SEWER DESIGN.

BY

H. N. OGDEN, C.E.,

ASSOC. M. AM. SOC. C. E.

Assistant Professor of Civil Engineering, Cornell University.

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PREFACE.

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.

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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 acompanying details of construction was patented under the name "Waring's Separate System," and under this patent Col. Waring has been paid large royalties by some of the cities for which he has 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 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.

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* 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."

In the second annual report of the Massachusetts State Board of Health, 1881, page 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 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 chemical examination goes, the real danger

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

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.

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.

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 large sewer should be the better ventilated.

While it is true that two systems, one for sewage and one for storm-water, 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. 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 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 dis-

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* Jour. Ass'n Eng. Soc., vol. III. p. 69.

charging 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 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 Record*, vol. XXVII. page 361, or by a special pipe, laid where the amount of ground-water makes it necessary and discharged into the sewer at a convenient man-hole. 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.

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 old map must alway 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. page 105.)

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, 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

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

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 beween 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 is 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 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. page 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 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.

The depth of rock and the character of the soil

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

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

	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	.620	100.0
Mapping 8 section maps	674.11	0.171	69.3
" 1 general "	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.

must be determined by borings or test-pits, the latter being preferable. Enough soundings should be taken to thoroughly explore 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. page 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 conveniently at hand. It may be run on wheels or slid on runners.

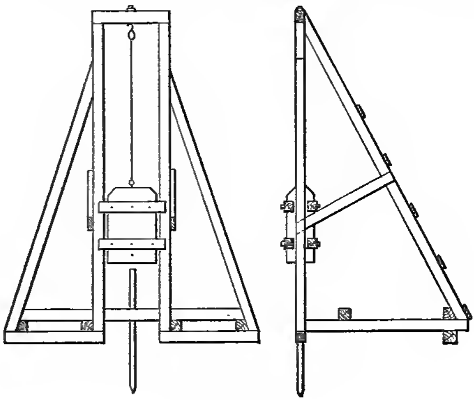

FIG. 1.

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.

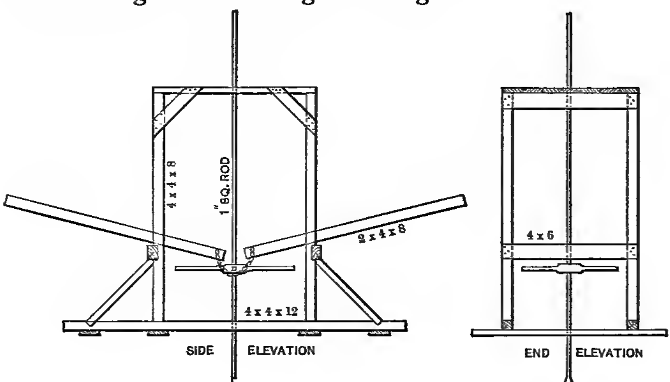

FIG. 2.

Fig. 2 illustrates a test boring-machine described in *Engineering News*, vol. XXI. page 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-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 $\frac{5}{8}$-inch pin for the lifting-chain to bear against.

Fig. 3 shows in detail another form of driver (*Engineering News*, vol. XXI. page 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

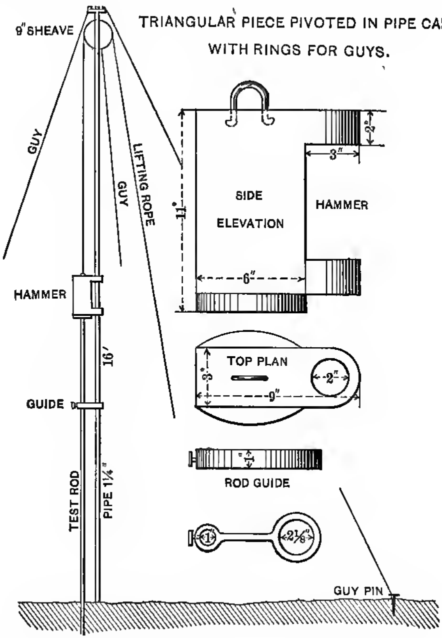

FIG. 3.

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 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 one foot. The profiles on the separate sheets

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

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.

These sheet-maps may be indexed on the large map, numbering the sheets to correspond with 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).

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.

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 2½ 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 ½ 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 24 hours and of 1 inch or more in 1 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-

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* *Weather Review* for 1890.

gages were stationed throughout the country. This number has since been increased to twenty-six, 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 cloud-bursts 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 over 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 down-pours are found only between the Sierras and the foot-hills 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 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 15 years before January 1, 1897. For Savannah, 62 in 8 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 following table is given:

TABLE I.

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	" " "
25 " "	3.60	" " "
30 " "	3.15	" " "
40 " "	2.75	June 10, 1876
50 " "	2.30	" " "
60 " "	1.98	" " "
120 " "	1.23	July 26, 1886

Table II from the Bulletin, given below, 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.

The first extended and detailed study of the excessive storms for a single locality was made in 1889 by

Table II.

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.
Bismark	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

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 employés 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 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 II 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 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. 4), two enveloping curves were drawn, one giving what might be called the very rare rainfalls, and the other

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

the ordinary maximum. The curves drawn were in both cases rectangular hyperbolas. After drawing the two curves their equations were determined to be

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

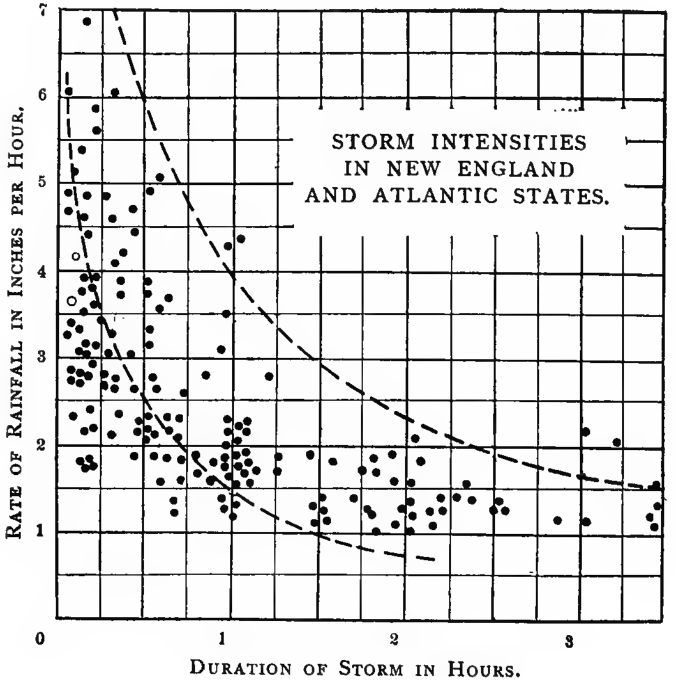

FIG. 4.

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 "	3.0	"	"
30	"	"	6.0 "	2.3	"	"
45	"	"	4.8 "	1.7	"	"
60	"	"	4.0 "	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 71 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 40 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 40 minutes will reach the values given by the curve $y = \dfrac{105}{x + 15}$ at least as frequently as two or three times in ten years."

In St. Louis Prof. Nipher has used the same method to 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 47 years.

Kuichling plotted (see Fig. 5) the local data of

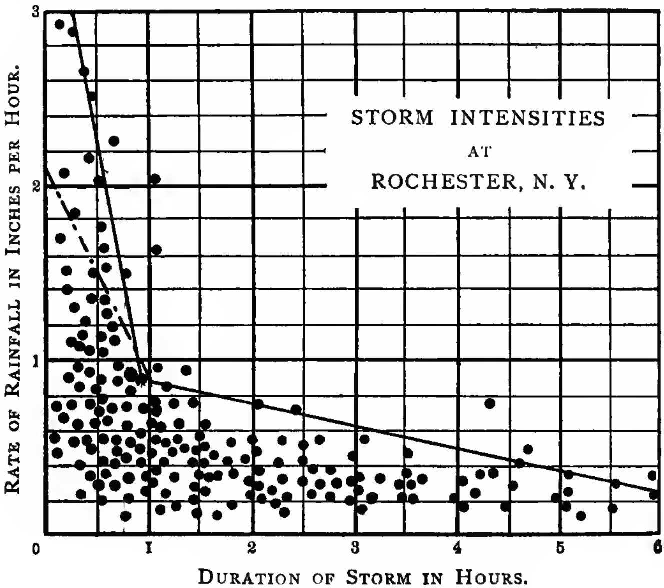

FIG. 5.

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.*

* See Trans. Am. Soc. C. E., vol. xx, page 1.

These two lines meet at a point of the diagram representing a duration of one hour and an intensity of .87 inches. The line to the left of the point of intersection is quite steep, while that on the right is more

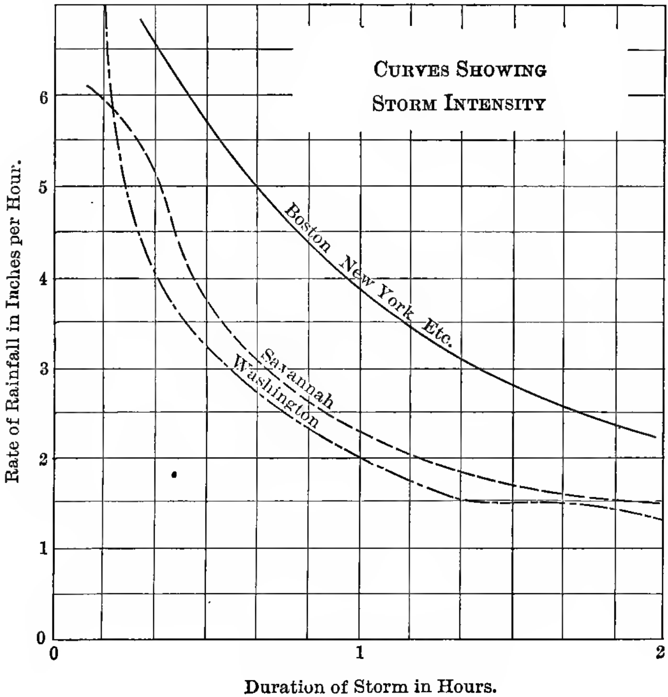

FIG. 6.

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 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

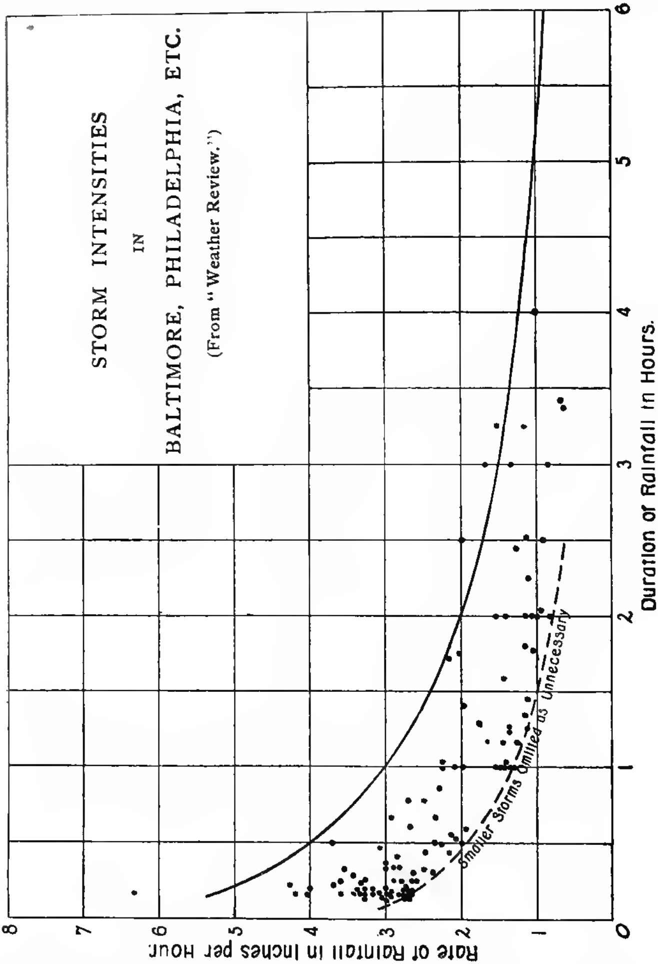

FIG. 7.

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 connection for which it was made. "It is," he says, "merely an attempt to utilize the avail-

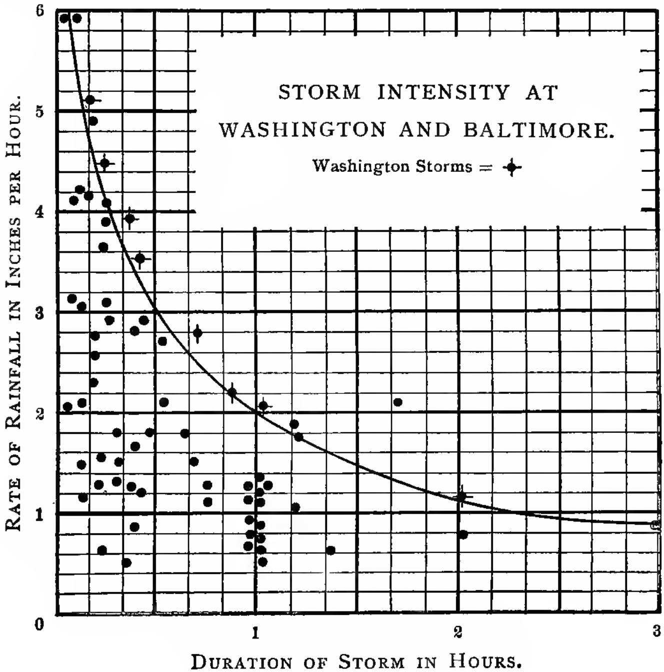

FIG. 8.

able data as to the local rainfall in a rational manner, and to remove the subject of urban sewerage from the realm of vague conjecture."

Figs. 7 and 8 show similar diagrams from the report of the Sewerage Commission of the City of Baltimore, 1897.

Bulletin "D" of the *Weather Review* shows three curves (see Fig. 6) constructed in this way; one for Washington, one for Savannah, and one from the combined records of Boston, Providence, New York, Philadelphia, and Washington. No algebraic expression for the curves is given, but for comparison we may take x equals 20 and 40 minutes, and compare in the following table the values of y with the three formulæ above given.

TABLE III.

TABLE SHOWING THE RELATION BETWEEN DURATIONS AND INTENSITIES OF STORMS ACCORDING TO THE SEVERAL FORMULÆ AND DIAGRAMS GIVEN.

Formula.	Intensities in Inches per Hour.	
	20 Minutes' Duration.	40 Minutes' Duration.
$yx = 360$	18.	9.0
$y = 2.10 - .0205x$	1.7	1.3
$y = 3.73 - 0.51x$	2.7	1.7
$y = \dfrac{360}{x + 330}$	7.2	5.2
$y = \dfrac{105}{x + 15}$	3.0	1.9
Washington diagram	3.9	2.8
St. Louis diagram	5.1	3.0
New York, Boston, etc., diagram	6.7	5.0
Baltimore diagram	3.6	2.5

The discrepancies in the above comparison where the rainfall for 20 minutes varies from 1.7 to 18 inches, and for 40 minutes varies from 1.3 to 9 inches, are partly to be accounted for by the different localities of the rainfall records and partly by differing judgments in the location of the envelope curve. In one case it may include all storms, and in another it may leave out very occasional ones where it is deemed unjustifiable to incur the cost of providing for such storms, considering the damage done by so rare a visitation.

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 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. Two feet per second may be taken as a minimum flow, and 15 feet per second as the maximum, but between the two the rate of flow will depend on the 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.

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

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 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 one 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 one 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 andRatcliff Street sewers, determined that from rainfalls of 2.9 inches in 36 and 25 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 36 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 5 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 one 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

Table IV.*

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, page 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.

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 preceding 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

* At Omaha, Neb.

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

Diagram 28 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. page 46, with like data of many other streams (see Table V).

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 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 V.

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	1.74	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	0.15	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	23.10	43.82	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 Koningsberg.

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."

Reviewing, then, the probable percentage of rainfall delivered to the sewer, it is seen that the percentage is not constant, but varies from several causes, and that a formula to take into consideration all the influencing factors would necessarily be complicated and require judgment in adapting the constants of such a formula to the district in question. Therefore it is more reasonable to study the district carefully, noting the amount of impervious surface, and from its characteristics determine the amount of run-off to be expected.

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, 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 24 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 24 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 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

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

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 VI.

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 VI 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 VII.

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 VI, we get finally the amount of water discharged from a given rain in terms of the varying densities, as follows:

TABLE VIII.

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. 9). The equation of this curve may also be found if desired. In applying the relation to cases where greater densities of population occur than are given in the table, it must be remembered that the rate of increase of the reduced impervious surface diminishes until that surface reaches a

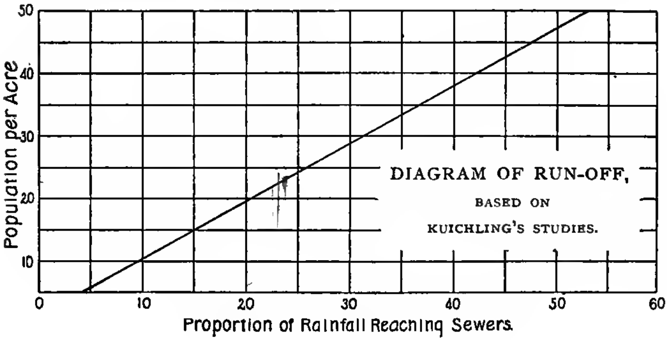

FIG. 9.

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 but 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.

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 one 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 one 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 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

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

Since the rainfall is assumed to be one 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

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

with $c = 1/2$, 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,

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

or

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

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, = one inch per hour, will reach the sewer during this period of time, he has

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

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,

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

Bürkli-Ziegler published in 1880 a paper on the discharge of sewers,* and in it proposed a variation of Hawksley's formula, to allow its use under other conditions than those of the London districts. In French units his formula was

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

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

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 hectares 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 .25 and .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

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

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

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

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

* Vol. XVI. page 179.

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 adds that the improvement over the Bürkli-Ziegler 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: $Q = c \,.\, A^{\frac{3}{4}} r^{\frac{3}{4}} s^{\frac{1}{4}}$; for $r = 1$, $cr = 3.95$.
Adams: $Q = c \,.\, A^{\frac{5}{6}} r^{\frac{5}{6}} s^{\frac{1}{12}}$; for $r = 1$, $cr = 1.03$.
Bürkli-Ziegler: $Q = c \,.\, rA^{\frac{3}{4}} s^{\frac{1}{4}}$;
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: $Q = cr \,.\, s^{\frac{1}{5}} A^{\frac{4}{5}}$;
for $r = 2:75$, $cr = 8.21$ for built-up areas;
$cr = 3.39$ for suburban areas.

N. Y. diagrams: $Q = crA^{.85}s^{.27}$;

$cr = 10.59$ 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:

Hawksley:	$Q = \text{const.} \times r^{.75}$.
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

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

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

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

As they are practically derived independently of a knowledge of the exact maximum rainfall, we may substitute for $c \,.\, r$ 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 $c \,.\, r$ 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.

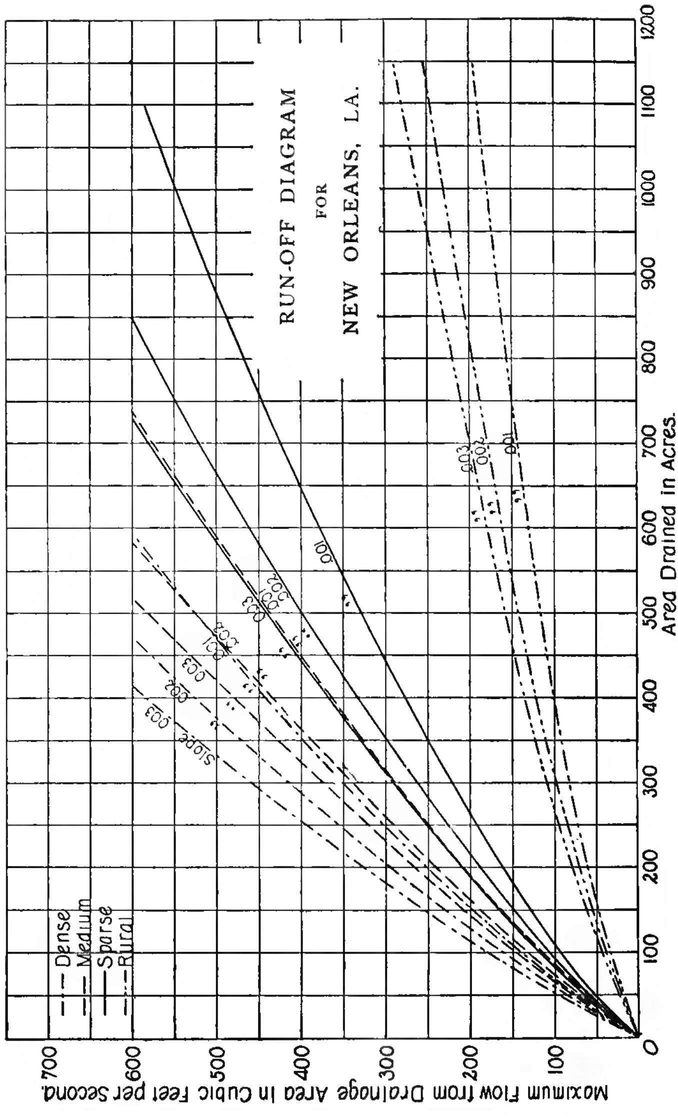
RUN-OFF DIAGRAM
FOR
NEW ORLEANS, LA.
Dense
Medium
Sparse
Rural
Slope .003
Area Drained in Acres.
Maximum Flow from Drainage Area in Cubic Feet per Second.

FIG. 1[illegible].

As a further example of work done in this direction, Fig. 10 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. 11 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, late 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.

Fig. 12 shows a diagram prepared by Mr. E. S. Dow and printed in the Journal of the Association of Engineering Societies, vol. X. page 353. It is based on the Bürkli-Ziegler formula, using the coefficients given in Gray's Providence report, and assuming a rainfall of one inch per hour. The broken appearance of the curves is due to a frequent change of scale, both vertical and horizontal, in order to keep the diagram condensed. It offers a ready means of making approximates, the constants of the formula being taken into account.

By either of the two ways just outlined, viz., by estimating the probable future population of each district of the city, and, by Table VIII of Chapter V,

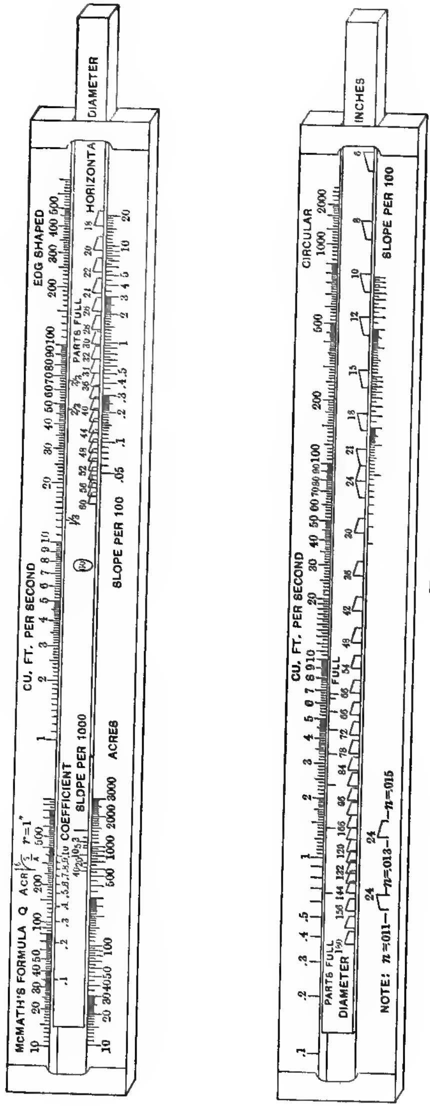
DIAMETER
INCHES
McMATH'S FORMULA
CU. FT. PER SECOND
HORIZONTA
PARTS FULL
SLOPE PER 100
COEFFICIENT
SLOPE PER 1000
ACRES
CIRCULAR
FULL
DIAMETER
NOTE: n=011 n=013 n=015

FIG. 11.

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

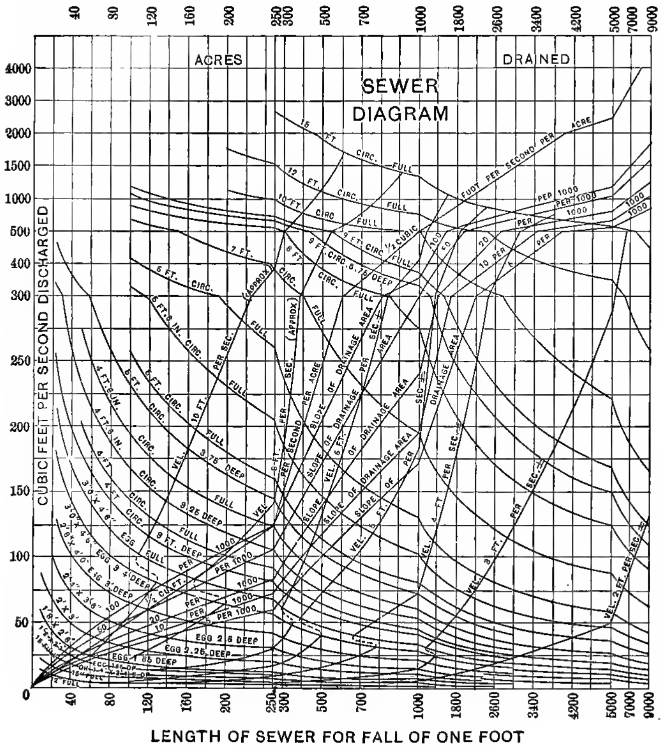

FIG. 12.

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.

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.

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

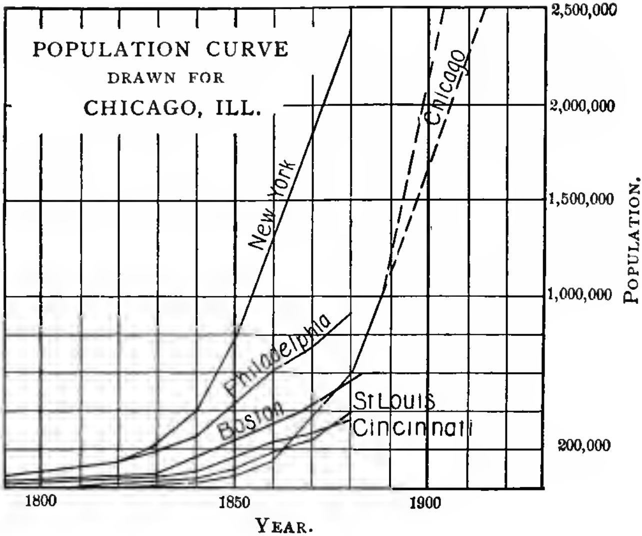
POPULATION CURVE
DRAWN FOR
CHICAGO, ILL.
New York
Philadelphia
Boston
St Louis
Cincinnati
Chicago
2,500,000
2,000,000
1,500,000
1,000,000
200,000
POPULATION.
1800
1850
1900
YEAR.

FIG. 13.

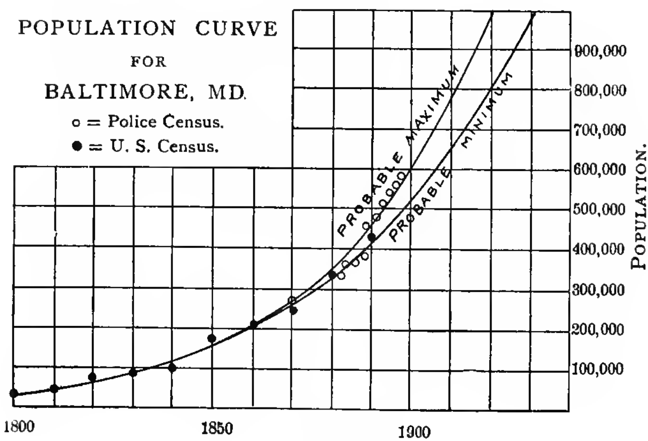
POPULATION CURVE
FOR
BALTIMORE, MD.
o = Police Census.
● = U. S. Census.
PROBABLE MAXIMUM
PROBABLE MINIMUM
900,000
800,000
700,000
600,000
500,000
400,000
300,000
200,000
100,000
POPULATION.
1800
1850
1900

FIG. 14.

District was estimated in this way, as shown in Fig. 13. 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. 14. 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 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 on the Metropolitan Water-supply made to the State Board of Health of Massachusetts in 1895, and is described as follows (see Fig. 15). 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

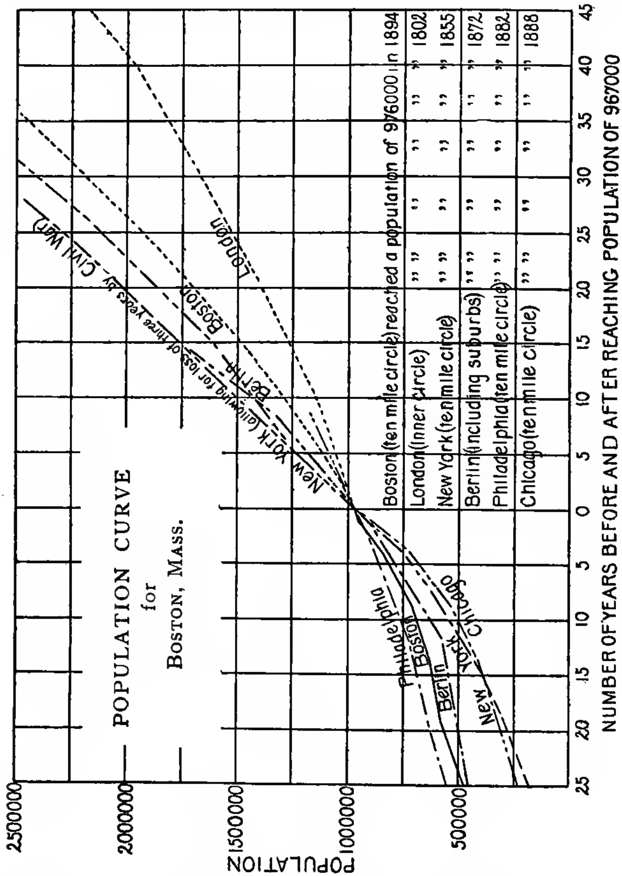

FIG. 15.

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 in-

POPULATION CURVE
drawn for
BROCKTON, MASS.

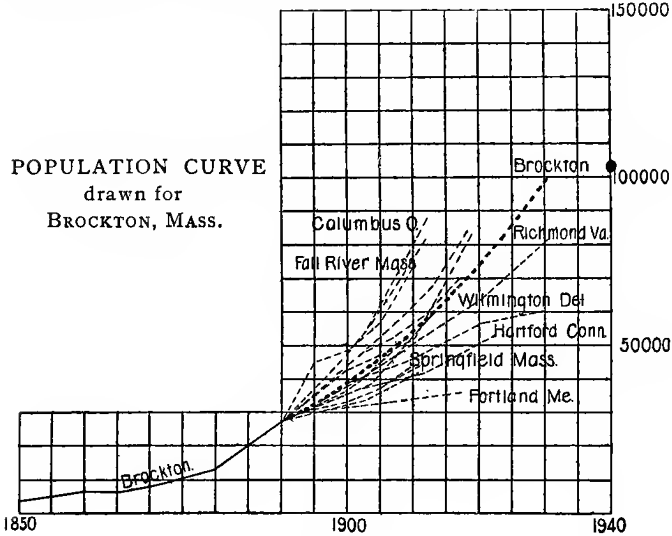

FIG. 16.

fluenced by no tremendous shock of pestilence, war, or business disaster its population line was drawn to follow approximately these other cities.

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

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 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 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 following table is taken from his report

TABLE IX.

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

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. 17 for the

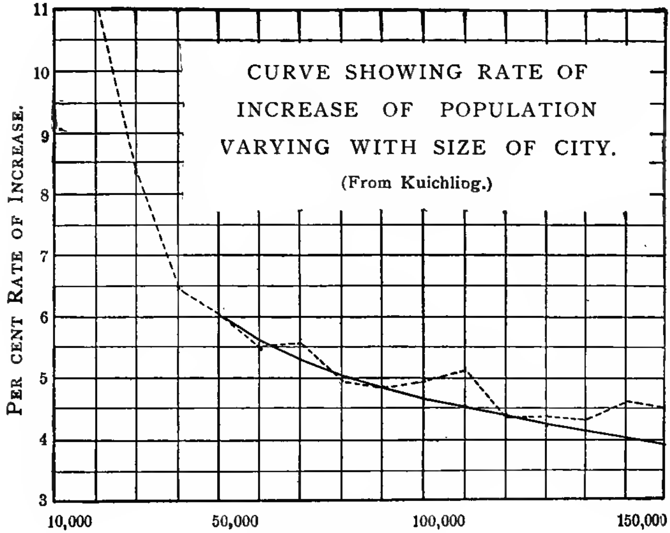

FIG. 17.

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 1890 and computed the rates of increase in a similar manner for cities of between 20,000 and 100,000, with results as shown in Table X and in Fig. 18. In this latter comparison

Table X.

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.
20000	19	66.02	0.50	29.13	
30000	17	48.75	1.59	12.57	
40000	17	40.41	0.67	10.21	
50000	11	41.67	1.41	7.68	
60000	9	56.46	1.00	12.36	
70000	5	54.74	0.74	8.83	
80000	6	51.47	3.30	7.57	
90000	6	50.96	2.80	7.07	

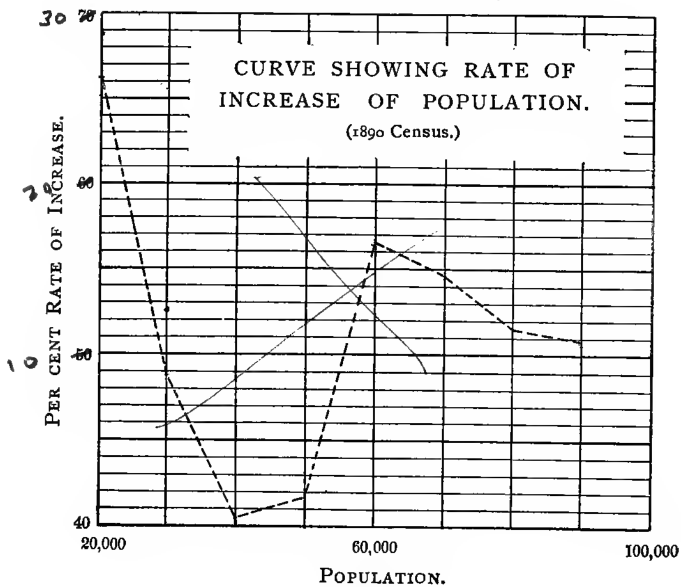

FIG. 18.

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 1880–1890, 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 or table the rate of increase corresponding to the present population, 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 X it is seen that cities of 20,000 inhabitants increased in population in 1880–1890 from 0.5 to 66 per cent, while the general law would indicate about 11 per cent. Should the city in question not be an average city, a large error would result from trying to apply this 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.

To further illustrate the method of securing an idea of the future population, Fig. 19 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 increase 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

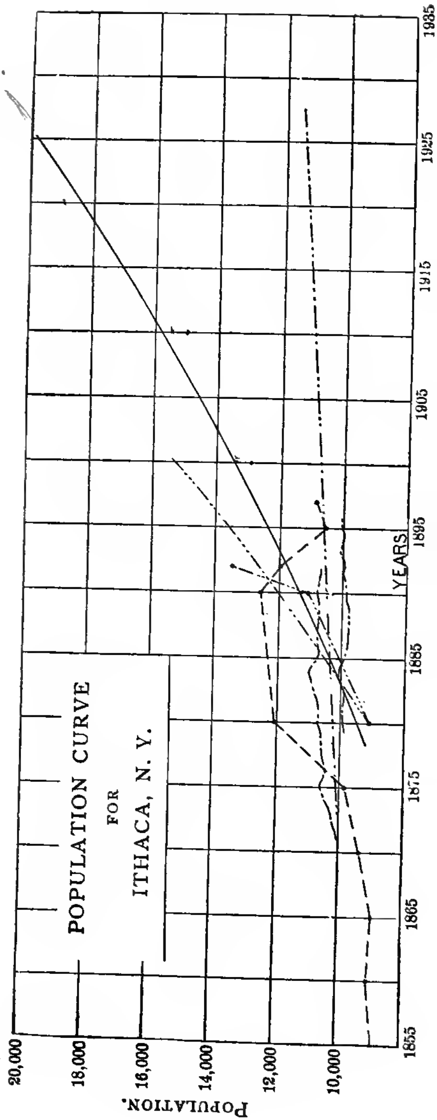
POPULATION CURVE
FOR
ITHACA, N. Y.
YEARS
1855
1865
1875
1885
1895
1905
1915
1925
1935
POPULATION.
20,000
18,000
16,000
14,000
12,000
10,000

FIG. 19.

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. 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 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.

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. 20 * 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-consumption for 1896. Fig. 21 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

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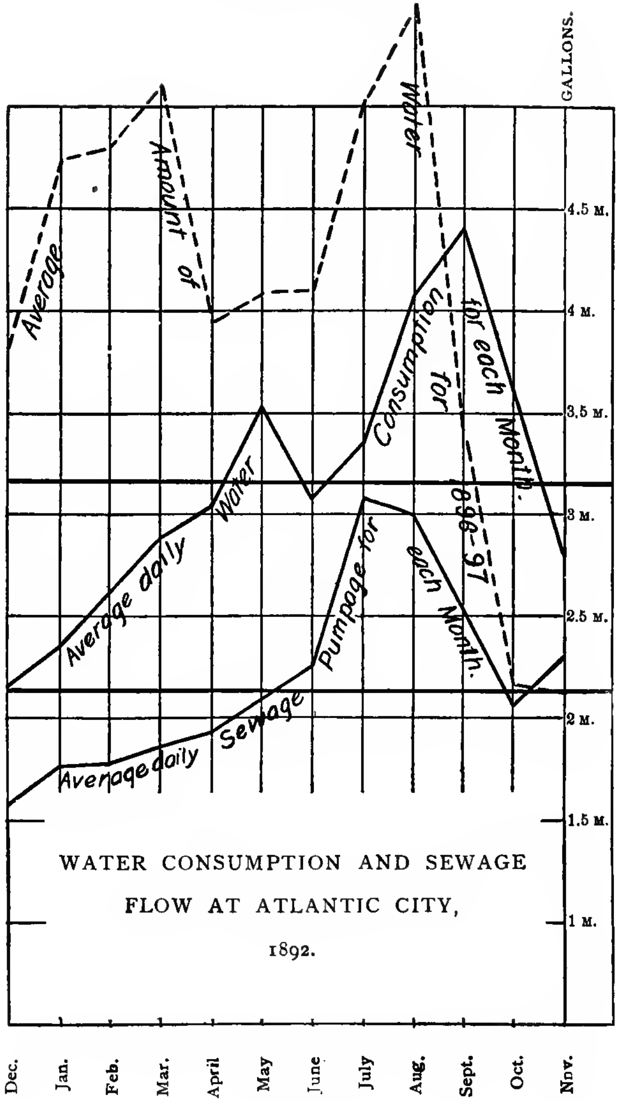
* From *Engineering News*, vol. XXIX. p. 124.

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



WATER CONSUMPTION AND SEWAGE FLOW AT ATLANTIC CITY, 1892.

FIG. 20.

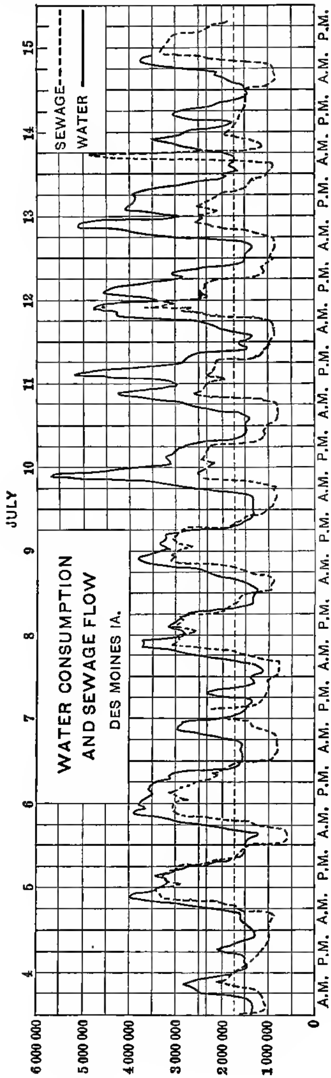

WATER CONSUMPTION
AND SEWAGE FLOW
DES MOINES IA.
SEWAGE
WATER
JULY
4
5
6
7
8
9
10
11
12
13
14
15
6 000 000
5 000 000
4 000 000
3 000 000
2 000 000
1 000 000
0
A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M.

FIG. 21.

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 table is given to show approximately how the average per capita consumption of water in different cities of the United States varies. The daily consumption is taken from the "Manual of American Water-works for 1897," and the populations taken from the same volume are for 1890. The table merely emphasizes the fact that there is no general law or rule as to the water-consumption of any particular city. It is to be noted in addition that the proportion of the total population using city water is not constant; that in many cities the water-consumption is not measured, but guessed at, and that therefore the per capita consumption as given in the table may be far from the actual amount.

Other legitimate causes affecting the average amount, yet generally masked by the waste, are the character of the city, whether residential or manufacturing, the amount of sewer-flushing and street-sprinkling done, the number of water-motors, hydraulic elevators, and other small hydraulic machinery. Any

TABLE XI

SHOWING CONSUMPTION OF WATER IN CITIES OF THE UNITED STATES.

Name.	Population.	Daily Consumption.	Daily Consumption per Capita.
Maine—			
Auburn	11,250	500,000	45
Bangor	19,103	3,000,000	158
*Waterville	12,107	1,000,000	83
New Hampshire—			
Dover	12,790	565,800	44
Manchester	44,126	3,500,000	80
Nashua	19,311	3,500,000	181
Vermont—			
Burlington	14 590	880,080	60
Massachusetts—			
Beverly	10,821	1,000,000	92
Boston	448,477	9,457,000 50,801,000	134
Brockton	27,294	1,098,560	40
Cambridge	70,028	6,002,142	85
Clinton	10,424	1,600,000	154
Fall River	74,398	3,166,500	43
Gloucester	24,651	1,000,000	40
Haverhill	27,412	3,000,000	110
Holyoke	36,637	3,561,643	100
Lawrence	44,654	3,005,624	67
Lynn	55,727	4,360,142	78
Marlborough	13,805	510,000	36
New Bedford	40,733	4,711,866	115
Newton	24,379	1,801,000	75
Pittsfield	17,281	1,700,000	100
Salem	30,800	2,147,437	70
Springfield	44,179	4,638,060	105
Waltham	18,707	1,221,842	64
Worcester	84,655	6,500,000	77
Rhode Island—			
Newport	19,457	2,100,000	108
Providence	132,146	8,905,000	67
Connecticut—			
Bridgeport	48,866	16,000,000	327
Hartford	53,230	7,500,000	142
Meriden	21,652	2,000,000	93
New Haven	81,298	13,700,000	170
New London	13,757	1,244,000	92
Stamford	15,700	1,500,000	94
Waterbury	28,646	4,000,000	140

* Including Fairfield.

TABLE XI—*Continued.*

Name.	Population.	Daily Consumption.	Daily Consumption per Capita.
New York—			
Albany	94,923	18,000,000	190
Auburn	25,858	3,500,000	135
Binghamton	35,005	5,217,703	150
Brooklyn	838,547	80,124,432	96
Buffalo	255,664	100,000,000	392
Elmira	30,893	3,000,000	97
Ithaca	11,079	400,000	36
Kingston	21,261	1,500,000	71
Middletown	11,977	1,800,000	150
Newburgh	23,087	3,500,000	152
New York	1,515,301	189,000,000	125
Oswego	21,842	2,330,500	105
Poughkeepsie	22,206	1,881,576	86
Rochester	133,896	11,000,000	82
Schenectady	19,902	4,750,000	237
Syracuse	88,443	7,500,000	85
Troy	60,956	8,599,977	141
Yonkers	32,033	3,230,000	101
New Jersey—			
Atlantic City	13,055	4,500,000	354
Camden	58,313	12,000,000	207
Elizabeth	37,764	4,000,000	105
Jersey City	163,003	19,300,000	118
Newark	181,830	22,200,000	122
Passaic	13,028	400,000	31
Plainfield	11,267	700,000	63
Trenton	57,458	5,500,000	96
Pennsylvania—			
Allegheny	105,287	28,000,000	266
Altoona	30,337	2,000,000	66
Bradford	10,514	1,330,500	127
Erie	40,634	5,426,000	133
Johnstown	21,805	6,000,000	273
Lancaster	32,011	5,700,000	180
Norristown	19,791	2,250,000	112
Philadelphia	1,046,964	215,000,000	205
Pittsburg	238,617	63,000,000	264
Reading	58,661	6,500,000	111
York	20,793	2,375,000	113
Delaware—			
Wilmington	61,431	5,829,040	95

TABLE XI.—*Continued.*

Name.	Population.	Daily Consumption.	Daily Consumption per Capita.
Maryland—			
Baltimore	434,439	57,000,000	131
Cumberland	12,729	174,240	13
Hagerstown	10,118	1,000,000	99
Washington, D.C.*	202,978	44,000,000	216
Virginia—			
Danville	10,305	509,817	50
Lynchburg	19,709	3,951,000	197
Norfolk	34,871	3,500,000	103
Portsmouth	13,268	400,000	30
Roanoke	16,159	1,966,278	122
West Virginia—			
Wheeling	34,522	7 596,871	222
North Carolina—			
Asheville	10,235	350,000	34
Charlotte	11,557	500,000	43
Raleigh	12,678	1,000,000	79
Wilmington	20,056	540,000	27
South Carolina—			
Charleston	54,952	1,500,000	27
Columbia	15,353	1,300,000	86
Georgia—			
Atlanta	65,533	4,541,334	69
Augusta	33,300	3,800,000	114
Macon	22,746	1,647,000	73
Savannah	43,189	6,000,000	116
Florida—			
Jacksonville	17,201	1,218,508	71
Pensacola	11,750	450,000	39
Alabama—			
Anniston	10,000	1,000,000	100
Birmingham	26,178	8,766,330	337
Montgomery	21,883	1,333,900	62
Mississippi—			
Natchez	10,101	275,000	27
Vicksburg	13,373	823,500	62
Louisville—			
Baton Rouge	10,478	500,000	48
New Orleans	242,039	9,000,000	37
Tennessee—			
Knoxville	22,535	1,931,891	86
Memphis	64,495	9,000,000	140
Nashville	76,168	12,204,576	160

* Including Georgetown.

TABLE XI.—*Concluded.*

Name.	Population.	Daily Consumption.	Daily Consumption per Capita.
Kentucky—			
Covington	37,371	2,624,320	70
Henderson	8,835	800,000	91
Lexington	21,567	1,200,000	55
Newport	24,918	2,250,000	90
Paducah	12,797	1,200,000	94
Ohio—			
Akron	27,601	3,000,000	109
Canton	26,189	3,000,000	114
Cincinnati	305,900	47,203,068	154
Cleveland	270,000	47,154,000	175
Columbus	88,150	14,000,000	159
Sandusky	18,471	3,781,000	207
Toledo	81,434	7,841,200	96
Indiana—			
Evansville	50,756	6,500,000	128
Fort Wayne	35,393	3,000,000	85
Indianapolis	105,436	9,000,000	86
Richmond	16,608	2,500,000	150
Terre Haute	30,217	3,000,000	99
Michigan—			
Bay City	27,839	3,075,000	110
Detroit	205,876	40,000,000	194
Grand Rapids	60,278	13,916,286	231
Kalamazoo	17,853	1,800,000	101
Muskegon	22,702	2,600,000	115
Saginaw	46,322	6,678,741	....
Illinois—			
Aurora	19,688	1,300,000	144
Bloomington	20,484	1,200,000	58
Chicago	1,600,000	251,839,816	151
Evanston	12,762	2,993,296	234
Peoria	41,024	4,000,000	98
Quincy	31,494	1,200,000	38
Springfield	24,963	3,708,566	148
Wisconsin—			
Eau Claire	17,415	1,400,000	80
La Crosse	25,090	3,484,400	139
Milwaukee	204,468	25,291,050	124
Oshkosh	22,836	2,100,000	91
Racine	21,014	2,000,000	95

attempts to reduce waste, whether by the introduction of meters or by house-to-house inspections, are always immediately noticeable in the daily consumption. The source of supply as affecting the interest in waste, that is, whether the water has to be pumped or whether it flows freely to the consumer by gravity, also affects the daily flow, while the normal consumption for purely necessary domestic use is about constant.

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, 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.

The examples cited in Boston are generally apartment- and boarding-houses, the average number of persons per house being 40. The consumption per

TABLE XII.

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.	Consumpt'n Gallons per Family.	Consumpt'n 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 supplied 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 w. c.
Worcester..	......	20,514	90,942	.....	16.8	Whole domestic consumption
" ..	......	81	327	80.2	19.9	Woodland 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

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.

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.

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.

After duly considering these quantities, 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 ex-

TABLE XIII.

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	Green houses	9,550

ample, require little beyond that needed for domestic use. Other cities, with industries using large amounts of water, may require more than the 35 gallons adopted for Boston.

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.

By measuring the flow of water through the water-mains 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.

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 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

TABLE XIV.

	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	15	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

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.

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

Fig. 22 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 amount for the five days is 220,440 gallons, as shown by the heavy straight line; the average daily maximum (272,400) being 24 per cent more than the average daily consumption.

Fig. 23, from a table given by Rafter and Baker, shows the water-consumption in Rochester from Hemlock Lake. As before, the average daily consumption is added and the maximum daily consumption is thus shown to be 46 and 58 per cent more than the average, for the two days shown.

A list of actual sewer-gagings, so far as have been made public, was compiled and published in *Engineering News*, vol. XXXV. page 131, in a paper on a sewer-gaging of Des Moines. The table is given below with gagings of the outlet sewer at Canton, O., and Chautauqua, 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,

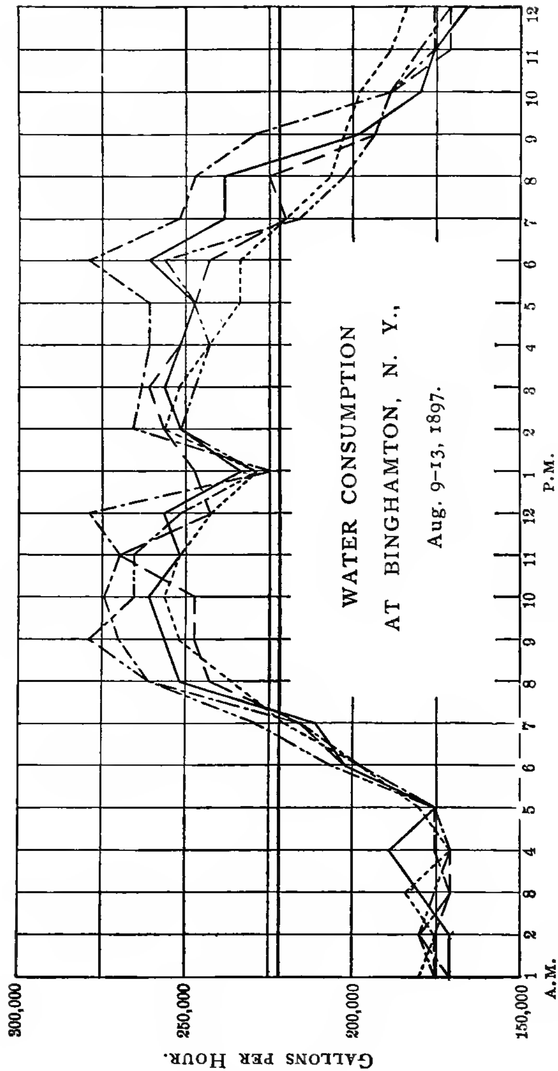

WATER CONSUMPTION
AT BINGHAMTON, N. Y.,
Aug. 9–13, 1897.
GALLONS PER HOUR.
300,000
250,000
200,000
150,000
A.M.
P.M.
1 2 3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12

FIG. 22.

and the second the same percentage should the quantities involved be reduced by the amount required to

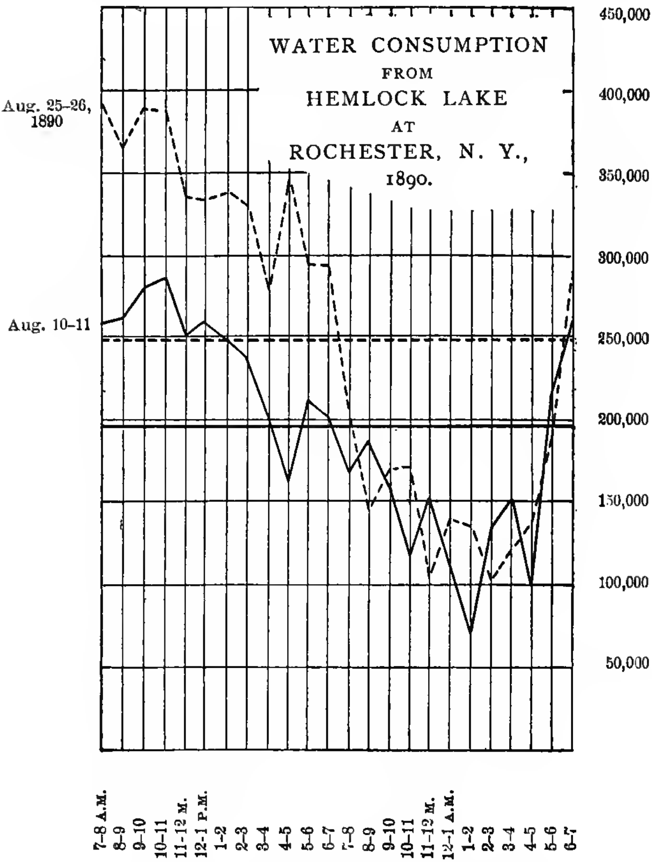


FIG. 23.

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

TABLE XV.

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

Sewer.	Date.	Population. Tributary.	Sewage Flow. Minimum.	Sewage Flow. Average.	Sewage Flow. Maximum.	Per Cent Max. above Average	Per Cent Max. above Average. Min. made O.
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 Coll..........	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

* 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. 24 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 of the other holidays of the year.*

Fig. 25 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.

* *Eng. News*, vol. XXIX. page 123. † *Ibid.*, vol. XXX. page 61.

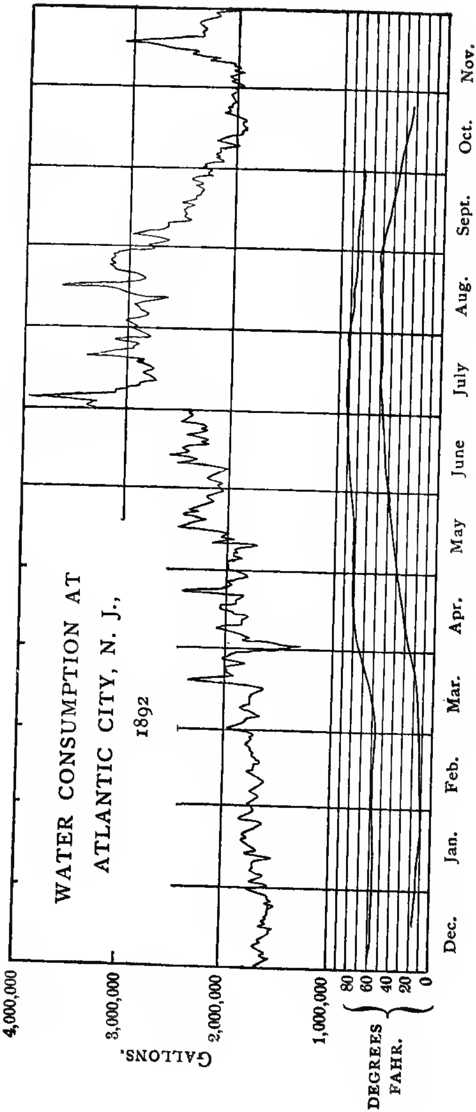

WATER CONSUMPTION AT
ATLANTIC CITY, N. J.,
1892
GALLONS.
4,000,000
3,000,000
2,000,000
1,000,000
DEGREES FAHR.
80
60
40
20
0
Dec.
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.

FIG. 24.

Fig. 26 shows the results of the gaging of the Schenectady outfall sewer as given by Mr. Landreth.* The average line is added (35,500 gallons), and the maximum found to be 59 per cent greater than the average flow.

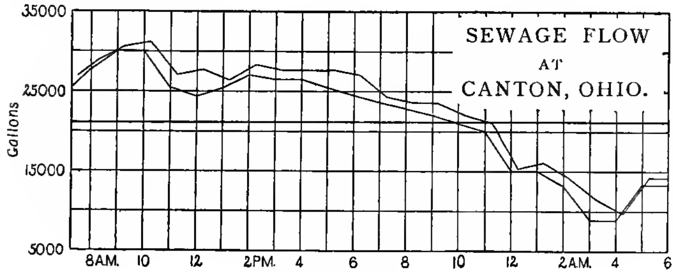


FIG. 25.

Fig. 27 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. 28 is a diagram taken from the paper of Mr. Grover already alluded to.

Fig. 29 gives a diagram of the sewage-pumping records at 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. 28 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

* *Eng. News*, vol. XXVII. page 305. † *Ibid.*, vol. XXXI. page 87.

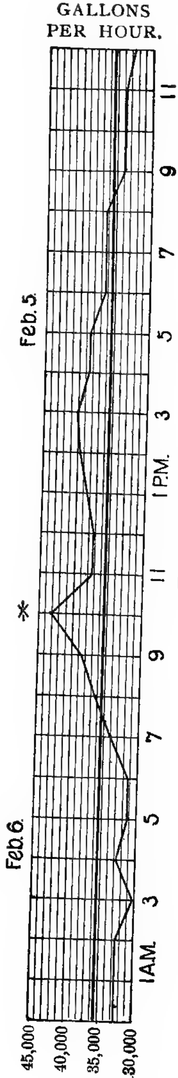

GALLONS
PER HOUR.
Feb. 5
Feb. 6
*
45,000
40,000
35,000
30,000
1 A.M.
3
5
7
9
11
1 P.M.
3
5
7
9
11

FIG. 26.

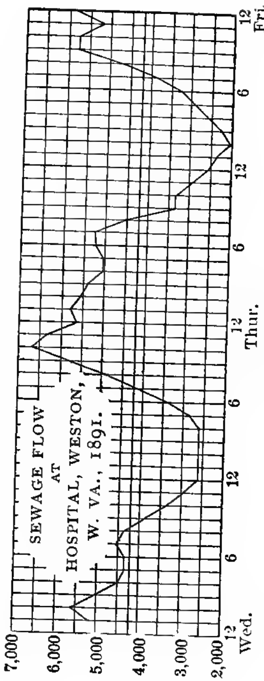

SEWAGE FLOW
AT
HOSPITAL, WESTON,
W. VA., 1891.
7,000
6,000
5,000
4,000
3,000
2,000
12
Wed.
6
12
6
12
Thur.
6
12
6
12
Fri.

FIG. 27.

greater than the average if the flow at 3 A.M. be called nothing, and the flow for the rest of the day reduced correspondingly.

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 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. Then assume that about ~~75~~ 100 per cent of this will be the sewage-flow at the same time. Then add 100 per cent for the maximum daily 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, and 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 "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

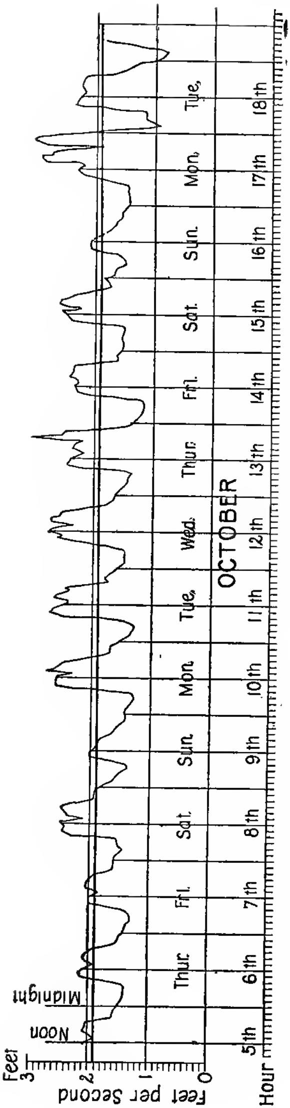

Feet per second
Feet
Noon
Midnight
Hour
OCTOBER
Thur. Fri. Sat. Sun. Mon. Tue. Wed. Thur. Fri. Sat. Sun. Mon. Tue.
5th 6th 7th 8th 9th 10th 11th 12th 13th 14th 15th 16th 17th 18th

Fig. 28.

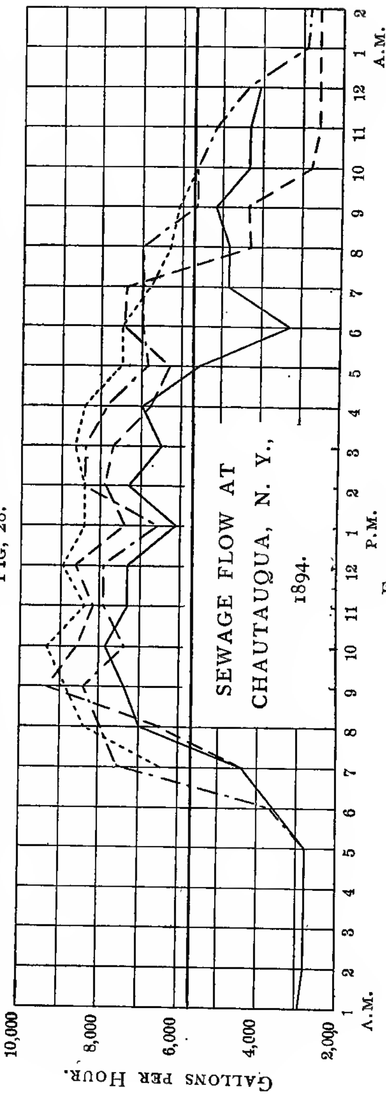

SEWAGE FLOW AT
CHAUTAUQUA, N. Y.,
1894.
Gallons per Hour.
10,000
8,000
6,000
4,000
2,000
A.M.
P.M.
A.M.

Fig. 29.

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 about 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	$10\frac{1}{2}$
Rochester,	"	10	$9\frac{1}{2}$
"	"	25	$8\frac{1}{2}$
Canton			$9\frac{1}{2}$
Schenectady			11
Hospital, Weston			9

* Baumeister.

Chautauqua, July 24..	$8\frac{1}{2}$
" " 31..	$8\frac{1}{8}$
" August 16... ..	$9\frac{1}{4}$
" " 18...	$9\frac{1}{8}$

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 XI it varies from 13 to 354 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 XII.

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.

CHAPTER IX.

GROUND-WATER REACHING SEWER.

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 four 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.

"*Schnectady, 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. page 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 a 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 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

* Trans. Am. Soc. C. E., vol. xxv. page 125.

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 works it will probably be safe to allow for an infiltration of 15 per cent of the flow of sewage proper."

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

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

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 on other details of the construction of the joint. In vol. XIII. page 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, made some experiments in which water was admitted to the joint at various intervals from the time of making. His results are given in the Trans. of the Association of Civil Engineers of Cornell University, 1897, page 113. 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 of pressure, 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 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 lengths of 6-inch pipe, and the mortar all mixed 1 : 1. His results 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 40,000 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 of 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 page 123.) For larger pipe the increase would probably be proportional to the area of the joint, or approximately to the squares of the diameters.

For example, a 12-inch pipe 2 miles long, laid in water, might be reasonably expected to carry as a minimum amount of subsoil infiltration $4000 \times 4 \times 2$ or 32,000 gallons of water, while the capacity of the pipe at a velocity of 2 feet per second is 102,000 gallons per day, or about three times the amount of ground-water. 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.

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.

It must be remembered, however, 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.

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. 30 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 this 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 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:

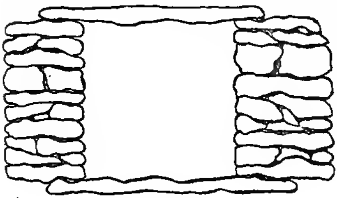


FIG. 30.

"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 the 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,

TABLE XVI.

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.	Bottom Velocity. Feet per Second.	Bottom Velocity. Feet per Second.	Feet per Second.	Feet per Second.	Bottom Velocity. Feet per Second.	Bottom Velocity. Feet per Second.
Soft earth		0.25							
Brick clay {allowed to settle ¼ 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

Gravel (size of peas)	0.62				2.000	2.46 (if disturbed)	0.62		
“ (size of beans)						2.95 (if disturbed)	1.066		
“ (diameter, .008 ft.)				0.82					
“ (diameter, .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 (.085 ft. diameter)							3.09		
“ (.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			
“ (.17 feet 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 (.33 ft. diameter)				4.92					
Laminated rock		6.00						6.00	6.00
Gravel (.56 ft. diameter)				6.56					
All gravel						6.56 (if disturbed)			
Gravel (.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		

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 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 XVI, 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 XVII.

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.

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

Baldwin Latham....	2 to 3 ft. per sec.
Beardmore.	2.5 to 3 " " "
Phillips.....	2.5 to 3 " " "
Rankin..	1 to 4.5 " " "
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, with a velocity of 3 feet, when flowing half full, has with the less depth a velocity of but 1.6 feet per second, which, by the table on page 135, is not sufficient to move anything except the smallest gravel.

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 1½ 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.

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 page 146).

Staley and Pierson say that a 6-inch lateral laid on a $\frac{4}{10}$ per cent grade works in a fairly saisfactory way, but Hering advises $\frac{5}{10}$ per cent if 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 the 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.

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 $v = CR\sqrt{H}$.

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. page 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

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

and in English units

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

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 XVIII.

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 recorded 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 surrounding 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

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

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 for 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 metres 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 preliminary, 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

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

* Recherches Hydrauliques.

or

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

To determine values of α and β, M. Bazin plotted for constant 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:

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

II. Smooth ashlar, brick, and wood:

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

III. Rubble masonry:

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

IV. Earth:

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

V. Carrying detritus and coarse gravel:

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

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 (page 159).

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 up-stream affects the shape of the parabola, bringing the axis nearly to the surface, so that the surface-velocity is the maximum velocity, while a down-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 90 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 C is .004, n is 1.74, and x is .67. 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.

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

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

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

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

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

$\dfrac{y}{1+\frac{x}{R}} = \dfrac{y}{\frac{R+x}{R}} = \dfrac{Ry}{R+x}$

transformed by taking the reciprocal of each side, the expression becomes

$\dfrac{R+x}{Ry} = \dfrac{R}{Ry}(1+x) = \dfrac{R}{Ry} + \dfrac{x}{Ry}$

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

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 $a + \dfrac{l}{n}$, and x to $a \cdot n$, so that C equals

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

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 Humphries 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 of 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

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

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

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

$$v = \frac{23 + \frac{l}{n} + \frac{0.00155}{S}}{1 + \left(23 + \frac{.00155}{S}\right)\frac{n}{\sqrt{R}}}\sqrt{R \,.\, S} \text{ in metric units,}$$

or

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

The values given for n are as follows:

I. Channels lined with carefully planed boards or with smooth cement............ .... 0.010

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 volume VIII of the Transactions of the Am. Soc. C. E., page 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

$$v = c\sqrt{R} \,.\, \sqrt{s}$$

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 $A \,.\, c\sqrt{R}$ 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 $12' \times 18'\,6''$ 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,

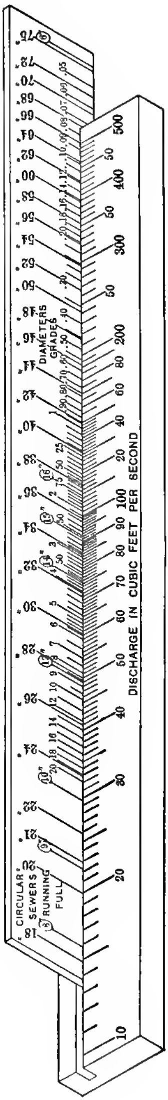


Fig. 31.

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.

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 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. 31.

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 be 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. page 383, very elaborate and complete and models of the sort.

In the same publication, vol. XCVI. page 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 edition, where Kutter's lines are printed in red on the same plate.

Among the diagrams compiled by J. Leland Fitz-Gerald, 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. page 212.)

The first diagram based on Kutter's formula was that published in the Trans. Am. Soc. C. E., vol. VIII. page 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. page 449.

A diagram computed by Mr. Moore of St. Louis is given in the Journal of the Association of Engineering Societies, vol. V. page 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. page 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 from its largest size, 24 inches, by Mr. F. S. Bailey, and is published in *Engineering News*, vol. XXXII. page 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. page 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. pages 116 and 119, are two diagrams by John W. Hill.

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.

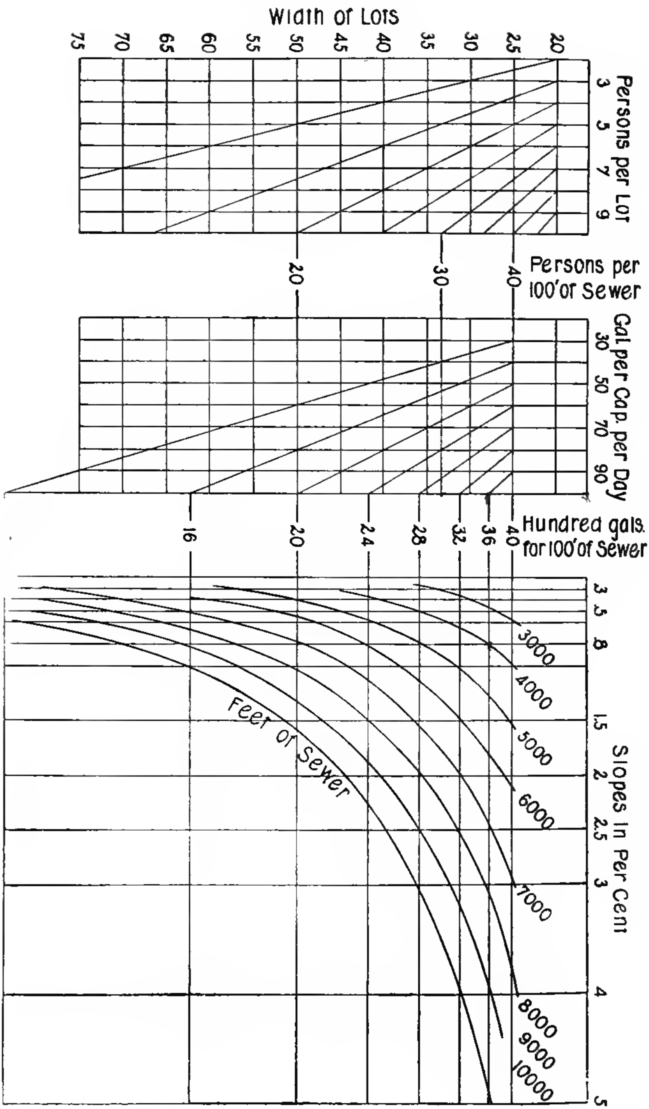

Width of Lots
75 70 65 60 55 50 45 40 35 30 25 20
Persons per Lot
3 5 7 9
Persons per 100' of Sewer
20 30 40
Gal. per Cap. per Day
30 50 70 90
Hundred gals. for 100' of Sewer
16 20 24 28 32 36 40
Slopes in Per Cent
.3 .5 .8 1.5 2 2.5 3 4 5
Feet of Sewer
3000 4000 5000 6000 7000 8000 9000 10000

FIG. 32.

The diagrams in Plates 3, 4, and 5 are given with the idea that they will serve all purposes of actual design. The abscissæ are grades in per cent, and the ordinates discharges 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. 32 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 arguments 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.

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 necessary 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. 5, we find the maximum rate of a storm lasting 11 minutes to be 3.18 inches per hour.

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 page 61, 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 3.1 cubic feet per second. By the same diagram, this will require a slightly larger pipe, but the grade being a little heavier the 15-inch pipe is allowed to stand. At the foot of the hill, 400 feet from Aurora Street, the grade changes to 0.5 per cent, and on this grade a 27-inch pipe is required, which will run the 400 feet to the corner. Here the drainage of 7 acres, or 1.8 cubic feet per second, enters, making 23.7 cubic feet in all. On the same grade of 0.5 per cent this takes about a 28-inch pipe, which will be made 30 inches. At Seneca Street will enter the water from the area between Seneca and State streets and west of Stewart Avenue. This amounts to 3.5

cubic feet per second, making 27.2 in all, still within the capacity of a 30-inch pipe. At Buffalo Street the contributing area is 13.7 acres, or 3.5 cubic feet per second, making a total of 30 7 cubic feet, requiring again a 30-inch pipe. 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 pipe from here to the creek, making it 36 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.

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 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

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* Baumeister.

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 the perpendicular system (Fig. 33).* 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

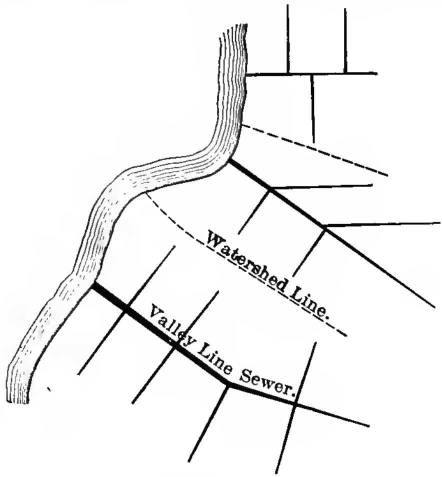

FIG. 33.

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

* Report of the National Board of Health, 1881, page 117 *et seq.*

large enough for satisfactory dilution and the sewage has all to be carried 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. 34. This is sometimes an after-thought

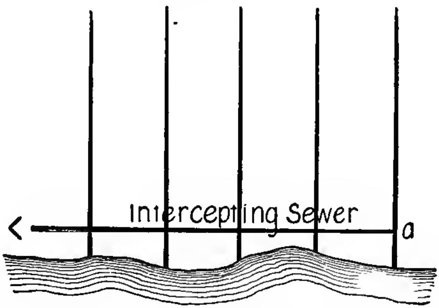

FIG. 34.

(as in Milwaukee and Chicago), in which case the different elevations of the main ends makes the construction of the intercepting sewer very difficult. The first system 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, it has the further advantage of allowing 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. 35).

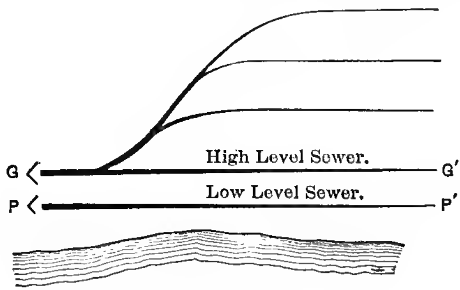

FIG. 35.

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.

(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. 36). 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 only city offering a good example, the sewage is cared for at a number of points in the circuit of the

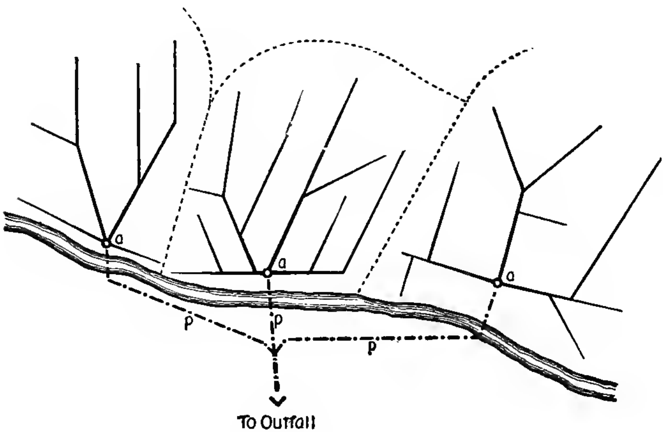

FIG. 36.

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. 37). 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.

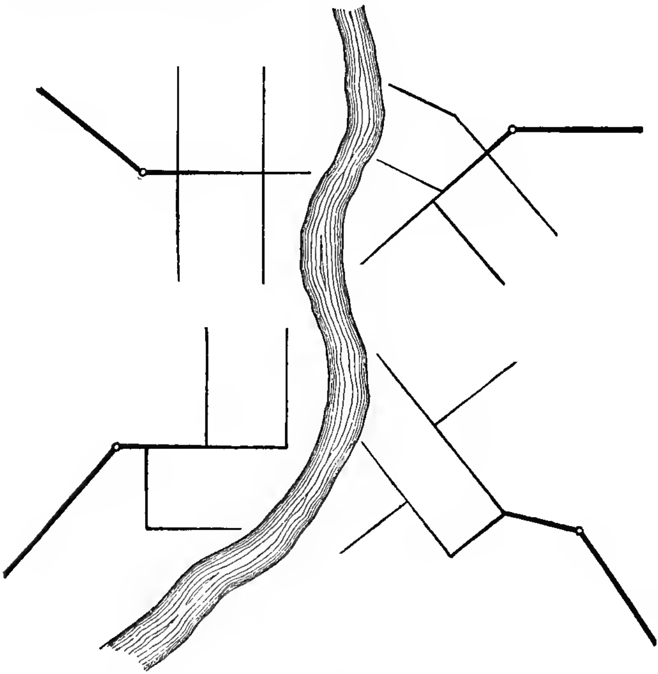

FIG. 37.

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. 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 depths 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 in the shortest path, though topographical conditions do not always admit of this. In Fig. 38

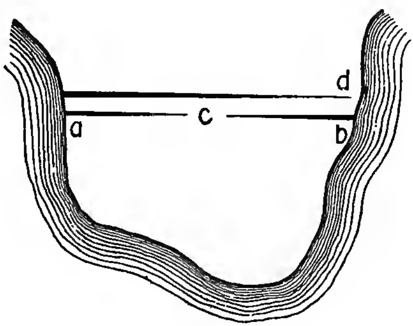

FIG. 38.

it is more expedient to build two sewers from *c* to the water at *a* and *b* than to take the 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. 39 and 40 illustrate the point, the length of

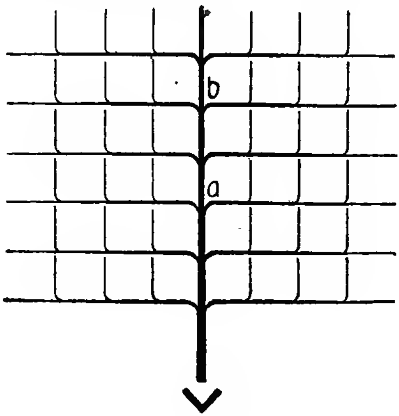

FIG. 39.

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. 40 than in Fig. 39, the latter is the more economical arrangement. Further, the laterals will have a better grade, that is, the grade will be placed where it is most needed, this resulting from the fact that, since the main in Fig. 39 is larger than that in Fig. 40, it will not require as great a grade for the same velocity. Compare also in Fig. 41 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

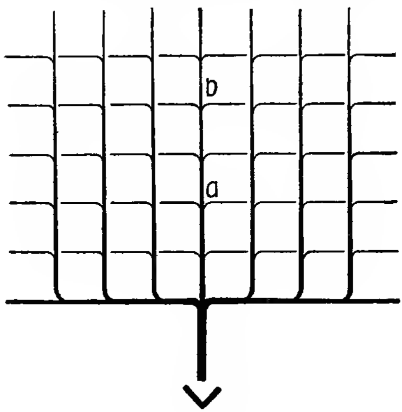

FIG. 40.

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. The comparative cost is here to be considered, though the single manhole or flush-tank probably gives better ventilation.

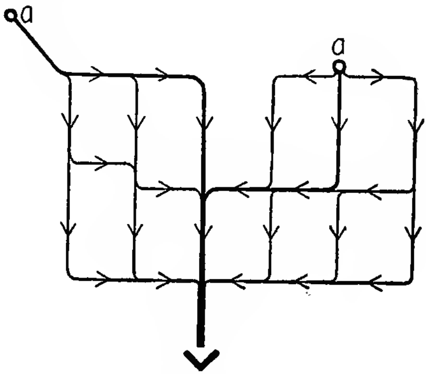

FIG. 41.

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.

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.

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

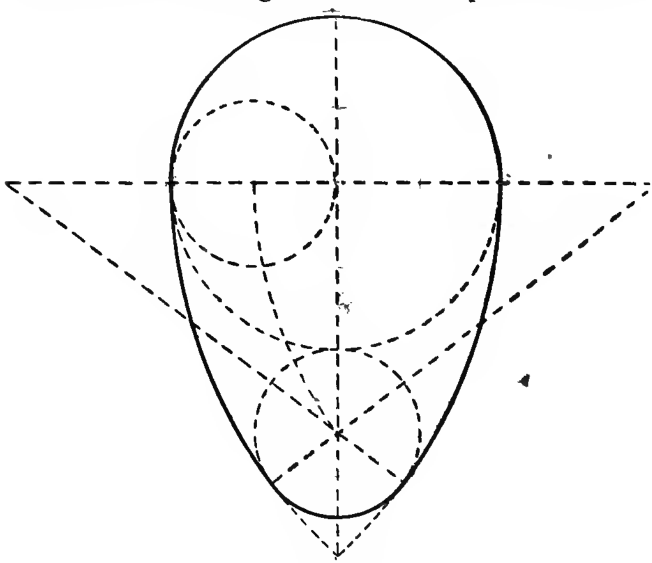

FIG. 42.

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. The egg-shape shown in Fig. 42 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, 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. 43, has a smaller

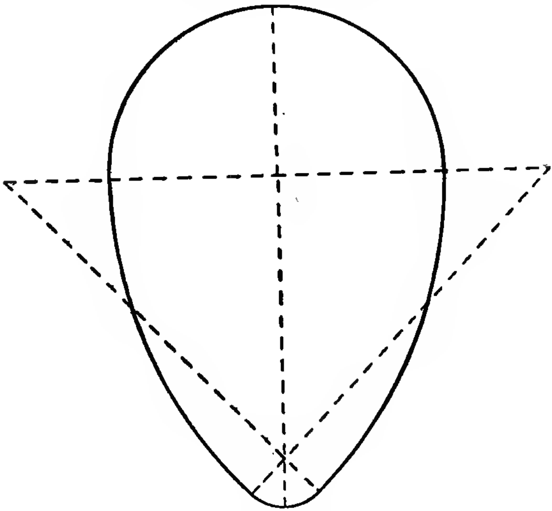

FIG. 43.

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.

Other forms are given by Latham and other engineers; some wide and shallow, designed chiefly for places where head-room is restricted, as under streams or railroads, while others deep and narrow are used to advantage in deep trenches where the excavation is made from the surface.

In order to obtain some comparison between the value of egg-shaped and circular sewers when the flow is small, the author has plotted two sections, reduced in Fig. 44, one of a circular sewer 6 feet in diameter

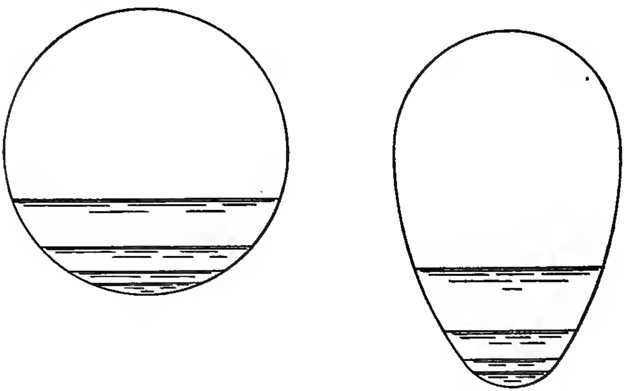

FIG. 44.

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	.35	.5	1.00	2.00	.32	.58	1.20	3.87
Area	.41	1.12	3.11	7.24	.37	.87	2.08	7.04
"C"......	.56	.66	.86	.97	.64	.73	0.82	0.96
Discharge.	.16	.64	3.53	12.00	.18	.63	3.20	11.32
Velocity ..	.39	.56	1.14	1.66	.50	.73	1.48	1.63
Per cent gain in velocity.............					.28	.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.

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 washing 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. 45):

"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 immediately 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-

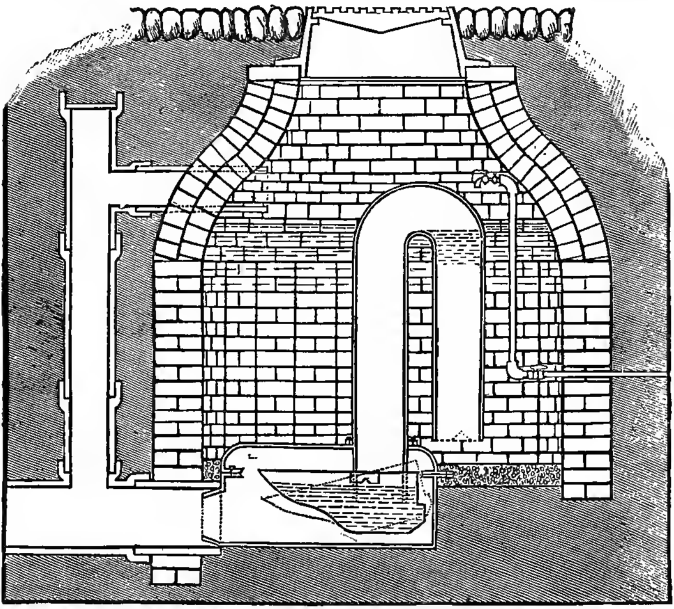

FIG. 45.

Waring Tank." The following description, together with a sectional drawing, is taken from the circular (see Fig. 46):

"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

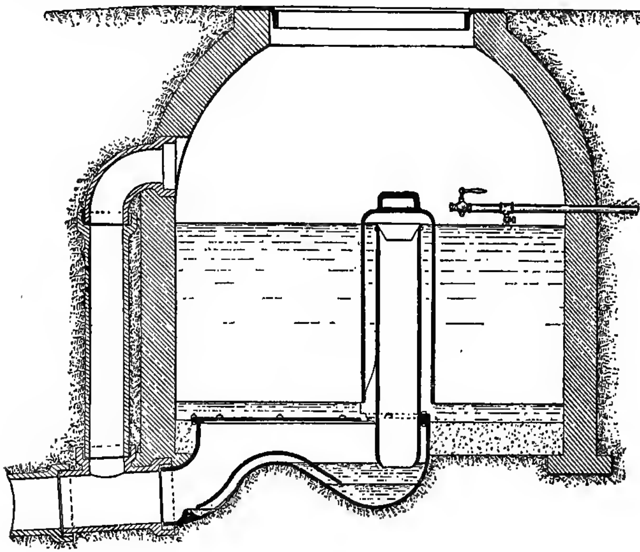

FIG. 46.

dam or obstruction to retard its breaking. 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. 47, 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.

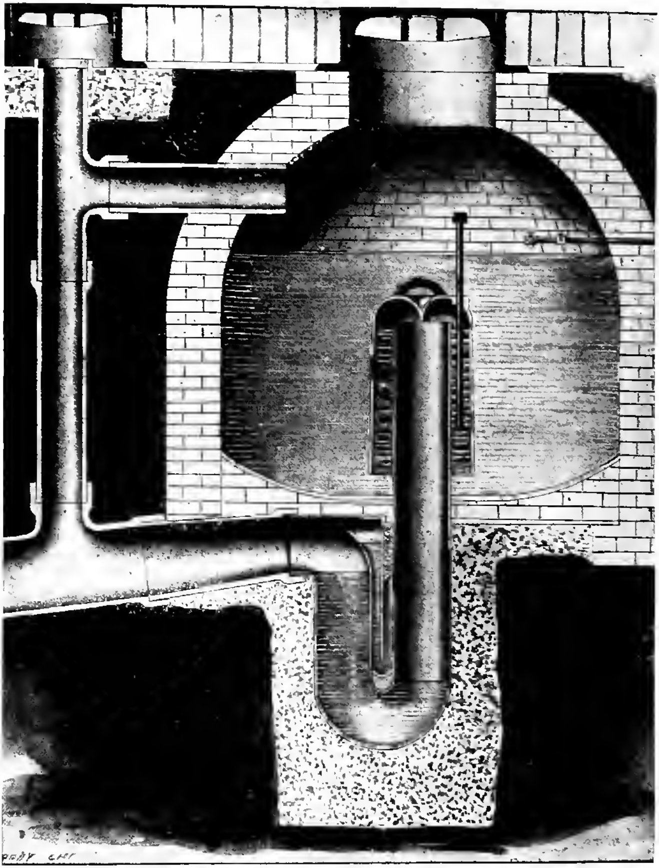

FIG. 47.

"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 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 in'aking limb the flow is checked, and the siphon is vented by the admission of air to it through the vent-pipe."

Table XIX.

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	4½	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

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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

* "Manual of American Water-works," 1897.

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. 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

* Trans. Am. Soc. C. E., vol. XXXIV. page 223.

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 2½ 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 XX and the diagram (Fig. 50) are

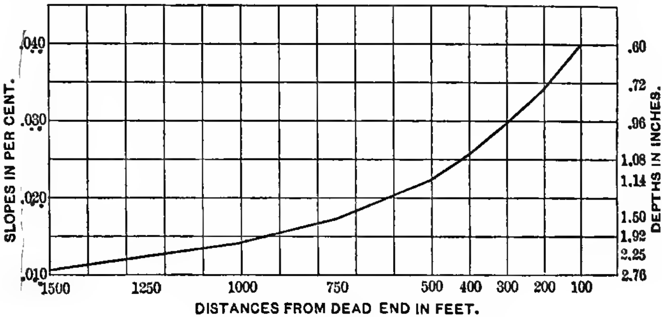


FIG. 50.

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.

The diagram (Fig. 50) 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

TABLE XX.

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

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 the 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 recorded as rapidly as possible (usually every seven seconds) the depth as marked on a thin vertical scale placed in the sewer. Figs. 51 to 54 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 considerable 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.

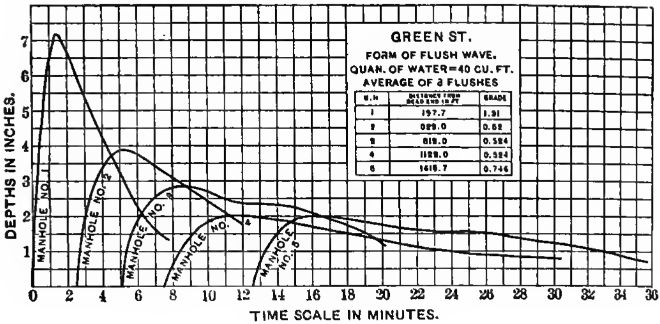


FIG. 51.

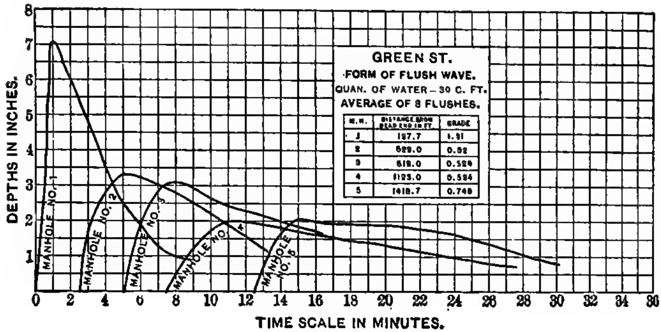


FIG. 52.

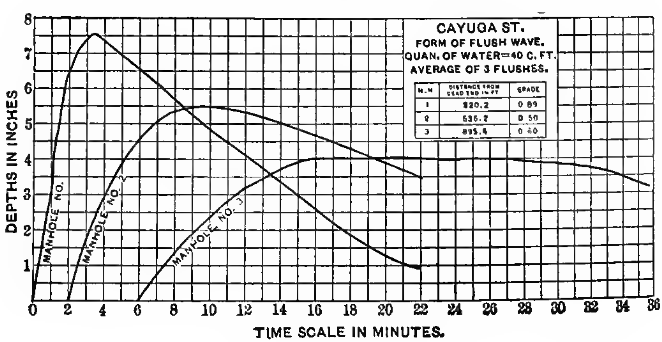


FIG. 53.

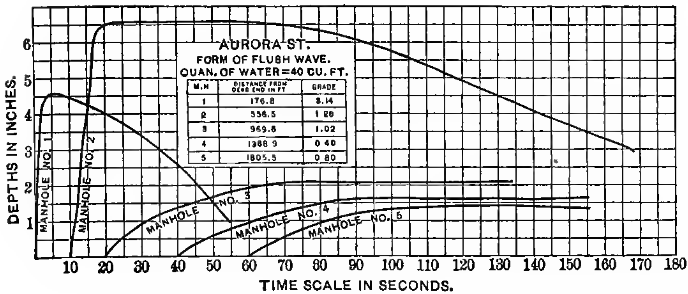


FIG. 54.

The order of procedure was to examine and note he condition of the line, and, after placing the gravel, etc., in the manholes, to make a number 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 XXI–XXIV give the results on the different lines.

TABLE XXI.

GREEN STREET SEWER.

Volume of Flush.	Effects at Manhole No. 1.	Effects at Manhole No. 2.	Effects at Manhole No. 3.	Effects at Manhole No. 4.	No. of Flushes.
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 "	"	"	"	"	2
120 "	"	"	"	"	3

Before commencing the work, the examination of the Green Street pipe showed it to be practically clean, with no ground-water, 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 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 XXII.

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 XXIII.

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 XXIV.

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 effect 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 XXV gives the distances and grades between manholes on the lines used in the experiments.

TABLE XXV.

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 about 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 discharge 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 householders, 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.

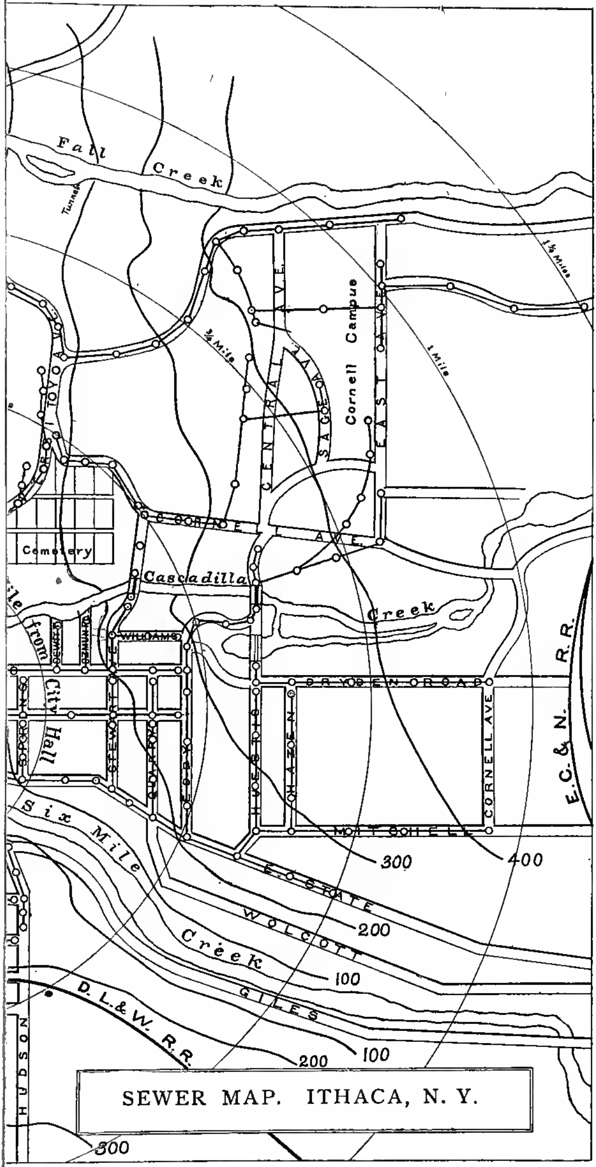
(7) If tanks are used on grades greater than 1 per cent, 15 to 20 cubic feet give as good results as larger amounts, with the same rule as to frequency of discharge.

(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.

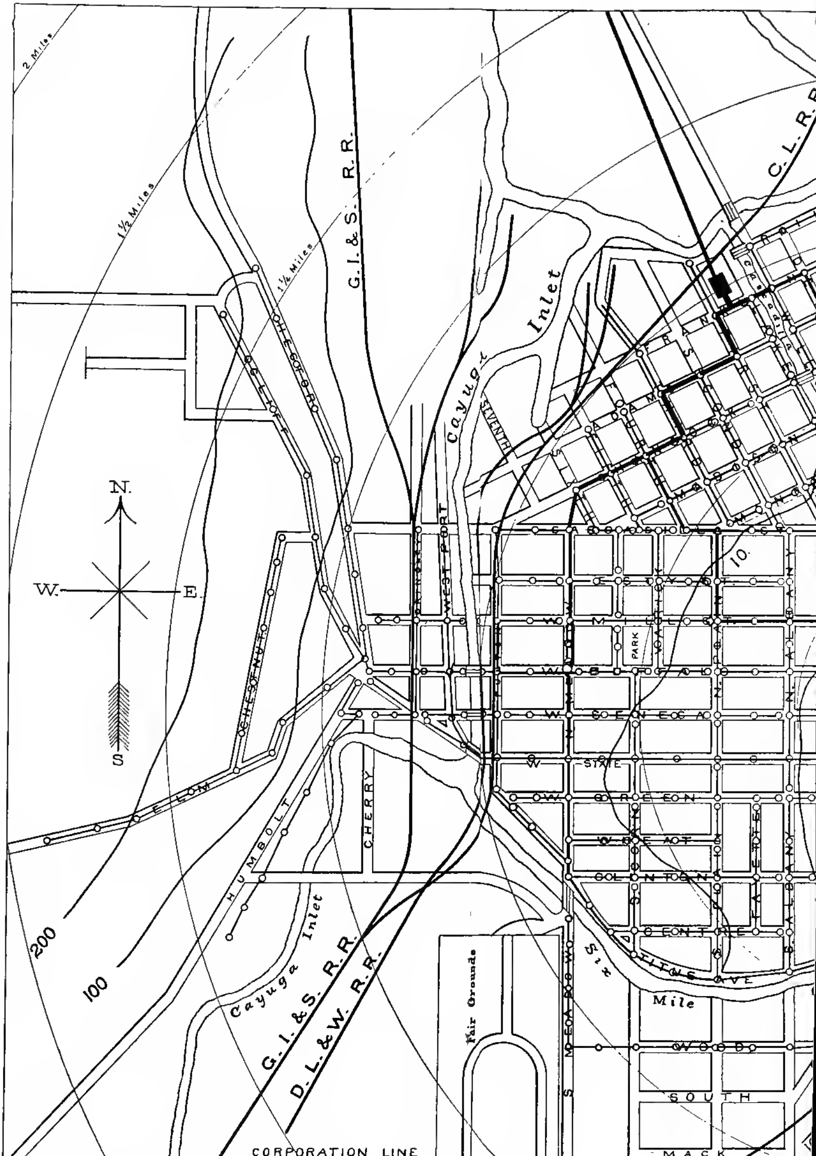
(9) On grades greater than 3 per cent flush-tanks are unnecessary, and their installation is a waste of money.

(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.

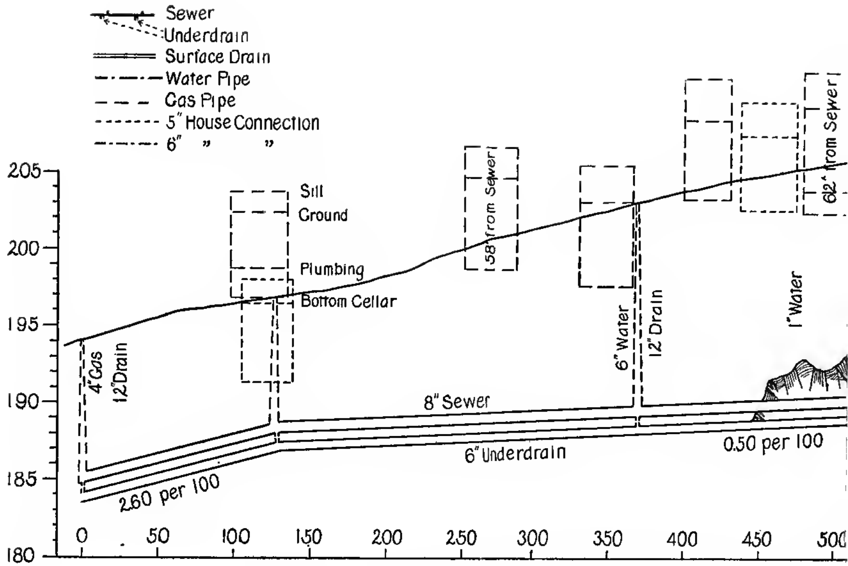
PLATE I.



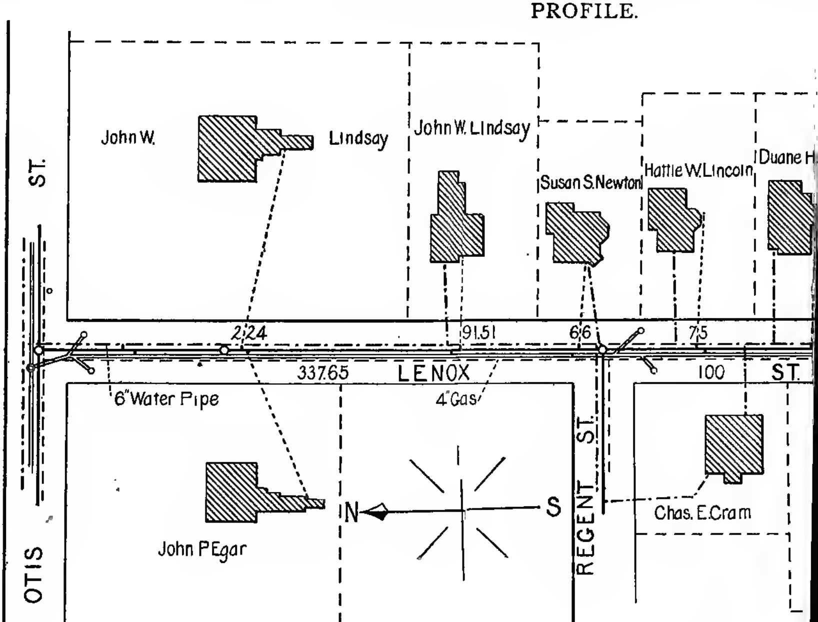
SEWER MAP. ITHACA, N. Y.



2 Miles
1½ Miles
1½ Miles
G. I. & S. R.R.
C. L. R.R.
Cayuga Inlet
SEVENTH
WEST PORT
N.
W.
E.
S
ELM
CHESTNUT
HUMBOLT
CHERRY
PARK
STATE
GREEN
CENTRE
TITUS AVE
200
100
Cayuga Inlet
G. I. & S. R.R.
D. L. & W. R.R.
Fair Grounds
Six
Mile
WOOD
SOUTH
MACK
CORPORATION LINE



PROFILE.



PLAN.

PLATE III.

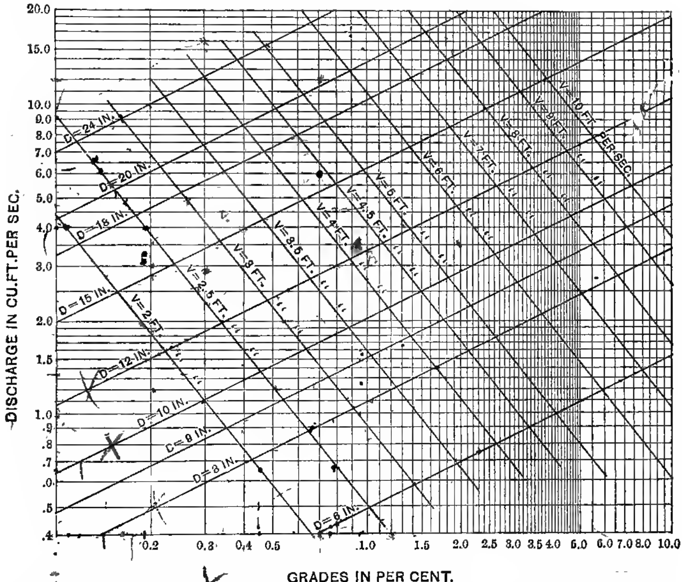


DIAGRAM BASED ON KUTTER'S FORMULA FOR VITRIFIED PIPES.

$n = .013.$

PLATE IV.

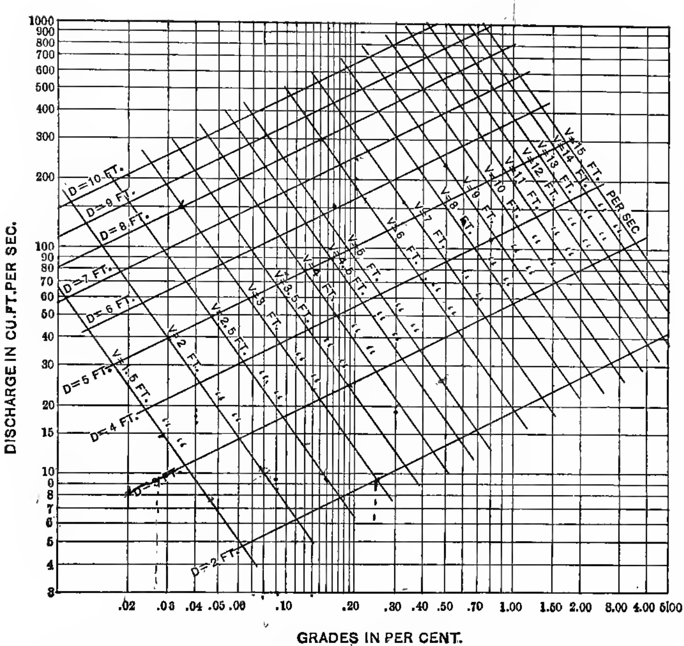
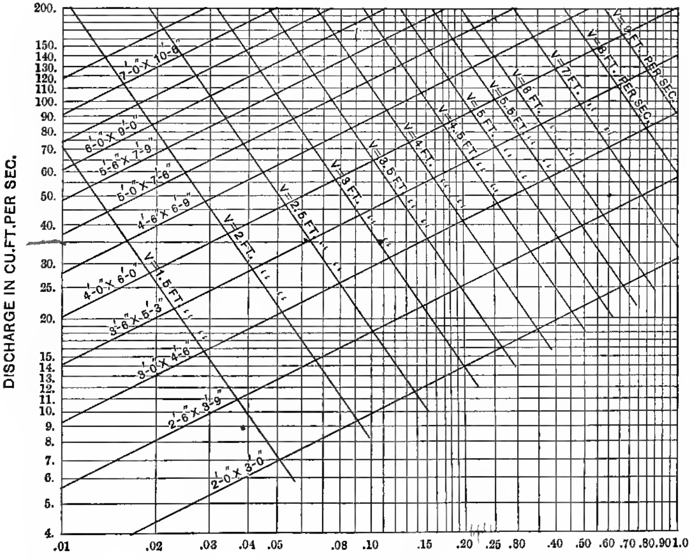


DIAGRAM BASED ON KUTTER'S FORMULA FOR CIRCULAR BRICK SEWERS.

$n = .015.$

PLATE V.



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