	78,800	11,000,000	140
Bangor, Me.	River, filter, pump	24,803	3,609,254	145
Auburn, N. Y.	Lake, pump	34,668	6,000,000	173
Newburgh, N. Y.	Stream, pump and gravity	27,805	5,000,000	180
Lewiston, Me.	Lake, pump	26,247	4,900,000	187
South Omaha, Neb.	Omaha Water-works	26,259	4,977,000	188
Muncie, Ind.	River, wells, pump	24,005	5,300,000	221
Anderson, Ind.	River, filters, pump	22,476	5,000,000	222

Table XV

SHOWING CONSUMPTION OF WATER IN VARIOUS LARGE CITIES OF THE UNITED STATES—1905

Name of City.	Source of Supply.	Population.	Daily Consumption.	Daily Consumption Per Capita.
Birmingham, Ala.....	Creek, pump, filters.....	132,683	5,028,900	37
Fall River, Mass.....	Lake, pump............	104,863	3,805,000	36
New Orleans, La.....	River, filter, pump......	287,100	13,820,000	48
St. Paul, Minn.......	Lakes, wells, gr., pump..	163,065	8,337,000	51
Providence, R. I.....	River, pump...........	175,597	10,130,000	58
Worcester, Mass.....	Imp. Res., gravity......	118,421	7,920,000	67
San Francisco, Cal....	Streams, gravity, pump..	342,800	25,000,000	73
Indianapolis, Ind.....	River, filter, pump......	169,164	13,400,000	79
Rochester, N. Y......	Lakes, gravity.........	162,608	13,500,000	83
Milwaukee, Wis......	Lake, pump............	285,300	24,000,000	84
Minneapolis, Minn...	River, pump...........	202,718	18,813,000	92
Newark, N. J........	River, gravity..........	246,070	24,000,000	97
Memphis, Tenn......	Wells, pump...........	102,320	10,000,000	98
Syracuse, N. Y......	Lake, gravity..........	108,374	11,000,000	100
Baltimore, Md.......	Creeks, gravity........	509,000	56,000,000	110
St. Louis, Mo........	River, pump...........	575,200	63,530,000	110
Paterson, N. J.......	River, pump...........	105,171	12,600,000	120
Cincinnati, O........	River, filter, pump......	325,900	39,600,000	121
Boston, Mass........	Creeks and river, pump..	560,900	80,000,000	141
Jersey City, N. J.....	River, gravity.........	206,433	32,020,000	155
Detroit, Mich.......	Lake, pump............	285,700	44,800,000	157
Los Angeles, Cal.....	Springs, gravity........	102,479	17,000,000	166
Pittsburgh, Pa.......	River, filters, pump.....	321,600	54,000,000	168
Washington, D. C....	River, filter, gravity.....	287,700	50,000,000	174
Omaha, Neb........	River, pump...........	102,555	18,000,000	175
Cleveland, O........	Lake, pump............	381,000	66,900,000	175
Columbus, O........	River, filter, pump......	125,560	23,000,000	183
Albany, N. Y........	River, filter, pump......	94,151	18,100,000	192
Philadelphia, Pa.....	Rivers, filters, pump.....	1293,700	287,188,000	222
Buffalo, N. Y........	Lake, pump............	352,400	92,365,000	262

The examples cited in Boston are generally apartment- and boarding-houses, the average number of persons per house being 40. The consumption per capita varied from 59 gallons in the more modern and expensive houses to 16.6 gallons in the cheaper apartment-houses.

Brookline, a wealthy residential suburb with a large number of private stables, conservatories, and lawns, had the large consumption of 44.3 gallons.

Table XVI

CONSUMPTION PER CAPITA FOR DOMESTIC USE IN BOSTON, BROOKLINE, NEWTON, FALL RIVER, AND WORCESTER, AS DETERMINED BY METER MEASUREMENT

City or Town.	Number of Houses.	Number of Families.	Number of Persons.	Consumption Gallons per Family.	Consumption Gallons per Capita.	Remarks.
Boston.....	31	402	1,461	221	59	Highest-cost apartment-houses in the city
"	46	628	2,524	185	46	First-class apartment-houses
"	223	2,204	8,432	123	32	Moderate-class apartment-houses
"	39	413	1,844	80	16.6	Poorest-class apartment-houses
"	339	3,647	14,261	139	35.6	Average of all apartment-houses supplied by meter
"	40	......	1,699	....	46.1	Boarding-houses
Brookline...	....	828	4,140	221.5	44.3	Average of all dwellings supplied by meter
Newton....	490	490	2,450	132.5	26.5	All houses sup lied with modern plumbing
" ...	....	619	3,005	....	6.6	These families have but one faucet each
"	....	278	1,390	34.5	6.9	Ditto
Fall River..	28	34	170	127.5	25.5	The most expensive houses in the city
" .	64	148	740	42.0	8.4	Average class of houses generally with bath and water-closet
Worcester..	....	20,514	90,942	....	16.8	Whole domestic consumption
" ..	....	81	327	80.2	19.9	Woodlan St., best class of houses
" ..	....	37	187	118.1	23.4	Cedar St., best class of houses
" ..	....	93	447	95.0	19.8	Elm St., houses of moderate cost
" ..	....	245	1,104	55.1	12.2	Southbridge St., cheaper houses
" ..	....	229	809	55 0	15.6	Austin St., cheaper houses

In Newton, 490 families, averaging five in a family, had an average consumption of 26.5 gallons per capita. The houses are modern, with every plumbing convenience, but small grounds.

The amounts used in Fall River and Worcester are very much less, partly from the manufacturing character of the cities and the resulting class of residents.

For the future water-supply of Boston the quantity required for domestic use, based on the table and facts above given and on the known local conditions, proportions, and numbers of the various classes of residents, and with due regard for future growth of each class, was assumed to be 30 gallons per capita.

Elaborate studies made in New York City upon the consumption of water in separate residences, seems to indicate that even the Brookline figures may be largely exceeded. Thus in 1900, Mr. J. J. R. Croes was permitted by the owners of 25 residences in Manhattan and of 12 residences in Brooklyn, to measure the amounts of water used in each house. Meters were used to secure the records, but no penalty was attached to the discovery of careless waste or wilful leakage. Table XV *

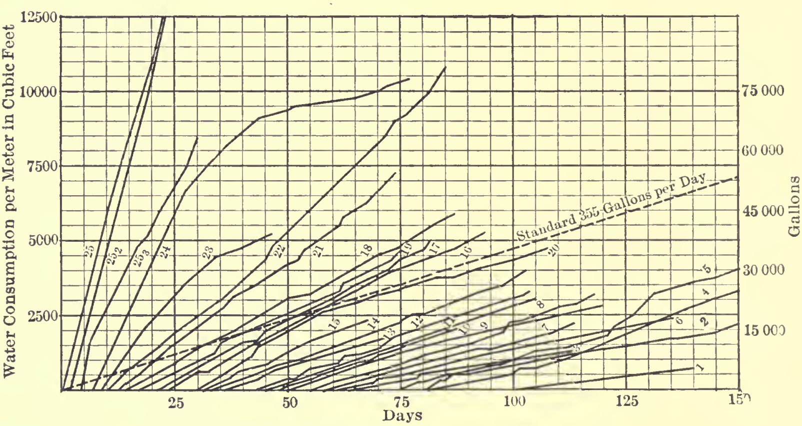
TABLE XV

SHOWING RECORD OF TWENTY-FIVE WATER-METERS IN RESIDENCES IN MANHATTAN. AVERAGE FOR THREE WEEKS IN JANUARY AND FEBRUARY, 1900.

No.	Nearest Street.	Nearest Avenue.	Gallons per Day.	Number of Occupants.	Gallons per Capita.	Baths.	Water-closets.	Faucets.
1	121	Manhattan	138.68	4	34.67	1	2	21
2	73	Boulevard	205.34	4	51.34	2	2	25
3	30	Lexington	205.70	7	29.39	2	3	12
4	88	Amsterdam	221.15	5	44.23	1	2	11
5	38	Third	236.10	5	47.22	1	2	15
6	15	Lexington	243.08	7	34.72	1	2	14
7	40	Park	286.05	4	71.51	2	3	21
8	69	Columbus	318.51	7	45.50	3	3	21
9	84	Eighth	354.35	4	88.59	2	3	25
10	48	"	364.70	6	60.78	2	3	13
11	10	Fifth	369.38	10	36.94	2	3	17
12	18	Third	371.91	8	46.49	1	2	9
13	44	Fifth	387.68	8	48.46	5	6	29
14	8	Greene	399.68	6	66.61	1	4	4
15	55	Sixth	480.21	8	60.03	3	4	13
16	72	Riverside	577.43	11	52.49	2	4	22
17	18	Third	578.21	11	52.57	2	3	7
18	77	Riverside	622.64	10	62.26	3	4	27
Average of 18 houses			353.38	6.95	50 84			
19	121	Lenox	519.33	6	86.56	2	3	23
20	48	Madison	743.68	9	82.63	2	3	28
21	94	West End	914.28	8	114.29	2	3	24
22	71	Boulevard	916.70	9	101.86	3	4	15
23	88	Riverside	1303.76	9	144.86	3	4	16
24	69	Eighth	2351.74	6	391.66	3	4	23
25	Houst'n	First	4521.91	21	215.33	....	3	6
Average of 7 houses			1610.20	9.71	165.75			
Average of 25 houses			705.28	7.72	91.36			

* Report to the Merchants' Association, p. 132.

gives the results of the measurements for Manhattan, and is so arranged that the last seven houses only are guilty of excessive and unnecessary water consumption, as determined by careful inspection of the premises. It will be seen that the per capita consumption varied from 29.39 to 88.59 gallons, the latter according to Mr. Croes being neither abnormal nor excessive. In the last seven houses, the leakage and waste was unmistakable. In No. 20, a leak was found in a flushing-tank, the repairing of which reduced the water used from 82.63 to 38.6



RECORD OF 25 WATER METERS ON RESIDENCES IN MANHATTAN, NEW YORK

FIG. 26.

gallons per capita per day. A similar leak in No. 23 caused a reduction in the use of water from 144.86 to 66.2. In the case of Nos. 24 and 25 water was found to be running continuously through water-closets, and the stoppage of these leaks reduced the flow from 392 and 215 to 73 and 40 gallons respectively. Fig. 26 shows these data, expressed graphically, The heavy line is drawn so that its inclination indicates the average rate of consumption, viz., 50.84 gallons per head per day. The relative inclination of the other line shows thus at a glance the agreement or non-agreement of the various houses. The effect of stopping the leaks in Nos. 23 and 24 is plainly seen, and that in the case of No. 25, the waste which was curtailed for two days was allowed to continue. It is pointed out also that the marked change in rate in the case of No. 5 was due to a visit of a small boy who delighted in sailing a toy steamboat in the bathtub of the house. During the ten days of his stay, he was drawing water for this purpose at the rate of 800 gallons per day, or he was instrumental in changing the per capita rate from 47 to 146 gallons per day.

From this table, Mr. Croes concluded that since the reasonble use, as indicated by the meter readings of the first 18 houses, was 351.9 gallons per house, the amount wasted was the average for the 25 houses, viz., 705.3 gallons less that amount, or 351.9 gallons, practically 50 per cent.

In the large tenement-houses of New York, however, the consumption per head is much less than the above would indicate. Thus the next house to No. 25 above, was a tenement with 36 occupants and the water-consumption was 10.6 gallons per head per day. Another tenement on the west side made from two old residences, had 50 occupants and used water at the rate of 5.6 gallons per day per capita.

The use of water for trade and manufacturing purposes shows a great variation in different communities. Brackett's report gives the actual amounts used in Boston; but without the number and size of the manufacturing industries his figures are of little value. The table is given, however, to show the relative amounts of water used by the different industries.

In New York it was found that the amount of water used in buildings where some business was carried on was remarkably uniform, after the well-known largest consumers had been required to install meters. It was considered safe to estimate that on the average each metered tap in New York would deliver 1450 gallons per day, this amount having been found to hold almost exactly, year after year.

After duly considering all the available data, Mr. Brackett found that the amount used in the Boston Metropolitan District for trade and mechanical purposes was about 25 gallons per capita. But he judged that, in view of the constantly increasing demand for water for these purposes, and also considering that an allowance of about 10 per cent should be made to cover shortage on meter measurements, at least 35 gallons per capita per day should be provided for these purposes. It is, of course, understood that this is applicable only to Boston, and that the amount will vary in different cities. Residential towns, for example, require little beyond that needed for domestic use. Other cities, with industries using large amounts of water, may require more than 35 gallons adopted for Boston. Of

Table XVI

METERED WATER USED FOR TRADE AND MECHANICAL PURPOSES IN BOSTON, CHELSEA, SOMERVILLE, EVERETT, AND CAMBRIDGE IN 1892

Name of Business.	Daily Average in Gallons.	Name of Business.	Daily Average in Gallons.
Offices, stores and shops....	2,458,700	Saloons..................	120,500
Steam railways...........	1,783,400	Laundries...............	91,660
Factories.................	1,414,000	Chemical works..........	87,270
Elevators and motors......	1,337,700	Iron works..............	83,730
Sugar-refineries..........	929,200	Mills and engines........	62,680
Hotels (transient).........	596,200	Marble and stone works..	52,950
Slaughter-houses..........	512,800	Wharves................	39,800
Street railways...........	422,900	Theatres................	36,100
Electric companies........	422,100	Fish stores..............	18,200
Breweries and bottling.....	420,940	Oil works...............	17,250
Gas companies............	355,530	Tanneries...............	16,800
Shipping.................	351,700	Bakeries................	13,030
Stables..................	309,600	Markets.................	12,050
Miscellaneous............	255,000	Distilleries..............	10,780
Restaurants..............	164,800	Greenhouses	9,550

course, the amount of water used in the various industries of a city has no relation to the population, and should be estimated from the amount and kind of manufacturing, although it can afterwards be reduced to a per capita basis for convenience.

For public purposes Mr. Brackett has divided the use of water as follows, the amounts being partly estimated and partly meter measurements:

Public buildings, schools, etc.......	2.30	gals.	per capita
Street-sprinkling..................	1.00	"	"
Flushing sewers....................	0.10	"	"
Fountains..........................	0.25	"	"
Fires..............................	0.10	"	"
	3.75	"	"

Of this amount 4 gallons per capita was allowed for public uses.

The amount of water wasted, that is, ignorantly allowed to escape from the mains and negligently allowed to escape from faucets and leaks, is very large.

A very striking proof that the pumping records do not show the amount of water used is furnished by one of the towns in the Metropolitan District. All the water used in the town was measured by a meter on the supply-main, and every service-pipe has a meter. The works were but four years old, had 18 miles of cast-iron mains, 376 services supplying about 2300 persons, and, with the exception of the water used for flushing mains, street-construction, street-sprinkling, and for fires, all of the water used was measured by the meters on the service-pipes. In 1893 the daily average amount registered by the meter on the supply-main was 128,560 gallons, while the total recorded by the service-meters was 65,380 gallons. Allowing 2000 gallons per day for blowing-off pipes and for fires, there remains 61,380 gallons, or nearly 50 per cent of the whole consumption, unaccounted for. In Newton 46 per cent of the water pumped was not accounted for by the service-meters, after making proper allowances for water not so registered. In Fall River, during the same year, with the most careful system of inspection to prevent waste, 37 per cent of the water pumped could not be accounted for.

In West Orange, N. J., the water company buys all its water by meter from another water company and then sells it by meter to its various consumers. According to Mr. J. H. Fuertes,* the company is never able to account for more than 80 per cent of the water, although by rigid inspections and immediate stoppage of all leaks they are able to keep close to this limit. If a small supply, distributed through only about 30 miles of street mains, with every inducement for both water company and consumer to keep down the leaks, is subject to such a loss, it may be regarded as ideal when 80 per cent of the water-consumption is accounted for. The following table from Mr. Fuertes' report† gives other data, showing how, from the best evidence possible, the amount not accounted for varies from 16 to 43 per cent.

Table XVII

SHOWING PERCENTAGES OF WATER-SUPPLIES WASTED OR NOT ACCOUNTED FOR BY REASONABLE USE

City.	Water Used by Consumers.		Public Uses.	Total.	Amount of Water Supplied	Unaccounted for.	Percentage.	Per Cent of Services Metered.
	Manufacturing.	Domestic.						
Brockton........	5.5	16.6	3.0	25.1	37.1	12.0	32	90
Boston..........	25.0	30.0	2.0	57.0	86.0	29.0	34	
Cleveland.......	40.0	26.0	10.0	76.0	96.0	20.0	21	49
Hartford........	3.0	30.0	5.0	38.0	62.0	24.0	39	99
Harrisburg (1891)	37.0	30.0	5.0	72.0	138.0	36.0	33	about 80
" (1904)	81.0	30.0	5.0	116.0	146.0	30.0	21	about 90
Lawrence.......	11.0	15.0	5.0	31.0	54.0	23.0	42	84
Milwaukee......	45.0	25.0	5.0	75.0	89.0	14.0	16	79
Syracuse........	39.3	31.0	18.0	88.3	108.3	20.0	19	72
Taunton........	14.7	21.5	3.0	39.2	64.0	24.8	39	45
Wellesley.......	0.4	28.6	2.5	31.5	55.0	23.5	43	100
Yonkers........	24.0	20.0	2.0	53.5*	94.0	40.5	43	100
Worcester.......	......	......	......	39.0	68.0	29.0	42	95

* So given by Mr. Fuertes.

By measuring the flow of water through the water-mains

* Report of James H. Fuertes, C.E., on "The Waste of Water in New York and its Reduction," p. 110.

† Pp. 45–47.

in Boston it was found that between 1 and 4 A.M., when little water should be used, there was still a consumption at the rate of from 30 to 35 gallons per capita. In Brookline, where the taps are nearly all metered, from June to December, 1891, the consumption from midnight to 4 A.M. was 44 per cent of the total consumption, or at the rate of 25.8 gallons per capita, and a careful inspection of every fixture only reduced this to 17.7 gallons. Other methods of comparison between the night flow and that used properly for domestic and city purposes led Mr. Brackett to sum up the question of waste as follows:

"That there exists a waste of from 40 to 50 per cent of the total consumption in most cities and towns where meters are not generally used is a fact accepted by those who have studied the question, but it is, I think, the popular idea that this enormous waste can be, and is, almost entirely prevented by the use of water-meters on the services. But the results obtained in the cities and towns where the largest number of meters are in use show that while the consumption per capita is smaller than in unmetered places of the same general character, still a very large proportion of the water supplied by the reservoirs or pumps does not pass through the service-meter."

His conclusion for Boston was that it is not possible even with the use of meters to reduce the waste below 15 gallons per capita; and that if some efficient system of waste-prevention is not adopted, the amount wasted will become, as it is now in some of our large cities, from 30 to 60 gallons per inhabitant.

As the result of this painstaking work, Mr. Brackett concluded that the future water-supply of Boston would need to provide 100 gallons per capita per day for the daily consumption, made up as already indicated: 35 gallons for domestic use, 35 gallons for trade and manufacturing, 5 gallons for public purposes, and 25 gallons for waste, the last amount being taken in view of the uncertainty of securing strict prevention of waste.

A common method of estimating the constant leakage and

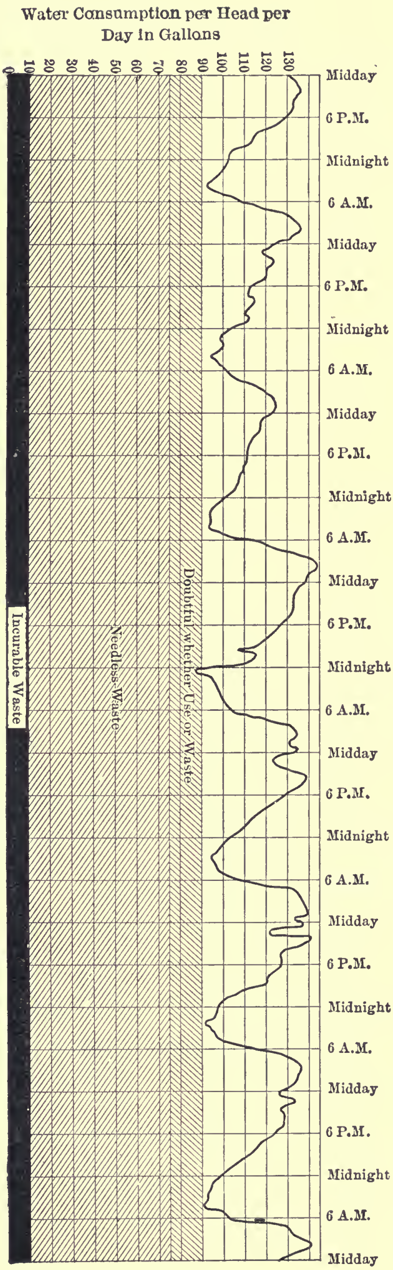


FIG. 27.

waste from a water system is to measure the night flow and assume that all that amount exceeding a small quantity for night use and unavoidable or incurable waste is a constant waste that can be cured if proper measures are adopted. Thus Mr. Freeman in his report to Hon. Bird S. Coler on the water-supply of New York determined the rate of flow from hour to hour, as shown in Fig. 27,* and showed graphically what a large part of the city's water-supply served no purpose, but was altogether wasted. In figures, his conclusion was that at the times when the demands for water were practically zero, that is, between 2 and 4 in the morning, the consumption still existed at the rate of 94 gallons per head per day. Making the liberal allowance of 19 gallons for night use and 10 gallons for incurable waste, Mr. Freeman

* Redrawn from Diagrams Nos. 5 and 6.

concluded that the needless waste in New York City was 75 gallons per head per day. His conclusion seemed to be justified by a comparison of the consumption curves of New York with those of Fall River and Woonsocket, Fig. 28,* the similarity being very marked, except for the constant excess in length of the New York ordinates.

Mr. Fuertes,† on the other hand, believes that a large night flow is a necessary accompaniment of a large, progressive, manufacturing city. He cites the night-consumption in Chicago (165 gallons per capita) for comparison and compares it with a night flow of 106 gallons per capita in a certain district in the Bronx. It was known that in this last, the mains were new, the buildings generally metered, and that this large night flow was due to its use in manufacturing. Mr. Fuertes says that it appears to him that data concerning night-consumption in large cities are really of little practical value in forming a foundation on which to build up demonstrations of excessive wastage.

But there can be little doubt of the value of this sort of study

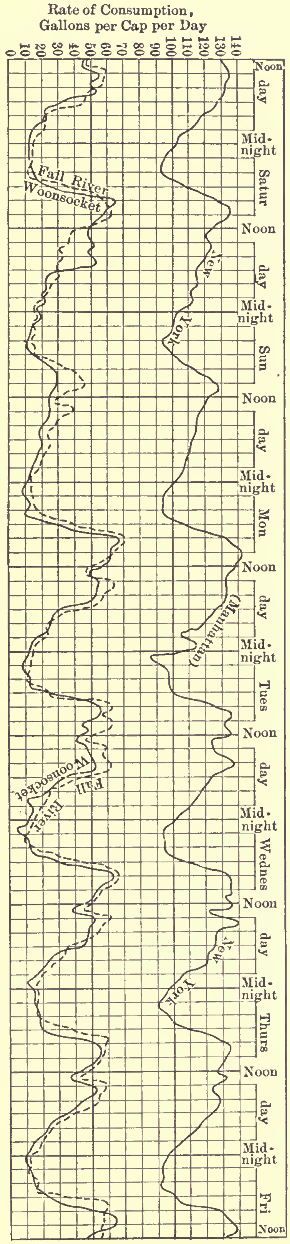


FIG. 28.

* Redrawn from Diagrams Nos. 5 and 6.
† Fuertes' Report, p. 124.

in fixing on the waste in strictly residential cities, and it would seem that if large night flows are due to manufacturing,

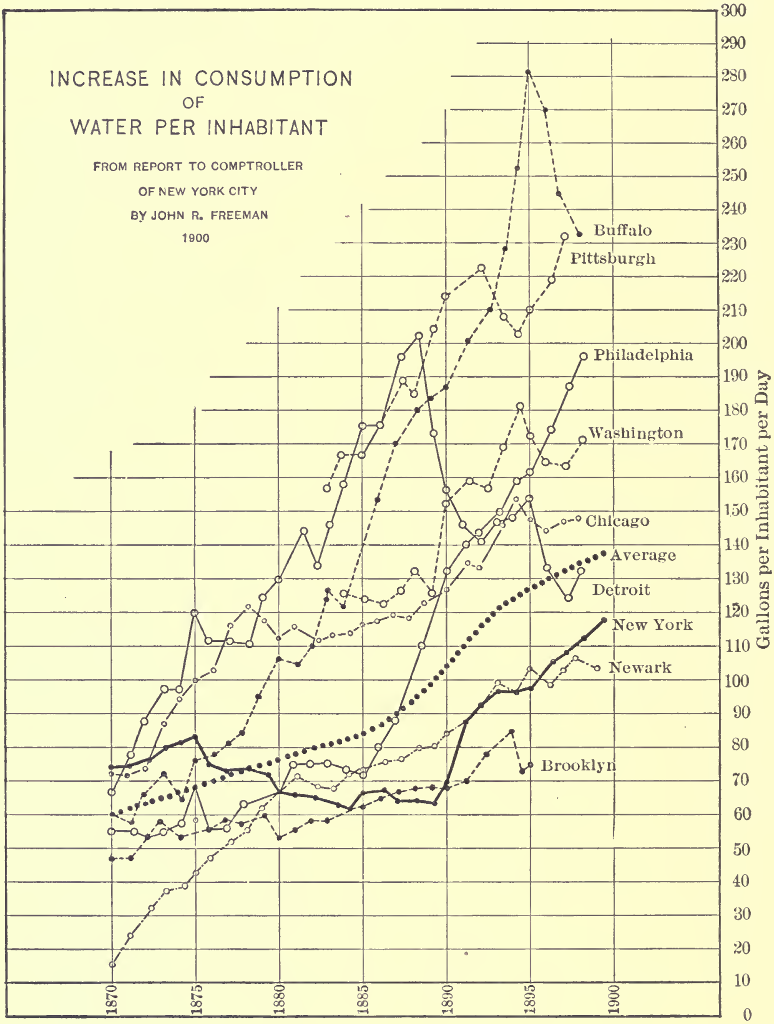


FIG. 29.

such industries as are concerned might be determined and allowed for.

Another factor, affecting the per capita consumption of water, and particularly noteworthy because sewers to carry the waste water are designed for the water-consumption at some distant time in the future, is that there is a tendency for all cities to increase the rate of per capita consumption, as they increase in size. Fig. 29, from the Freeman Report,* shows this tendency unmistakably, although the exact rate varies largely in different cities and in a few cities the law is apparently contradicted. Nor is this inconsistent with the statement already made that there seems to be no relation between the per capita consumption and the size of cities. In any one city, the rate increases as the city grows, and the figure would indicate that, as an average value for large cities, the increase is about 30 gallons every 10 years. Even in cities where meters are generally used, the law holds, though the rate of increase is smaller.

The possible effect of a general introduction of water-meters and the effect of their use on the rate of consumption must be considered also in predicting future water-consumption. To actually say that a city in which water-waste has been abnormal will instal meters and that, as a consequence of this predicted action, the sewers may be made only a fraction of the size otherwise necessary, would not be a safe engineering venture. But the probable result of installing meters may be pointed out and the consequent saving in the cost of building sewers used as an argument in favor of their purchase and use. Fig. 30, from Freeman's Report,† shows in a general way the effect upon the per capita consumption of the use of meters. The curve of the average is not exact, but is drawn as summing up the general tendency and as a convenient guide to the eye in studying the comparative effect of various percentages of meters in the different cities. The effect of meters is unquestionably to reduce the careless waste of water that occurs in about 25 per cent of the house-services. At first, it has a tendency to reduce the reasonable consumption, but

* Diagram No. 4. † p. 70.

such a tendency is soon suppressed, or offset by the effect of the growth of the city. Thus Fall River, as shown by the diagram of Fig. 31,* had in 1874, a per capita consumption

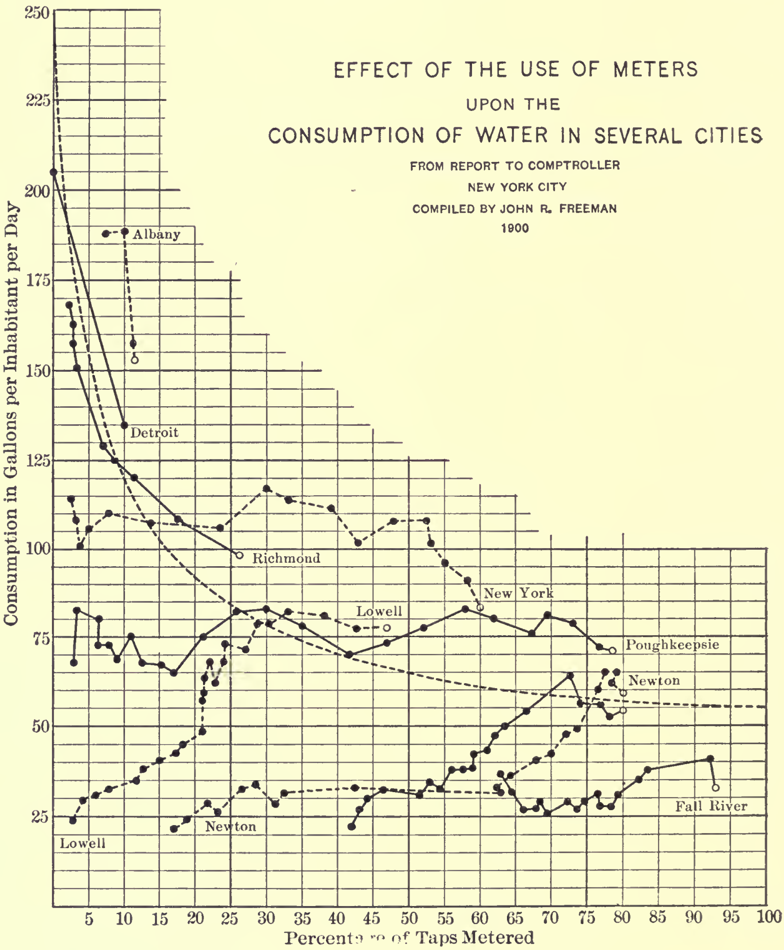


FIG. 30.

of 84.5 gallons. This decreased steadily with the introduction of meters until in 1887, with 70 per cent of the services metered,

* From figures quoted by Mr. Fuertes, p. 155.

the consumption was only 26.9. Since 1887, there has been a slight but nearly constant increase in the rate. Fig. 32,

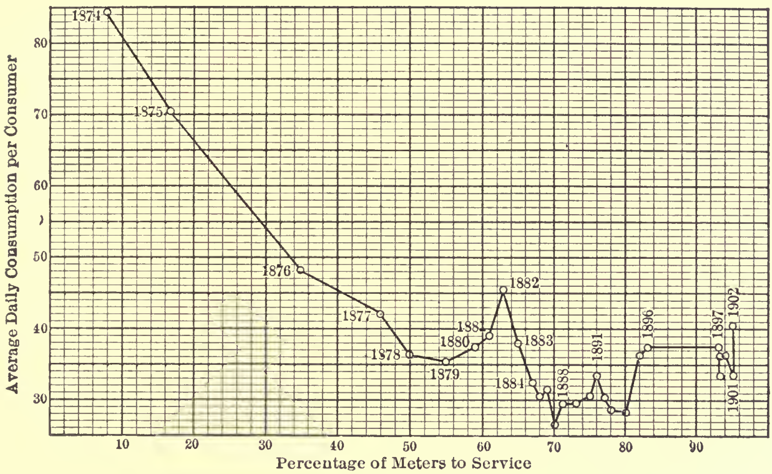


FIG. 31.

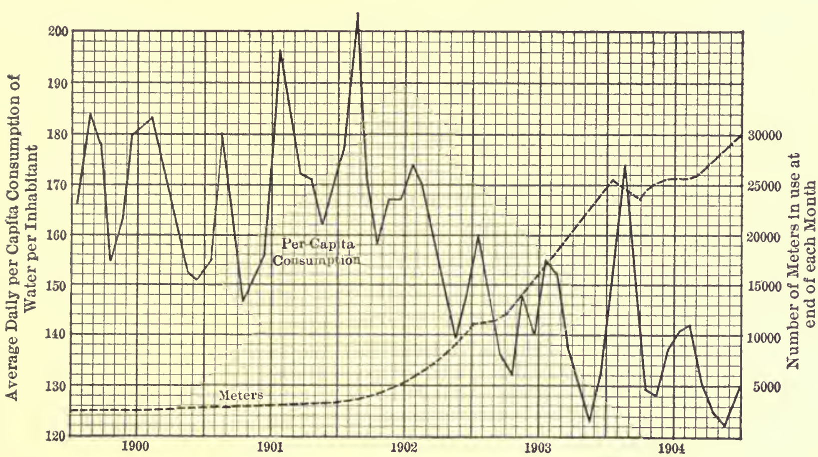


FIG. 32.

from Fuertes' Report,* shows the effect upon the water consumption of the gradual installation of meters in Cleveland.

* p. 143.

The decrease in the per capita water-consumption as the number of meters in use increases is clear evidence of the effect of the latter in eliminating leakage and waste.

As a further example of a method of ascertaining the relation between the amount of water used and the character of the population, reference is made to Vol. VI, No. 1, of the Journal of the New England Water-works Association, where the relation is shown between the number of fixtures in a house and the amount of water used. The following table, since partly amended, is taken from that report. It shows that by actual

TABLE XVIII

	Per Cent of First-faucet Flow.	Gallons per Capita per Day.
First faucet	....	7.0
Second faucet	20	1.4
First bath	50	3.5
Second bath	15	1.1
First water-closet	100	7.0
Second water-closet	40	2.8
Set tubs	20	1.4
Hose	1	1.1
Stores, etc	50	3.5
Schools	50	3.5
Churches	10	0.7
Boilers	130	9.1
Laundries	200	14.0
Greenhouses	90	6.3
Stables	50	3.5

meter-readings in Newton, on houses having but one faucet, 7 gallons per capita per day was the average amount used, the minimum being 5 and the maximum 11; that when a house has two faucets, 20 per cent of additional water is used; for the first bath, 50 per cent additional, etc. All these are based on a family of five, the average number in Newton. If boarding-houses or tenements are considered, these numbers will be increased by about 7 per cent per person.

The average water-consumption, however, no matter how

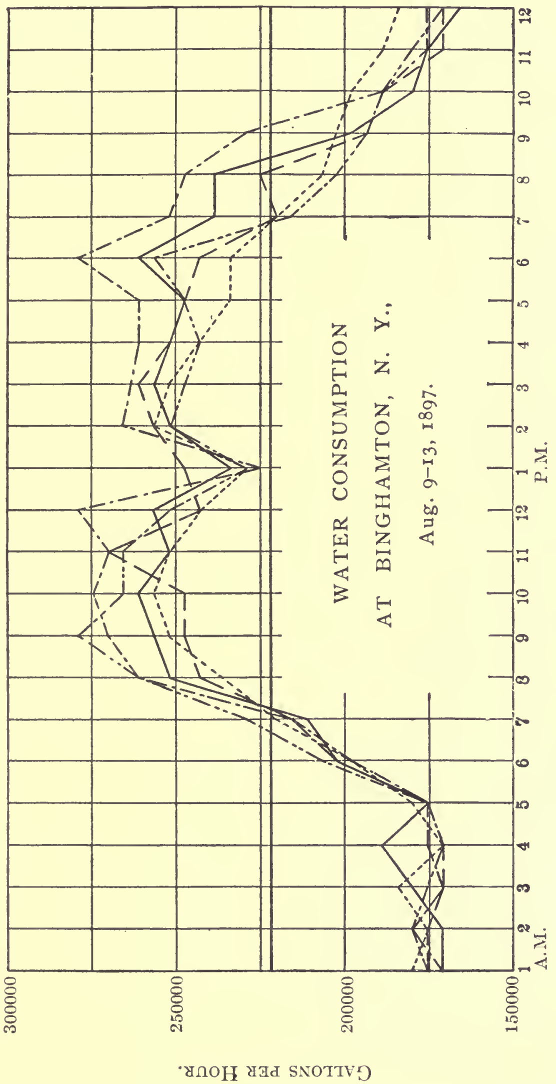


Fig. 33.

carefully estimated, is not a sufficient guide to the rate of sewage-flow. Further knowledge is needed on the subject of the variation of flow from hour to hour. Evidently more water is used and the flow to the sewers is greater during the day than during the night, and if the size of the sewer was based

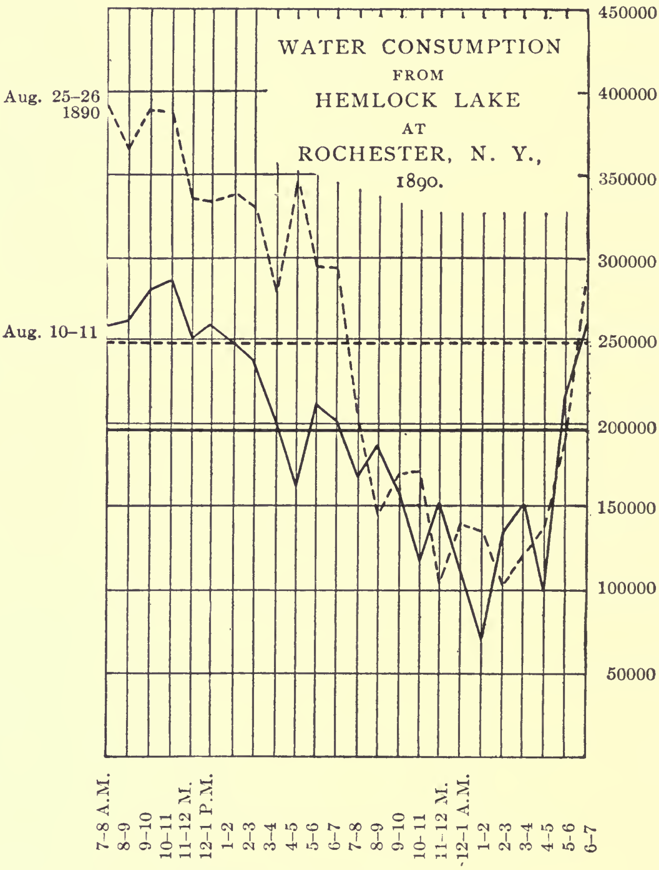


Fig. 34.

only on the average flow through the twenty-four hours, it would be too large at some times and altogether too small at others. To fulfil its purpose, the sewer must plainly be large enough to readily carry off the water, at that time of the day when the rate of flow is largest, even if it requires an area two or three times that necessary for the average flow.

To show actual examples of hourly and daily variation in the water-supply, the following diagrams are given:

Fig. 33 shows the variation in the daily water-consumption of the city of Binghamton, N. Y., the details being furnished to the author through the kindness of Mr. John Andersen, secretary of the Water Board. The average daily rate for

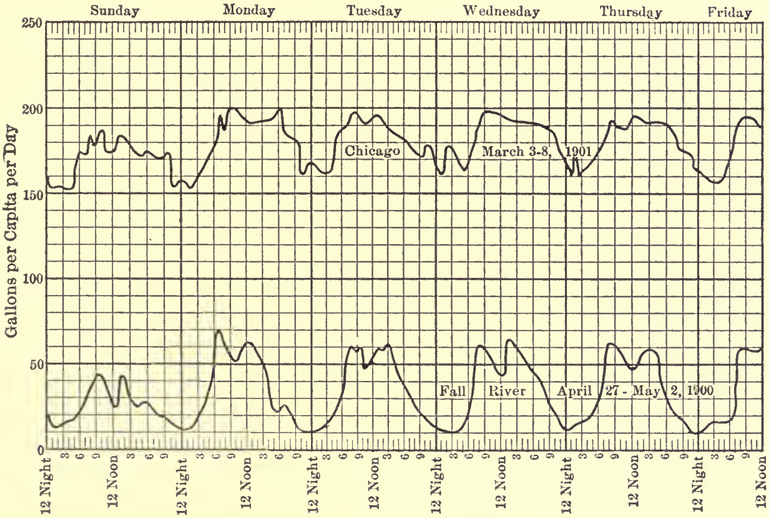


FIG. 35.

the five days is 220,440 gallons per hour, as shown by the heavy straight line; the average daily maximum (272,400) being 24 per cent more than the average daily consumption.

Fig. 34, from a table given by Rafter and Baker, shows the water-consumption in Rochester from Hemlock Lake. As before, the average rate of consumption is added and the maximum consumption is thus shown to be 46 and 58 per cent more than the average, for the two days shown.

In Fig. 27, the variation in the water-consumption has already been shown. The maximum rate there is 143 gallons per capita, and the average 115, for the maximum rate is only 24 per cent more than the average. If, however, 65 gallons of waste were to be eliminated, so that the average rate would be 50 gallons and the maximum 78 gallons, the latter would then be 56 per cent greater than the average.

TABLE XIX.

GAGINGS OF DRY-WEATHER FLOW OF SEWAGE AT DES MOINES AND ELSEWHERE

Sewer.	Date.	Population. Tributary.	Sewage Flow.			Per Cent Max. above Average.	Per Cent Max. above Average. Min. made O.
			Minimum.	Average.	Maximum.		
Compton Av., St. Louis....	1880	....	65	102	149	46	130
College St., Burlington, Vt. .	1880	325	65	115	140	30	70
Huron St., Milwaukee, Wis.	1880	3,174	......	......	120		
Memphis, Tenn...........	1881	20,000	61	......	140		
13 sewers, Providence, R. I.	1884	33,825	......	78			
16 sewers, Toronto, Can....	1891	168,081	......	87			
Hospital, Weston, W. Va...	1891	1,000	40	91	151	66	118
Schenectady, N. Y........	1892	*10,000	72	86	103	20	71
Canton, Ohio.............	1893	40,000	54	129	180	40	68
Chautauqua..............	1894	7,000	6	20	30	50	71
Iowa Agricultural College..	1894	289	0	32	77	141	141
Des Moines, Iowa, East side	1895	3,100	22.5	74	142	92	131
" " West side	1895	19,400	54	129	180	40	68
Cadillac, Mich............	1906	2,992	152	202	253	15	100
Gloversville, N. Y.........	1908	20,642	61	101	143	42	105

* Estimated.

The variation shown in the next to the last column is evidence of the effect of ground-water flow. The larger the minimum flow, the smaller the effect of the daily variation. The last column shows the percentage by which the maximum is greater than the average if the minimum flow be made zero and the average and maximum flows reduced by the same amount.

Fig. 35 * shows the hourly variations in the cities of Chicago and Fall River. In the former, the leakage is very large, and the maximum rate of consumption only 15 per cent more

* Fuertes' Report, p. 123.

than the average. In the latter the leakage is very small

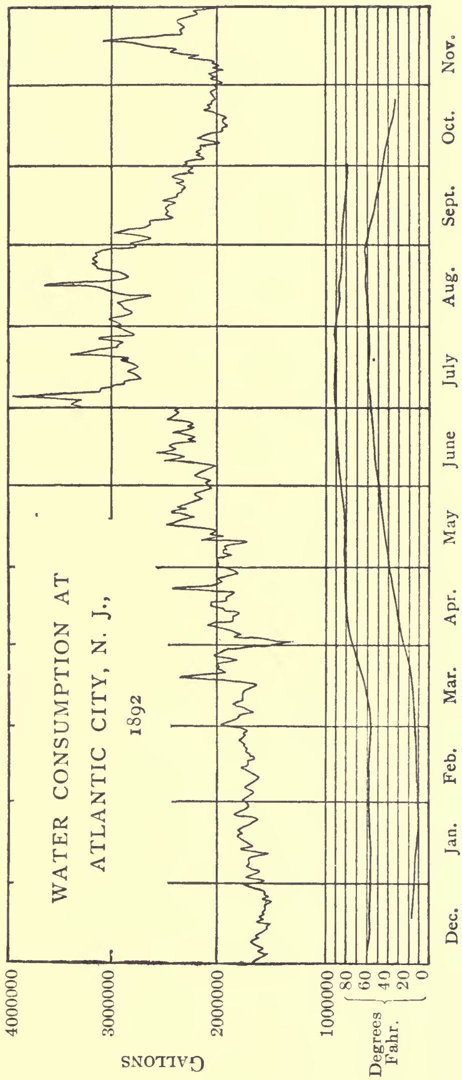


Fig. 36.

and the maximum rate is at least 100 per cent more than the average.

A list of actual sewer-gagings, so far as have been made public, was compiled and published in *Engineering News*, Vol. XXXV, p. 131, in a paper on sewer-gaging of Des Moines. The table is given on p. 130, Table XIX, with gagings of the outlet sewer at Canton, O., Chautauqua, N. Y., Cadillac, Mich., and Gloversville, N. Y., added. Two columns have been added to the table, one giving the percentage by which the maximum flow is greater than the average, and the second the same percentage, should the quantities involved be reduced by the amount required to make the minimum zero. The percentage will be affected largely by the amount of ground-water running in the sewer and by the amount of water used

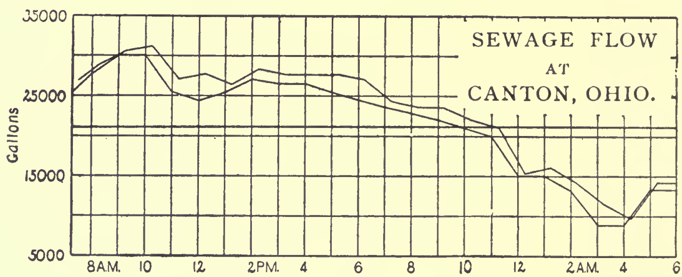


FIG. 37.

for manufacturing purposes and discharged into the sewer. No definite information on this point, however, is to be found.

Fig. 36 shows a sewage-pumping diagram for Atlantic City, N. J., for the year 1892. The effect of the summer season is plainly seen, as well as of the other holidays of the year.*

Fig. 37 shows a diagram prepared from the results of a gaging of the outfall sewer of Canton, O., in the spring of 1893.† The average daily amount (21,100 gallons) has also been calculated and added, the maximum amount being 43 per cent greater than the average.

* *Engineering News*, Vol. XXIX, p. 123.
† *Ibid.*, Vol. XXX, p. 61.

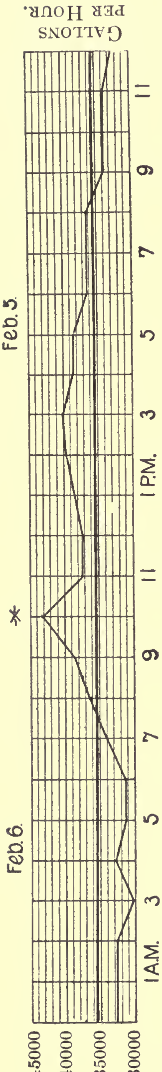

GALLONS PER HOUR.
45000
40000
35000
30000
Feb. 6.
Feb. 5.
1 A.M.
3
5
7
9
11
1 P.M.
3
5
7
9
11

FIG. 38.

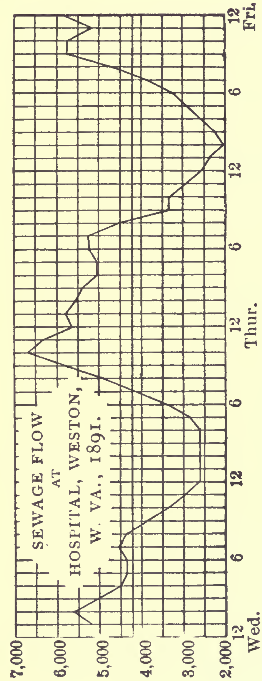

SEWAGE FLOW AT HOSPITAL, WESTON, W. VA., 1891.
7,000
6,000
5,000
4,000
3,000
2,000
12 Wed.
6
12
6
12 Thur.
6
12
6
12 Fri.

FIG. 39.

Fig. 38 shows the results of the gaging of the Schenectady

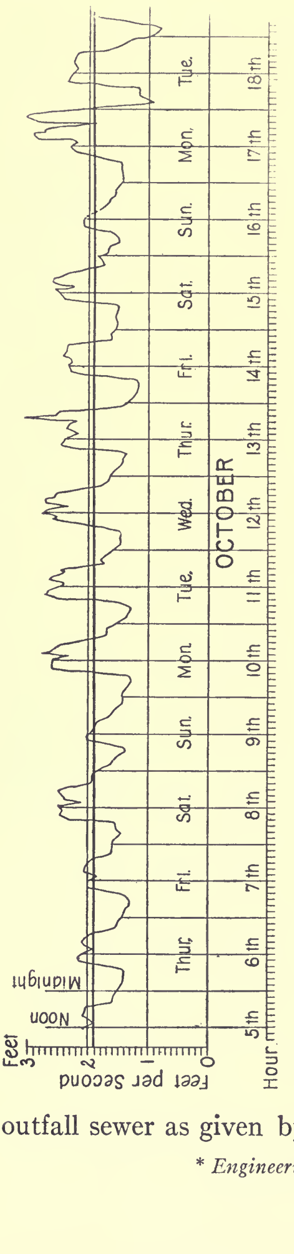


FIG. 40.

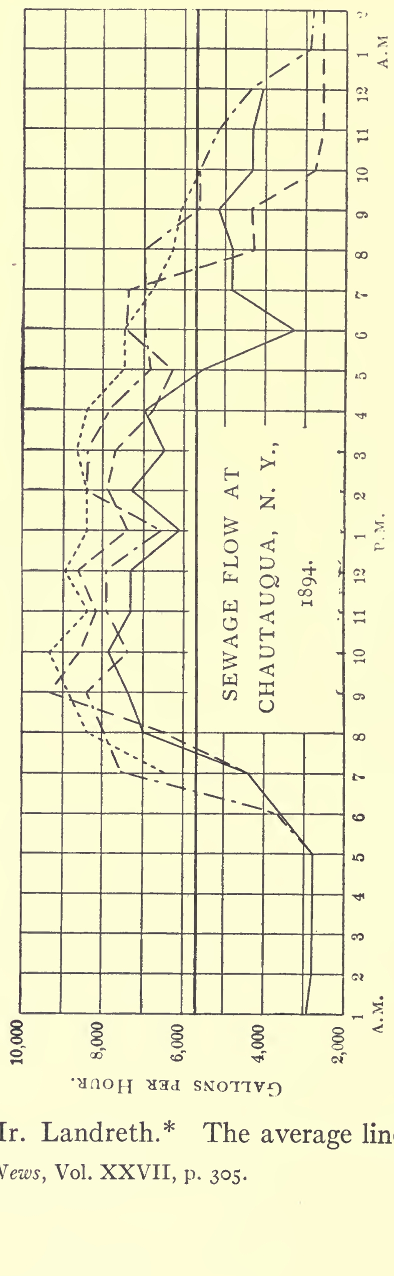


FIG. 41.

outfall sewer as given by Mr. Landreth.* The average line is

* *Engineering News*, Vol. XXVII, p. 305.

added (35,500 gallons), and the maximum found to be 59 per cent greater than the average flow.

Fig. 39 shows the results of the gaging of the sewer at the Insane Hospital at Weston, Va., made under the direction of Mr. Rafter and quoted in Rafter and Baker's "Sewage Disposal." The maximum here is found to be 22 per cent greater than the average.

Fig. 40 is a diagram taken from the paper of Mr. Grover already alluded to.

Fig. 41 gives a diagram of the sewage-pumping records at

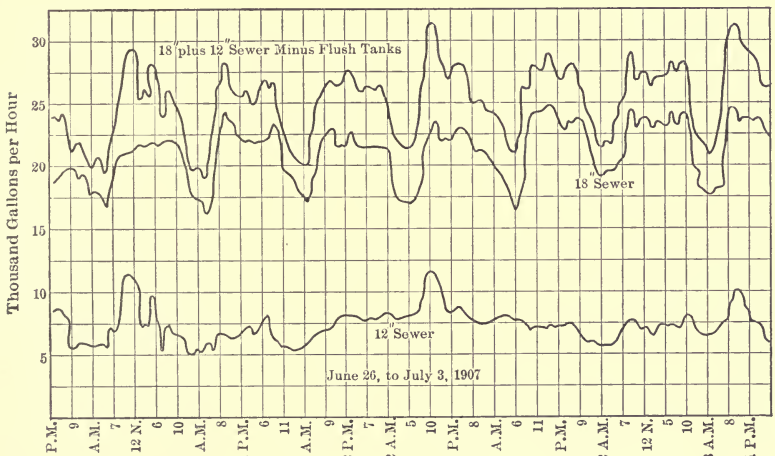


FIG. 42.

Chautauqua, N. Y., on the days indicated.* Here the average for the four days is 5700 gallons, the average maximum flow being 50 per cent more than the average.

In Fig. 40 the effect of constructing poor pipe-lines is plainly seen from the large flow at the times of day when there should be little or no flow in the sewers. While the maximum flow is only 56 per cent greater in amount than the average flow, yet it is 117 per cent greater than the average if the flow at 3 A.M.

* *Ibid.*, Vol. XXXI, p. 87.

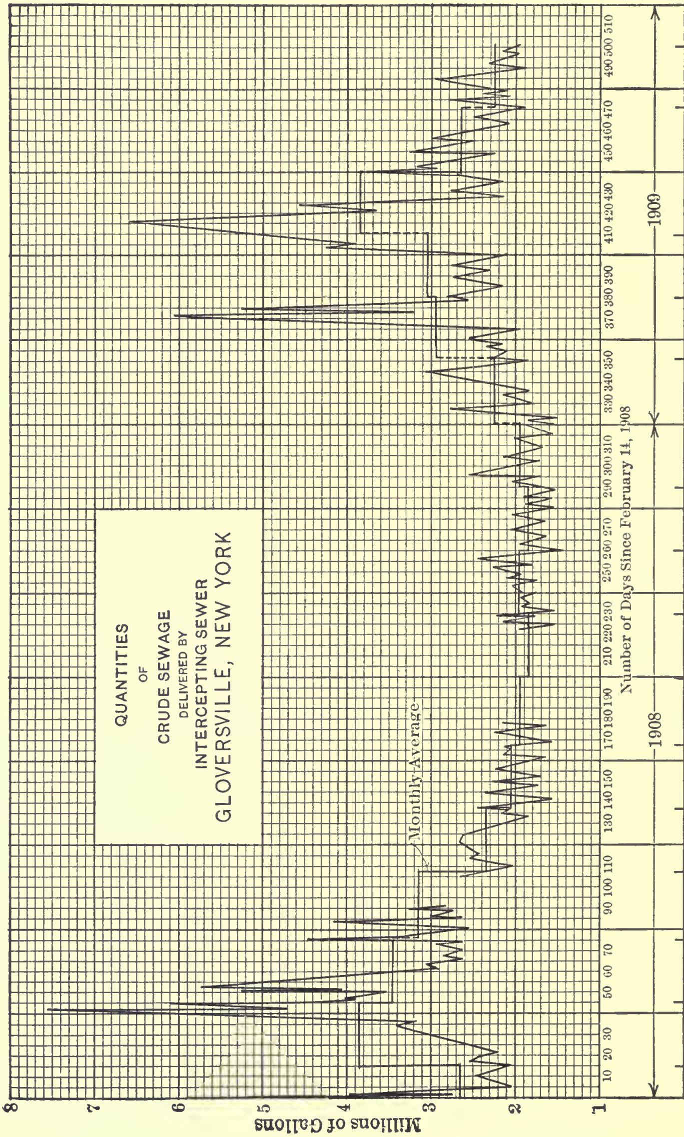

QUANTITIES
OF
CRUDE SEWAGE
DELIVERED BY
INTERCEPTING SEWER
GLOVERSVILLE, NEW YORK
Monthly Average
Millions of Gallons
1 2 3 4 5 6 7 8
Number of Days Since February 14, 1908
10 20 30 50 60 70 90 100 110 130 140 150 170 180 190 210 220 230 250 260 270 290 300 310 330 340 350 370 380 390 410 420 430 450 460 470 490 500 510
March 1 April 1 May 1 June 1 July 1 August 1 Sept. 1 Oct. 1 Nov. 1 Dec. 1 Jan. 1 Feb. 1 March 1 April 1 May 1 June 1 July 1
1908
1909

FIG. 43.

be called nothing, and the flow for the rest of the day reduced correspondingly.

Fig. 42 shows three curves based on gagings made of the two main sewers of the small city of Cadillac, Mich. The lower curve is for the 12-inch main, serving 475 persons. The middle curve is for the 18-inch main, serving 2517 persons. The upper curve is the result of adding together the measurements in these two sewers but correcting for the discharge of thirteen flush-tanks. The 12-inch pipe, without flush-tank water, had a minimum flow of 221 gallons, an average of 318 and a maximum of 558, the latter being 75 per cent greater than the average even with the high minimum flow. For the 18-inch pipe the minimum rate was 138 gallons, the average, 178, and the maximum, 239, or 34 per cent above the average. The large flow is said to be due almost wholly to defective plumbing, and the results are interesting as indicating the large amount of waste in a relatively small place.

Fig. 43* shows a long-time measurement of the flow of sewage in the intercepting sewer at Gloversville. The large flows in the spring of both years is undoubtedly due to groundwater entering the pipes, although the system is intended strictly for house sewage only. The minor variations from day to day are very marked and the excess in March is plainly responsible for a flow more than double the average.

Fig. 44, also of Gloversville, shows the hourly variation for a typical day. The effect of the mill wastes is most striking and serves, in this city at any rate, to show that the high night flow of sewage is not due to manufacturing use.

For the sewage alone, the average hourly rate was 62,000 gallons and the maximum 78,000, or 26 per cent increase. With the mill wastes added, the average flow is 84,000 gallons and the maximum 119,000, an excess of 42 per cent.

It follows from a study of the tables and diagrams given above that while the amount of flow or the per capita flow varies between wide limits, affected largely by the amount of

* Report of Harrison P. Eddy and Morrell Vrooman to Mayor of Gloversville.

water wasted, a law of daily variation is to be found for all places, and that the maximum flow is from 50 to 100 per cent greater than the average flow. The obvious method, then, for determining the proper amount of house-sewage flow is to determine the probable daily water-consumption per capita at the future time for which the sewers are designed. About 75 per cent of this might then properly be assumed to be the

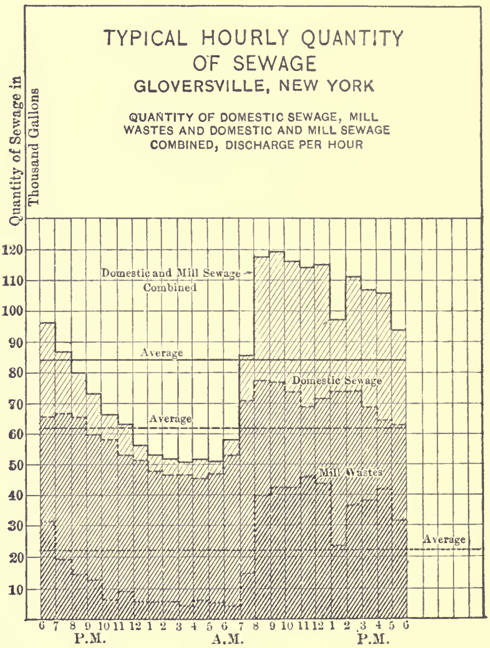


Fig. 44.

sewage flow. It will be safer, however, to assume that the maximum water rate will be the maximum sewage flow, and that any extraordinary excess in water rate will be partly compensated for by the fact that really only about three-quarters of such excess will reach the sewers. Therefore, add 100 per cent to the average water-consumption for the maximum rate of flow, and the result is the amount of flow for which the sewer must be designed if it is to be limited to house-sewage. This does not include, it is to be noted, any ground-water flow nor any large manufacturing enterprises which may affect the daily variation.

Analyses, more in detail, have been made of the variation in the flow, considering not only the daily maximum, but also seasonal variations, taking the monthly maximum and adding it to the weekly maximum, to the daily maximum, and to the hourly maximum. This method was given by Fanning in his "Water-supply," and was quoted by Staley and Pierson. Baumeister says that the days of greatest consumption require one and a half times as much water, and hence the sewers must be designed to carry off one and a half times the normal flow. The hourly maximum is one and a half times the hourly mean. Hence the capacity of the sewer must be such as to remove hourly $\frac{1\frac{1}{2}\times 1\frac{1}{2}}{24}=\frac{1}{11}$ of the average daily quantity, or more than twice the amount calculated on the supposition that the same quantity was supplied each hour of the year. In one of the recent German books on water-works by Franzius and Sonne the daily maximum is fixed at $1\frac{1}{4}$ and the hourly at $1\frac{2}{3}$, making the average between the latter and the daily average $\frac{1\frac{1}{4}\times 1\frac{2}{3}}{24}=\frac{1}{12}$ approximately.*

A common method of defining the maximum flow is to say that one-half of the daily flow will run off in 6 to 8 hours, and the sewer must be designed for this rate of flow. From the diagrams given, and taking the midday hours when the flow is greatest, the number of hours required to carry off one-half of the daily flow is as follows:

Binghamton,	August	9	$10\frac{1}{2}$
"	"	10	$10\frac{1}{2}$
"	"	11	$9\frac{1}{2}$
"	"	12	10
"	"	13	$10\frac{1}{2}$

* Baumeister.

Rochester, August 10	$9\frac{1}{2}$
" " 25	$8\frac{1}{2}$
Canton	$9\frac{1}{2}$
Schenectady	11
Hospital, Weston	9
Chautauqua, July 24	$8\frac{1}{2}$
" " 31	$8\frac{1}{8}$
" August 16	$9\frac{1}{4}$
" " 18	$9\frac{1}{3}$

indicating that to assume that one-half of the daily flow will flow off in 8 hours is a safe assumption. It further indicates that a capacity of twice the average flow is a larger allowance than necessary.

What the daily average will be must be left to the judgment of the engineer. By Table XII it varies from 19 to 222 gallons per capita per day, according to published records. Mr. Brackett shows that for the vicinity of Boston from 11.2 gallons to 44.3 gallons per capita per day are legitimately used for domestic purposes, and that these amounts must be increased for public purposes and for manufacturing and trade.

Mr. Whitney of Newton shows that the amount of water for domestic use varies with the number of fixtures in the house, from 7 gallons per capita per day for one faucet to 22.8 gallons for two faucets, two water-closets, and two baths, and that other uses increase the amounts as given in Table XVIII.

It remains for the engineer, after studying the character of the population and the possibility of manufacturing interests, to fix such a per capita allowance as is appropriate for that community.

PROBLEMS

42. From Fig. 24 determine the ratio of the sewage flow to the water-consumption for the months of July and September.

43. From Table XIII, plot points showing the relation between total population and per capita consumption. See if the relation follows any law.

44. If 75 gallons per head per day in New York City are wasted, and can be eliminated, what would be the ratio of the maximum daily rate to the average, after the waste is stopped? What is it now, by Fig. 27?

45. From Fig. 33 determine the average and maximum rates, if they are corrected on the assumption that the night flow is hiefly waste and can be reduced to 1000 gallons per hour. What percentage then will the maximum be of the average?

46. The average daily water-consumption of a city is found to be 5,000,000 gallons. What rate of flow, in cubic feet per second, should be taken for the sewage flow? Use Baumeister's rule, and then compare with the suggestions of p. 138 and p. 139.

CHAPTER IX

GROUND-WATER REACHING SEWERS

THE amount of rain-water entering a sewer has been discussed, and also the amount of water from domestic uses, the latter including the amount used for manufacturing and other municipal purposes. It now remains to determine the amount likely to come from the ground-water through which the sewer-line passes. This amount will depend on the material of which the sewer is made, on the kind of joints, on the method used in making them, and on the distance and head of ground-water in which the sewer is exposed to the infiltration. The last condition has considerable variation even in the same line, both because of irregular variation due to rain and because of periodic seasonal changes. In constructing the filter-beds at Brockton, it was found that there was a seasonal variation of 4 feet in the height of the ground-water, the height being greatest in May and least in November. Such a rise in the elevation of the ground-water might increase the length of sewer exposed to ground-water by some miles, especially if the hydraulic grade of the underground stream followed nearly parallel to the sewer-grade.

In his report on the Sewerage of Ithaca, Mr. Hering says: "In addition thereto [60 gallons per capita per day, assumed for average water-supply], 10 per cent of this quantity has been added to allow for ground-water which will probably find its way into the sewers in spite of the most careful workmanship."

In the report on the Sewerage of the Mystic and Charles rivers, January, 1889, the engineer, Mr. F. P. Stearns, has collected the following information:

"*Kalamazoo, Mich.*—Some ground-water finds its way into the system, estimated from data taken before the sewers were open for public use, to be 20 per cent of the capacity of the sewers.

"*Norfolk, Va.*—No accurate estimate made, but ground-water forms at least 60 per cent of pumping. From information given elsewhere in the returns, the maximum flow is found to be about 167 gallons daily per inhabitant connected with the sewers. Of this, the ground-water, estimated at 60 per cent, equals 100 gallons.

"*Schenectady, N. Y.*—The sewers are laid through wet ground and quicksand in some instances. The Erie Canal seepage also affects them to a small degree. Measurements made at about the time that the system was completed indicate that the infiltration of ground-water amounts to about 5 per cent of the capacity of the mains."

Mr. Stearns also says that he has recently examined two new systems of pipe sewers which were built with the intention of excluding the ground-water, and in both cases the amount of water collected by the sewers was considerable. In one of the cases, where the population connected with the sewers was small, the amount of ground-water was probably in excess of the sewage proper.

In the sewerage works of Canton, O., built in 1893, a 20-inch outfall with no connections was gaged for subsoil water, and in a length of 2400 feet a flow was found, due to infiltration, of 31,712 gallons in 24 hours, or at the rate of 70,000 gallons per mile per day. In the same system of about 11 miles there was a flow to the disposal works between midnight and 6 A.M. of about 73,000 gallons, which is at the rate of 26,500 gallons per mile per day (*Engineering News*, Vol. XXX, p. 61).

In the design for Taunton, Mass., 20 per cent for infiltration was added to the estimated flow in a 24-inch pipe passing through a swamp.

In North Brookfield, Mass., 1580 feet of 12-inch pipe was found to leak at the rate of 2500 to 5000 gallons per day, a rate of about 17,000 gallons per mile per day.

At Rogers Park, Ill., Mr. Broughton, engineer for The Shone Co., by means of special precautions (deep sockets and careful ramming) reduced the leakage in 9200 feet of 6-inch pipe under a head of from 1 foot 6 inches to 9 feet 6 inches of water, to 15 gallons per minute, or 1240 gallons per mile per day.

In Winona the same engineer made all sewers, which lay in water with a head of more than 5 feet, of cast iron.

At Brockton, Mass., the ground-water flow was said to be 400,000 gallons from 16 miles of sewers, or 25,000 gallons per mile per day.

At Altoona, Pa., the flow from 6100 feet of 27-inch pipe was 47,181 gallons, or at the rate of 40,814 gallons per mile per day.

From 3190 feet of 30-inch pipe the flow was 52,352 gallons, or at the rate of 86,592 gallons per mile per day.

From 5030 feet of $33\frac{1}{4} \times 44$-inch brick and concrete sewer the ground-water flow was 252,342 gallons, or at a rate of 264,000 gallons per mile per day. It should be noted, however, that this last flow has since been largely reduced by the contractor working under the direction of the engineer.

In the East Orange sewerage works,* where the conditions for producing a water-tight sewer were unusually severe, a large part of the line being 10 feet or more under water and laid in quicksand, but where at the same time unusual precautions were taken to prevent leakage, the amount of ground-water entering the sewer from 29 miles was found to be 650,000 gallons. The house-sewage flow after three years' use was 620,000 gallons, and the flush-tank flow 30,000 gallons.

Rafter and Baker, after noting that at East Orange some of the sewers were laid under 20 feet of ground-water, and that a brick sewer with its many joints and porous material was

* Trans. Am. Soc. C. E., Vol. XXV, p. 125.

used for 4000 feet in a location most unfavorable for tight work, and that with these exceptionally adverse circumstances the infiltration was only 50 per cent of the total quantity, or an amount equal to the domestic flow, say: "The results obtained under the extremely unfavorable conditions existing at East Orange of a leakage of only 2.5 gallons per second (215,000 gallons in 24 hours) from 25 miles of vitrified-tile sewers, with 66,000 joints, is indicative that, under favorable conditions and with careful workmanship, a system of such sewers may be made nearly impervious, though in designing disposal work it will probably be safe to allow for an infiltration of 15 per cent of the flow of sewage proper."

In connection with studies made on the pollution of Boston Harbor, the State Board of Health of Massachusetts had tests made on the infiltration of ground-water into sewer systems of recent construction. These showed * that into 137 miles of sewers, ranging in size from 8 inches to 36 inches diameter, the leakage was at the rate of 40,000 gallons per mile per day.

In the course of his report, Mr. Goodnough, the chief engineer, said that so far as he could judge by determinations of the amount of night flow throughout the State, the leakage into any large sewer system might easily amount to 70,000 gallons and at times when the leakage is extraordinary to as much as 80,000 gallons per day per mile of sewer. He adds that, in extensions of the Metropolitan Sewer System it seems desirable to make provision for a leakage of as much as 80,000 gallons per day per mile of sewer.

In the case of the joint trunk sewer from Elizabeth and Newark to Staten Island Sound, unusual care was taken in the construction, special jointing material (sulphur and sand) being used instead of cement and in the 150 miles of main sewer and tributary systems, the infiltration was at the rate of 25,000 gallons per mile per day.†

* Special Report on the Discharge of Sewage into Boston Harbor, 1900.

† Report Passaic Valley Sewerage Commission, Dec., 1907, p. 13.

Mr. J. N. Hazlehurst,* refers to a lawsuit, entered into because while the specifications limited the leakage into a sewer system under contract to 5000, measurements made after all the pipes were laid showed an average leakage of 44,600 gallons per mile per day. In the course of this article, he refers to measurements made at Malden, Mass., where tests made directly after the completion of the sewerage system showed a leakage at the rate of 50,000 gallons per mile per day. This was, he intimates, in spite of the fact that all the work was underdrained and cement used almost extravagantly in the hope of securing water-tight work.

Mr. Hazlehurst also refers to the measurements made in New Orleans, where an original estimate of 31,800 gallons per mile per day (0.003 cubic foot per second per acre) was found to be approximately correct after the sewers had been built.

Such values as have been given above seem to establish the fact that many sewers can be found that are imperfectly constructed and that as a consequence admit more or less ground-water into the pipes. It is, moreover, probable that the measurements recorded are the results of attempts to reduce leakage to more moderate and reasonable figures and are therefore unduly high.

Mr. Hazlehurst implies, however, that these leakages are characteristic and says definitely that there can be no such thing as a water-tight pipe line and that if an engineer, in his specifications, sets a limit to the amount of leakage, it behooves him to "be very sure of his ground, both literally and figuratively."

Granted, however, that it is doubtful whether a sewer can be made water-tight under ordinary conditions and methods of construction, it remains to be seen what is a reasonable amount of infiltration.

If the sewer is of brick, assuming first-class construction,

* *Engineering News*, Vol. L, p. 179.

the amount of ground-water entering may be restricted to that due to the porosity of the brick and mortar. Various methods are found in pocket-books for making brick walls impervious, and many statements are made to the effect that brick masonry in engineering construction allows considerable water to pass through. No definite data, however, seem available for the exact amount of such percolation under different conditions of construction. In the case of sewers, experience seems to show that in the same ground more water comes through a brick than through a pipe sewer; but nothing definite is known on the subject.

The author has seen water under a head of a few feet pass, in numberless small streams, through a 12-inch brick wall. He has experimented with tanks of brickwork, with 8-inch walls, with and without plaster coats, and is convinced that, with the work of ordinary housemasons, it is beyond all reason to expect an 8-inch brick wall to be water-tight, even with an ordinary plaster coat. It is essential for water-tightness that the voids of the plaster coat be filled either with several coats of cement wash, with asphalt, or with some one of the many water-proofing paints or plasters available. Mr. John N. Brooks * has suggested that a more suitable unit for infiltration for brick or concrete sewers is gallons per day per square yard of interior surface, and from some experiments of his own on a concrete sewer 9 miles long, he finds the leakage for 4- to 6-foot sewers, in first-class condition with special waterproofing of three-ply felt and pitch, to be at the rate of 0.8 gallon per square yard, or 6000 gallons per mile for the 4-foot sewer.

The only two instances on record of leakage through the walls of brick sewers, are at Orange, N. J., and at Malden, Mass., already referred to, and instanced by Mr. A. P. Folwell,† secretary of the American Society of Municipal Improvements. The leakage was found to be at the rates of 570,000 and 800,000

* Proc. Am. Soc. C. E., Vol. XXVIII, p. 1705.

† *Municipal Engineering*, Vol. XXV, p. 348.

gallons per mile for the two places, respectively, or 122 and 136 gallons per square yard. It is absurd, however, as Mr. Folwell himself points out, to expect that such leakage as this is reasonable, and it only serves to illustrate what large flows may occur through badly built brickwork.

If the sewer is of vitrified pipe, ground-water enters the pipe through the joints, and the amount to be expected depends, *assuming perfect workmanship*, on the kind of cement, depth of joint, and other details of construction of the joint. In Vol. XIII, p. 71, of the Journal of the Association of Engineering Societies, are given the results of some tests by Freeman C. Coffin, C. E., made to investigate this very point. His results are given as follows:

"In the standard form of pipe-socket, with well-made joints of either Portland cement, neat or 1 : 1, or of Rosendale cement 1 : 1, with over-filled joints, the leakage would not be serious, probably not exceeding 1000 gallons per mile per day, with the level of the ground-water from 2 to 8 feet over the pipe.

"In pipe with deep sockets the tests indicate that if the joints are well made the leakage will be about as follows: In Rosendale cement neat it will be very large, perhaps over 100,000 gallons per mile per day. In Rosendale cement mixed with sand 1 : 1 the leakage would not exceed 700 to 800 gallons per mile per day. In Rosendale 1 : 2 it would approximate 1000 or 1200 gallons per day; with Portland cement neat, about 150 gallons per day; with Portland 1 : 1, about 500 or 600 gallons."

The sockets of these pipes were very small to reduce the area of cement as much as possible, and Mr. Coffin thinks that even with the best intentions the difficulty of filling these joints in a trench would be insurmountable, and he therefore gives the figures above as representing only what can be done in a laboratory experiment.

In the discussion of the above conclusions, Mr. Coffin says that it would seem that Portland cement, either neat or mixed 1 : 1, or Rosendale 1 : 1, would make work that was sufficiently tight for all practical purposes, provided the joints could be well filled and could remain undisturbed by water or jarring until sufficiently set to resist. Unfortunately, in practical construction joints in a sewer-pipe are never made with the same care as in a laboratory experiment; and further, it seldom happens that the joints are allowed to stand from 12 to 48 hours before being covered with water, as was done in these experiments. In a wet trench the cement is not always forced into the joint, and water is admitted to the joint before the cement is thoroughly set, tearing off the coating and leaving an opening into the pipe.

To approximate actual conditions as nearly as possible, F. S. Senior, as his thesis work, under the direction of the author, made some experiments in which water was admitted to the joint at various intervals from the time of making. His experiments* brought out the following points: First, that there is to be expected a gradual improvement in the tightness of cement-joints from the time that they are first laid, amounting to from 40 to 80 per cent, and that the decrease in leakage is greater for Rosendale cement than for Portland. Second, that there is more leakage under high heads, and that the increase with the head is nearly proportional to its square root. Third, that there is a great advantage in using quick-setting cement if there is any probability of having the joints covered with water; and further, that a quick-setting cement will reduce the length of time necessary to pump from a wet trench, since the amount of infiltration after the cement has taken a hard set is inconsiderable. Fourth, that in a wet trench a gasket is of great value; and whereas without it a line on which water has risen before the cement in the joints was hard would admit water to the extent of half filling a 6-inch pipe, yet with gaskets the leakage would be no more than if the water had been kept off till the cement had set. This last is in contradiction to Mr. Coffin, who concluded that a

* Trans. Ass'n Civil Eng'rs, Cornell Univ., 1897, p. 113.

gasket, by taking up in the joint space that should be filled with cement, was a detriment rather than a benefit. Mr. Senior's experiments were all made on six 2-foot lengths of 6-inch pipe, and the mortar all mixed 1 : 1. His results in detail were as follows:

When the water was turned onto the joints within 3 or 4 minutes after the cement had begun to set (15 minutes after the first joint was made), the leakage through Rosendale cement was at the rate of 150,000 gallons per mile per day, decreasing to 30,000 after 72 hours. The Portland cement, under the same conditions, showed a first leakage of 120,000 gallons, decreasing to 4500 after 72 hours.

When, however, the cement was allowed to stand 30 minutes before water was admitted, the leakage through Rosendale joints was at first 70,000, reduced to 25,000 after 72 hours, and for Portland the leakage was 60,000, which became 4,000 after the same time.

In using gaskets, water was turned on in 10 minutes, or before the cement was set, so that the full benefit of the gaskets was brought out. With Rosendale the first leakage was only 26,000 gallons, and after 72 hours it had decreased to 8000 gallons per mile per day. With Portland, while the first leakage was 11,000 gallons, after 72 hours it was but 5500 gallons, or a small and insignificant amount. The results of this last work as well as that of Mr. Coffin show that even under the best conditions there is some leakage; but that if the joints are well made this amount can be reduced to about 4000 gallons of water per mile per day for a 6-inch pipe. (Compare with p. 144.) For larger pipe the increase would probably be proportional to the area of the joint.

If the joint space increased regularly both in width and length, this area would be approximately proportional to the square of the diameter. But the widths are uniform at $\frac{3}{8}$ inch up to and including 10-inch pipe and all other sizes have a $\frac{1}{2}$-inch space for standard pipe. For deep and wide socket pipe, the space is always $\frac{5}{8}$ inch. The area then depends on the circumference or directly as the diameter, except that there is an abrupt change between the 10- and 12-inch pipes.

As an example, a 12-inch pipe 2 miles long, laid in water, might be reasonably expected to carry as a minimum amount of subsoil infiltration $4000 \times 2 \times 2$ or 16,000 gallons of water, while the capacity of the pipe flowing half-full at a velocity of 2 feet per second is 650,000 gallons per day, or the leakage is about $2\frac{1}{2}$ per cent of the capacity of the pipe. This might be somewhat reduced by allowing 12 hours for the cement to set before the water is turned on, but it takes into account not a single bad joint nor one which is not fully filled, of which there are always many in actual construction. If the leakage were at the rate of 10,000 gallons for a 6-inch, or 20,000 gallons for the 12-inch pipe, it would be less than most of the examples cited and would be only 6 per cent for the 2 miles.

In Vol. XIX of the Journal of the Association of Engineering Societies is a valuable paper by Mr. F. A. Barbour of Brockton, Mass., on the strength of sewer-pipe, and incidentally on some tests of the tightness of pipe-joints. He gives, however, no conclusions as to amounts, saying that the results have been decidedly unsatisfactory from the standpoint of a written report, and no tabulations of the figures will be given.

The evident lesson so far as ground-water is concerned is that, instead of adding a certain percentage to the desired capacity of the sewers, a more rational method is to consider in detail the lengths of pipe to be laid under a head of ground-water, and to increase those lines and the mains lower down by a certain amount of leakage per mile, the amounts to be arrived at by actual experience and by the experiments quoted.

Mr. Alexander Potter, Engineer of the Joint trunk sewer of New Jersey, concludes* that even with rigid inspection it is not safe to count on less than about 25,000 gallons per mile per day leakage with cement joints, but with sulphur-sand joints it may be reduced to about 5000 gallons per day. These

* *Engineering Record*, Vol. LX, p. 377.

amounts seem large in view of the experimental data, but it must be remembered that the amounts given in the experimental data are for leakages through perfect joints, and that in construction any workmanship except the best, which it is practically impossible to secure in trench-work, will materially modify and increase the amounts given.

PROBLEMS

47. The annular space for the joint in a 10-inch pipe (3-foot lengths) is $\frac{3}{8}$ inch. If the thickness of the pipe is $\frac{7}{8}$ inch and the leakage is at the rate of 15,000 gallons per mile per day, determine the rate of infiltration through the cement in gallons per square yard of area.

48. The annular space of a 6-inch pipe is $\frac{3}{8}$ inch and for a 12-inch pipe $\frac{1}{2}$ inch. If the thickness of the pipes are $\frac{5}{8}$ inch and 1 inch respectively, compute the relative permeability, assuming the latter to depend on the area of the joint.

49. If the leakage through the walls of a brick sewer, 4 feet 6 inches diameter, is at the rate of 5 gallons per square yard of interior surface, compute the leakage per mile of sewer.

50. If the leakage into a sewer system is at the rate of 25,000 gallons per mile per day under a certain head, what would be the probable leakage if the head is doubled, judging by Senior's experiments.

51. Assuming the leakage into 6-inch pipe to be at the rate of 8000 gallons per mile per day, determine the total leakage into 10 miles of 6-inch pipe, 5 miles of 8-inch pipe, 2 miles of 10-inch pipe, and 0.75 mile of 12-inch pipe.

CHAPTER X

GRADES AND SELF-CLEANSING VELOCITIES

In the early days of sewer-construction the fact that sewers could be kept clean by any other method than by periodic sweeping was scarcely appreciated. Sewers were a subject not fit even for discussion, much less for the professional interests of any except the meanest laborers. Sewers were necessary, were to be taken for granted, but were not to be made a topic of public conversation. To this public attitude towards sewers it is undoubtedly due that the principles of hydraulics, early studied in the case of rivers by the most eminent scientists, have been so tardily applied to sewage-flow, and have only recently been recognized in determining the size, shape, grade, etc., to make the sewer best suit its intended purpose.

Baldwin Latham gives examples of defective house-drains said to be still in use in London houses. Fig. 45 shows a defective section of a sewer carrying storm-water and sewage, which has been in use in Ithaca for many years. Many examples could be found of similar faulty construction, of broad flat inverts, of rough surfaces, of open joints, and of grades not sufficient to carry along the matter in suspension. It is, however, enough to point out that such imperfectly constructed sewers have been the rule in the past, and that only within the latter half of the last century has the relation between the hydraulic elements concerned and a clean non-depositing sewer been recognized. Now it is known that with a sufficient velocity and depth any material that has

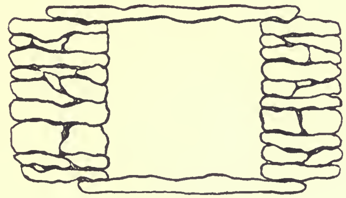


FIG. 45.

been deposited there may be scoured out from the bed of a sewer or stream, or it may be held in suspension and so prevented from accumulating deposits. To what laws or by what means this power of water to hold material of greater density in suspension is due is not clearly known. The subject, however, is of great importance, because if a sewer is to be kept clean without intermittent hand labor, it must be through the transporting power of the water which hurries along with and in itself all solid matter. The admirable compilation by Mr. E. H. Hooker on this subject, presented as a thesis at Cornell University in 1896, and published later in the Trans. Am. Soc. C. E., gives the following propositions, applicable to sewers, as expressing the main facts so far as they are known and necessarily underlying any broad theory of the cause of the suspension of sediment:

"1. The movements of solids by water may take place by dragging, by intermittent suspension, or by continuous suspension.

"2. Motion in each of the three ways is increased with increase of depth; yet the depth itself can only affect the intermittent suspension.

"3. Motion in each of the three ways is increased by increase in the mean velocity.

"4. The presence of the sediment in the stream-flow decreases its mean velocity.

"5. Dragging as well as suspending power increases with heaviness of the liquid and with its greater coefficient of viscosity.

. . . .

"10. Increase of vortex motion increases the power of transport.

. . . .

"13. Bodies suspended in flowing water, either intermittently or continuously, tend to acquire a velocity greater than that of the water surrounding them."

The theories offered to explain the facts or propositions just given are summed up by Mr. Hooker with the statement that the suspension of sediment in flowing water may be attributed to three causes acting together, or in rare cases separately:

"First. The resultant upward thrust due to eddies, conditioned upon the fact that the earth's (bed of stream) profile offers more rugosities than the air profile, and the effort exerted by a current upon a solid varies as the square of the relative velocities.

"Second. The resultant upward motion of solids due to the fact that an immersed body tends to move faster than the mean velocity of the displaced water, and in such motion tends to follow the line of least resistance.

"Third. The viscosity of the water."

The law of Airy, that the transporting power of flowing water varies as the sixth power of the velocity, Mr. Hooker passes over without comment, but he gives curves showing the increase of suspending power with velocity.

By these laws as given, it is evident that a certain velocity and depth are necessary to keep material from sedimentation. The exact relation between velocity and depth to secure the best transporting power is not known. In the case of sewers it is generally assumed that for a given quantity of water the maximum transporting power is secured with the maximum velocity, and that therefore a sewer section in which the volume of flow is variable should be designed so as to keep the velocity of flow for all depths equal, or as nearly equal as possible, to that obtainable from the section most favorable for that quantity if considered alone. Since the maximum velocity for a constant quantity is obtained when area divided by wetted perimeter is a minimum, the section generally used as giving the greatest velocity is circular, and in sewers of varying flow the section is egg-shaped as being the best possible. Should it, however, be found that the depth of flow is more important as a function of the transporting power than is now thought, the maximum velocity will be no longer sought, since

TABLE XX

TABLE GIVING VELOCITIES OF CURRENT AT WHICH DRAGGING BEGINS

Authority	Dubuat.	Telford.	Blackwell	Sainjon.	Login.	Rhine Measurements.	Zschokke.	Verein Hütte.	Bouniceau
Reference	Hydraulique. Paris, 1786, p. 94.	Partiot in Annales des Ponts et Ch., 1871, I, p. 34.	Proc. Inst. Civil Engineers, Vol. 82, pp. 47–50	Partiot in Annales des Ponts et Ch., 1871, I, p. 33.	Stevenson's Canal and River Engineering, p. 315.	Zeitschrift des A. und I. Vereins zu Hannover, 1884, p. 176.	Lectures, Zürich, 1895.	Ingenieurs Taschenbuch.	"Etudes sur la Navigation." 1845, p. 19.
Remarks.	Bottom Velocity. Feet per Second.	Bottom (?) Velocity. Feet per Second.	Feet per Second.	Botto Velocity. Feet per Second.	Bottom Velocity. Feet per Second.	Feet per Second.	Feet per Second.	Bottom Velocity. Feet per Se ond.	Bottom Velocity. Feet per Second.
Soft earth		0.25							
Brick clay {allowed to settle ½ hour in water}					0.25				0.26
Potter's clay	0.27							0.26	
Soft clay		0.50						0.52	0.49
Fresh-water sand					0.667				
Large sand	0.71								0.72
Vegetable soil					0.833				
Sand		1.00							0.98
Firm sand								1.02	
Sea sand					1.103				
Gravel (size of anise seed)	0.36								0.36
" (size of peas)	0.62				2.000	2.46 (if disturbed)	0.62		
" (size of beans)						2.95 (if disturbed)	1.066		
" (diameter, 0.008 ft.)				0.82					

Gravel (diameter, 0.03 ft.)				1.64					
Brickbat (contents, 2.59 cu.in.)			1.75–2.00						
Gravel		2.00							
Oolites (2.39 cu.in.)			2.00–2.25						
Slate (2.38 cu.in.)			2.00–2.25						
Sea pebbles (1.06 in. diameter)	2.13							2.30	2.13
Brickbats (4.76 cu.in.)			2.25–2.50						
Flints (1.96 cu.in.)			2.50–2.75						
Brickbats (18.5 cu.in.)			2.75–3.00						
Oolites (17.68 cu.in.)			2.75–3.00						
Slate (9.06 cu.in.)			2.75–3.00						
Flints (10.37 cu.in.)			3.00–3.25						
Pebbles		3.00							3.28
Boulders								3.08	
Gravel (0.085 ft. diameter)							3.09		
" (0.13 ft. diameter)				3.28					
" (size, hazel to walnut)						3.48 (if disturbed)			
" (size, pigeon's egg)						3.67 (if disturbed)			
Smallest gravel						3.87			
Gravel (walnut size)						4.92			
" (0.17 ft. diameter)							5.15		
" (weight, 2.0 grams)						4.92 (if disturbed)			
Broken stone		4.00							3.94
Gravel (weight, 2500 grams)						5.90 (if disturbed)			
Conglomerate (soft schist)		5.00						4.90	5.00
Gravel (0.33 ft. diameter)				4.92					
Laminated rock		6.00						6.00	6.00
Gravel (0.56 ft. diameter)				6.56					
All gravel						6.56 (if disturbed)			
Gravel (0.18 cu.ft.)							7.22		
" (1.25 ft. diameter)				9.84					
Hard rock		10.00						10.36	9.84
Gravel (1.09 cu.ft.)							10.29		
" (2 20 ft. diameter)				13.12					
" (2.18 cu.ft.)							15.44		

now it is used only as an index of the transporting power. As Mr. Hooker intimates, the whole subject is far from being on a satisfactory basis, and observation and experiments are much needed to put the matter in its true light.

The available experiments on the velocity required to take up into suspension or to drag along material in running water are not many. Table XX, taken from Mr. Hooker's article, gives what there are.

It is seen that a velocity varying from 16 to 60 inches per second is required to take up material, and Baldwin Latham gives the following table showing how the specific gravity of the material affects that velocity. The experiments on which this is based were made by Mr. T. E. Blackwell, C.E., for the government referees, in the plan of the Main Drainage of the City of London.

TABLE XXI

Material.	Specific Gravity.	Commenced to Move at a Velocity of
Coal	1.26	1.25 to 1.50 ft. per sec.
"	1.33	1.50 to 1.75
Brickbat	2.00	1.75 to 2.00
Chalk	2.05	1.75 to 2.00
Oolite stone	2.17	2.00 to 2.25
Brickbat	2.12	
Chalk	2.00	
Broken granite	2.66	
Chalk	2.17	2.25 to 2.50
Brickbat	2.18	
Limestone	1.46	
Oolite stone	2.32	2.50 to 2.75
Flints	2.66	
Limestone	3.00	

Evidently other conditions than the specific gravity are concerned, and as no dimensions are given, it is probable that there was a variation in the size of the pieces tested and, what is probably of more active importance, in the shape of the pieces. In a thesis on Flushing-waves, in 1894, Messrs. Fort and Filkins of Cornell University note that a piece of brick nearly cubical in shape, weighing 22 ounces and having a volume of over 16 ounces, was carried by a flushing-wave more than 1000 feet, while under the same conditions a mere flake of the same brick, having a volume of not more than 2 cubic inches, could not be moved more than 600 feet. Also a piece of limestone nearly cubical, weighing 7.75 ounces, was carried 1400 feet, while a piece weighing 4.75 ounces, but nearly flat in shape, was carried only 470 feet.

It is seen, then, from the above old, meagre, and variable data that a flow of water requires a certain velocity to carry along solid material, and that the suspension of the material depends also on its size, shape, and specific gravity.

Material deposited at the same place will be lifted by a flow of water and carried to different distances; those pieces whose shapes are such as to withstand the current, offering a thin and sloping edge to it, being last taken up, as the velocity increases, and soonest dropped. In a similar way a large stone too heavy to be carried along has been found to shelter smaller ones which otherwise might have been taken up by the current. Small irregularities in the channel serve as shelters for the fine material, and piles of sand, etc., are likely to accumulate behind projecting bits of mortar. It is plain, then, that neither theoretical determinations of the velocity required to carry matter in suspension, nor yet the results of experiments on different materials of varying sizes and specific gravities, are sufficiently like the conditions prevailing in sewers to determine the velocities required in the latter, and it is only from experience in sewers themselves, where the material to be transported is that natural to a sewer and where the conditions of rugosity of bed and variation in the velocity in the different laminæ are those peculiar to a sewer, that any reliable recommendations must come.

As an indication of the extreme lower limit of the range of velocities, reference may be had to the recent work on sedimentation where studies have been made on the velocity of settlement of particles of varying size. Thus, Hazen has found* that in still water particles 1 mm. in diameter settle at the rate of .33 foot per second, but that particles .1 mm. in diameter reduce this rate to .028 foot per second and those .01 mm. to .005 foot per second. That is, the size of particle, particularly in the case of fine suspended matter, plays an important part in the rate of settling and it is quite possible for the particles to be so small that they will not settle at all. In the separate system the size of the suspended particles is continually changing on account of the organic decomposition of the material and at the lower end of a long line the reduction in size may be so great that a much smaller velocity will suffice to keep the sewer clean than would be needed at the upper end. The relatively low velocities in Table XX shown in the cases of soft earth and clay are undoubtedly due to the small size of the particles involved. Experiments with more finely divided material might show even lower velocities. The question then is not merely what velocities will prevent sedimentation, but what are the sizes of particles of suspended matter found in sewage and what velocities are necessary to prevent sedimentation of such particles.

Since the size of particles carried in suspension in sewers, either for storm water or for domestic wastes, is largely conjectural, it follows that it is hopeless to attempt to apply any definite law of hydraulics to the problem, but that rather the engineer must be contented to accept the results of experience and secure in new work such velocities as have been found adequate to prevent undesirable subsidence.

The following required velocities are those suggested by different prominent engineers and have been tabulated by Staley and Pierson:

Baldwin Latham	2 to 3	ft. per sec.
Beardmore	2.5 to 3	" "
Phillips	2.5 to 3	" "

* Trans. Am. Soc. C. E., Vol. LIII, p. 63.

Rankin	1 to 4.5	ft. per sec.
Adams	2.5 to 3	" "
Philbrick	2.5 to 3	" "
Gebhard	2 to 3	" "

Baldwin Latham gives a little more detail, saying that in his experience he has found that in order to prevent deposits in small sewers or drains, such as those of 6 or 9 inches diameter, a velocity of not less than 3 feet per second should be secured. Sewers from 12 to 24 inches diameter should have a velocity of not less than $2\frac{1}{2}$ feet per second, and in sewers of larger diameter in no case should the velocity be less than 2 feet per second. This statement would evidently imply an expected or experienced increase of transporting or scouring power in the current with an increase of depth.

A fact still further contributing to the general uncertainty of this subject is that the velocities given above are those for the pipes flowing full or half full. Since a small pipe sewer rarely flows half full, and since the velocity decreases rapidly as the depth in the pipe decreases, it follows that the bottom velocity on which the scouring power depends must be much less than the $2\frac{1}{2}$ or 3 feet per second which by the table seems necessary. For example, an inch flow in an 8-inch sewer, laid on such a grade that it has a velocity of 3 feet per second when flowing half full, has with the less depth a velocity of but 1.6 feet per second, which, by the table on p. 156, is not sufficient to move anything except the smallest gravel.

When it is remembered that sewers are designed for a period of years in advance and that the full capacity of the sewer is to be reached only at that future time and then only at that hour of that day throughout the entire year when the flow will be the greatest, it is readily seen that in fact at all times the flow is less than that estimated and that therefore the velocity will always be less than that assumed to exist.

In examining in the Ithaca sewers the velocities with small depths, the author has found velocities of 0.98 foot per second apparently carrying along the solid matter and requiring no more flushing than is usual. It would seem, then, that the stated velocities are not the actual flow velocities, but are those required at half-depth in order to get the needed velocities with the usual flow, the actual velocities needed being from 1 foot to $1\frac{1}{2}$ feet per second.

The velocity required being known, it can only be secured by sufficient grade, and the minimum grades are those just sufficient to produce the velocities given above. By the rule of Latham, the larger the sewer the less need be the velocity and grade, but it assumes that the amount of flow is sufficient to keep the sewer flowing half full.

Thus it is possible to carry away a definite amount of sewage either by a large pipe and a small velocity or by a small pipe and a correspondingly high velocity. According to Latham, the following sewers laid at the grades given will all have the same velocity flowing half full, but the amounts carried must be in the ratios of 100, 25, 4, and 1:

A sewer	of	10	ft. diameter,	grade	.038	per cent
"	"	5	"	"	.076	"
"	"	2	"	"	.190	"
"	"	1	"	"	.380	"

The velocity required is therefore a function of the quantity as well as of the grade.

The fact that large sewers may be laid on a light grade and yet maintain the necessary velocity is sometimes responsible for an attempt to reduce the necessary grade for a small pipe by the substitution of a larger one. Thus one of the author's students in a Thesis design planned 24-inch pipes for a number of laterals, because his grades were very light and he mistakenly thought that by using a larger pipe, the grades for that pipe if half full, could be used. The error is at once apparent, a small flow in a large pipe requiring a greater slope for the pipe than if the flow was confined in a smaller cross-section.

An ingenious arrangement was adopted some years ago by Mr. J. H. Fuertes in the case of the Paxton Creek intercepting sewer for Harrisburg, Pa. The permissible grade for this pipe line was so light that the resulting velocity was altogether inadequate, so the diameter was increased from about 18 inches to 5 feet and a full cross-section was insured by automatically admitting water from the creek so that this water added to the sewage would secure the needed velocity. The increased cost of the sewer was here justified by the elimination of pumping which would otherwise have been necessary.

For sewers flowing constantly, either full or half full (the velocity is the same at both points, increasing at a point $\frac{8}{10}$ full to a maximum of 112 per cent), at a velocity of 2 feet per second, there are required, according to Kutter's formula, grades as follows ($n = .013$):

6″	8″	9″	10″	12″	15″	18″	20″	24″	size
.7	.5	.4	32	.22	.16	.11	.10	.09	grade

Latham's tables, based on Weisbach's formula, give the following:

6″	8″	9″	10″	12″	15″	18″	20″	24″	size
.34	.26	.23	.20	.18	.14	.11	.10	.09	grade

—showing a large difference for the smaller sizes as explained in the discussion of the two formulæ (see p. 167).

Staley and Pierson say that a 6-inch lateral laid on a $\frac{4}{10}$ per cent grade works in a fairly satisfactory way, but Hering advises $\frac{5}{10}$ per cent is possible. At Ithaca, where all the sewage has to be pumped and where all the sewers in the valley are laid on the minimum grade, the grades adopted were:

6″	8″	10″	12″	15″	18″	20″	24″	size
.5	.5	.45	.40	.35	.25	.20	.20	grade

Examination of the plans of engineers throughout the country discloses that fact that there is considerable latitude in the grades adopted as the minimum, implying either that the knowledge of the grade needed for the minimum velocity given above is indefinite, or that ideas of what the minimum velocity is, vary. At the lower end of a main where there are no house-connections and where, should the sewer get blocked, backing up of the sewage would do no harm and would probably accumulate a head which would force out the obstruction, a grade or velocity less than those given might be tolerated. And at the upper ends of laterals where, although the amount of flow is probably small, a flush-tank can be placed to wash out periodically whatever might form a stoppage, light grades can be used, however undesirable. Between these two extremities of the line, grades less than those given are unwise and a source of continual trouble. Advantage is sometimes taken of natural aids to get intermediate flushing, as proximity to some stream, or to breweries or swimming-tanks, and on this account the grades are lessened. The whole subject gives an excellent opportunity for experimental work in sewers actually in use, and is open to much more enlightenment.

PROBLEMS

52. If a 12-inch sewer flowing half-full has a velocity of 3 feet per second, what would be the velocity of flow with a depth of 1 inch? (Use tables from Trautwine's or Kent's Pocketbooks for values of area and wetted perimeter.)

53. If a combined sewer, 4 feet diameter, is so laid as to secure a velocity of flow of 2.5 feet per second flowing full, what would be the velocity of flow for the house-sewage with a depth of 4 inches?

54. Using the Chezy formula ($v = c\sqrt{R \cdot S}$), find the value of S to give a velocity of 3 feet per second in a 24-inch pipe, half-full ($c = 100$).

55. If the estimated sewage-flow is 2 cubic feet per second and the available fall is only $\frac{3}{8}$ inch in 100 feet, find the necessary diameter of sewer, flowing full, to give a velocity of 2.5 feet per second, assuming creek-water can be added. Use formula $v = c\sqrt{R \cdot S}$ with $c = 120$.

56. If a 6-inch lateral is laid on a $\frac{4}{10}$ per cent grade, what is the theoretical velocity? (Use $v = c\sqrt{R \cdot S}$ with $c = 70$.)

CHAPTER XI

DEVELOPMENT OF FORMULÆ FOR FLOW

THE first attempts to discover the law by which the velocity of running water depends on the fall and cross-section of the channel is supposed to have been made in 1753 by Brahms, who observed that the acceleration which we should expect in accordance with the law of gravity does not take place in streams, but that the water in them acquires a constant velocity. He points to the friction of the water against the wet perimeter as the force which opposes the acceleration, and assumes that its resistance is proportional to the mean radius, R; that is, to the area of the cross-section divided by the wet perimeter, or $v=CR\sqrt{H}$, with C equal to a fraction multiplied by $\sqrt{2g}$.

In 1775 Brahms and Chezy, the latter a celebrated French engineer, whose most famous work was the Burgundy Canal, made the next advance, and altered the relation between the velocity and the mean radius. These engineers are to be regarded as the authors of the well-known formula usually known as the Chezy formula, viz.,

$$v=C\sqrt{\frac{A}{P}\cdot S}=C\sqrt{R\cdot S};$$

or velocity equals a constant multiplied by the square root of the hydraulic radius and by the square root of the slope.

Dubuat, 1779, undertook to determine experimentally some of the laws governing flowing water, and for that purpose he made an elaborate series of gagings of some French canals and of artificial channels. His results are summed up in these two laws:

1. The force which sets the water in motion is derived solely from the inclination of the water-surface.

2. When the motion is uniform the resistance which the water meets, or the retarding force, is equal to the accelerating force.

He also showed that the resistance is independent of the weight or pressure of the water, so that its friction upon the walls of pipes and channels is entirely different in its nature from that existing between solid bodies.

Coulomb's investigations, a little later, indicated that the resistance offered by the perimeter of a channel is represented by two values, the first of which is proportional to the velocity, and the second to the square of the same. Upon this principle, de Prony based his formula

$$R \cdot S = av + bv^2;$$

in which a and b are constants to be derived from experiments. From thirty measurements by Dubuat and one by Chezy, de Prony found, for metric measures, that a equals 0.000,044 and b equals 0.000,309. Somewhat later, Eytelwein, after comparing the above thirty-one experiments with fifty-five others by German hydraulicians, suggested that a should equal 0.000,024 and b equal 0.000,366.

This formula of Eytelwein is a familiar one, and reference is made to Proc. Inst. C. E., Vol. XCIII, p. 383, for extensive tables on sewer design, based upon it.

Many authorities, seeking to simplify the expression, held that it would be permissible to shorten the formula by neglecting the term $a \times v$, which is very small for large streams especially, reducing the form to that of the Chezy formula again.

For this modified formula the value of b is given as 0.0004, later taken by Eytelwein as 0.000,386, and it has been much used in Germany and Switzerland until recently. It gives in metric units

$$v = 50.9\sqrt{R \cdot S},$$

and in English units

$$v = 92.2\sqrt{R \cdot S}.$$

It was noticed, however, that while this formula agreed with the experiments for certain conditions of slope and velocity, it would not hold for others; so that, as an improvement, Ruhlman and Weisbach deduced from the same experiments varying values of the constant to correspond with the varying values of velocity. The following table gives the values of c varying with v as given by Weisbach.

TABLE XXII

Veloc. V = ft. per sec.	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.5	2.0	3.0	5.0	7.0	10.0	15.0
Const. C =	72.8	76.6	79.3	81.1	82.6	83.8	84.6	85.4	87.8	89.1	90.4	91.5	92.0	92.4	92.7

The table shows the values of c, for velocities common to sewers, to lie between 89 and 91, which are undoubtedly correct for certain kinds of channels. But, as will be seen later, the physical conditions of the channel also affect the values of c, so that, without knowing the conditions of the experimental channels on which these values of c are based, the results are uncertain for general use.

Baldwin Latham gives very elaborate tables based on these values of c, giving grades necessary to produce velocities of from 2 to 10 feet per second in pipes flowing $\frac{1}{3}$, $\frac{1}{2}$, $\frac{2}{3}$, and full, the pipes being both circular and egg-shaped. Similar tables are given for discharge. Except that diagrams are now so largely used, a reproduction of these tables with better values of v might well be made, for their convenience and general adaptability are remarkable.*

According to Dubuat, de Prony, and all hydraulicians up to their time, differences in roughness in the wet perimeter, or irregularities in the direction of the stream, had no effect on the value of the coefficient. It was assumed by Dubuat that a layer of water adheres to the walls of the pipe or channel, and is therefore to be regarded as the wall proper sur-

* The elaborate treatise on Sanitary Engineering, by Col. E. C. S. Moore, published early in 1899, contains such tables as are here suggested.

rounding the flowing mass. According to Dubuat's experiments the adhesive attraction of the walls seems to cease at this layer, so that differences in the material of the walls produce no perceptible change in the resistance. That this reasoning is not good we now know; but since the early experiments on the value of the coefficient were made under conditions in which the wall-surfaces differed but little, and since no new experiments were made until the middle of this century, engineers, however much convinced of the unreliability of these early formulæ, were not in a position to construct a more accurate one. It was left to M. Darcy, Inspecteur Général des Ponts et Chaussées, to whom the city of Dijon owes her excellent water-supply, to open the way to a better understanding of this subject. In the Dijon water-pipes M. Darcy noticed,* as had been observed by others, that those pipes which presented the smoothest inner surface furnished the greatest quantity of water in a given time, or, in other words, that the greatest velocity was found in the smoothest pipes. He argued that a similar phenomenon must take place in open channels, and undertook to make a thorough and extensive series of experiments upon this point. By special authority of the government, he had constructed near Dijon, on the Canal de Bourgogne, a special canal 6 feet wide, 3 feet deep, and about 1850 feet long. It received its water from the canal and discharged it into the River l'Ouche. The water was supplied by two reservoirs at a constant head, and the amounts were measured by a series of carefully calibrated weirs. The canal was furnished successively with different linings, viz., neat cement, 1 : 3 mortar, boards, brick, fine and coarse pebbles, and laths, nailed transversely to the direction of the current, .01 and .05 metre apart. The grades were varied from .001 to .009 per unit of length. Besides these experiments, all known data were collected and compared. Just as M. Darcy had completed these arrangements, most of them pre-

* Recherches expérimentales relatives au Movement de L'Eau dans les Tuyaux, par Henry Darcy, 1857.

liminary, he died, in 1860, and the carrying on of the experiments and drawing up of the conclusions fell to his assistant, M. Bazin. It was the latter who arranged and conducted the gagings and extended them to several branches of the Canal de Bourgogne, who collected and digested the numerous results, and who has written an elaborate book on the subject, embodying the results of years of investigation and study.* Bazin made two general deductions:

1. The coefficient c of the formula for the determination of the mean velocity in canals and rivers of uniform flow varies with the degree of roughness of the wetted surface.

2. These coefficients c vary much more nearly with R than with v.

He further noticed a change in c corresponding to a change in s, but he did not consider it of sufficient importance to be taken into account.

From his study and the knowledge thus gained M. Bazin established a new formula, making it applicable to his experiments and having v change with the differences in the roughness by having four classes of surfaces, with special coefficients for each class, and putting every channel into one of these four classes. He takes the abbreviated formula of Eytelwein,

$$RS=bv^2,$$

and makes the constant

$$b=\alpha+\frac{\beta}{R},$$

or

$$RS=\left(\alpha+\frac{\beta}{R}\right)v^2, \text{ or } v=\sqrt{\frac{1}{\alpha+\frac{\beta}{R}}}\sqrt{RS}.$$

To determine values of α and β, M. Bazin plotted for con-

* Recherches Hydrauliques, entreprises par M. H. Darcy: continuées par M. H. Bazin, 1865.

stant slope and constant roughness a series of experiments, using formula in the form

$$\frac{RS}{v^2}=\alpha+\frac{\beta}{R},$$

the y-ordinates being values of $\frac{RS}{v^2}$ or $\frac{1}{c^2}$, and the x-ordinates, values of $\frac{1}{R}$. α then gives the distance of the origin from the point where the Y-axis is cut by the straight line, drawn as nearly as may be through the points; and β is the tangent of the angle which the line makes with the X-axis.

In this way four sets of coefficients were obtained, here given in English measure.

I. Cement and carefully planed wood:

$$\alpha=.000,046; \quad \beta=.000,0045.$$

II. Smooth ashlar, brick, and wood:

$$\alpha=.000,058; \quad \beta=.000,0133.$$

III. Rubble masonry:

$$\alpha=.000,073; \quad =\beta.000,0600.$$

IV. Earth:

$$\alpha=.000,085; \quad \beta=.000,3500.$$

V. Carrying detritus and coarse gravel:

$$\alpha=.000,122; \quad \beta=.000,7000.$$

The last class was added later by Kutter.

In his treatise M. Bazin says: "One must regret the substitution for a single simple formula of a new formula with variable constants in the coefficients; but the indeterminate character of the coefficients is an inconvenience peculiar to the nature of the phenomena, and further progress in hydraulic theory can never remove it. There are, moreover, very few physical laws the formulated expression of which does not include like indeterminates."

The experiments of M. Bazin and the coefficients derived from them are standards in hydraulic history, whose accuracy has never been questioned, and are to-day used throughout France in determining the velocity in open channels. Tables based on this formula are given by Flynn in his Hydraulic Tables (p. 180).

About the same time that M. Bazin was making these important experiments in France, Humphreys and Abbot were making their well-known experiments on the velocity of flow in the Mississippi River.* They deduced the following formula:

$$v=\left(\sqrt{0.0081m+\sqrt{225R_1\sqrt{s}}}-0.09\sqrt{m}\right)^2$$

where $m=\dfrac{1.69}{\sqrt{R+1.5}}$ for small streams and $m=0.1856$ for large streams.

By experiments made more recently in which this formula has been tested, it has been proved to be not generally applicable, both from the fact that the limits of m are not wide enough and that the influence of slope as introduced is not accurate.

The variation in the velocities of different laminæ of the stream were, however, well brought out by the work on the Mississippi River, and the earlier results of M. Bazin verified. It appeared, especially from the later and more extensive work, that the velocities in a longitudinal vertical plane would form the abscissæ of a parabolic curve with the axis parallel to the surface and at the depth of maximum velocity. This depth, when the air is still, is, according to the Mississippi work, about $\frac{3}{10}$ of the entire depth below the surface. A wind blowing

* Report on the Mississippi River, 1876.

down-stream affects the shape of the parabola, bringing the axis nearly to the surface, so that the surface velocity is the maximum velocity, while an up-stream wind drops the axis below the mid-depth. In the last case the bottom and top velocities were about the same, while the up-stream wind reduced the bottom velocity to about 85 per cent of that at the surface.

It is to be noticed that before the construction of the Kutter formula the most advanced development of the primitive formula $v=c\sqrt{RS}$ was embodied in the formula of M. Bazin, who made the coefficient c vary with (1) the degree of roughness of the wetted perimeter, decreasing with the increase of roughness; (2) the value of the hydraulic mean radius, increasing with its increase; and (3) the slope, decreasing with its increase in large channels and increasing with its increase in small channels.

It remained for Ganguillet and Kutter to combine all these variables into one algebraic expression for the value of c, a discussion of which follows in logical order.

Before taking up the discussion of the Kutter formula, however, the two latest English formulæ may well be noticed. The first, by Henry Robinson, was arrived at, says the author, by Mr. Edgar Thrupp, the author's chief assistant, and is said to be based on the results of direct experiments in sewers, made by himself and by a great many other observers during the last forty years and up to the present time.

The formula is

$$v=\frac{R^x}{C\sqrt[n]{S}},$$

where v is the velocity in feet per second;

R is the hydraulic radius;

S is the length of sewer in which it falls one foot;

C is a coefficient of roughness; and

x and n constants.

The index x, the root n, and the coefficient C depend on the nature of the surface of the channel. For brick sewers in good condition the value of the index x is .61, n is 2, and C is .00746. For cement plaster x is .67, n is 1.74, and C is .004. A diagram given at the end of the book on "Sewage Disposal," by Mr. Robinson, allows velocities and discharges to be read directly.

The other English formula, Santo Crimp's, is given as

$$v = 124\sqrt[3]{R^2} \times \sqrt{S};$$

the same meaning being given to the letters, except that S is the fall divided by the length. It would seem that, with no possible variation for the coefficient, this formula could not equal the others in accuracy, though it is suitable for conditions similar to those upon which it was founded, that is in brick sewers.

With the use of logarithmic plotting, a new interpretation of old data was possible and it remains to refer to two exponential equations based on the old experiments but made possible only by the aid of logarithmic paper.

Sullivan's formula, presented to the world by Mr. M. E. Sullivan of Denver, Colorado, in a small book called "The New Hydraulics," expresses the relation between the three variables in the form $v = CR^{.75}S^{.50}$ in which C varies only as the roughness and as nothing else, and so is absolutely, according to Mr. Sullivan, a correct index of the roughness. For cast-iron and for terra-cotta pipe, a numerical value of 141 is advised for C, and for brick conduits a value of 120.

Messrs. Williams and Hazen soon after produced their little book, explaining their slide rule for solving the various problems of water-flow, and deriving their exponential equation which has the form $v = CR^{.63}S^{.54}$. They say that if exponents could be selected agreeing perfectly with the facts, the value of C would depend on the roughness only, and for any degree of roughness C would then be a true constant. They advise in their formula a value of 140 for new cast-iron pipe, of 100 for old cast-iron pipe, 110 for vitrified pipe and 100 for brick conduits.

PROBLEMS

57. With the aid of Table XXII (first assuming a value of C and deriving v in order to select a proper C from the table) determine values for v when $d=6$ inches and $s=.5$ per cent and for $d=36$ inches and $s=.1$ per cent.

58. If Latham's book on Sanitary Engineering is available, look up values in those tables for comparison with results of Problem 57.

59. Compute, using Table XXII, the value of v when $d=48$ inches and $s=6$ inches in 100 feet. Compare result with tables in Moore's "Sanitary Engineering."

60. Determine the value of C from Bazin's formula, using the constants for brick and assuming that $D=12$ inches. Compare with the constant obtained for Table XXII.

61. In the Williams-Hazen formula, take $C=110$, $D=30$ inches and the fall of the water surface 7.98 feet in 4160. What is the velocity of flow and what the discharge with the pipe half-full?

62. In the Sullivan formula, take $C=125$, $D=2.5$ feet and the hydraulic grade, 11.85 feet in 6170. What is the velocity of flow and what the discharge, if the pipe flows full?

63. Determine the values v and Q, by the Santo Crimp formula if the slope and diameter are respectively .00192 and 30 inches.

64. Determine the values of v and Q in the Robinson formula if $C=.004$, $n=1.74$ and $x=.67$, the other data being identical with Problem 62.

CHAPTER XII

KUTTER'S FORMULA

In the endeavor of Ganguillet and Kutter to construct a new formula which by proper variation of its constants should be applicable to all streams, small as well as large, to pipes as well as to rivers, the original Chezy formula was used as a basis. The way in which their complicated formula was made up is here roughly outlined, both to indicate more clearly the meaning of and reason for its terms, and also to show a method by which other empirical formulæ can be constructed. Their method was to assume and demonstrate that the value of "c" would be best formulated by an expression of the form used by M. Bazin, but into which the element of roughness should be introduced. M. Bazin's formula was

$$v=\sqrt{\frac{1}{\alpha+\frac{\beta}{R}}}\sqrt{R\cdot S};$$

or, making $1/\alpha$ equal y and β/α equal x, it would read

$$v=\sqrt{\frac{y}{1+\frac{x}{R}}}\sqrt{R\cdot S}.$$

Ganguillet and Kutter at first thought that y could be made constant, that is, independent of roughness, which would be made a function of x, equal to $n\cdot y$, or $n\cdot y^2$, etc. To check this assumption and at the same time find the value of y, a number of the gagings of Bazin were selected and curves drawn with values of $1/\sqrt{R}$ for abscissæ and $1/c$ as ordinates. Average lines were then drawn through the points thus plotted. Then, since the expression c equals $\dfrac{y}{1+\dfrac{x}{\sqrt{R}}}$, can be transformed by taking the reciprocal of each side, the expression becomes

$$\frac{1}{c}=\frac{1}{y}+\frac{x}{y}\times\frac{1}{\sqrt{R}},$$

or it has the form of an equation of the first degree and of a straight line in which if values of $1/c$ are plotted as ordinates and of $1/\sqrt{R}$ as abscissæ, then $1/y$ must be the constant term and x/y the tangent of the angle of inclination. If $1/y$ is constant, it will appear by all the lines passing through the same point on the axis of y. Far from doing this, the lines plotted cut the axis at points unmistakably far apart, and the divergence was especially noticeable when small flows with steep grades and large flows in rivers with low grades were compared. It seemed then that, in the formula, y must be made a variable as well as x. After repeated trials to get the plotted points to agree with the curves drawn by the formula, y was made equal to $a+\frac{l}{n}$, and x to $a \cdot n$, so that C equals

$$\frac{a+\dfrac{l}{n}}{1+\dfrac{an}{\sqrt{R}}}$$

the above values or relations being determined by series of gagings made with the same slope as nearly as possible. Up to this point in the development, c had been made to vary with R and with n, and it remained to make it vary with S. It was known from the experiments of Bazin and from those of Humphreys and Abbot that it must be brought in in such a way that c would increase as the slope decreased for large rivers, and would decrease as the slope decreased for small channels and pipes. In other words, there must be a point or a certain value of R or a certain sized stream in which the slope had no effect on c, but that for that size the value of S could increase or diminish without affecting the value of c. From the available data a series of points were plotted with values of c and R as variables, and all with the same slope as near as possible. In this way a series of lines were obtained, each representing a certain slope, and it was found without doubt that they intersected at a point whose value for $1/\sqrt{R}$ was one metre approximately, or whose value of $1/c$ was .027 in metric units. The element of slope was introduced, arbitrarily, by making y equal to $a+l/n+m/S$; then, preserving the relation $x=ny-l$, x was made equal to $\left(a+\frac{m}{S}\right)n$. Then, to determine the values of l, points were plotted from streams whose value of $1/\sqrt{R}$ was 1, and the roughness of whose channels was similar, and the value of l was found to be 1.00. Then, to get a, points were plotted with values of $1/S$ as abscissæ and of y as ordinates, and the point where the line intersected the axis gave a equal to 23, and m, or the tangent of the angle, equal to .00155. The constants a, l, and m being determined, it remained to find values for n for different channels. This was done by again plotting points of actual gagings for different streams and finding corresponding values of n. In this way the values were found to range from .009 to .040.

To sum up, then, from the original formula

$$v=c\sqrt{R\cdot S},$$

in which c equals

$$\frac{y}{1+\frac{x}{\sqrt{R}}},$$

where y equals

$$a+\frac{l}{n}+\frac{m}{S},$$

and where x equals

$$\left(a+\frac{m}{S}\right)n,$$

$$v=\frac{23+\frac{l}{n}+\frac{.00155}{S}}{1+\left(23+\frac{.00155}{S}\right)\frac{n}{\sqrt{R}}}\sqrt{R\cdot S} \text{ in metric units,}$$

or

$$v=\frac{41.66+\frac{1.811}{n}+\frac{.00281}{S}}{1+\left(41.66+\frac{.00281}{S}\right)\frac{n}{\sqrt{R}}}\sqrt{R\cdot S} \text{ in English units.}$$

or

$$v=c\sqrt{R\cdot S}.$$

The values given for n are as follows:

I. Channels lined with carefully planed boards or with smooth cement 010

II. Channels lined with common boards or surfaces carefully plastered with cement-mortar, one-third sand, in good condition, also for iron, cement, and terra-cotta pipes, well jointed and in best order, and for other surfaces equally rough 011

III. Channels lined with unplaned timber or rough cement-mortar 012

IV. Channels lined with ashlar and well-laid brick-work, ordinary metal, earthenware, and stoneware pipe in good condition but not new, cement and terra-cotta pipe not well jointed nor in perfect order, plaster, and planed wood in imperfect or inferior condition, and other surfaces equally rough 013

V. Channels in rubble masonry 017

VI. Channels in earth; rivers 025

VII. Streams with detritus 030

The unwieldy nature of the formula given above has led to almost general use of graphical methods of solution. In the first notice of the formula in the Swiss exhibit in Philadelphia, 1876, there was shown with the printed exposition a diagram familiar to us in its English units, by means of which c could be graphically determined. It is printed in the back of the translation of Kutter's book by Hering and Trautwine, and can be found in some of the pocket-books. Mr. Hering in his translation gives tables for x and y, by means of which the diagram can be replotted at any time. Similar tables are given in Jackson's translation.

But even with this diagram to aid in finding c, several algebraic reductions need to be made before the real purpose of the formula, that is, the value of v, is known. Trautwine in his pocket-book devotes four pages to tabulating c for different values of S, R, and n, when values of v might have been given. After c is known the square root of the product of R and S must be multiplied by c to get v.

In Vol. VIII of the Transactions of the Am. Soc. C. E., p. 1, Mr. Hering gives a method by which the velocity can be at once read from the diagram constructed for c. His reasoning is very simple. The equation

$$v=c\sqrt{R}\cdot\sqrt{S}$$

can be written

$$\frac{v}{\sqrt{R}}=\frac{c}{\sqrt{1/S}},$$

or the four terms are in a simple proportion, so that by plotting the values of c and of $\sqrt{1/S}$ on one side, and of v and of $\sqrt{R}$ on another side, of an angle, the corresponding relations will be represented by similar triangles. In Kutter's diagram the coefficients c are already plotted on the vertical and the values $\sqrt{R}$ on the horizontal axis; by plotting an additional scale of grades on the former and of velocities on the latter axis the graphical solution is complete by merely drawing parallel lines. The article referred to gives the diagram with tabulated values of x and y, and of the relation of $\sqrt{1/S}$ to g, or the grade per hundred. Several numerical examples are also given.

But even this graphical determination of v is not enough. In sewer-design, except at the limits, the value of v is useful only as it enters into the value of Q. A diagram, then, to be thoroughly useful should give at once the value of Q from the physical data, viz., slope and size of pipe, and the next chapter is devoted to the construction and use of such diagrams. It remains in connection with Kutter's formula to mention the set of tables which, except in the form of a graphical diagram, give the formula most conveniently for use. Reference is made to Flynn's tables, published as Nos. 67 and 84 of Van Nostrand's Science Series. These are made possible in their form by establishing the fact that, within the ordinary limits of use for pipes, sewers, and conduits, the value of s affects the value of c almost not at all, and therefore s may be taken as constant. c then varies only with R and n. Tables are calculated for any one value of n, values of c being given for values of R. Instead of tabulating the values of c, however, it is noted that the equivalent of Q, viz., $Ac\sqrt{R \cdot S}$ can be broken up into the two factors $Ac\sqrt{R}$ and $\sqrt{S}$, and the value of v can be taken as the product of $c\sqrt{R}$ and $\sqrt{S}$.

Further, since for pipes flowing full the value of R is proportional to the diameter of the pipe, diameters are written instead of values of R; the tables then give (for a certain value of n) diameters of pipes from 5 inches up to 20 feet, and for those diameters the corresponding values of A, of R, of $c\sqrt{R}$, and of $Ac\sqrt{R}$.

For any slope, its square root, given in another table, multiplied by $c\sqrt{R}$ or $A \cdot c\sqrt{R}$ for the desired diameter gives the resulting velocity or discharge.

No. 67 gives tables for circular sewers from 5 inches to 20 feet in diameter with $n = .015$; tables for egg-shaped sewers (old form) $1' \times 1'\,6''$ to $12' \times 18'$, flowing full, two-thirds full, and one-third full, with $n=.015$; tables of S and $\sqrt{S}$, ranging from $S=1$ in 4 to $S=1$ in 2640, or from 25 to .028 per cent.

No. 84 gives, with other tables and discussions, tables for circular pipes flowing full, the diameters ranging from 5 inches to 20 feet, and with values for n of .011, .012, and .013. A table of slopes is given varying from $S=1$ per cent to $S=.053$ per cent, decreasing by small amounts, so that the table is very convenient.

To illustrate the use of Flynn's tables the following examples are given, using No. 84:

1. What are the velocity and discharge of an 8-inch sewer flowing full on a grade of .4 per cent, n being assumed at .013?

From the table for $n=.013$, and for a value of $d=8$, $c\sqrt{R}$ is 31.00 and $Ac\sqrt{R}$ is 10.822. From a table of square roots, $\sqrt{S}$ is .06325. Then v equals $31.00\times.06325$, or 1.96 feet per second, and Q is $10.822\times.06325$, or .68 cubic foot per second.

2. What will be the size of sewer required to carry off a flow of 3.6 cubic feet per second, the grade being .125 per cent, n being taken at .013?

From the table for $n=.013$ a value of $Ac\sqrt{R}$ must be found which multiplied by $\sqrt{.00125}$ shall equal 3.6. This is 3.6 divided by .0354, or 101.7, which corresponds to a diameter of 1 foot 6 inches, the value required. Similarly the velocity will be the product of $c\sqrt{R}$ found in the same line, or $57.80\times.0354$, that is, 2.05 feet per second.

3. On what grade must a 24-inch pipe be laid to secure a velocity of 2.5 feet per second, n being taken at .011?

From the table for $n=.011$, and a diameter of 24 inches, $c\sqrt{R}$ is found equal to 87.36, which multiplied by $\sqrt{S}$ must be 2.5. $\sqrt{S}$ is therefore .0285, and S, .0008, or .08 per cent.

4. What grade is necessary to discharge 8.5 cubic feet of sewage through a 20-inch pipe, and what will be the velocity, n being .012?

From the table for $n=.012$, and a diameter of 20 inches, $Ac\sqrt{R}$ is 150.61, which multiplied by $\sqrt{S}$ must be 8.5; $\sqrt{S}$ must therefore be .05637 and S is .00267, or .267 per cent.

In using Kutter's formula, or tables or diagrams prepared from it, it must be remembered that the resulting value of v depends, even with known values for s and R, upon the judgment of the engineer in selecting the proper value for n. Kutter gives a value of .011 for cement and terra-cotta pipe in good condition, and .013 for stoneware pipe in good condition but not new, and for cement and terra-cotta pipe not well jointed. These values, from experiments made by the author in pipe sewers, seem to be true only for perfectly clean pipes; and whenever accumulations of silt occur, or in pipes with any projecting cement, these values are too small. Probably .013 for pipes and .015 for brickwork would agree more closely with actual sewer-gagings than the values given above.

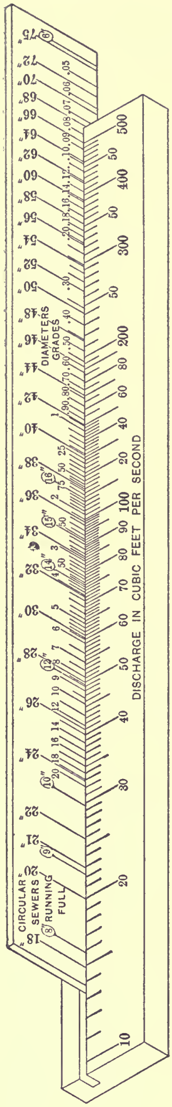


Fig. 46.

It may, however, be well to note that there is some evidence tending to show that the values of n as just given are not constant, but change with the depth of flow. In a thesis by Glenn D. Holmes written in 1897 under the direction of the author, are given some values of n found experimentally for clean sewer-pipe on different grades. The values found varied from .007 to .021 for the differing grades (.56 to 2.51 per cent) and for the varying depths. As the depths increased in the experiments, reaching the half-full point as a limit, the values of n were increased for the higher grades and decreased for the lower, with the evident meeting-point at $n = .013$, agreeing with the common assumption. Further experiment in this direction would seem desirable.

As a convenient and ingenious method of finding values of Q from the grade and size of pipe, "Colby's Sewer Computer" is of value. Based on Kutter's formula, with $n = .013$, the logarithms of the grades, discharges, and diameters are laid off on the rule and runner, so that by proper setting of grade and diameter the discharge can be at once read off. No velocity is given, although the runner could have additional divisions for this purpose. The rule is shown in Fig. 46.

PROBLEMS

65. Determine numerically the difference in the value of C between values of $n = .013$ and $n = .015$; for values of s between .005 and .0005, and for values of R between $R = .125$ and $R = 1.25$.

If a diagram is based on a value of $n = .013$, by what per cent would v be increased or diminished for $n = .015$.

66. For ordinary range of values used in sewer work show numerically how much variation in s would affect the value of c.

67. Compute average values of C for ordinary sizes of pipes, and for ordinary range of s, with $n = .013$.

68. Given an open ditch section, of side slopes 2 horizontal to 1 vertical with bottom an arc of 18″ radius tangent to the sides, determine the depth of flow, on a grade of .005 per cent, to carry 35 cubic feet per second. Ditch is lined and n may be taken at .015.

69. Find the size of a trough whose width is double its depth that will deliver 180 cubic feet per minute. Assume the slope to be 2 feet in 1000 feet and the coefficient of roughness, n, to be .013.

70. An outfall flume is to discharge 40 cubic feet of sewage per second. It is to be built of plank ($n = .011$) on a slope of 1 in 3000. What should be the dimensions to give the minimum amount of lumber.

71. If a 36-inch brick sewer has a coefficient of roughness $n = .015$ and a 36-inch pipe sewer one of .013, compute the grades in the two cases necessary to give velocities of 2 feet per second. If the sewer were a mile long, how much deeper in the ground would the brick sewer be?

CHAPTER XIII

SEWER DIAGRAMS

WHILE the earlier formulæ were not so complex that their solution was especially tedious, the later ones, and especially Kutter's, are of that nature. It is therefore not only in keeping with the general tendency of the times to reduce all computations to graphic or other approximate and time-saving methods, but it is almost a necessity if the formulæ are to be of practical use.

Diagrams, to be of service, must fulfil the following conditions: they must deal directly with the quantities of interest, not with some function of those quantities; they must be on a scale large enough so that the error of reading may be within the allowable error of the result; they must be equally serviceable for all sizes, velocities, etc.; they must be so constructed as to give well-defined intersections at all parts. An advantage of the diagram, besides the time and labor saved, lies in the possibility of comparing of the quantities involved, and this should not be overlooked.

The diagrams that have already been printed may divided into two classes: first, those based on Latham's tables or Eytelwein's formula; and second, those based on Kutter's formula or on modifications of it, such as Flynn's tables.

Of the first class may be cited the extensive diagrams of Mr. W. T. Olive printed in the Proc. Inst. C. E., Vol. XCIII, p. 383, very elaborate and complete and models of the sort.

In the same publication, Vol. XCVI, p. 268, are diagrams giving discharges as before, and also giving the relations between the velocities and discharge at different depths in both circular and egg-shaped sewers.

The diagrams in the "Separate System of Sewage," first edition, by Staley and Pierson, are made up from Latham's tables, and the difference between these values and those from Kutter's formula are well shown in the second and later editions, where Kutter's lines are printed in red on the same plate.

Among the diagrams compiled by J. Leland FitzGerald, reprinted on a plate in Baumeister's "Sewerage" (First Edition), is one also based on Latham's tables, giving the discharge of circular and egg-shaped sewers. (*Engineering News*, Vol. XXIV, p. 212.)

The first diagram based on Kutter's formula was that published in the Trans. Am. Soc. C. E., Vol. VIII, p. 1, by Rudolph Hering, and reprinted by the Society for general use. It has discharges for ordinates, and slopes in feet per hundred for abscissæ. The intersecting curves are those for velocities and diameters, and a separate sheet is required for different values of n. One such sheet ($n = .013$) is given in *Engineering News*, Vol. XXXII, p. 449.

A diagram computed by Mr. Moore of St. Louis is given in the Journal of the Association of Engineering Societies, Vol. V, p. 360, whose ordinates are diameters, and abscissæ discharges in cubic feet per second. The intersecting curves then are velocities and grades, given in fall per hundred feet. This is not as well adapted for use as the first, both in that the intersections are more oblique and that in order to read for small pipe a supplemental diagram of the corner has to be redrawn on a larger scale.

In the *Engineering News* for August 11, 1892 (Vol. XXVIII, p. 127), is a carefully drawn diagram by Professor Talbot of the University of Illinois. Here the discharges were made ordinates, and the gradients in per cent the abscissæ. The square roots of the gradients were plotted instead of the gradients themselves, and in order to get better intersections the axes of the diagram are inclined towards each other. The line of equal diameters becomes a straight line, and in order to get the 6- and 8-inch pipes on the diagram their discharges are made ten times the true value. The diagram is supplemented by Mr. F. S. Bailey, in order to show larger sizes and is published in enlarged form in *Engineering News*, Vol. XXXII, p. 403, the construction being as before except that the value of n is taken at .015.

A rather complicated set of formulæ has been published by Messrs. Adams and Gemmell (see *Engineering News*, Vol. XXIX, p. 396) for sewers from 6 inches up to 5 feet in diameter. All the values are kept in one diagram by placing one part of the diagram to a certain scale over another part already drawn to a larger scale, and by reading ordinates for one part of the diagram on one side and for another to a different scale on the other. The result is a very compact diagram, but one likely to lead to confusion. These are the diagrams given in the very convenient "Sewerage Engineer's Note-Book" by Albert Wollheim, London, who thinks them the "most handy diagrams yet printed."

In the *Paving and Municipal Engineer*, Vol. VII, pp. 116 and 119, are two diagrams by John W. Hill.

In order to make more definite the errors involved by using the tables of Latham or diagrams based on those tables, values of v, for various slopes and sizes of pipes have been computed and are shown, side by side, in Table XXIII.

Table XXV

SHOWING VALUES OF v, FOR DIFFERENT SLOPES AND SIZES OF PIPE BY FORMULÆ OF WEISBACH AND KUTTER

Slope.	6" Diameter.		12" Diameter.		18" Diameter.		24" Diameter.		36" Diameter.		48" Diameter.		60" Diameter.	
	Weisbach	Kutter	Weisbach	Kutter	Weisbach	Kutter	Weisbach	Kutter	Weisbach	Kutter	Weisbach	Kutter	Weisbach	Kutter
1 : 10	11.1	8.0	14.9	13.6	17.6	15.0	....	....	....	....	....	....	....	....
1 : 100	3.6	2.5	5.1	4.3	6.2	5.9	7.2	7.2	8.7	9.8	10.0	....	11.1	....
1 : 200	2.4	1.8	3.6	3.1	4.4	4.1	5.1	5.1	6.2	6.8	7.2	8.3	8.0	9.6
1 : 500	1.4	1.1	2.2	1.9	2.7	2.6	3.1	3.2	3.9	4.3	4.5	5.2	5.1	6.1
1 : 1000	0.9	0.8	1.4	1.3	1.7	1.8	2.2	2.3	2.7	3.0	3.2	3.7	3.6	4.3

It will be noticed that for values of $R = .5$, i.e., for 24-inch pipe, the velocities are identical, but that for smaller sizes, Weisbach's formula gives velocities that are too large, and for larger sizes, velocities that are too small. The difference is more marked in a comparison of quantities, especially for the larger sizes. For example, the difference for a 60-inch sewer on a grade of 1 : 500 is that between a discharge of 100.2 and 119.8 cubic feet per second, or nearly 20 cubic feet per second, which is about the capacity of a 24-inch pipe.

A recent book (1910) on water supply and sewerage, in spite of the discrepancies between the values of the old formulæ and the more modern ones, has included in its text a plate giving values of v and Q for varying values of d and s, basing the diagram on Latham's old tables. From the diagram, the values given in Table XXV in the columns marked Weisbach can be checked and the differences shown there would be found also in any designs based on the diagram referred to.

All the diagrams consider the four quantities, size of pipe, velocity of flow, grade, and discharge, while the element of roughness is left out, considering that it is the same for all the quantities included in the diagram; and if, as when the sewer changes from pipe to brick, it is necessary to change n, another diagram has to be made. It is possible to construct a diagram having any two of the above quantities as ordinates and abscissæ, while the other two quantities appear as curves crossing the axial lines, and each other at various angles. Further variations can be made by constructing the diagrams in parts, each to a different scale; by using a logarithmic scale for one or both axes; by laying off, instead of the diameters or the corresponding values of R, the values of the square root of R. The grade may be expressed and drawn in per cent, in feet per mile, or in number of feet for a fall of one foot. Separate diagrams have to be prepared for brick sewers and for pipe, for circular sewers and for egg-shapes, so that for completeness three diagrams, however made, should be provided.

The diagrams in Plates 3, 4, and 5 are given with the idea that they will serve all purposes of actual design. The abscissæ

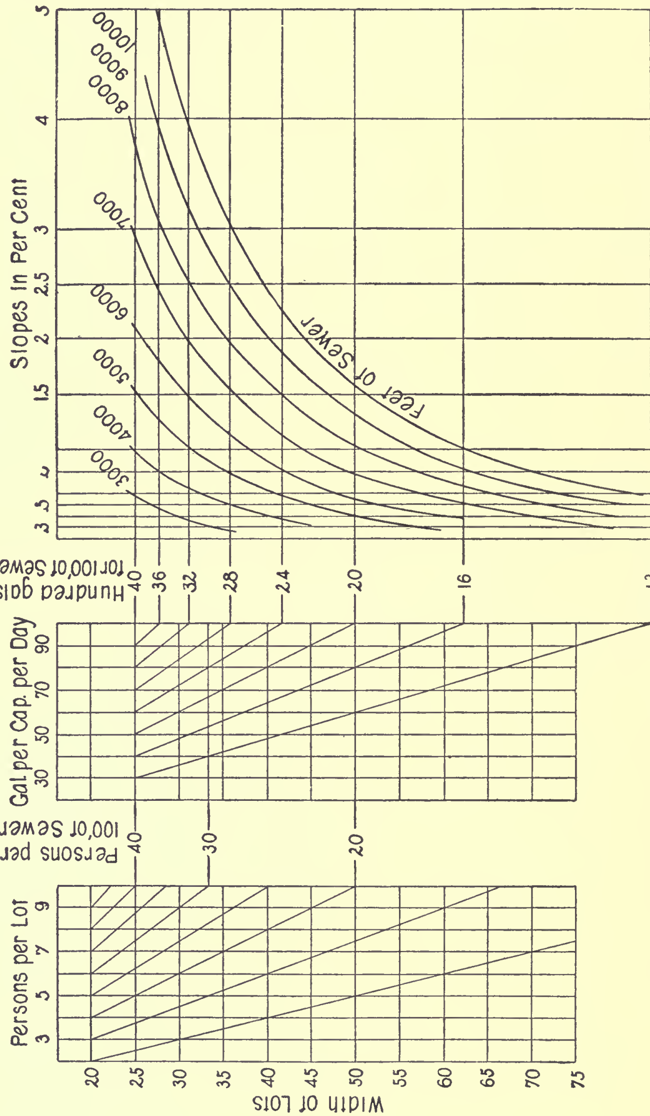


FIG. 47.

are grades in per cent, and the ordinates discharge in cubic feet per second, the logarithms of both quantities being plotted instead of the numbers themselves. The advantage of this method of plotting, shown in the good intersections, is evident.

The diagram shown in Fig. 47 is given as being convenient for laying out laterals. By its use the greatest length possible for a 6-inch pipe flowing full to be laid for contributing house-drains can be read off at once. As the diagram shows, the first factors are the width of lots and the probable number of persons per lot. This is changed into the number of persons for one hundred feet of sewer and combined with an assumed number of gallons per head per day. This gives gallons per hundred feet of sewer, which, taken with the grade of the sewer, gives gallons capacity of length of sewer, to which the assumed contribution is made. A similar diagram can easily be made for 8-inch pipe.

PROBLEMS

72. Using values found in Flynn's Tables, construct a diagram on cross-section paper, having slopes in feet per thousand for abscissæ and discharge in million gallons per day for ordinates.

73. Construct three curves, showing the relations between sizes of pipes and slopes in per cent by which velocities of 2.0, 2.5 and 3.0 feet per second may be obtained according to Kutter's formula.

74. Construct a diagram on logarithmic paper for circular pipes flowing half-full, with slopes in feet per thousand for abscissæ and discharge in cubic feet per second for ordinates. Show curves for both sizes of pipes and for velocities.

75. Using the Williams-Hazen formula, construct a diagram to show relation between slopes and discharges for pipes 6 to 24 inches diameter on logarithmic paper.

76. Construct a diagram for elliptical pipe (or for a basket-handle section) showing discharges for various sizes and slopes.

77. Construct a diagram for conduits whose section is a right triangle, vertex down, showing discharges for various depths and slopes.

CHAPTER XIV

USE OF DIAGRAMS

In order to understand better the use of Kuichling's method for determining the amount of rain-water to be considered and the proportion of the fall reaching the sewers, the following hypothetical example is given of its use in Ithaca, N. Y., the plan of which city is given on Plate I.

It is assumed that the surface-water above Eddy Street will be taken care of by a drain discharging from the end of that street into Six Mile Creek, and that all the storm-water falling on the area between Eddy Street and Aurora Street at the foot of the hill is to be taken care of by a drain running from State Street north to Cascadilla Creek. Evidently three main laterals will lead into this drain—one coming down the hill on State Street, one on Seneca Street, and one on Buffalo Street; while a fourth line, smaller than the others, will enter from Mill Street.

To determine the rate of rainfall the duration required for a maximum flow at the outfall is necessary, that is, the length of time for rain to get from the upper end to that point. The point farthest from the outfall is at the corner of State and Eddy streets, and, scaling from the map, it is 2375 feet to the corner of Aurora and State streets and 1325 feet on Aurora Street to the creek. Down the hill the average grade, from the contours, is about 7 per cent, and if we assume a 12-inch pipe, according to the diagram on Plate 3 the velocity will be 10 feet per second. That is, it will take water about $2375 \div 10 = 237$ seconds, or four minutes, to reach the bottom of the hill.

On the flat there are 1720 feet, and the velocity will be that due to a fall shown by the contours to be 10 feet in that distance, a .58 per cent grade.

By the diagram on Plate 4, assuming that a 3-foot pipe will be needed, we find the velocity to be 6 feet per second, so that it will take 254 seconds, or a little more than 4 minutes, for the water to get to the creek—a total time from the farthest point of 8 minutes. Adding 3 minutes for the time necessary after the storm starts for the rain to reach the gutters and catch-basins, the duration of the storm which will give a maximum discharge is 11 minutes.

Now consulting Kuichling's diagram, Fig. 6, we find the maximum rate of a storm lasting 11 minutes to be 3.18 inches per hour.

On larger areas it will be necessary to determine the time of concentration separately for the different parts of the same area in order that the rate of rainfall may decrease as the combined parts give larger and larger total areas. In the present case, however, the entire area is so small and the time of concentration to the bottom of the hill is so short that no greater accuracy would be secured by computing the rate of rainfall separately for the smaller district. For large areas, however, the times of concentration must be completed in order.

We assume, then, that there is a rain falling at the rate of 3.2 inches per hour which is to be cared for, and note that by the topography no water from the upper side of Eddy Street or above will enter this drain; and that as the velocity is high and the buildings are residences, so that a temporary filling of the gutters will not be an annoyance, the pipe need not begin until the water has reached the corner of Stewart Avenue and State Street. At that point there is a contributing area, scaling it from the map, of $1750 \times 580 = 1,015,000$ square feet, or 23.3 acres. An inch an hour is practically the same as a cubic foot per acre per second, so that the discharge from these 23.3 acres will be $23.3 \times 3.2 = 74.5$ cubic feet per second, provided it all flows off. The area has a population of about 25 per acre, and, by the table given on p. 74, 25.3 per cent of the rainfall will flow off. 74.5×25.3 equals 18.8

cubic feet per second, or the amount of run-off to be cared for by the drain at that point. It may be noted here that on steep hillsides 5 to 10 per cent is sometimes added, but as this area is entirely unimproved and likely to remain so, having a large proportion of lawn and no paved streets, nothing need be added.

By diagram on Plate 3, to discharge 18.8 cubic feet per second on a grade of 10 per cent, which the hill from this point down is seen to be, will take a 15-inch pipe, which will run to Spring Street. Here an area of 540,950 square feet, or 12.4 acres, discharging 25.3 per cent, adds 10.0 cubic feet per second. Since the diagram does not show the intersection of the 30 cubic-foot line with the 10 per cent grade line, the size must be computed. It is found to be an 18-inch pipe. At the foot of the hill, 400 feet from Aurora Street, the grade changes to 0.5 per cent, and on this grade a 30-inch pipe is required, which will run the 400 feet to the corner. Here the drainage of 7 acres, or 5.6 cubic feet per second, enters, making 34.4 cubic feet in all. On the same grade of 0.5 per cent this takes about a 33-inch pipe. At Seneca Street will enter the water from the area between Seneca and State streets and west of Stewart Avenue. This amounts to 11.2 cubic feet per second, making 45.6 in all, requiring a 39-inch pipe. This may be taken from the diagram on Plate 4 and will be either brick or concrete. At Buffalo Street the contributing area is 13.7 acres, or 10.9 cubic feet per second, making a total of 56.7 cubic feet, requiring a 42-inch sewer. On account of the amount of sediment brought down the hill, and the large deposits where the velocity is so retarded, it will be wise to increase the size from here to the creek, making it 48 inches for the remaining distance.

Reviewing the velocities, the farthest point is at the corner of Quarry and Buffalo streets. From here to State Street, with a grade of 6 per cent, the water flowing in an open gutter will have a velocity of about 9 feet per second, requiring, for the 1085 feet, 120 seconds or 2 minutes. Still flowing in the gutter it will take about three-fourths of a minute more to reach Stewart Avenue. In the 15-inch pipe on the 10 per cent grade the velocity is between 12 and 15 feet per second for 300 feet, adding a quarter minute, or 3 minutes in all. For the remaining distance the time will be 6 feet per second, requiring nearly 5 minutes more, or 8 in all. This agrees with the time assumed, and therefore a second rate of rainfall based on this time just found will not be necessary.

For a possible maximum with a shorter storm, if the Buffalo Street lateral is considered, it will take only about 6 minutes for its water to reach the outfall, including the time necessary for the water to reach the gutter, and the corresponding rainfall is 3.4 instead of 3.2 as used before. For the limited area drained by this lateral it is plain that there can be no maximum flow brought down by this pipe.

To illustrate the method of determining the sizes of pipes for domestic sewage, assume that it is required to find the size for the Northern Main in the city of Ithaca, that is, the pipe coming directly to the pumping-station and taking the sewage from the region north and east of Cascadilla Creek. According to the map this is an area of 138 acres, and is populated at the rate of about 30 per acre. The area taken is all that can ever drain into the system, and represents all the future population in the district. The population to be considered is 4140. After studying the water-supply it is assumed that a future provision of 70 gallons per head per day should be provided, that is, an average daily flow of 289,800 gallons. If half of this is supposed to flow off in 8 hours, the rate of flow in those hours will be 18,110 gallons per hour, or 302 gallons per minute, equal to 40 cubic feet per minute. A knowledge of the territory will justify the assumption that fully 2 miles of the pipe-line will be covered with ground-water, which, at the rate of 20,000 gallons per mile per day, will add 4 cubic feet per minute to the flow, making 44 in all. By the topographical conditions the grade will be the minimum, and will be determined by the requirements of velocity. Looking on Plate 3, we find that for 44 cubic feet per minute, with a velocity of 2.0 feet per second, a 12-inch pipe is required. This pipe will run from the lower end to the first lateral, where the volume of flow will be diminished by the amount there contributed. The smaller pipe is continued until the next lateral is reached, and so on.

It is to be noted that while the size of pipe as just taken is calculated so that the estimated flow will fill the pipe only half full, this factor of safety is not always necessary, and in the lower ends of large mains, where the flow is large and comparatively constant, the size of pipe as determined for the exact flow is taken, or the pipe increased by one or two sizes only, as it is evidently bad engineering to require the expenditure of a large amount of money unnecessarily. In studying the grades and sizes for a city, various ways of tabulating and simplifying the large amount of computation can be devised. The relative effect of the grades of one line on another is best seen by plotting the profiles one above another so that the lower end of every lateral is in a straight line directly above the starting-point.

PROBLEM

78. Design the size and grades for an outfall sewer along the route shown on Plate VI. The following suggestions should be followed:

Determine the volume of storm-water and domestic sewage that will have to be taken care of at the point *A* from District I. Assume that the sewer will be 10 feet below the surface at this point and that it will be laid on a slope to give a velocity of 2.5 feet per second to *B*. Assume the drainage from all of District II to enter at *B*. Assume a minimum grade again to *E*, and that the drainage from three-quarters of District III will enter at *D*. From *E* to *F* the grade will be greater, the elevation of the invert at *E* being +5. From *F* to *G*, the grade is 1 per cent and the drainage of one-quarter of District III will enter at *G*. From *G* to the river the grade is that necessary to give a velocity of 2 feet per second. Assume a ground water leakage of 8 cubic feet per day per square foot of interior surface.

CHAPTER XV

SEWER PLANS

The location of the outfall is the prime element among the factors brought together to decide how the mains and laterals of a city shall be arranged. The outfall itself, leading to the place of disposal, is located to agree with the method of disposal chosen, a discussion of which is not here taken up. The outfall may lead to the seashore, to the banks of a stream or lake, to broad farm-lands for irrigation, to a well-adapted area for filtration, or to some low out-of-the-way place for chemical treatment. If the place of disposal is the sea, tides, currents, and winds largely determine the location of the outfall. If onto irrigation-fields, the sewage must be taken wherever suitable land is available, whether down the valley from the town or on the top of a hill above it. The filtration-area must be chosen where proper soil is to be found; unless the area is artificial, when it can be placed more advantageously as regards distance and grade. If chemical treatment is to be practised, only enough land for the buildings and tanks is necessary, and a location to which the sewage can be led by gravity should be obtained if possible. Thus the position of the outfall is not in all cases to be decided by the topography, but is conditional on the final disposal. However, when the sewage is to be turned into a river or lake, the valley lines to the shore are usually followed. If the combined system is used, provided the sewage has to be treated or led away from the nearest point of the river to a point farther down-stream, it is usual to let the storm-water overflow into the river, while the house-sewage is carried along to the desired point. This is done by automatically arranged outlets. By an ingenious device the height of the overflow-weir is so arranged that the overflow begins to discharge when the amount of dilution has reached that point previously decided to be allowable. It is advisable, and these relief-outlets make it practicable, to discharge the storm-water at as many points as possible, keeping the size of the storm-sewers small, carrying the water on the streets as long as possible and avoiding the pouring of a large mass of water and sediment into the river at one point.

The outfall being located, it is possible to arrange the mains and laterals according to one of five systems;* and although

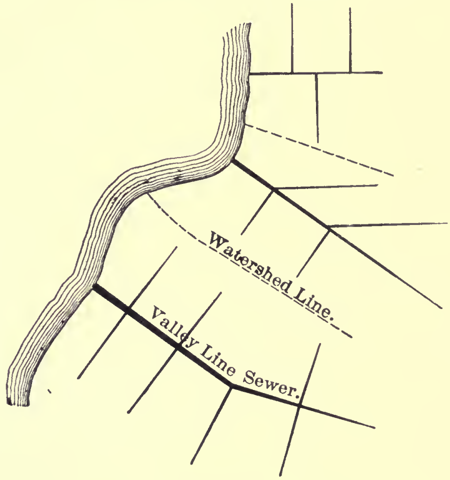


FIG. 48.

there may be combinations of these, so that it is sometimes difficult to recognize the system adopted, yet in the first study of the topography it is advisable to keep these separate arrangements in mind.

(A) Perpendicular system. When the city lies on the bank of a large river or bay, as New York, Philadelphia, or Portland, Me., where the volume of flow in the stream or the change of water at each tide is sufficient to keep the sewage diluted so as to be inoffensive, the only aim is to get the sewage into the water by the shortest path. In this we have what is called

* Baumeister.

the perpendicular system (Fig. 48).* The mains follow down the beds of the separate valleys with laterals running from the ridge-lines between. There are as many mains as there are subordinate valleys, the grades are the best possible, and the sections of the sewers are small. Wherever a flat area is adjacent to the stream, and sewers from higher land must cross this to reach the water, it is possible that in heavy rains the low land may be flooded from the gorged sewers; this, however, is a question of the design and can be avoided.

(B) Intercepting system. If the stream is not large enough for satisfactory dilution and the sewage has all to be carried

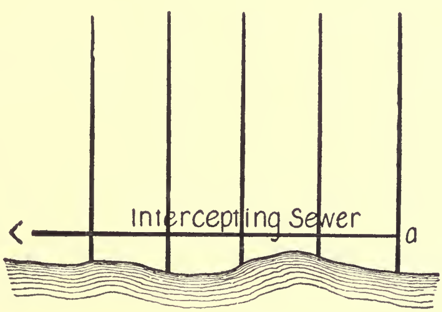


FIG. 49.

to a single point for treatment, or if the river-water is used for domestic purposes, so that the sewage has to be carried down-stream below the intake of the river-water, then the ends of the mains of system "A" are picked up by an intercepting sewer, the combination making the intercepting system, Fig. 49. This is sometimes an after-thought (as in Milwaukee and Chicago), in which case the different elevations of the main ends makes the construction of the intercepting sewer very difficult. Sometimes this intercepting sewer may be designed on such a scale as to pick up the ends of outfall sewers from separate villages or cities. Thus along the Charles River, just outside of Boston, is an intercepting sewer which takes

* Report of the National Board of Health, 1881, page 117 *et seq.*

the sewage from Waltham, Newtonville, and other places which formerly had sewer systems discharging directly into the river. Similarly, in New Jersey along the Passaic River and in New York, along the Bronx River, are sewers which act as interceptors for villages and cities instead of for lines of sewers all within one city. The first system, i.e., the perpendicular, can always be designed so that, when the necessity occurs, the interceptor may be put in with the elevations of the mains properly adjusted. This large sewer is usually expensive to build, being in the lowest ground, often below the stream-level, in gravel or soft mud. Besides preserving the stream from pollution, this system has the further advantage of allowing

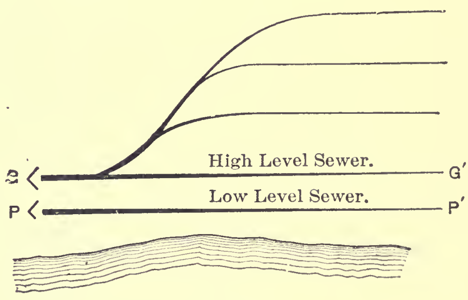


Fig. 50.

all the sewage to be brought to one point for pumping in case this is necessary, so that one large pump can take the place of several small ones.

(C) Zone system. In case the sewage has to be pumped it may happen that a large part of the contributing territory is high enough so that the sewage from that part will flow to the outfall by gravity, and in this case the sewers may be arranged to form the Zone system, that is, a double intercepting system. An intercepting sewer is laid nearly following a contour so that it may discharge all the sewage from the land above it to the outfall by gravity, while the second interceptor collects only that part of the sewage which would in any case have to be pumped (Fig. 50). The advantages are the reduced amount of water to be pumped, the decreased probability of flooding the lowest part of the city, and, in the case of land-disposal, the possibility of using the sewage at the place of treatment at different levels, as is done in England at some of the irrigation-areas. In this case, as in " B," the sewers leading into the interceptors may be arranged according to the perpendicular system or to the fan system.

One of the best examples of the Zone system may be seen in the city of London, England, where three different zones have been arranged to be drained by a low-level sewer, a mid-

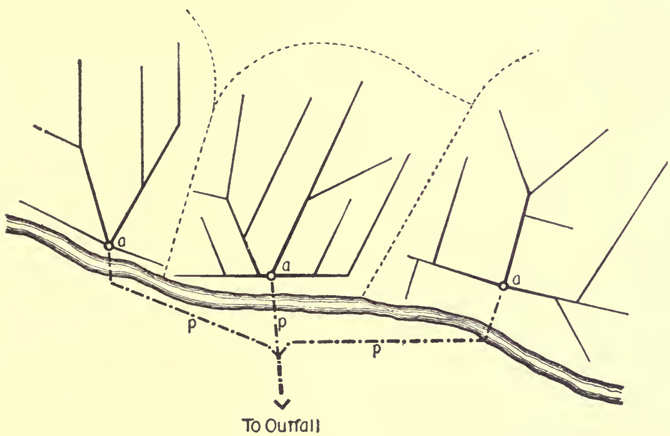


FIG. 51.

dle-level sewer, and a high-level sewer. The sewage from the low-level district has to be pumped three times, that from the middle-level only once, and that from the high-level sewer not at all.

It should be noted that the possibility of making use of the zone system economically depends upon the cost of the high-level intercepting sewer being less than the capitalized cost of pumping the sewage not so intercepted. It is easily possible for a high-level intercepting sewer to cost so much for construction as to far exceed the expense of handling the relatively small amount of liquid by pumps or otherwise.

(D) Fan system. In this the mains radiate from the outfall to serve different parts of the city, each main having its own branches and laterals (Fig. 51). Whether this or the perpendicular system is used will evidently depend on the topography and on the requirements of the outfall.

(E) Radial system. In this system, of which Berlin is the

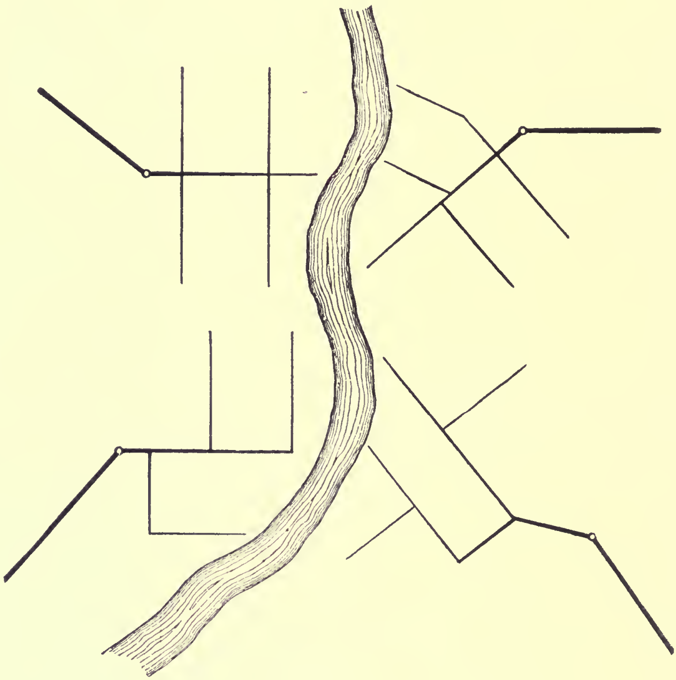


FIG. 52.

only city offering a good example, the sewage is cared for at a number of points in the circuit of the outskirts, and the sewage is brought to these points by different mains. The drainage is thus from the centre outward in several directions, and the sewage is cared for on filtration areas in these several localities (Fig. 52). Baumeister notes the great advantage of this method in that the sewers in the centre of the city are laid after that part is built up, so that there is little possibility of the section growing and needing larger sewers, while the part on the outskirts which will grow is near the pumping-stations or disposal works and can be served at comparatively small cost and without interference with the rest of the system. In the other systems, as the outskirts away from the outfall grow, the whole system must be increased, the intermediate lines being designed to carry off only the amount first considered.

The ideal topography for the radial system would be a gently sloping conical hill with the city at the apex. Such an ideal condition is really never found, so that the practical use of this system is limited to level areas for the drainage of which pumping will have to be resorted to by any system. At Berlin the outfall sewers leave the city in four different directions and the sewage is lifted by pumps and discharged onto the several irrigation fields which constitute the celebrated sewage farm of Berlin.

Many combinations of these systems occur, and the outlines given are to be considered only as guides to judgment in the individual case. The arrangement and combination must be adjusted to the topographical conditions. In the intercepting system, if there are a number of subordinate mains and one of those farthest up-stream is low, it follows that the interceptor in order to take the sewage from this and still have a grade down-stream must at the last contributing main reach a point much lower than otherwise necessary.

It is also often possible to take up a few hundred feet of the original subordinate main and, by relaying on a lighter grade, be able to raise the intercepting sewer throughout its entire length and thereby considerably reduce its cost. A small auxiliary pumping-station may sometimes be introduced to care for the sewage from the single low main and so reduce the entire cost of construction. In the new intercepting sewer for Chicago, where the depth is determined both by the depth of the present mains and by the requirements of grade, tunnel-work is resorted to as being cheaper than open cut, and large intermediate pumping-plants are required to avoid excessive depths at the outfalls.

When possible, the laterals should be laid to reach the mains

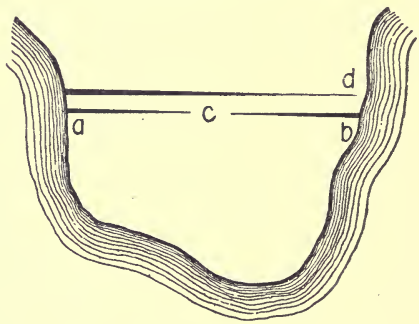


FIG. 53.

in the shortest path, though topographical conditions do not always admit of this. In Fig. 53 it is more expedient to build two sewers from *c* to the water at *a* and *b* than to take the

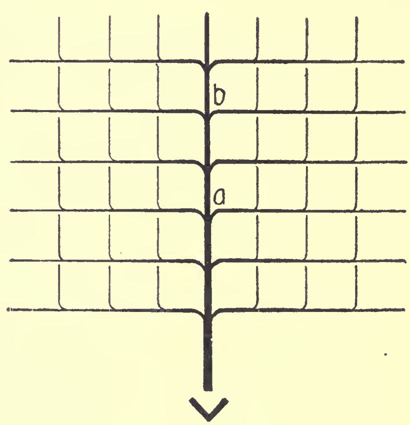


FIG. 54.

sewage from the whole area from *d*, both because the grades would be greater from the centre both ways than from water to water, and because the longer sewer would require greater size and greater depth of cutting. Where there are a number of laterals and lines of equal size it is best to combine them into a main as soon as possible, rather than to have a number of lines of about the same capacity. In algebraic terms, it is cheaper to build a single line of nx capacity than to build n lines of x capacity.

Figs. 54 and 55 illustrate the point, the length of the sewers being the same in both cases; but as the length of the small laterals is greater and that of the mains less in Fig. 55 than in Fig. 54, the former is the more economical arrangement. Further, the laterals will have a better grade, that is, the grade will be

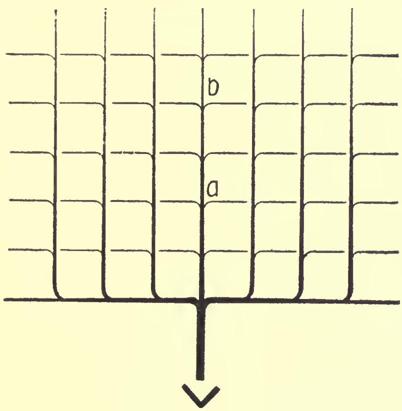


FIG. 55.

placed where it is most needed, this resulting from the fact that, since the main in Fig. 54 is larger than that in Fig. 55, it will not require as great a grade for the same velocity. Compare also in Fig. 56 the two sides of the diagram, illustrating two ways of laying out the pipes.

On the other hand, in order to maintain in the sewers as uniform a velocity as possible, and in order to avoid deposits, wherever the sewage from a hillside discharges into a flat it is better to carry the sewage along contours than perpendicular to them. This will not increase the length nor, as a rule, the sizes of the sewers, since every street must have a sewer, and since where this arrangement is desirable the grades are ample for an almost indefinite length. In the case of storm-water, the circuitous path has the further advantage that the gradual accumulation of storm-water in the lower parts of the city will not require such large sewers as if the sewage were brought down in the short time required on the steepest streets.

Other points that may be noted are as follows:

Since manholes or flush-tanks are usually built at the ends of laterals, it is often possible to run the ends of two laterals into the same manhole, thereby saving the cost of one manhole and flush-tank, though increasing the length of the sewer.

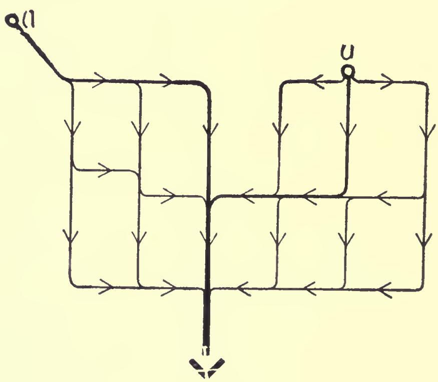


FIG. 56.

The comparative cost is here to be considered, though the single manhole or flush-tank probably gives better ventilation.

It is generally recommended that, since the flow of air is along the top of the pipes, wherever the sizes of the pipes are changed the smaller pipe be raised enough to bring the crowns of the pipes on a continuous line, in order to have a continuous ventilation upward through the sewer. For example, a 12-inch pipe emptying into a 15-inch pipe should be 3 inches higher on account of the ventilation. On long lines laid on a minimum grade this is a serious matter and requires that the lower end of a main be a foot or more deeper than the grades themselves would call for. Since the pipes are expected to run only half full, leaving half of the pipe for ventilation, it seems to the author that both of these factors of safety are not necessary, and that, except in rare cases where the pipes are expected to run full, where the cutting is deep and the line long, this requirement of grade can be omitted.

The Rawlinson principle of straight lines between manholes is rigidly insisted upon except for sewers large enough for a man to walk through, and all curves and changes of direction are made in the manholes. It has been recently recommended that even the house-drains, which are generally made to enter the sewer through a Y branch, should connect through a T in order to facilitate inspection. The direction of the flow imparted by a Y branch is said to be imperceptible, especially when the branch has a fall of 6 inches or more, and the possibility for inspection is very desirable.

To compensate for the increased resistance to the flow in the short curves made in the manholes, it is usual to add a small fall in this curve, amounting to an inch or so for an 8- to 15-inch pipe.

Recently in one of the engineering periodicals, there has appeared a theoretical consideration of the actual loss of head incurred by the flow of sewage around such curves. The variation in the conditions such as the relative height of the flow line in the main sewer and in the lateral, and in the clean condition of the pipes is such that no exact solution can be hoped for. Probably an inch fall in the bend is more than enough, but it does no harm and prevents deposition of sediment at the end of the lateral.

In order that the streams from two or three sewers meeting at the same manhole may have as little eddy-forming effect as possible and may meet and continue to flow with the least deposit of sediment, it is desirable that the streams all have the same velocity in order that in no one shall the velocity be checked. It is desirable also that the sewage-level in each of the joining sewers shall be at the same height. This last of course is not possible for all stages of all intersecting lines, but it may be so for the depth of flow for which the sewers are designed. At the manholes the confluent streams are made to merge gradually by means of tongues formed in the bed of the manhole by which means the streams are guided together as smoothly as possible. If one sewer at a small depth enters a main at great depth, it is better to allow the lateral a straight drop with a bend at the bottom than to let the flow shoot down a steep incline to have its solid matter deposited at the foot. When two laterals enter the main from opposite sides it is especially desirable that the streams be guided into the main rather than that their opposing currents meet and form eddies which will tend to the formation of deposits.

It is sometimes possible to use the volume of water brought down by a lateral as a source of flushing, a gate or storage reservoir in the manhole being arranged, and in this case an incline is better than a straight fall.

PROBLEMS

79. Consult collections of plans of sewer systems such as may be found in the report of the New York State Department of Health, 1900–1904, and select distribution systems to illustrate, as well as possible, the different systems described in this chapter.

80. Make a numerical comparison of the results to be obtained by using designs of either Figs. 54 or 55. Assume the length of each square of the figure to be 500 feet, that the amount of contributing sewage is 10,000 gallons per day for each such unit distance. Assume a uniform velocity of flow of 2 feet per second and compare the relative elevations of the two outfalls.

81. If a flush tank installed costs $60, and two sewers are leaving a ridge street on the same cross street, would it be more economical to start the two lines from one flush tank in the middle of the ridge street or to build two flush tanks 180 feet apart on the cross street? Assume the pipe to be 8 inches, costing 14 cents a foot, and the excavation to cost 60 cents per cubic yard.

CHAPTER XVI

SEWER CROSS-SECTIONS

As has already been stated, in this country sewer-pipe are invariably made circular, though attempts have been made in England to make pipe of oval form. Experience shows that the difficulties of burning (any eccentricity or deformation, which in a circular pipe can be avoided by turning axially, spoils the oval pipe), will probably prevent any similar attempts in this country. The general advantage of circular pipe lies in the fact that when half full or full there is a greater velocity for the same grade than in any other pipe-section, and for the amount of material in the pipe the circular section has the greatest area; or, geometrically, for the same perimeter the circle, of all polygons, has the greatest area. If, therefore, sewers were always to flow full, they should, whether of pipe or brick or concrete, be built of circular form, in order to economize material. But with a variable flow the circular section loses its value, and the less the flow the poorer the section for its purpose. According to the equation of flow, $v = c\sqrt{RS}$, the velocity varies with the square root of the hydraulic radius, and in a sewer where the depth of flow changes from hour to hour the velocity decreases as the depth decreases, since the ratio of $A : p$ continually decreases. This applies especially to the combined system, where the sewers are large to accommodate the rainfall, so that the house-sewage flows in a wide shallow stream with a velocity much less than that at the half-full point. To avoid this difficulty the cross-section of the brick sewer has been changed in an attempt to make the ratio of $A : p$ as nearly constant as possible for every depth, that is, to make the area of flow nearly semicircular for every depth. The gain over the circular sewer in increased velocity for low depths is considerable, although the velocity can never be made constant, since with the same grade the larger circle will have the greater velocity.

Many different shapes have been tried, though there are now but two in common use for this purpose. The egg-shape shown in Fig. 57 was introduced in England by Mr. John Phillips in 1846, and is used to-day with the same proportions then advised. The vertical height is equal to one and a half times,

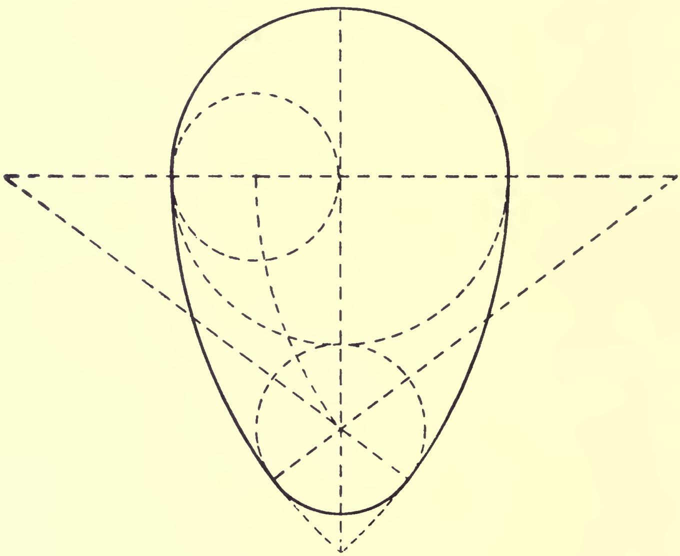


FIG. 57.

the radius of invert is equal to one fourth, and the radius of the sides to one and a half times the transverse diameter. The other form of egg-shape, Fig. 58, has a smaller invert and is therefore better adapted to sewers where the depth of flow may at times be very small. The vertical height is one and a half times the transverse diameter as before. The radius of invert is one-eighth of the transverse diameter, and the radius of the sides one and a third times. Latham says that this new form is stronger than the old, and that with small volumes of flow it is better adapted to be self-cleansing than the earlier form.

In order to obtain some comparison between the value of egg-shaped and circular sewers when the flow is small, the

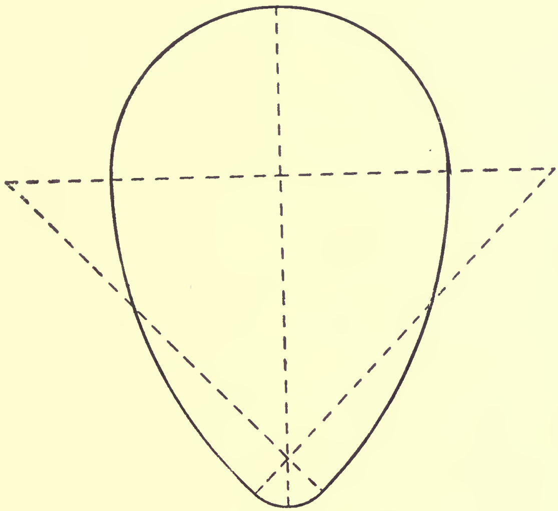


FIG. 58.

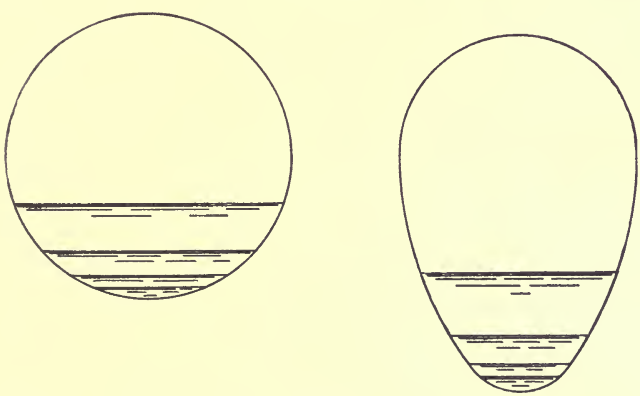


FIG. 59.

author has plotted two sections, reduced in Fig. 59, one of a circular sewer 6 feet in diameter showing depths of flow of 3, 6, 12, and 24 inches, and one of an egg-shape, with the same discharges in both cases. The grade was assumed at .03 per cent for both sections, and by repeated trials the depths in the egg shape necessary to give the same discharges as the circular were found. The benefit then is seen in the increased value of v in the former case.

	Circular Sewer				Egg-shape			
	I	II	III	IV	I	II	III	IV
Depth	0.35	0.5	1.00	2.00	0.32	0.58	1.20	3.87
Area	0.41	1.12	3.11	7.24	0.37	0.87	2.08	7.04
"C"	0.56	0.66	0.86	0.97	0.64	0.73	0.82	0.96
Discharge	0.16	0.64	3.53	12.00	0.18	0.63	3.20	11.32
Velocity	0.39	0.56	1.14	1.66	0.50	0.73	1.48	1.63
Per cent gain in velocity					0.28	0.30	0.30	......

According to the table, a gain of about 30 per cent in the velocity is obtained by using the egg-shaped sewer.

Since egg-shaped sewers are less stable and substantial than circular sewers, since for the same area of cross-section they require more masonry, and since they are more difficult of construction, it is of value to note the alleged advantage in velocity of this form and compare it with the increased cost of construction.

Fig. 60 shows a diagram by which the discharge and the velocity of flow in the circular pipe can be read directly in terms of the discharge and velocity when the pipe is flowing full. The ordinates give the proportionate depths of flow and the horizontal line through any given or desired proportionate depth extended to meet the two curves given show by the abscissæ the proportions of the discharging velocity when the pipe is flowing full. Similar curves may easily be made for the two forms of egg-shaped sewer referred to above or for any other section which is being used.

Interesting curves of this sort applied to the sections suggested for the main intercepting sewers at Boston and called respectively, the basket-handle section, the gothic section, and the catenary section, will be found in the report on the Boston

Metropolitan sewerage systems, by Howard Carson, Chief Engineer.

While the circular section is always employed for terra cotta pipe, in the case of larger sewers, made of brick or concrete, there is not the same necessity for adhering to the circular section. It is sometimes economical not to do so and a large number of peculiar and interesting sections may be found by reference to the larger text books and to the files of the periodicals. A floor of nearly flat area saves on the cost of forms,

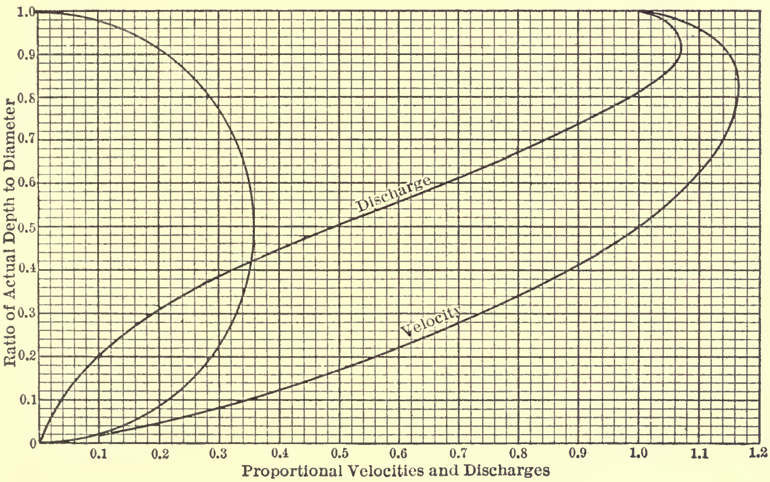


FIG. 60.

these floors being laid very much like sidewalks. Then, in firm earth where good lateral support may be had, light sidewalls can carry an arch cover at the minimum of expense. In deep cuttings, sometimes elliptical or oval sewers have been used with a view of saving excavation by limiting the width of the trench. Where, in other places insufficient head room has been encountered, the circular form has been changed to an ellipse with the long axis horizontal or to a section having very short sidewalls and with flat arcs for the top and bottom.

It is not possible in these pages to take up in detail the particular uses of the various sections which have been used or the needs of various conditions nor would it be desirable to consider the hydraulic properties or the structural weaknesses of such sections without reference to their uses. It is only possible, therefore, here, to point out that these various sections are used and that for large outfalls where the flow of sewage is reasonably uniform, the economy of construction of other forms should be compared with the economy of material which for the same capacity always remains as an advantage to the circular form.

PROBLEMS

82. Compare the velocities and discharges of two conduits laid on the same grade ($s = .0003$), one being circular, 6 feet diameter, flowing full, and the other of the same area but rectangular with width twice the depth.

83. Using the diagram of Fig. 60, determine the velocities of flow in a 36-inch tile sewer, laid on a 0.2 per cent grade, with depths of flow varying by 2-inch intervals from 0 depth to half-full. Plot a curve showing variation of velocity with depth.

84. A brick sewer is to be laid on a .0002 grade and is to carry 100 cubic feet per second flowing half full. If the flow sometimes drops to 5 cubic feet per second, how much gain in velocity would be had by using an old-style egg-shaped sewer?

85. Using Kutter's formula, find the value of V, for a conduit of wooden plank nailed together edgewise to form a trough of triangular section. Assume the depth at the centre to be 12 inches and the grade to be .05 per cent.

86. An outfall sewer has a section formed by a semicircular invert, surmounted by vertical side walls. If the radius of the invert is 3 feet and the total depth of flow 6 feet, find the slope, by Kutter's formula, necessary for a velocity of 2 feet per second.

CHAPTER XVII

FLUSHING

Notwithstanding the fact that of late years the grades and sizes of sewers have been more carefully determined and more accurately proportioned to the work required of them, and that they are now so built that the scouring and suspending power of the running sewage at no time gets below a predetermined minimum, yet accumulations of silt and filth often occur which must be cared for by some special means. There are two ways by which such deposits may be removed, by flushing or working out the obstruction with a strong flow of water, and by scraping or dragging it out with a suitably designed hoe or scraper.

The water for flushing may be obtained in several ways. Where the topography admits of it, water from some stream may be introduced at the upper parts of the system and discharged into the same stream at a lower level; in the case of a seaside city the high tide may be allowed to enter the sewer and flow out at some point where the tide is lower; a reservoir may be filled at high tide, and discharged after the tide has fallen; rain-water, waste water from baths, factories, etc., may be accumulated for a time, and then discharged into the sewer; the public water-supply may be used; or, finally, the sewage itself may be dammed up and made to act as flush-water. In planning the flushing arrangements, it must be borne in mind that a quiet flow of sewage or water, however large, is of little effect in removing obstructions once formed, and that to be effective the flush-wave must be sudden, of large volume, and introduced within a short distance of the obstruction. This wave-action, in all cases except where the stream-flow is always sufficient to fill the pipes, must be formed by a sudden discharge through a gate or other device. This may be done either automatically, or by hand; at fixed intervals, or whenever deemed necessary. In this country a reservoir accumulating water from the public water-supply and discharging through an automatic gate (the so-called automatic flush-tank) is the flushing method in general use. In many cases, however, it would seem a sad lack of judgment to neglect to provide, when it can be easily and cheaply done, other means of washing out the mains and laterals of a system.

When hand-gates are used, limited, on account of weight, to pipes of about 20 inches diameter, either the water-supply or sewage may be used. For this purpose the brickwork on the lower side of the manhole beyond which it is suspected that deposits may occur is brought up in a plane around the pipe from the bottom, and a bearing-surface for the gate bolted on; or a frame in which the gate may slide up and down may be secured to the manhole wall. The end of the pipe may form the bearing-surface, or the pipe may be closed by a plug.

Large sewers, especially storm-water sewers in which the flow-volume varies largely, require gates too large and heavy to be raised directly by hand, and a screw or windlass must be provided. If such a gate is located at a point in the sewer where an overflow into some stream can be arranged, it provides for the contingency of a gate sticking or broken mechanism or the negligence of attendants.

Automatic flush-tanks, generally used with 6- and 8-inch sewers, in this country are of two types, viz., operating through some movable part or through the starting up of a large siphon. Of the first type is that made in Schenectady, N. Y., by the Van Vranken Flush-tank Co., the following description and drawing of which is taken from the circular (see Fig. 61):

" The tank consists of a siphon, of which the interior diameter ranges from 5 to 8 inches in the various sizes, a trap at the bottom, and a cast-iron case connected with the sewer or drain. It is this trap that forms the essential feature of the Van Vranken siphon. Instead of being fixed, it is hung on trunnions under the longer leg, being so balanced that when nearly full its centre of gravity is brought forward and a portion of the contained water poured out. As the water had previously risen in the outside reservoir to a height above the lower bend of the siphon equal to the depth of water in the trap, the sudden change of level in the latter causes the longer leg to be imme-

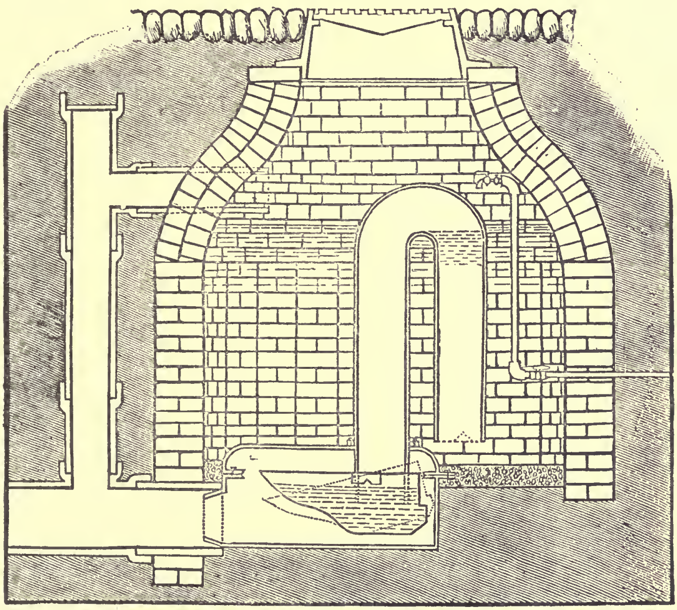


FIG. 61.

diately filled with a stream under about 4 inches head, so that the siphonic action commences at once without waste of water."

The siphon-tank was invented by Mr. Field, so far as its present form is concerned, was afterwards improved by Col. Waring, and is known as the "Field-Waring Tank." The following description, together with a sectional drawing, is taken from the circular (see Fig. 62).

"The siphon invented and patented by Rogers Field and improved by Col. George E. Waring, Jr., consists (in the form shown) of an annular intaking limb, and a discharging limb at the top of which is an annular lip or mouthpiece, the bottom of which is tapered to less diameter. The discharging limb terminates in a weir-chamber which when full to its overflow-point just seals the limb. Over the crest of the weir is a small siphon whose function is to draw the water from the weir-chamber and thus unseal the siphon. At the lower end of the small siphon is a dam or obstruction to retard its breaking.

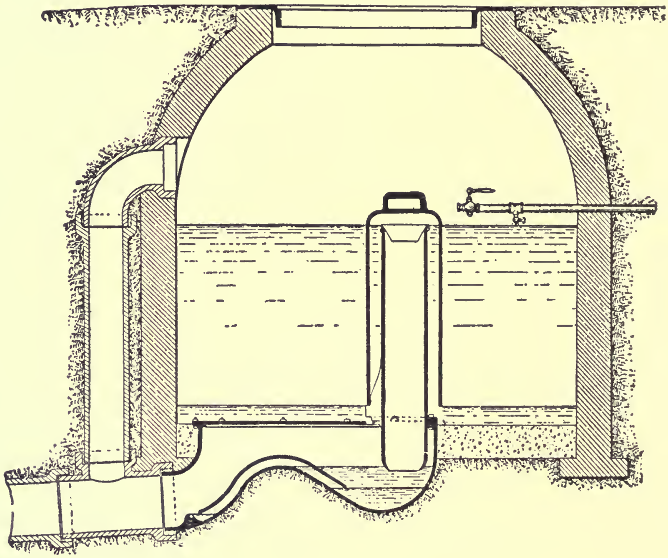


FIG. 62.

The main siphon is brought into action (on the tank being filled) by means of a small stream of water flowing over the annular mouthpiece and falling free of the sides of the discharging limb. As soon as the lower end of the discharging limb has been sealed by filling the weir-chamber the falling stream of water gathers up and carries out with it a portion of the contained air, thus producing a slight rarefaction.

"This rarefaction causes the water to rise in the intaking limb higher than in the basin outside, and hence increases the stream of water flowing over the mouthpiece, which in turn increases the rarefaction, and the siphon is soon brought into full play.

"On the tank being emptied to the bottom of the intaking limb the flow is checked, and the small siphon over the crest of the weir draws the water from the weir-chamber, air enters the discharging limb, and the siphon is vented ready for the tank to again fill.

"These siphons are largely in use and are giving excellent satisfaction; made in two sizes for flushing sewers."

A slight modification of this tank was made by Benezette Williams, and the improved tank was manufactured under the name of "The Rhoads-Williams Siphon." It has been much used in the West and has proved very satisfactory. The catalogue gives the following description and table, which latter will serve as a general index of the capacity of flush-tanks:

"The Rhoads-Williams Siphon, as illustrated in Fig. 63, consists of an annular intaking limb or bell, and a discharging limb terminating in a deep trap below the level of the sewer. Below the permanent water-line in the discharging limb is connected one end of a blow-off, or relief trap, having a less depth of seal than the main trap, the other end of which joins the main trap on the opposite side at its entrance to the sewer and above the water-line of the trap.

"The bell has a vent-pipe terminating at a given point above the bottom of the bell, and extends above the high-water line. The pipe which extends above the bell has a cap on it with the proper size sniff-hole for venting the siphon.

"As the tank fills (the main trap being full) the water rises in the intaking limb or bell, even with the level of the water in the tank, until, reaching the end of the vent-pipe, a volume of air is confined in the two limbs of the siphon between the water in the intaking limb and the water in the main trap. As the water rises higher in the tank the confined volume of air is compressed, and the water is depressed in the main trap and in the blow-off trap. This process goes on until the water in the tank reaches its highest level above the top of the intaking limb, at which time the water is depressed in the blow-off trap to the lowest point and the confined air breaks through the seal, carrying the water with it out of the trap, thus releasing

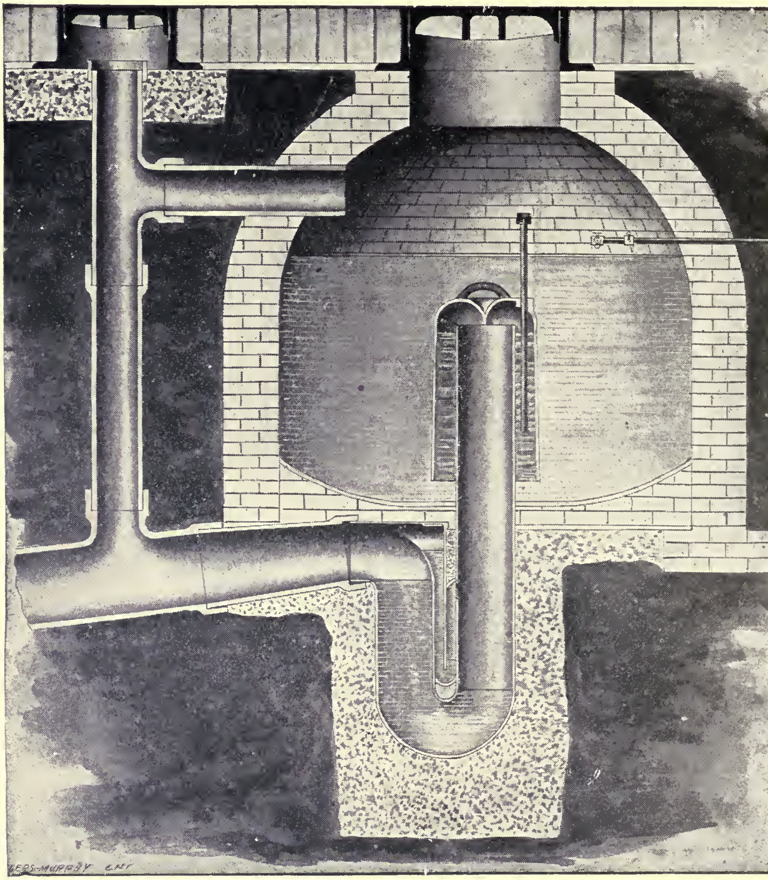


FIG. 63.

the confined air and allowing an inflow from the tank, putting the siphon into operation.

"On the tank being discharged to the bottom of the intaking limb the flow is checked, and the siphon is vented by the admission of air to it through the vent-pipe."

TABLE XXVI

RHOADS-WILLIAMS AUTOMATIC SIPHON

Diameter of Discharging Limb. Inches.	Diameter of Sewer. Inches.	Size and Capacity of Tanks, with Siphons of Standard Length.			Water required to fill 100 Lineal Feet of Sewer. Cubic Feet.	Price at Factory for Siphons of Standard Length.
		Diameter. Feet.	Discharging Depth. Inches.	Discharging Capacity. Cubic Feet.		
5	6	4	26	27	20	$26.00
6	8	$4\frac{1}{2}$	31	40	35	30.00
8	10	5	36	59	55	40.00
10	12	6	36	85	79	60.00
12	15	7	40	128	122	90.00

The Miller tank is the latest development and is probably the best and most reliable tank to be had to-day. The following description from the catalogue explains the workings of the several parts:

" The Standard Design Miller Siphon, as shown by accompanying illustration (see Fig. 64), consists of but two parts: the discharging limb or deep-seal trap (with the discharge mouth integral therewith), and the intaking limb or bell, which is placed over the longer leg of the siphon and held securely in place by its own weight, both parts being plain castings with no machine work whatever.

" This siphon has no moving parts to get out of order, no joints to leak, and no small tubes to clog up or choke, and is universally acknowledged to be the simplest and most durable device of its kind ever made.

[From London *Engineering*.]

" . . . The action of this siphon is as follows: As the water entering the tank rises above the lower edge of the bell it encloses the air within, the lower portion of the trap being, of course, filled with water. As the water-level of the tank rises the confined air gradually forces the water out of the long leg of the trap, until a point is reached when the air just endeavors to escape around the lower bend. Now as the difference of water-level in the two legs of the trap equals the difference of the levels between the water in the tank and the water within the bell, it will be seen that the column of water

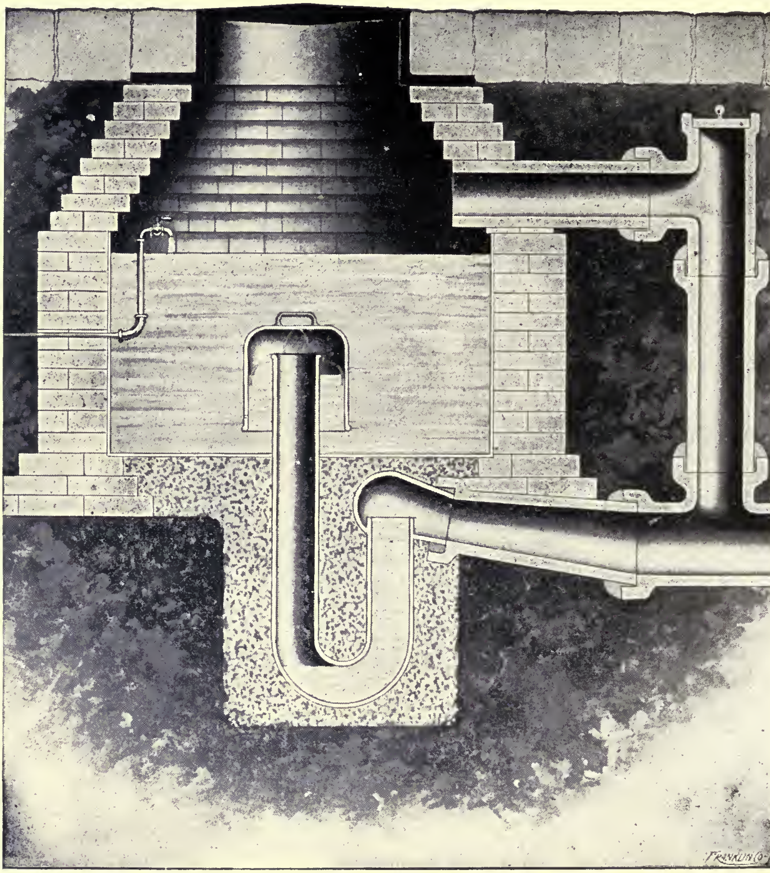


FIG. 64.

in the short discharge leg has practically the same depth as the head of water in the tank above the level at which it stands in the bell. The two columns of water therefore counterbalance each other at a certain fixed depth in the tank. As soon as this depth is increased by a further supply, however small, a portion of the confined air is forced around the lower bend, and by its upward rush carries with it some of the water in the short leg, thus destroying the equilibrium and the siphon is

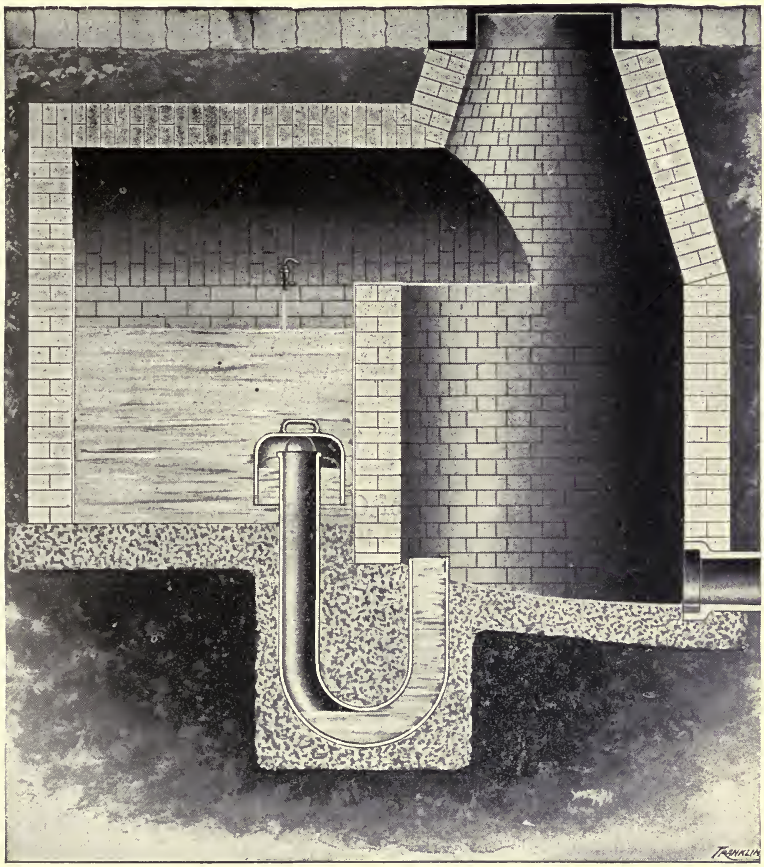


FIG. 65.

brought into full action immediately. The water is then drawn out of the tank to the bottom of the bell, the siphon vented by the admission of air through the sniff-hole, and the operation repeated. The secret of this invention is the free projection of the overflow edge of the short leg of the trap, which allows of the instantaneous escape or falling away of the heaved-up water. Thus if the discharge mouth were formed as an ordinary bend, the siphon would not act (although the confined air rushes around the lower bend), for the simple reason that the heaved-up water has no means of instantaneous escape, and therefore the equilibrium is not sufficiently disturbed. It will thus be seen that the action of this siphon depends, not on the escape of air, but on the sudden reduction of a counterbalancing column of water.

"Repeated trials with a 6-inch (Miller) siphon have shown that it will discharge full bore a 500-gallon tank, fed so slowly as only to be filled in fourteen days.

"There being no internal obstruction, the discharge is extremely rapid.

"We have had the opportunity of seeing one of these siphons at work in the excellent Sanitary Museum at Hackney, and, though severely tried, the siphon worked perfectly."

A special form of the "Miller Tank," designed by Andrew Rosewater for use in the city of Omaha, Neb. (see Fig. 65), is now manufactured. It is claimed that it discharges 40 per cent faster than any other siphon of the same size. It does not take the place of the inspection manhole, but affords easy access for inspection during the working of the siphon.

Fig. 66 * shows an automatic discharging siphon made by the Merritt Company, of Camden, N. J., and embodying a different principle. The main discharge pipe is built in the form of a "U" tube, the longer leg containing an auxiliary small air pipe, with a return bend at its lower end. When the chamber starts to fill, this small pipe bend or seal is filled with water, so that the rising water confines and compresses air in both the large and small "U" pipes. In time, and at any desired height, determined by changing the relative lengths of the parts of the small-pipe siphon, the seal is broken and the air escaping draws air enough from the large pipe to start it

* From Ogden and Cleveland's "Practical Method of Sewage Disposal."

in action. The method has an advantage in that it requires no deep excavation, and the mechanism can be set after the manhole is built.

Of late years the Pacific Flush-tank Co., having obtained

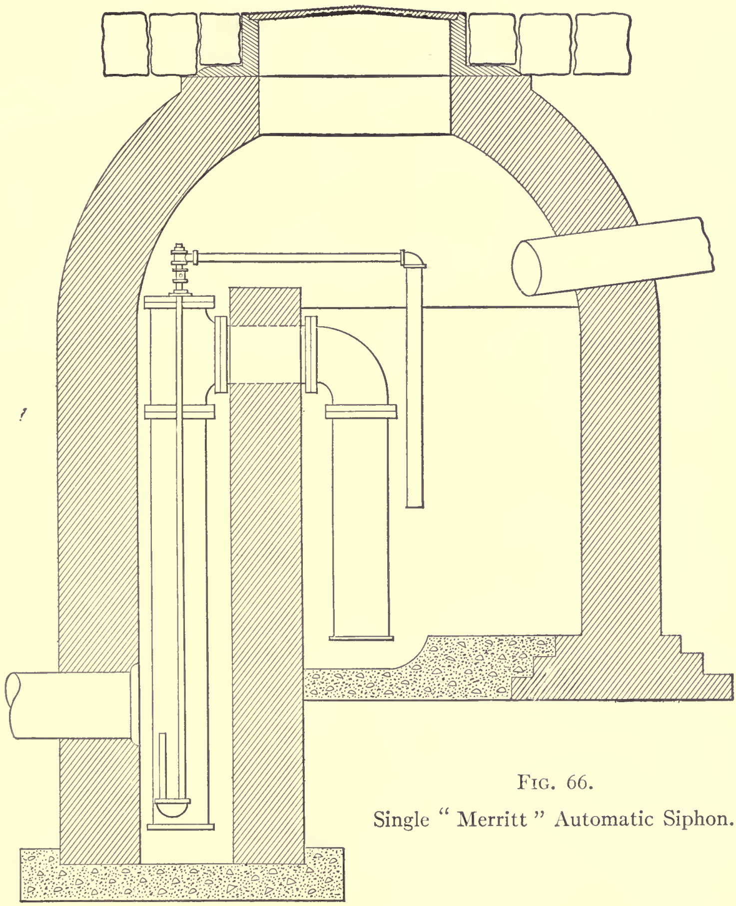


Fig. 66.
Single "Merritt" Automatic Siphon.

control of the Rhoads-Williams Tanks, and having made a number of modifications, combinations and improvements on its own tanks, is able to supply flush-tanks of a great variety of forms and sizes. The use of automatic siphon control in connection with the discharge of sewage onto filter beds has given a great impetus to the use of flush-tanks, and their reliability has been much improved. The question of the propriety of their use for the purpose of cleaning the upper ends of laterals is discussed in the next chapter.

PROBLEMS

87. In the tipping bucket of the Van Vranken flush-tank, assume a definite geometrical shape and locate the proper centres of support, considering the bucket both empty and full.

88. In the Miller tank, determine the relation between the head of water in the tank and the length of the lower limb of the siphon. Is any other factor involved in fixing the depth of water in the tank at the instant of discharge?

89. Determine the time (approximately) for the discharge of a flush-tank, 4 feet diameter, $2\frac{1}{2}$ feet deep through a 6-inch siphon. (Consult catalogue for dimensions).

CHAPTER XVIII

USE OF FLUSH-TANKS

THE following paper, read by the author before the American Society of Civil Engineers, in May, 1898, offers a discussion on the suitable use of flush-tanks, their proper capacity, frequency of discharge, etc.

The use of flush-tanks in connection with small pipe sewers, which has been made an integral part of the "Separate System" and generally adopted in systems caring only for house-sewage, is attended with much uncertainty. In such systems it is generally specified that a flush-tank be placed at the head of every lateral, each tank being so regulated as to discharge at least once in 24 hours. The relation between the size of the sewer-pipe and the amount of water used in a flush is not given, nor is the influence of grade discussed. The general law is laid down that all laterals, regardless of size, grade, or contributing population, must be supplied with flush-tanks in order to secure a self-cleansing flow in the laterals and to maintain the integrity of the system.

The financial burden of such a requirement is evident. As an example, it may be cited that in the plans for the sewerage system of Ithaca, N. Y., in which plans this requirement of flush-tanks was thoroughly complied with, even for the 12 per cent grades, no less than 131 flush-tanks were required in 25.3 miles of sewers, or one for every 1020 feet. The relative importance of the flush-tanks may also be seen by comparing the actual cost of the sewers with the estimated cost of the tanks. The cost of the sewers, viz., the sum of the amounts of the several contracts, was $81,000, and, estimated at $50 each, the flush-tanks would cost $6550, or more than 8 per cent of the cost of the system. It would seem, then, that the cost of flush-tanks is by no means insignificant, but that their use increases the cost of the separate system by nearly one-tenth, besides introducing a permanent charge, both for water used and for intelligent care in maintenance. That these annual charges are no bagatelle will be apparent by again referring to the case of Ithaca. Assuming that the tanks required are of a capacity of 150 gallons, a minimum amount, discharging but once a day, the water required is 19,650 gallons a day. Twenty cents per 1000 gallons (the amount charged in Ithaca *) is a fair average amount, and at that price the daily charge for water is $3.93, or $1434.45 per year. Adding to this $600 per year as the wages of a mechanic, whose constant attention is found by experience to be necessary in examining and readjusting the tanks, the total annual charge is $2034.45. This, capitalized at 6 per cent, gives $33,908, and, added to the $6550, gives $40,458 as the total expenditure on account of flush-tanks in a sewer system costing for pipe laid $81,000. Surely the item of flush-tanks is an important one, and should be carefully examined, so that if the conditions of the sewer-grade, for example, modify the necessity for tanks, or if the amount of water is a function of the time-interval between flushes, or of the size of the pipe, it may be known in order that the large proportionate cost of flushing may be reduced to what has been found by careful investigation to be an absolute minimum.

That the requirement given above is felt by present-day engineers to be largely in excess of necessity is sufficiently evident from a study of the paper by F. S. Odell, M. Am. Soc. C. E., entitled "The Separate Sewer System without Automatic Flush-tanks," † and the subsequent discussion, in which the author says that at Mt. Vernon, N. Y., no flush-tanks are used, and that, while hand-flushing by means of fire-hose is practised at intervals of six months, even this infrequent flushing does not appear necessary, as examination of the sewers invariably shows a very wholesome and satisfactory condition.

* "Manual of American Water-works," 1897.

† Trans. Am. Soc. C. E., Vol. XXXIV, page 223.

In the discussion very little positive evidence is given, but the experiences recorded go chiefly to show that while automatic flush-tanks do not in themselves make the separate system practicable, there is, nevertheless, a need, under certain conditions, for flushing, those conditions being as yet not fully determined.

The questions, answers to which are essential for an intelligent disposal of flush-tanks on a sewer system, are four, viz.:

1. What is the relation, if any, between the grade of the sewer and the necessity for automatic flush-tanks?

2. Assuming a need for automatic tanks, how does the grade of the sewer affect the amount of water required, and what is the proper amount to be used?

3. How often should tanks be discharged?

4. What effect does the substitution of a 6-inch for an 8-inch lateral have on the necessity for tanks and on the amount of water to be used?

Before attempting to answer these questions, it will be well to look at the subject broadly, and consider the hydraulic problem involved. Sewage is water carrying in suspension less than 1 part in 1000 of solid matter, and sewers are supposed to be so laid that the resulting velocity of flow is sufficient to keep this solid matter in suspension. This suspending and scouring power probably depends on the velocity, and on the depth, of the sewage stream, and if either gets below a certain point, sedimentation will follow and a deposit take place. It is generally stated that a velocity of about $2\frac{1}{2}$ feet per second is required; but the effect of depth is neglected. At the lower end of a 6-inch lateral the depth and velocity are assumed to be sufficient to prevent this sedimentation, but as the contributing population grows less toward the upper end, the depth and velocity decrease and the transporting power of the stream falls so low as to allow the solid matter, brought into the sewer by the house-drains, to become stranded. This deposit increases by gradual accumulation until the sewer is blocked, until the head from the backed-up sewage is sufficient to carry away the obstruction, or until the discharge of the flush-tank (and here is seen its true function) takes up the obstruction and carries it to a point where the depth and velocity of the sewage will hold it in suspension. Table XXVI and the dia-

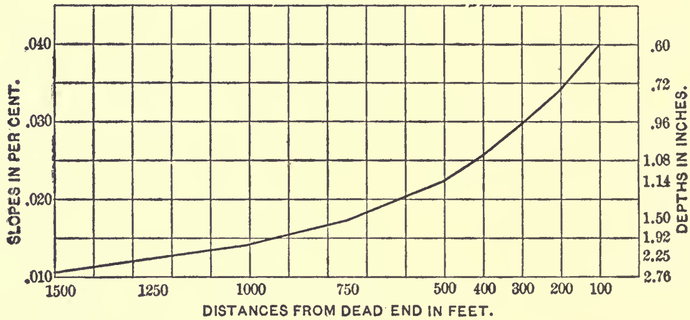


FIG. 67

gram (Fig. 67) are given to show the requirements in grade to maintain a velocity of $2\frac{1}{2}$ feet per second in a 6-inch lateral, assuming a constant contributing population of 76 persons per 100 feet of sewer, with a daily flow of 60 gallons per capita, and with the assumption of one-half flowing off in 6 hours.

TABLE XXVI

Distance from Dead End in Feet.	Discharge in Cubic Feet per Second.	Slope in Feet per Foot.	Depth of Flow in Inches.
1750	0.245	0.0103	3.00
1500	0.210	0.0104	2.76
1250	0.175	0.0123	2.25
1000	0.140	0.0140	1.92
750	0.105	0.0174	1.50
500	0.070	0.0225	1.14
400	0.056	0.0256	1.08
300	0.042	0.0302	0.96
200	0.028	0.0342	0.72
100	0.014	0.0400	0.60

The diagram (Fig. 67) shows that, taking n equal to 0.013, and computing velocities by Kutter's formula, a grade of 1 per cent is required for a 6-inch pipe half full for a velocity of 2.5 feet per second, and that if the amount of flow constantly decreases, the depth of flow decreases also, and the grade, in order to maintain the same velocity, must be increased according to the diagram. The diagram is given for two reasons: first, to show that by the accepted laws governing the transportation of material in flowing water, lateral sewers could be laid, theoretically, on such grades that no flushing would be necessary, since, with grades which continually increase toward the upper end, the corresponding velocities will always be equal to that required to transport matter in suspension; second, to show that as the grade of the sewer increases, the distance from the upper end to the point where the stream reaches the velocity required to carry matter in suspension decreases, and so the aid required from flush-tanks is less. No value can be placed on the grades given, as the diagram is based on the assumption of a house with five persons every 66 feet, and this is not always the case; but it is believed that there is a grade at or beyond which flush-tanks are not required, and if the distance to which the flushing power extends is a function of the amount of water discharged, then this amount should be less on steep grades.

Referring again to Mr. Odell's paper, it is first noted that at Mt. Vernon, with grades of from 0.5 to 6 per cent, no flush-tanks are used, and a good hand-flushing twice a year answers every purpose.

In the discussion, Mr. Hering says that on light grades flushes of 200 to 300 gallons generally lose their flushing power after passing a few hundred feet through the pipe, and that sometimes after 500 feet he has been unable to detect any difference in the flow due to the discharge of the tank.

Mr. Kiersted writes that in one system designed by him he recommended flush-tanks only on laterals of less than 0.5 per cent grade, and for five years the system has been in operation with but few stoppages.

Mr. Folwell writes that in his experience he has omitted flush-tanks on grades from 6 to 12 per cent, and on the 6 per cent grades no stoppages were discovered, nor were there any odors.

Mr. Le Conte intimates that flush-tanks as built do not answer their purpose, for where grades are light and the flush most needed, they do the poorest work; and the large quantity of water needed to be effective must be obtained by some other means.

Mr. Odell maintains that flushes of 200 gallons or less fail to flush a sewer properly, especially on flat grades where flushing is most needed.

A table by Mr. Allen shows that on grades greater than 0.5 per cent a velocity of more than $2\frac{1}{2}$ feet per second is maintained over 1000 feet from the flush-tank, but on lesser grades the velocity drops to 2 feet or less within 600 feet.

In order to obtain an insight into general engineering practice in the matter, and, at the same time, reap the benefit of any experience which was to be had, the author sent out, on January 17th, 150 reply postals, reading as follows:

"Ithaca, N. Y., January 17, 1898.

"Dear Sir:

"To aid me in deciding as to the necessity for flush-tanks for our sewer system, will you kindly answer the following:

"I. Do you find flush-tanks a necessity, or is periodic hand-flushing sufficient to keep sewers clean?

"II. Does the element of grade affect the question, and within what limits of grade are tanks required?

"III. Does your experience show any relation between the minimum amount of water required for effective flushing and the grade of the sewer?

"Thanking you in advance for your kind assistance in this matter,

"I am, yours very truly,

"H. N. Ogden,

"*Engineer, Ithaca Sewer Commission.*"

These postals were sent to those cities of between 10,000 and 60,000 population, in the New England and Middle Atlantic States especially, which were reported in the "Manual of American Water-works" for 1897 as having separate or sanitary sewers. Eighty answers were received, and the courtesy and good-will expressed in all was much appreciated. It was the same story in nearly all cases. "I would be pleased to answer your questions fully, but this is the best that I can do for you," or "This is only my idea, while I can readily understand that what you want is the result of actual experience," or "I cannot give you the desired information, but would be thankful to you if you would let me know the result of your inquiry." The results given below in a brief summary show chiefly how uncertain and vague is the knowledge on the subject, and how necessary are some experiments and investigations.

Of the eighty engineers who sent replies to question No. 1, whether flush-tanks are necessary, seventeen had no opinion on the subject; twelve had experience only with combined systems, but had, according to their replies, found no trouble in keeping the ends of their 10- and 12-inch laterals clean with rain or with hand-flushing; twenty-six of the eighty used periodic hand-flushing and found it to answer every purpose, keeping the sewers clean and free from obstructions; twenty-five either used flush-tanks or considered them a necessity for small pipe sewers. It was not possible in these last answers to separate actual experience from personal conjecture on the question, so that this number may include many hearsay opinions.

The evidence is not very clear. The fact that twenty-six used hand-flushing satisfactorily indicates that such flushing is sufficient. That it must be properly and regularly done, however, is made plain by the fact that, out of twenty-five believing in flush-tanks, nine had tried periodic hand-flushing, found it uncertain and irregular, and had put in flush-tanks, to secure proper attention. On the other hand, of the twenty-six believing in hand-flushing, two came to that opinion after becoming disgusted with the uncertainty of tanks.

On the second question, only twenty-three of the eighty ventured an opinion. Of these, eight thought that the grade did not affect the question, but that flush-tanks were as necessary on steep as on flat grades. One engineer explained his position by saying that while the velocity on the steep grades might be greater, yet as the depth would be less, the transporting power would be less, and therefore tanks were equally necessary. Of the fifteen who thought that tanks are not needed above a certain grade, six merely ventured it as an opinion, and nine fixed the limit at from 0.5 to 3 per cent; four of these gave 1 per cent as the limit; one, 3 per cent; and the other four less than 1 per cent.

Only six replies were given to the last question, whether the amount of water in the flush-tank should be varied with the grade of the sewer. Of these six, two engineers thought that no difference should be made; three thought that less water could be used on the steep grades, but had no definite opinion as to the relative amounts; while one well-known engineer, who has thoroughly studied the workings of the sewer system under his care, writes that he finds one flush daily on a 2 per cent grade as effective as two flushes daily on a 0.5 per cent grade, each flush of 300 gallons.

The general conclusion from the replies is that on low grades, probably below 1 per cent, occasional flushing is needed at the upper ends of laterals; that this may be accomplished either by hand-flushing or by the use of automatic tanks; that if tanks are used, less care and vigilance are required in inspection and oversight, but, on the other hand, the periodic examination of the system, which should not be omitted, is apt to be irregular, and if a tank fails to work or if an obstruction occurs below the effect of the flush, a serious nuisance may result; that if hand-flushing is used, a constant and regular inspection must be practised, although actual flushing may be required but once a month or even less. The amount of water needed in flush-tanks is not known, nor the relation between amount and grade.

With a view of obtaining more information on this apparently unstudied subject, the author carried on some experiments in the spring of 1897, in which he was assisted by Mr. I. W. McConnell, C.E. The results of the experiments have been recorded by Mr. McConnell in a thesis for the degree of Civil Engineer in Cornell University.

The sewers on which the experiments were made, chosen so as to afford a variety of grade, with as long lines as possible, were all 8-inch pipe, and each had at the upper end a manhole about 4 feet in diameter at the bottom. Flush-tanks of the usual commercial size discharge at a rate of about 1 cubic foot per second, and, by repeated experiment, the opening from the manhole into the sewer was reduced to such a size (about 5 inches) that the rate of discharge varied from 0.89 cubic foot per second for 4 feet head in the manhole to 1.1 cubic feet per second for 6 feet head. These conditions it was thought approximated closely enough to the workings of a flush-tank. A 5-inch opening was cut in a pine board held firmly against the end of the 8-inch pipe; then a flat rubber-faced cover, 6 inches in diameter, was placed over the opening and held there by a light stick braced against the back of the manhole, making an effective plug. The manhole was filled to any desired depth by means of fire-hose attached to neighboring hydrants, and then, by means of a cord fastened to the stick and to the cover, the contents of the manhole were discharged into the sewer. The capacity of the manholes at depths varying by 6 inches was determined by measurement, so that by filling to the proper depth any desired amount of water could be discharged. The effect of the flush-waves was then noted at the successive manholes down the line. No determinations of the velocity of the wave were made, the effect being judged by the depth of the wave, and by the force shown in moving gravel, etc., placed in the different manholes. The wave-depths were read by observers stationed in the manholes, where they re-

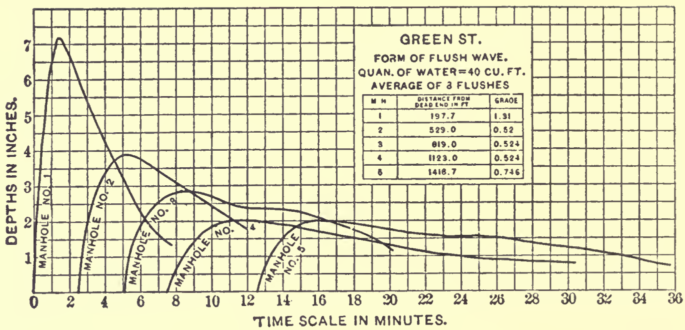


FIG. 68.

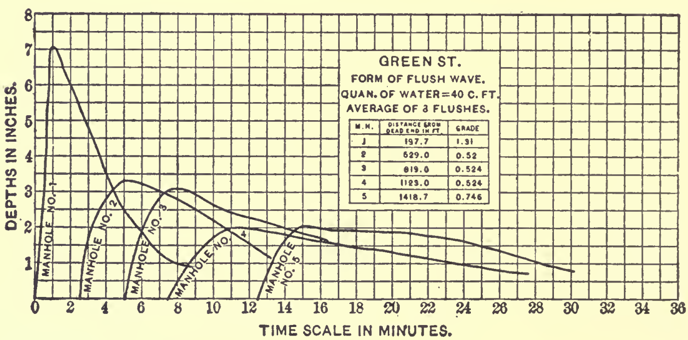


FIG. 69.

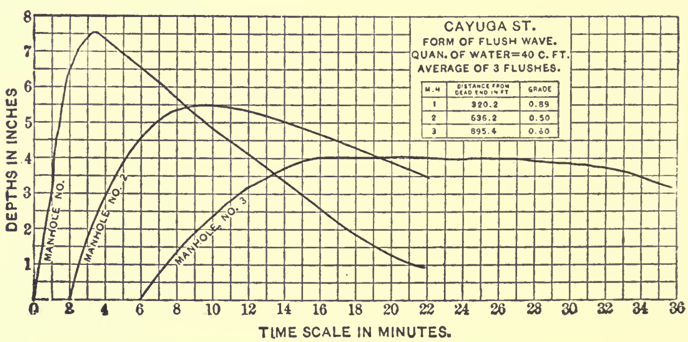


FIG. 70.

corded as rapidly as possible (usually every seven seconds) the depth as marked on a thin vertical scale placed in the sewer. Figs. 68 to 71 show the wave-forms and the progressive flattening as the wave gets farther and farther from the flush-tank.

To test the transporting power of the wave small brickbats and gravel of various sizes, coated with paint so as to be recognizable, were placed in the inverts at the manholes. A considable growth, apparently of vegetable origin, had become attached to the sides and bottom of the pipe, and the value of the flush in removing this growth was also noted. The order of procedure was to examine and note the condition of the line, and, after placing the gravel, etc., in the manholes, to make a number

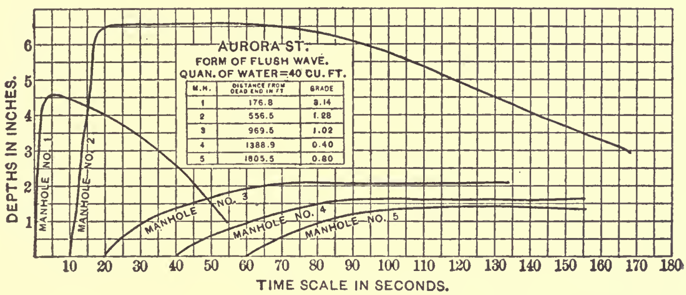


FIG. 71.

of flushes, each of 20 cubic feet, and note the results. Then, increasing the amount discharged to 30, 40, 50, and 60 cubic feet, the respective results were noted. Then either the whole pipe was scraped by a rubber-edged piston-like cleaner, or merely the manhole inverts and about 6 feet each way into the pipe, and the flushing repeated. Tables XXVII–XXX give the results on the different lines.

Before commencing the work, the examination of the Green Street pipe showed it to be practically clean, with no groundwater, except between the third and fourth manholes, where there was a stream about one-fourth inch deep. No house-connections had been made, but there was a small depth of silt, and bits of cement left from construction, also a slight vegetable

Table XXVII

GREEN STREET SEWER

Volume of Flush.	Effects at				No. of Flushes.
	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	
25 cu.ft.	Scoured clean	Scoured clean	No effect	No effect	1
30 "	"	"	"	"	1
40 "	"	"	Several stones started	"	8
60 "	"	"	Small gravel generally started	"	2
80 "	"	"	do	"	2
120 "	"	"	"	"	3

growth on the sides and bottom of the pipe. Gravel of all sizes placed in the pipe at the flush-tank was carried through to manhole No. 1 in two flushes of 25 cubic feet each, the first flush alone not being sufficient. The gravel scoured out of the bottom of manhole No. 1 by the first flush was not brought to No. 2 until the 80-cubic-foot flush was put in, and no gravel scoured out of No. 2 was brought to No. 3 by any of the flushes. After the seventeenth flush as above, the pipe was thoroughly scraped and cleaned, and flushes eighteen to twenty-eight were made. Similar results were obtained, except that the flushes carried the gravel about 200 feet farther than before and seemed effective for that distance.

Table XXVIII

CAYUGA STREET SEWER

Volume of Flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
30 cu.ft.	Scoured clean	No effect	No effect	No effect	3
40 "	"	Disturbed but not cleaned	"	"	7
60 "	"	Partly scoured	Some vegetable growth passed through	"	2
80 "	"	Cleaned	"	"	3

In Cayuga Street there were a few connections and little flow, so that the condition of the pipe was very foul; there was also a heavy vegetable growth in the pipes.

On Linn Street no comparative records could be made. The pipe was clean from the flush-tank to manhole No. 1, and in this length there were no connections. From No. 1 to No. 2 it was slightly foul, and very foul the remainder of the length. There were two house-connections on the line. Five flushes of 20 to 60 cubic feet were made. Each was very effective, one apparently as much so as another. All obstructions introduced were removed at once from manholes Nos. 1 and 2. A steady flow 1 inch deep from the hose carried everything forward at once to a point beyond No. 2 and to the flatter grade.

Table XXIX

AURORA STREET SEWER

Volume of Flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
40 cu.ft....	Cleaned	Cleaned	No effect	No effect	3
60 "	"	"	Disturbed	Water dirty; some vegetable growth came through	7
80 "	"	"	"	A few stones disturbed	2

Table XXX

FIRST STREET SEWER

Volume of Flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
40 cu.ft.......	Cleaned	No effect	No effect	No effect	5
60 "	"	"	"	"	3
80 "	"	"	"	"	2

On the Aurora Street line the pipe was very foul, chiefly from a hospital connection at the upper end. The vegetable growth was excessive, and the accumulations of organic matter very evident.

On Buffalo Street, where the grade is about 12 per cent, the effective of the flush was amazing. Where any sewage at all flows in the pipe, it is sufficient to remove all obstructions. A flush of any volume rushes down the hill at a high velocity, with piston-like action, and sweeps everything before it.

Table XXXI gives the distances and grades between manholes on the lines used in the experiments.

Table XXXI

DISTANCES AND SLOPES BETWEEN MANHOLES

Description.	Green St.		Cayuga St.		Aurora St.		First St.		Linn St.	
	Distance in Feet.	Grade percentage.	Distance in Feet.	Grade percentage.	Distance in Feet.	Grade percentage.	Distance in Feet.	Grade percentage.	Distance in Feet.	Grade percentage.
Dead end to manhole No. 1......	298	1.31	320	0.89	177	3.14	371	1.00	331	2.94
Manhole No. 1 to manhole No. 2..	231	0.52	316	0.50	390	1.28	341	0.50	278	2.70
Manhole No. 2 to manhole No. 3..	290	0.52	259	0.60	413	1.02	394	0.57	317	0.50
Manhole No. 3 to manhole No. 4..	305	0.52	.....	.....	419	0.40	393	1.00	.....	.....
Manhole No. 4 to manhole No. 5..	296	0.75	.....	.....	417	0.80	.....	.....	.....	.....

The manager of the Van Vranken Flush-tank Company gives his practice in proportioning the sizes of flush-tanks for any particular sewer as follows: The capacity of the reservoir should be equal to one-half that of a length of sewer in which the grade produces a rise equal to the diameter of the pipe; so that the Green Street line, 8 inches diameter and 0.5 per cent grade, should have a discharge of half the volume of the pipe, $\frac{4}{3}\times 100$ in length, or 23 cubic feet; and for a 1 per cent grade one-half of that, or 11.5 cubic feet. He says, further, and the statement has been confirmed by the author's work, that an 8-inch pipe on a 0.4 per cent grade will flow one-third full at a distance of 300 to 400 feet from the tank discharging the above amount; and that on a 5 per cent grade the water will come down as a solid piston for any dischrage greater than 14 cubic feet.

The manager of the Pacific Flush-tank Company writes that as a rule he does not interfere with engineers in their design for tanks, but, in his opinion, a flush of 175 gallons on a 1 per cent grade is sufficient, and on any flatter grade twice that amount of water should be used, or, as he says, " long lines or flat grades require greater capacity of tanks than steep grades or short lines."

Conclusions.—The following conclusions are based upon previously published data on this subject; upon the experience of engineers in different parts of the country; upon the flushing diagrams recently published by J. W. Adams, and upon observation and the special experiments made in Ithaca; and it is believed that they are justifiable and a safe guide in the use of flush-tanks.

(1) Flushing of some sort is required at the upper ends of laterals, the frequency and amount depending on the number of house-connections, on the carefulness or prodigality in the use of water by the house-holders, on the grade and size of the sewer, on the character of its construction, and on a mysterious something which defies definition, but which produces frequent accumulations in one line and does not affect another, apparently like the first.

(2) This variety in the conditions prevents any exact statement of a relation between the quantity of water which should be discharged from a flush-tank and the grade of a sewer, but it plainly indicates that the advantage of automatic flush-tanks lies in a general guarantee or insurance against accumulations in the upper part of the laterals, while periodic hand-flushing must be depended on only when in charge of a responsible, indefatigable, and intelligent caretaker.

(3) Judging by the experience at Ithaca, and despite the

statements of other engineers, it seems to the author that on grades of less than 1 per cent automatic flush tanks are an economic necessity, even where water has to be paid for, the added expense of frequent hand-flushing more than offsetting the possible discharge of flush-tanks when not absolutely necessary.

(4) The volume of water discharged should not be less than 40 cubic feet, and the effect of the flush can hardly be expected to reach more than 600 or 800 feet. Below this point accumulations may occur which must be removed by hand-flushing and carried on to a point where the sewage-flow has the necessary transporting power.

(5) On flat lines and where obstructions occur below the influence of the flush-tank, a second flush-tank, placed about 800 feet from the first, will be more effective than increasing the first tank to a capacity of three times its original discharge.

(6) The frequency of discharge should depend on the local conditions, but it is probable that the maximum interval depends on the practical working of the siphon, so that the usual prescription of once in 24 hours is a safe rule.

(7) If tanks are used on grades greater than 1 per cent, 15 to 20 cubic feet give as good results as larger amounts, with the same rule as to frequency of discharge.

(8) However, economy is best served, on grades above 1 per cent, by omitting flush-tanks, and resorting to periodic hand-flushing at such intervals as experience shows to be necessary on the different lines. In most cases semi-annual or quarterly flushings, with a hose, are sufficient.

(9) On grades greater than 3 per cent flush-tanks are unnecessary, and their installation is a waste of money.

(10) Hand-flushing should be performed and tanks discharged at night, as a flow of even an inch in a sewer offers a large resistance to the flushing action; while with a pipe flowing half full the effect of a flush-tank is scarcely visible.

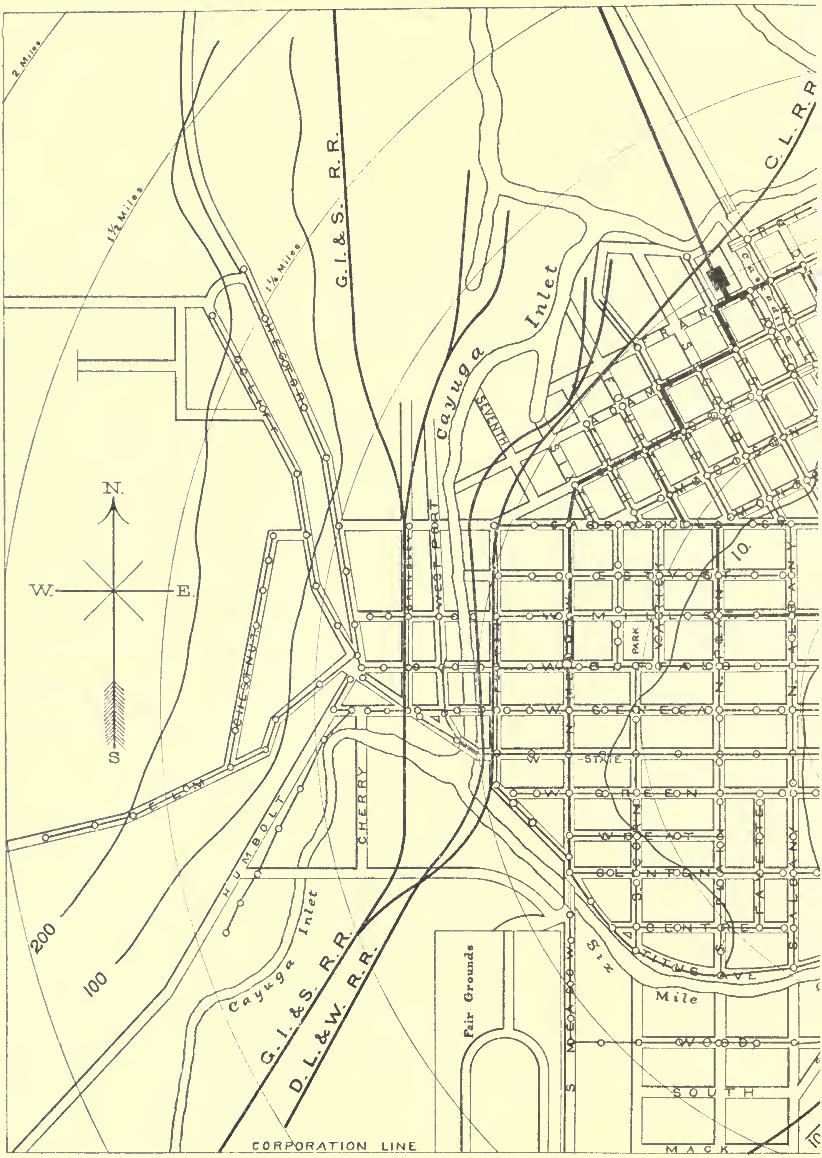
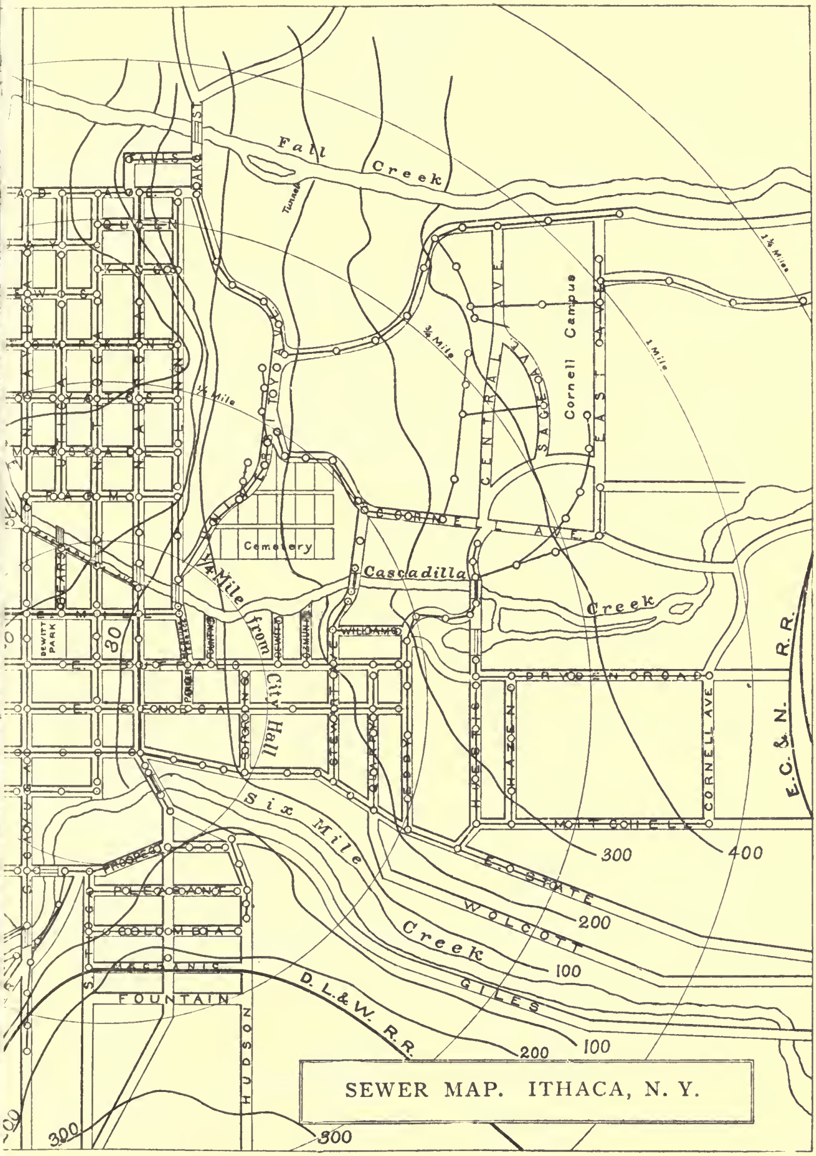
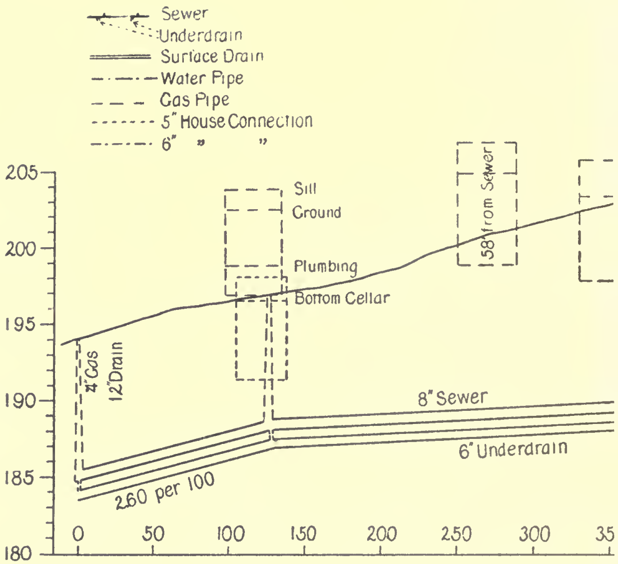

2 Miles
1½ Miles
1¼ Miles
G. I. & S. R.R.
C. L. R.R.
Cayuga Inlet
SEVENTH
WEST PORT
N.
W.
E.
S
ELM
CHESTNUT
HUMBOLT
CHERRY
STATE
PARK
10.
Six Mile
Fair Grounds
TITUS AVE
CENTRE
SOUTH
MACK
200
100
Cayuga Inlet
G. I. & S. R.R.
D. L. & W. R.R.
CORPORATION LINE

PLATE I.

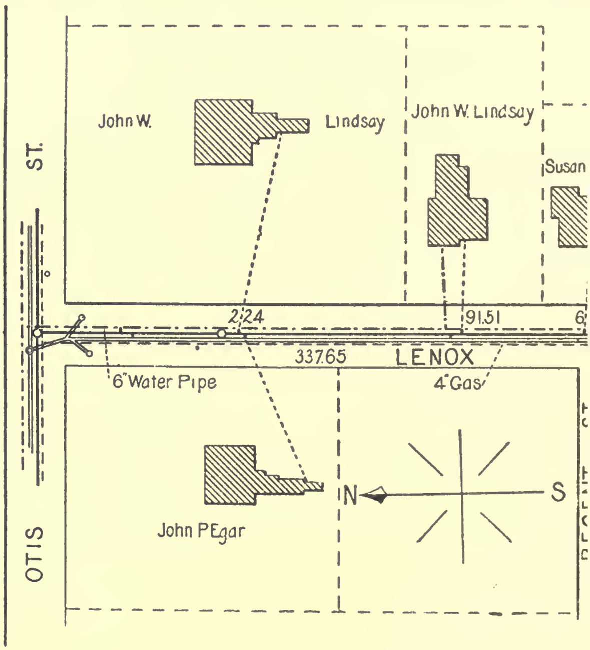


SEWER MAP. ITHACA, N. Y.



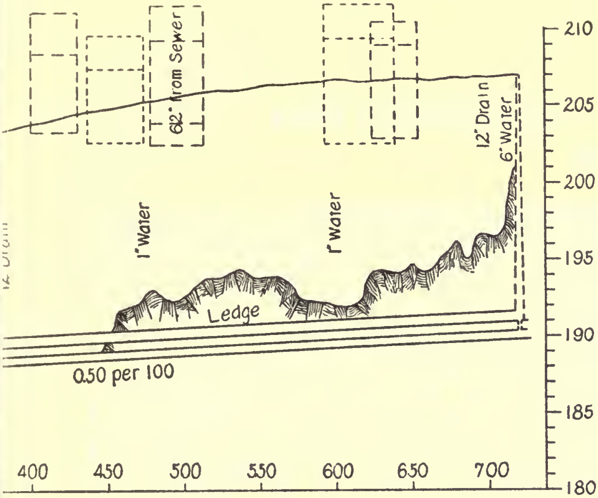
Sewer
Underdrain
Surface Drain
Water Pipe
Gas Pipe
5" House Connection
6" " "
205
200
195
190
185
180
Sill
Ground
Plumbing
Bottom Cellar
58' from Sewer
4" Gas
12" Drain
8" Sewer
6" Underdrain
2.60 per 100
0
50
100
150
200
250
300
35

PRO

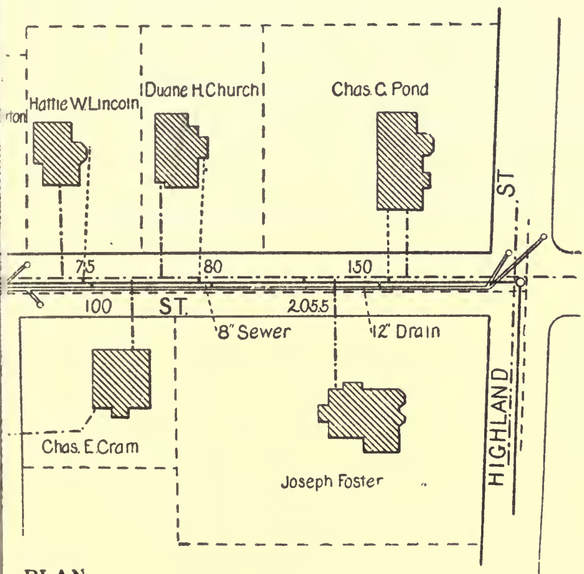


OTIS ST.
John W. Lindsay
John W. Lindsay
Susan
2.24
91.51
337.65
LENOX
6" Water Pipe
4" Gas
John P. Egar
N
S

PLATE II.



E.



PLAN.

PIPES FLOWING FULL

PLATE III.

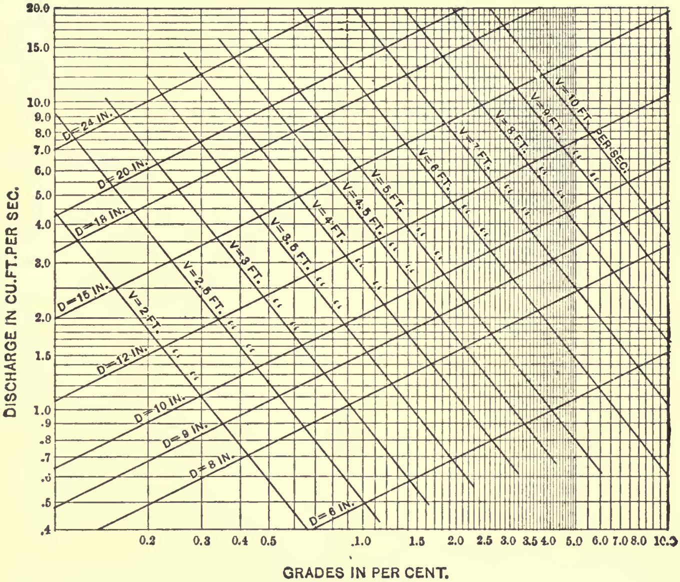


DIAGRAM BASED ON KUTTER'S FORMULA FOR VITRIFIED PIPES.

$n = .013.$

PIPES FLOWING FULL

PLATE IV.

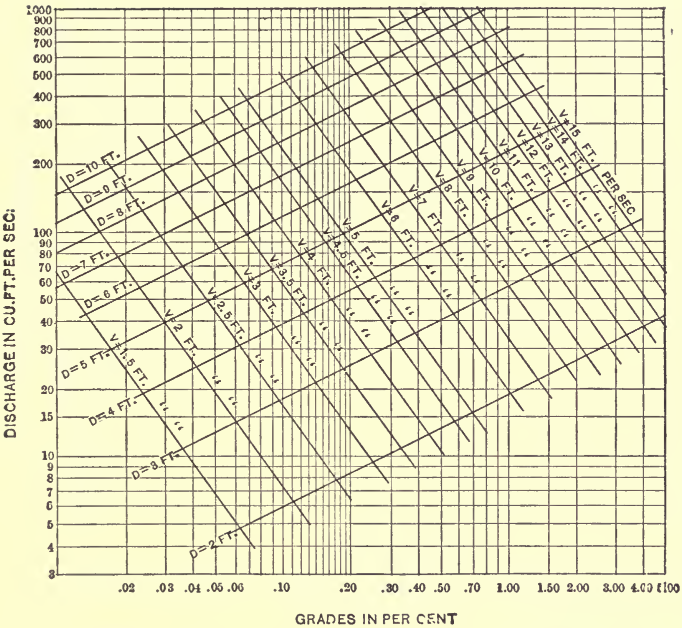


DIAGRAM BASED ON KUTTER'S FORMULA FOR CIRCULAR BRICK SEWERS.

$n = .015.$

PIPES FLOWING FULL

PLATE V.

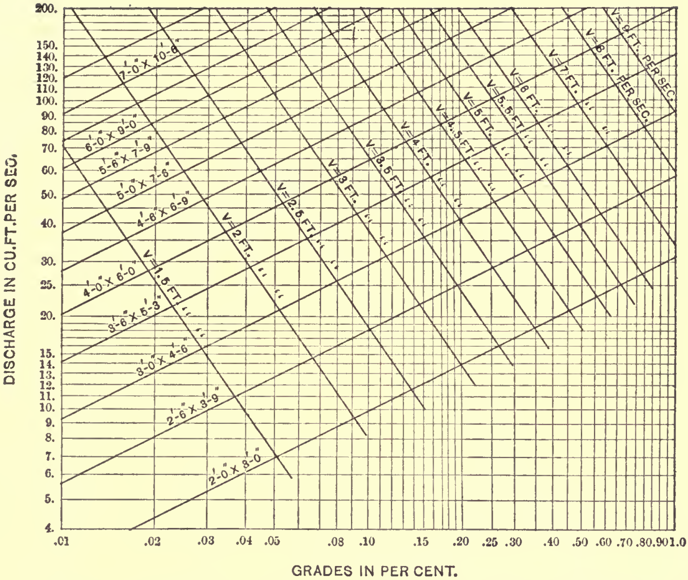


DIAGRAM BASED ON KUTTER'S FORMULA FOR EGG-SHAPED BRICK SEWERS.

$n = .015.$

BINGHAMTON, N.Y.

MAP SHOWING

TOPOGRAPHY

AND

PROPOSED OUTFALL SEWER

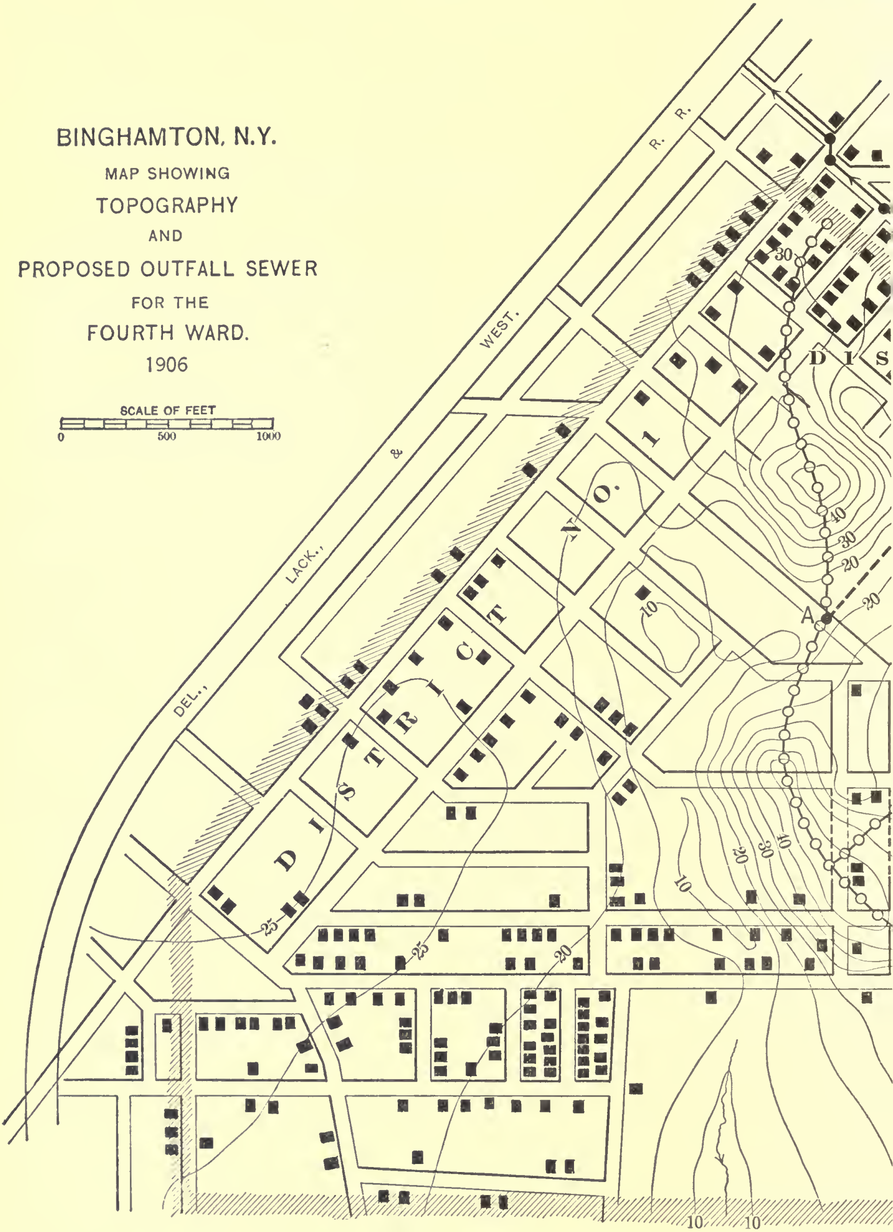
FOR THE

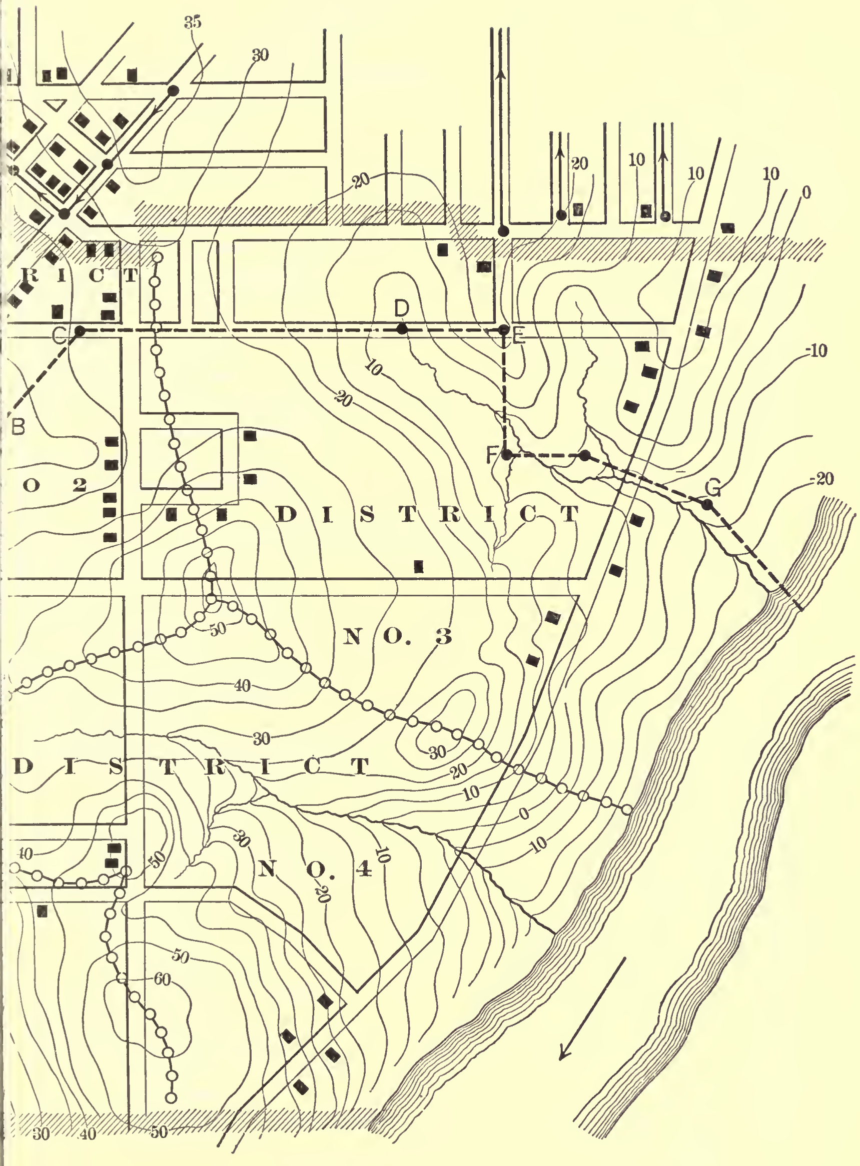
FOURTH WARD.

1906

SCALE OF FEET

0 500 1000




R I C T
O 2
D I S T R I C T
N O. 3
D I S T R I C T
N O. 4
B
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