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Volume 40 OCTOBER, 1953 Number 4

PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

CONTENTS AND INDEX—VOLUME 40, 1953

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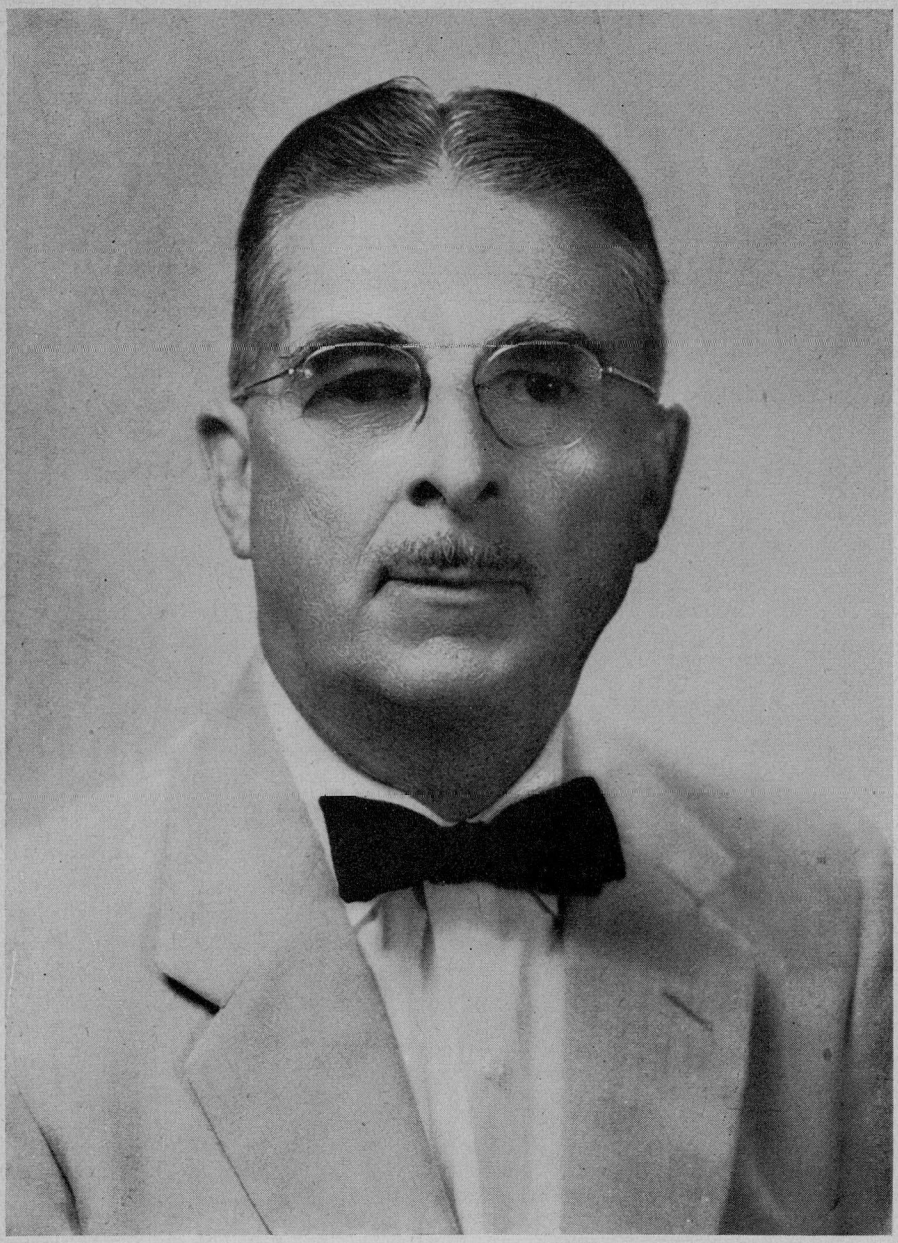

CHESTER J. GINDER
President
Boston Society of Civil Engineers
July 1953 - March 1954

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 40 OCTOBER, 1953 Number 4

THE BOSTON CENTRAL ARTERY*

(Presented at a joint meeting of the Boston Society of Civil Engineers, the Northeastern Section, A.S.C.E., and the Structural Section, B.S.C.E., held on April 25, 1953.)

PART I—GENERAL CONSIDERATIONS

BY J. B. WILBUR,† *Member*

IN FEBRUARY of 1948 a joint board submitted to Governor Bradford the Master Highway Plan for the Boston Metropolitan Area, based on traffic surveys furnished by the Massachusetts Department of Public Works, and for which Charles A. Maguire and Associates, in cooperation with DeLeuw Cather and Company of Chicago and the J. E. Greiner Company of Baltimore were the consulting engineers. The most significant feature of this master plan is a series of expressways, devised to assist in solving the complex traffic problem of Metropolitan Boston.

During the spring of 1949, the General Court authorized a 100 million dollar bond issue for highway construction in Massachusetts, of which 92 million dollars was made available to the Department of Public Works, and 8 million dollars to the Metropolitan District Commission. Of the funds available to the Department of Public Works, not over 37 million dollars was to be spent in Metropolitan Boston, with the determination of projects within this limitation to be determined by Commissioner William F. Callahan, and with all work financed by this first bond issue to be under contract by June 30, 1951.

Commissioner Callahan determined that the major portion of

†Consultant, Fay, Spofford & Thorndike, Boston, Mass.

*Since the presentation of this paper this project has been officially designated as the "John F. Fitzgerald Expressway".

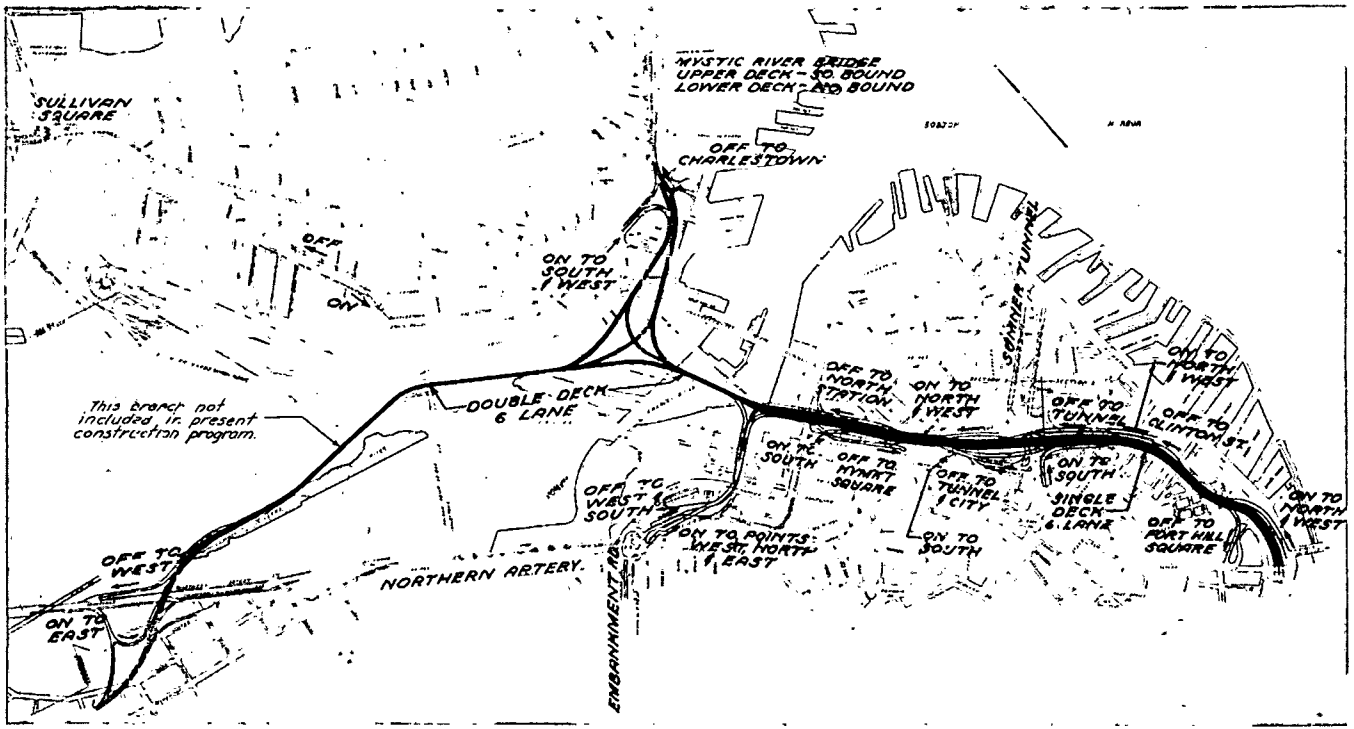

FIG. 1.—SECTION I OF THE BOSTON CENTRAL ARTERY.

this 37 million dollars should be used to begin construction on the Boston Central Artery, and retained the engineering firms of Charles A. Maguire and Associates and Fay, Spofford and Thorndike as consultants, on a joint venture basis, to design the first section of the project.

Historically the Boston Central Artery has come to be known as a cross-city expressway extending from the Charles River, near the North Station, southward across the city of Boston to Massachusetts Avenue near Southampton Street. It will be noted that Section One of the Artery extends from the Charles River as far south as Fulton Street, and includes connections to the Mystic River Bridge in Charlestown, to the Northern Artery in Cambridge,* to the Embankment Road in Boston and, as originally planned, with temporary ramps leading from Fulton Street to Atlantic Avenue near State Street. Later studies indicated the desirability of extending the Artery to the vicinity of Fort Hill Square, and of omitting the temporary ramps to Atlantic Avenue.

For the Department of Public Works, Commissioner Callahan designated Mr. George H. Delano as Project Engineer, who, as the project developed, has been assisted by Mr. John Clarkeson, special consultant to the Department.

It is perhaps needless to point out that a project of this kind involves a consideration of many complex aspects, encompassing a great variety of objectives and viewpoints that must be integrated and balanced in attempting to arrive at a satisfactory solution.

The social aspects of the problem find their source in land use. The Artery, of necessity, displaces both homes and businesses, and, to a minor extent, recreational areas. On the other hand, any aid to the freer flow of traffic in Metropolitan Boston will improve living and working conditions in that area. While it is relatively easy for the engineer to think in terms of the greatest good to the greatest number, this viewpoint is often of limited solace to the man who is about to lose his business, or the family faced with the loss of its home. Minimizing displacements of this kind is therefore, a cardinal objective in a project such as the Boston Central Artery.

The legal aspects of this project are varied. For example, the navigation clearance under the Charles River Bridge had to be ap-

*Since the presentation of this paper, construction of the Northern Artery Connection has been postponed.

proved by the Port of Boston Authority and by the Department of the Army. This was a major point, since the elevation of this structure was the key to grades for an important portion of the project. Again, legal factors in connection with the acquisition of property and access, as well as in connection with property damages, and changes in existing structures and utilities have an important influence on what can and what cannot be done.

The importance of functional considerations can scarcely be overstressed, but since this aspect will be discussed by Mr. Keane, it will not be enlarged upon at this time. Suffice to state that the purpose of the Boston Central Artery is to act as a distributing and collecting system for vehicular traffic. If the structure does not accomplish this purpose effectively, it cannot be considered satisfactory, regardless of whatever other merits it may possess. Many people have been misled by the belief that the primary purpose of the Artery is to permit through traffic to cross the city efficiently. While this is a secondary benefit, it is the efficiency of the Artery as a distributing system that has controlled our basic design.

The economic aspects of this design are also of major importance. The possibility of a central artery for Boston has been discussed for years, and the funds for its construction have been made available only after repeated efforts. It is the desire of the Department of Public Works to build as much of the Artery as is possible with the funds allocated to Section One, and we have been guided by this basic policy. Any desire to build a monumental structure or to expend funds for unnecessary items have been held in check. The responsibility of the Department to the taxpayers of Massachusetts has been foremost in the minds of both the Department and its consultants.

It is interesting to note that the cost of property damages amounts to about one-third of the total cost in a project of this kind, so that in any economic study a balance between construction costs and property damages must be struck. An illustration of this is found in comparative studies, made in the early stages of the project, of the relative merits of a single level versus a double level structure. At first thought it would appear that the double level structure, with traffic moving in one direction on the lower level, and in the opposite direction at an upper level, would prove to be economical. This is

because, while the construction cost for a two level structure is about five per cent higher, one would expect the property damages to be lower, due to the narrower right of way occupied. This is correct in portions where there are few if any ramps, such as in the connecting link over the Boston and Maine tracks leading to the Northern Artery in Cambridge. However, when ramps are frequent, as in downtown Boston, the longer ramp lengths from the upper level occupy so much land that the property damages may become even greater for a two level structure than for the one level system. Hence, from the economic viewpoint, decisions regarding the number of levels to be used must be examined with relation to the factors that actually exist in a given area.

Technical problems affect choice of type of both superstructure and substructure, as well as detailed structural design. A detailed discussion of the structural phases of this project might better be reserved for a meeting of the Structural Section of the Society, so that only one matter of this nature will be brought up at this time—namely, the choice of type of bridge at the Charles River crossing.

This bridge crosses the river on a skew. To avoid higher pound prices, it was decided to increase the span somewhat and thus eliminate skew construction. However, to prevent an undue lengthening of

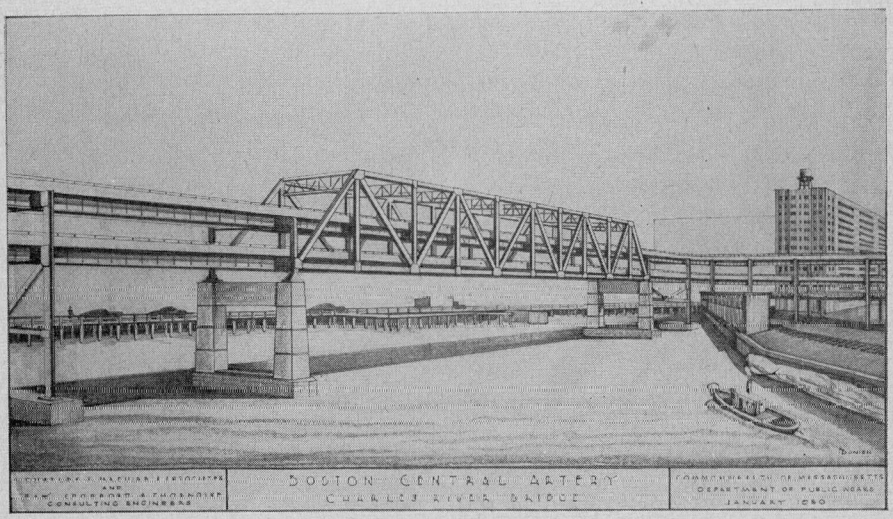

FIG. 2.—CHARLES RIVER BRIDGE.

span to avoid skew, two-level construction was indicated, since this involved a smaller width of structure. The adoption of a two-level bridge also worked out well, since it joined with the two-level connection to the Northern Artery on the Charlestown side, and with a transition from a two-level section to a one-level section of the Artery in the narrow opening between the Industrial Building and the Charles River Warehouse on the Boston side.

Because of the prominent location of this bridge, consideration was given to various structural types that would be pleasing in appearance. However, curves in the adjacent structure, as seen in plan view on both ends of the bridge, made it impossible to use flanking spans as would be required for either a cantilever bridge or a continuous truss. Both the under clearance requirements and the foundation conditions were obstacles to an arch. There remained then, by the process of elimination, only the simple truss as the structure functionally suited to the site. For the span of 375 feet a curved top chord would normally be used, both for economy and appearance. However, in this case, the depth of the truss, because of double deck construction, was already deeper (53 ft.) than was desirable for either economy or appearance. Hence it seemed best to use a parallel chord truss.

Because of the importance of this project, we have from its early stages given serious and sympathetic consideration to the appearance of the Boston Central Artery. To this end we have worked closely with Mr. Harold Knight, who is and has been for some years associated with Fay, Spofford and Thorndike.

Our procedure in developing the design has been that of making numerous preliminary engineering studies to determine the best type of structure by studying possible span lengths and arrangements, foundations, clearances, grades and other limiting features, and from these studies to select the layout considered most feasible, from the engineering viewpoint.

Mr. Knight has then studied this layout from the architectural viewpoint, considering the proportions, possible ornamentation and other architectural features to ensure that the structure, as laid out primarily on the basis of functional and economic considerations, received the best possible architectural treatment.

The most important duty of an organization responsible for the

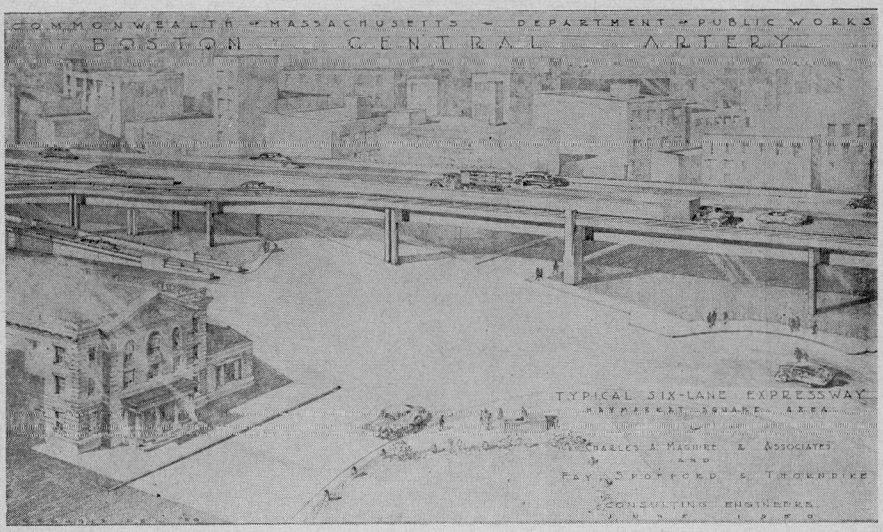

FIG. 3.—THE EXPRESSWAY AT HAYMARKET SQUARE.

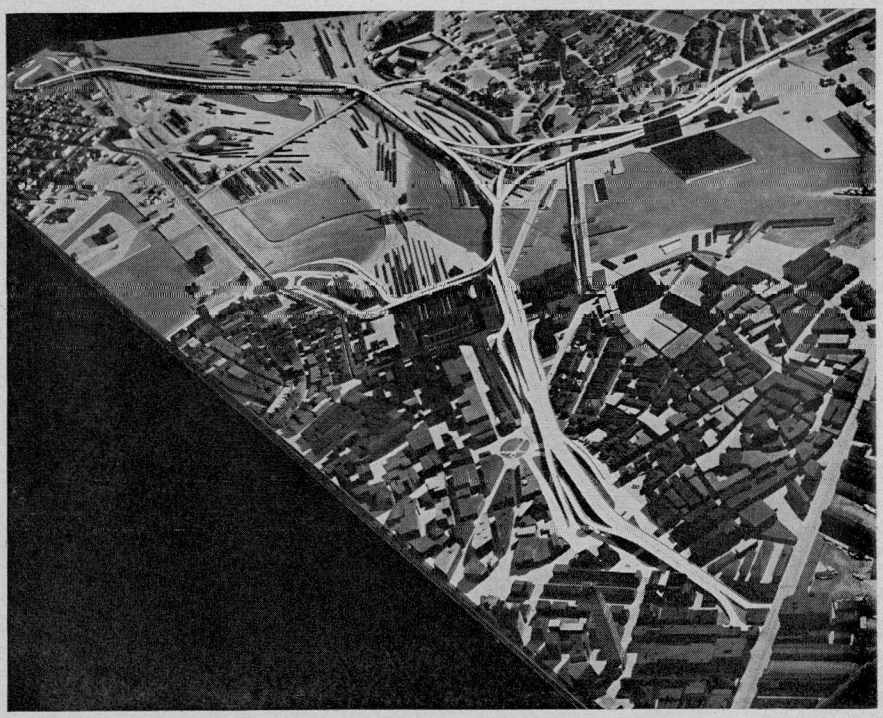

FIG. 4.—SCALE MODEL OF SECTION I OF BOSTON CENTRAL ARTERY.

design of a structure such as the Boston Central Artery is that of arriving at a design which, to the best of the ability of that group, represents the proper balance of the various factors involved. Unfortunately, one cannot hope to arrive at a design that achieves optimum results from all points of view. For example, what is functionally best is not likely to be the most economical—or again, what is aesthetically best may not be consistent with what is technically best or economically most desirable. The design finally reached is usually a compromise between the various objectives, representing to the best of one's ability, the most satisfactory balance that he is able to achieve.

This we are doing. We would be the last to claim that our design is above criticism. We can only say that it is the result of a prolonged and intensive study carried out by a group who are working objectively in an effort to integrate the many factors involved into the best possible over-all design.

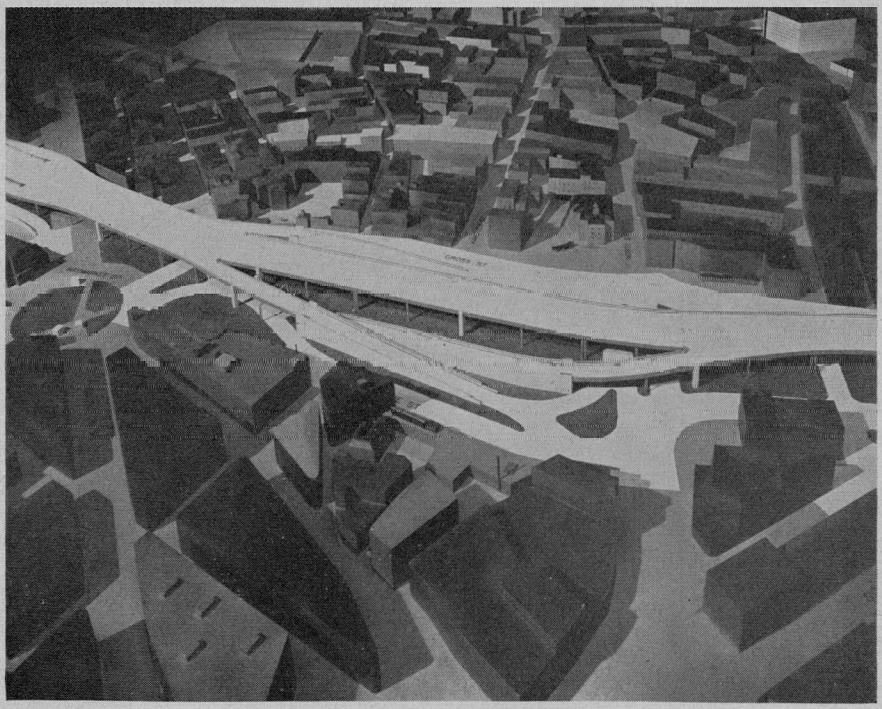

Fig. 5.—Portion of Scale Model Showing Arrangement of Ramps Near Entrance to Sumner Tunnel.

Part II—Location Planning and Highway Design Features

By E. C. Keane,* *Member*

At the outset I should like to discuss briefly the reasons why this type of urban expressway work is being done, not only in Boston, but in most of the major cities in the United States.

Around 1930 the rural highway system of the country had reached the point where there were reasonably good hard-surfaced roads between all urban centers. The highways were not good by present-day standards, but one could drive from city to city in fair comfort. Within the cities, however, very little progress was being made. There were, of course, numerous minor improvements in cities and a few spectacular projects like the West Side Highway in New York, but on the whole we struggled through the thirties with city streets constructed in the horse-and-buggy era. In Boston at that time there were only about two miles of State highway.

Meanwhile, motor vehicle registrations were climbing at an amazing rate. In 1921 there were about 10.5 million motor vehicles, and automobile salesmen were even then asking each other if the saturation point was about to be reached. Looking back, that question now seems amusing, for eight years later, in 1929, the registration figures had reached 26.5 million, or more than 2.5 times the 1921 figure. The increase was slight through the depression and war years, and by 1945 we still had only 30.6 million vehicles. The came the war's end, and the lid was off. By 1950 the total registration reached 50 million, and the end was not in sight.

The average travel per vehicle has recently been running close to 10,000 miles per year. That means the total annual travel, country-wide, is now about 500 billion vehicle miles.

Around 1940 it came to be realized that the tremendous mileage consists very largely of trips to town and back—to work, to shop, to carry on business, to see the doctor, to enjoy a meal out, and so on, and that city-to-city mileage is really minor as compared with the day in, day out mileage of trips to town and back.

Many people ask, "Why do we permit all these people to drive into the cities? Why do we not make them use public transportation?" The answer is that there is no law against driving; people simply

*Engineer, Fay, Spofford & Thorndike, Boston, Mass.

prefer to drive, and they do so. Transit system executives can argue that mass transportation is cheaper; that it is (or can be made) faster than private transportation; that it is less wasteful of our natural resources (oil, steel, rubber, etc.); that a dollar spent on rapid transit facilities is a much better investment for the public than a dollar spent on highways, and so forth. But the people have not been convinced, and they have deserted the transit systems in droves. In only three years, from 1945 to 1948, the number of transit riders in urban centers declined 20 per cent, while in the same years the number of motor vehicles increased 33 per cent.

Police powers of city and state governments could undoubtedly be extended to exclude non-essential vehicles from downtown areas, but it will actually be done only if it is the will of the people. There seems to be little chance that the people will approve any proposal to hamper their freedom of action in this way, at least in our generation. It is possible that restrictions on intown driving may come about at some future date; however, that date now seems a long way off. If this does come about some day, there need be no fear that the highways now being built will be wasted. They will be needed in any case to facilitate the movement of essential vehicles such as trucks, busses, and the cars of doctors and others who perform vital services.

City governments have an important stake in the new express highways. It is well recognized that the values of business properties (and hence the taxes that can be collected on them) vary directly with their accessibility. It is worth noting that the assessed valuations in Boston's Ward 3, which includes the central business district, decreased from 693 million dollars in 1926 to 450 million in 1950, a loss of 243 million dollars or about 35 per cent. Can anyone doubt that this great loss was due in large measure to the failure to meet the growing demands for highways to the downtown area and for parking facilities in it?

Despite the losses, there is still a prosperous central business district in Boston. This district still offers the widest selection of merchandise, the highest caliber of professional services, and the most desirable locations for carrying on many types of business. Thousands more will visit the area daily, with a salutary effect on business and on property values, when highways and parking areas are constructed to restore the accessibility that existed before people turned in such

large numbers to the use of automobiles. It is true that the city will suffer an initial loss in property tax revenue due to the right-of-way takings, but this loss is as nothing compared to the losses which have been suffered in the past, due largely to traffic strangulation, and which will continue unless the trend is reversed.

A question frequently asked is, "How can we possibly afford these huge highway expenditures?" The answer is, through the operating savings that can be made on modern limited-access highways. The potential savings for the millions of vehicles per year using the new expressways are very large. Applying the data on vehicle costs as of 1948, presented by Lawrence Lawton in his excellent paper in "Traffic-Quarterly" for January 1950, to a 6-lane downtown expressway operating at capacity, we find that the savings to vehicle owners can amount to something like five million dollars annually per mile of expressway. In the computation of savings, expressway speeds are assumed at 40 miles per hour and speeds on congested city streets at 10 miles per hour. The net savings to the public, after allowing for interest, sinking fund and maintenance charges, when capitalized in the usual way, would justify an initial expenditure, per mile of expressway, much larger than the mile of expressway would cost even under the most difficult conditions. Therefore, I believe that most, if not all of the urban expressway projects that are underway all over the country could be shown to be economically justified on the basis of the net savings to the public. At any rate the public now seems to be in the mood to authorize very large appropriations, and it appears probable that urban expressways will continue to bulk large in the total highway program of America.

The collection of tolls on urban expressways is not practical, due to the many exit and entrance ramps that have to be provided, and the projects are usually financed through bond issues which are amortized by an increase in the gasoline tax. This is the method used in Massachusetts.

Now to the particulars of the Boston Central Artery: As noted by Doctor Wilbur, a Master Highway Plan for the Boston Metropolitan Area was recommended in February, 1948. The basic plan and the studies leading up to it were described in a paper by Joseph K. Knoerle, published in the Journal of the Boston Society of Civil Engineers for October 1948, and I shall try not to go over the same

ground. Suffice it to say that the Boston Central Artery is a portion of the belt expressway recommended in the master plan. Figure 1 shows the location of the portion that has been designed to date. Figure 6 shows how this portion fits into the belt expressway plan, and the relation of the Central Artery to other major highways in

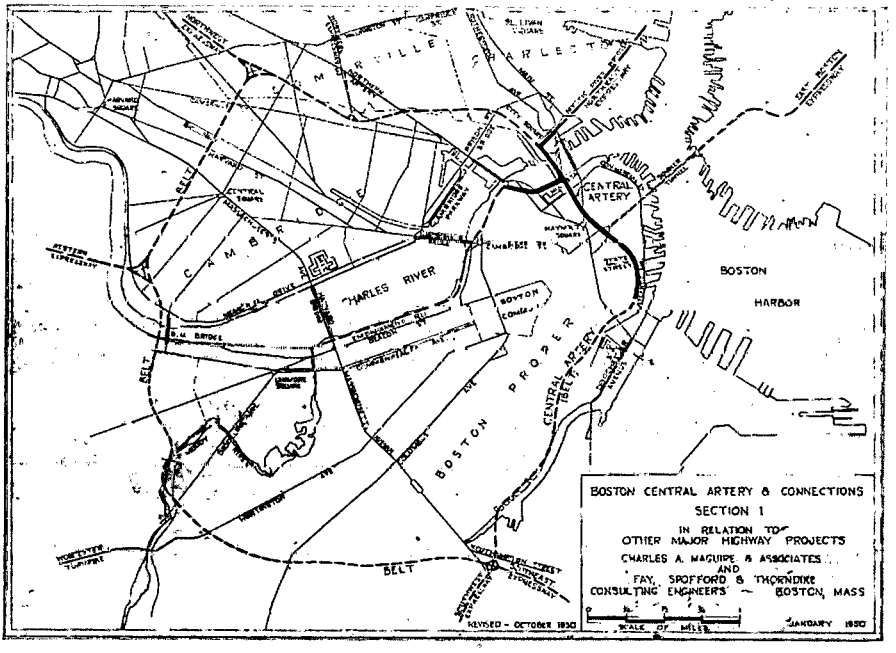

FIG. 6.—BOSTON CENTRAL ARTERY IN RELATION TO OTHER MAJOR HIGHWAY PROJECTS.

Boston. The Mystic River Bridge will, at first, be the principal feeder to the Artery from the north, but ultimately it will be only one of the eight major radials that will feed into the belt. The new Embankment Road, along the south side of the Charles River, is essentially a diameter of the belt, and it will be relieved of some of its traffic load when the whole belt becomes available.

It should be emphasized that the belt (including the Central Artery) is a distributing highway, not a means of by-passing Boston. According to the origin and destination study that preceded the 1948 Master Plan, only about 15 per cent of the traffic, that now touches downtown Boston, wishes to pass through without stopping. The

other 85 per cent of the vehicles wish to stop at some point within the downtown area. It is fundamental that we must locate the belt as closely as possible to the areas where origins and destinations are heavy; otherwise it will not fulfill its function. We must also provide frequent exits and entrances in the form of ramps, and distribute the ramps along the expressway somewhat as the vehicle desires are spread in the zones served. Since the desires are heaviest in the downtown area, the ramps must be spaced closely there. In the stretch from Causeway Street to Fort Hill Square the design includes twelve ramps. Since it takes four ramps to make one set (off from north, on to north, off from south, and on to south) the twelve ramps mentioned will be the equivalent of three sets. In addition to the three sets of ramps to downtown streets, there is a direct connection across the North Station tracks to the northerly end of Embankment Road which is being constructed by the Metropolitan District Commission; also two ramp connections to streets in Charlestown, as well as direct connections to the Mystic River Bridge. Design of the portion of the belt from the interchange near City Square, Charlestown, westerly to the Northern Artery, is being held in abeyance, since it appears probable that available funds will be used for an extension of the Central Artery toward the south. However, when the westerly branch toward the Northern Artery is constructed at a future date, provisions will be made for cutting in ramps from Rutherford Avenue.

During the preliminary design period there was considerable pressure from some of the North End businessmen to relocate the downtown portion of the Central Artery to the northerly edge of the Boston peninsula, along Commercial Street and Atlantic Avenue. The change was not made, and the fundamental reason for the decision was that an artery in that location would not provide ramp service at points where the ramps are needed for proper distribution and collection of traffic, for example at key points like Haymarket Square and Sumner Tunnel. Also, the construction cost would be greater by some eight million dollars, property damages would be at least as great, and more families and businesses would be displaced.

The six lanes of the main expressway should have a capacity of about 90,000 vehicles per day. As of 1955, with only Section I constructed, it is expected that the daily traffic at the most travelled point will be over 70,000 vehicles per day.

Following are the geometric design standards for the elevated highway:

DESIGN STANDARDS FOR ELEVATED HIGHWAY
BOSTON CENTRAL ARTERY AND CONNECTIONS — SECTION I

General:	Conform with "Interstate" standards of the American Association of State Highway Officials.
Design Speed:	Expressways, 35 m.p.h. Ramps, 7/10 of expressway speed or 25 m.p.h.
Minimum Radii:	Expressways — 400′ Ramps, at expressway connection — 200′ Ramps, remainder — 80′
Superelevation:	Per AASHO formula, but not in excess of ½″ per ft.
Pavement Widths:	12′ per lane, plus 2′ shoulder where adjacent to vertical curb; i.e. 40′ for 3-lane roadway and 28′ for 2-lane roadway.
Vertical Clearance:	14 feet minimum.
Lateral Clearance:	Right side, 6′ from edge of traffic lane. Left side, 4′6″ from edge of traffic lane.
Gradients:	Expressways, 3.0% preferred, 4.5% maximum. Ramps, 5.0% preferred, 6.0% maximum.

The rather high standards shown above, especially as to lane widths and lateral clearances, are believed justified in order to save lives and property. The economic loss on account of traffic accidents in the United States is now about three billion dollars a year. There are, of course, no traffic lights, no stop signs, and no intersections at grade on the elevated structures of the Central Artery. Pedestrians will not be allowed on the structures.

The "design speed" referred to above is not the maximum permissible operating speed, but is a basis for determining minimum radii and for correlation of superelevation with alignment. Actual operating speeds will no doubt be somewhat higher.

The number of traffic lanes used on the various parts of the structure is in accordance with the findings of the Master Plan. All of the roadways are for mixed traffic, not for passenger vehicles ex-

clusively. The truck traffic is expected to comprise about 25 per cent of the total. It is believed that the economic benefit to the trucks, even though they number only 25 per cent of the traffic, will exceed the economic benefit to the 75 per cent that are passenger cars.

Although it is not the purpose of the paper to cover the structural design of the Central Artery, it may be appropriate to touch on the manner in which field conditions affected the fitting of the structure to the site. The type of viaduct decided upon was a steel structure with reinforced concrete deck and rigid-frame bents. From the point of view of economy in the structure, and also from the point of view of appearance, it would have been desirable to use bents with a uniform geometric outline, uniformly spaced. In most parts of the project, however, site conditions made it impossible to employ this uniform structural arrangement. Figure 7 shows the location of one of the roadways of the Charlestown interchange near City Square. The plan shows existing physical features, including underground conduits insofar as we have been able to discover them, and also the footing

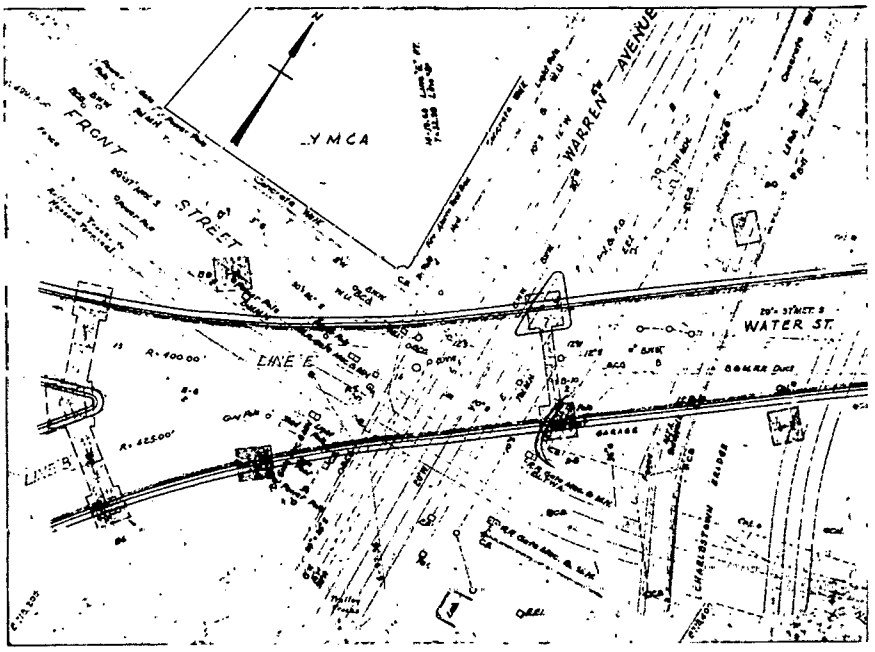

FIG. 7.—PORTION OF SURVEY PLAN, CHARLESTOWN.

locations for the expressway columns. In addition to the usual problem of obtaining an acceptable alignment for the elevated roadway, we were obliged in this particular area to give considerations to the following: the YMCA building; the Charlestown Bridge; the Boston & Maine Railroad tracks to Hoosac Terminal and the Boston Naval Shipyard; MTA trolley tracks on Warren Avenue and on the Charlestown Bridge; the MTA rapid transit elevated structure on Charlestown Bridge; the preservation of traffic lanes on city streets; and 13 different kinds of underground conduits, some of which were represented by three or four separate pipes. To add to the difficulty it was necessary to reserve space for a proposed rapid transit tunnel from Charlestown to Boston, which will be located in Warren Avenue just east of the YMCA. It will be noted that the longitudinal bent spacing and the transverse spacing of expressway columns are both non-uniform, as necessary to meet all the conditions mentioned.

Column location problems of this sort were encountered in every part of the project, and in order to solve them it was necessary to adopt an adaptable structural system, rather than the somewhat simpler system that would have been possible under less difficult conditions.

Highway lighting on the structure will be by means of mercury vapor lamps designed to furnish an average illumination of 1.4 foot-candles on the roadway surface. The luminaries will be attached to steel standards of special design, set in the line of the guard rail.

In downtown Boston where steam is available from the public utility company, the lower portions of ramps, which are on earth fill between retaining walls, will be radiant heated to melt snow as it falls and thus provide safe braking areas on down ramps and sure acceleration on up ramps. Steam purchased from the public utility will be passed through heat exchangers to warm fluid which will be pumped through pipe grids embedded in the pavement. The mechanical design is conventional, but the installation is noteworthy because of its size. This is believed to be the largest roadway heating installation made anywhere up to this date.

Time does not permit a discussion of the many other design features that might be of interest, or to tell of the experiences, most of them quite agreeable, in dealing with the railroad companies, owners of utilities, and the many public and private agencies concerned with

this project. However, it would perhaps be well to say a few words on the subject of terminal facilities, that is, parking spaces, for the vehicles which wil be enabled to reach the downtown area in greater numbers because of the expressway construction.

The legislation authorizing the highway work did not cover the provision of the parking facilities, without which the full benefit of the expressway will not be realized. Other legislation does permit the City of Boston to make a start on an off-street parking program, and in fact one multi-story parking garage is already under construction and others are contemplated. The Central Artery will undoubtedly increase the demand for such facilities, and there is every prospect that the parking program will be enlarged, either through additional legislation or through the entrance of private capital into the picture. There are many areas near the expressway that are now covered by over-age, uneconomic buildings, and if, as we believe, there is to be a redevelopment of these areas, it is reasonable to suppose that a goodly percentage of the areas will be devoted to parking. The problem does not appear to be insuperable.

In closing I should like to say that it has been a pleasure to work with the many capable career men in the Massachusetts Department of Public Works and in the other agencies concerned. With their cooperation, we are producing a design which, we believe, will stand the test of many years.

SEWER CAPACITY DESIGN PRACTICE

WILLIAM E. STANLEY,* *Member*
and
WARREN J. KAUFMAN.**

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, held on December 7, 1949.)

INTRODUCTION

THE design of sewers includes three major phases: (1) the capacity design, i.e. the determination of the flow rates of sewage for which capacity in sewers should be provided, (2) the hydraulic design, i.e. the application of hydraulic science to the determination of the proper size, slope and other characteristics of sewer pipes and structures to provide the capacity to handle the sewage flow rates computed in the first phase, and (3) structural design, i.e. providing proper materials, thickness, and structural strength to sewer conduits and structures.

Camp[1] has recently presented a masterful discussion of the hydraulic design of sewers, including some recent developments in fluid mechanics which influence sewer design. He restricted his paper to hydraulic design.

This present discussion will be restricted to methods of estimating the sanitary and storm sewage flows for which hydraulic capacity should be provided in sewers.

Methods used to date to estimate sewage flow rates for sewer capacity design have been empirical with wide variations in the values of the several basic factors.

A study has been made of engineering practices in the estimating of sewage flows for the capacity design of sanitary, storm, and combined sewers, based on data received by correspondence, from Engineers of Sewer Design in a number of cities in the United States and Canada.

The junior author (as a basis for an S.M. degree thesis at

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[1]"Design of Sewers to Facilitate Flow"—S.W.J. (January 1946, p. 2).

M.I.T.) has given particular attention to the relatively new "hydrograph" method of computing storm sewer capacity requirements. There are interesting possibilities that this method may come into greater use.

Some knowledge of the basic sewer capacity quantities which have been used by experienced and competent sewer designers in a number of representative communities should provide a valuable guide to judgment for any engineer preparing the design of a specific sewer system, if some common base for measurement of sewer capacity were available. Sewer capacity in terms of gallons per day or cubic feet per second per tributary acre appears suitable as such a comparative capacity index. One of the objectives of this paper is to translate sewer design factors used by the several sewer designers into such capacity indices.

It would be an unwise procedure for any sewer designer to take the per acre quantities used in one community and apply them blindly to the design of sewers in another community. Sewer capacity design properly executed is an art involving mature and experienced engineering judgment applied to many factors pertinent to any specific sewer area.

The sewer designer's task involves forecasts of the magnitude of three intangible factors which greatly influence the proper capacity design of sewers: (1) he must anticipate the territorial development for upwards of 40 years in the future and be prepared to reasonably evaluate zoning restrictions which quite likely will be modified during the anticipated capacity life of his sewers; (2) he must foresee future sewer operational practices which affect probable sewage flows; and (3) he must anticipate the probable future desires of sewer users as to adequacy of service. None of these can be ignored—none of them are subject to exact determination.

PART I—CAPACITY DESIGN FOR SANITARY SEWERS

General Statement—Sanitary Sewers

Illustrative procedures for capacity design of sanitary sewers which have been used in 20 or more municipalities are summarized in Appendix I. Certain basic considerations are outlined in the following paragraphs.

Sanitary sewers (sometimes designated *separate* sewers) presumably should receive no storm water. Actually, the extent of storm water entrance during rains and the uncertainty as to underground infiltration are major factors which determine the adequacy of the capacity of many sanitary sewers. Also roof drains may or may not be legally permitted to be connected into sanitary sewers.

Basic Factors

The principal basic factors in estimating sewage flow rates for capacity design of sanitary sewers include:

a) *Population Density*—per acre—has been used in general to represent the future extent of development anticipated by the sewer designer.

b) *Per Capita Domestic Sewage Contributions*—those fractions of the community water supply reaching the sewers as soiled water—future yearly average rates and maximum or peak rates of sewage flow have been generally estimated with some relation to water consumption rates.

c) *Commercial Area Contributions*—sewage flows from stores, hotels, and business establishments—sometimes these have been included in the per capita sewage flows for smaller communities. In larger communities per acre allowances have been considered a more reasonable basis of determining sewage flow rates for which sewer capacity should be provided.

d) *Industrial Area Contributions*—sewage flows from manufacturing establishments have been provided for by (1) an allowance per acre of area for industries in general, and (2) a special rate of sewage flow from certain known industries with large process wastes of a character permissible to be discharged into sanitary sewers.

e) *Infiltration*—"the quantity of ground water which leaks into a sanitary sewer through defective joints"[2]—an important factor not easy to evaluate often the cause of overloaded sanitary sewers.

f) *Storm and Surface Water*—more or less storm water—from roof leaders, court yard drains, basement drains, and some street inlets—finds its way into sanitary sewers—sometimes surface water streams drain

[2]Glossary—Water and Sewage Control Engineering (March 1949, p. 122).

into sanitary sewers. Some designers have included storm and surface water flows under the designation of "infiltration." Some cities forbid roof drainage into sewers, other city ordinances require or permit it.

Sewer capacity practices in various cities differ considerably in the attention given to these several factors. Some designers have considered the importance and uncertainties of infiltration (including storm water) so great compared to the other factors that little attention has been given to dividing computed sewage flows into the various classifications. Several city sewer designers have combined all fractions of sewage flow into a coefficient of sewer capacity—usually in terms of cfs per acre, or cfs per 100 acres, the value of the coefficient differing for various classifications of tributary area.

Historical Guidance Aspects

Ancient sewers were built for storm and surface water only; it was not until the middle of the 19th century that domestic sewage could be admitted legally into sewers.

Separate sanitary sewers, first used at Croydon, England (about 1850) and in the United States at Pullman, Illinois, and Memphis, Tenn., were quite generally overloaded due to insufficient capacity. Waring at Memphis, and in many other cities, installed many 4-inch and 6-inch sewers resulting in a great deal of stoppage troubles. His unfavorable aggressive promotional activities and the failure of many of his small sewers due to stoppages (1880-1890) together with a report by Dr. Hering (1886), based on his investigation of European sewerage practices, were important factors which directly influenced the selection of combined sewers by many cities toward the end of the 19th century.

The need of sewage treatment, increasingly appreciated during the last twenty or more years, is swinging the use of separate sanitary sewers back into favor. Greater attention to the capacity design of sanitary sewers can be expected in the future.

Population Densities

Population densities represent a measure of the anticipated future development of an area for which sanitary sewers are to be provided. Generally some definite future density has been anticipated

by the sewer designer. Table 1 includes representative data for a number of cities.

TABLE 1

Population Densities for Sanitary Sewer Capacity Design

City	Year of Data	Population per Acre	Remarks
Baltimore	1949	100 to 135	Closely built small houses
		20 to 50	Suburban cottages
		50 to 70	Suburban groups
Washington Suburban District	1946	5	Outlying estates
		18	General suburban areas
		62	Apartment areas
Boston	1949	100	Boston City, largely combined sewers
Buffalo	1946	25 to 75	City 95 % combined sewers
Cranston	1943	17 to 60	1970 conditions
Dallas	1949	10	Average in residential areas
Des Moines	1949	10 to 30	Zoning Ordinance
Los Angeles	1951	19	R-1, One family zone
		36	R-2, Two family zone
		76	R-3, Multiple dwelling, boarding houses, etc.
		102	R-4, Multiple dwelling—apartments
		177	R-5, Multiple dwelling—clubs, hotels
Madison, Wis.	1937	30	Light residential
		50	Apartment districts
Milwaukee	1945	25	Outlying areas
Rochester, N. Y.	1946	23.65	Avg. 4.3 people per lot; 5.5 lots per acre
Springfield, Mass.	1949	75 to 30	75 for 10 A. log line decrease to 30 for 1000 A.

PER CAPITA SEWAGE FLOWS

Some sewer designers have computed domestic sewage flows on the basis of average rates and then have multiplied by a factor to obtain a maximum rate for sewer capacity design. In other cases a maximum per capita rate has been used. Representative data from a number of cities (Table 2) show quite a variation in the per capita quantities anticipated by sanitary sewer designers.

TABLE 2

Per Capita Sewage Flows for Sanitary Sewer Capacity Design

City	Year of Data	Per Capita Sewage Flows gallons per day: Average Rate	Sewer Design Basis	Remarks
Baltimore	1949	135	2 to 4 times average*	
Washington Sub. Dist.	1946	100	2 to 3.3 times average*	
Boston	1949	75 150	150 300	Sewers flowing half full [Sewers flowing full]
Cleveland	1946	100		
Cranston, R.I.‡	1943	119	167	
Dallas	1949	150	575	Including storm water & infiltration
Des Moines	1949	100	200	
Los Angeles	1949	—	—	Included in a per acre capacity coefficient
Madison, Wis.	1937	—	300	Max. hourly rate
Milwaukee	1945	125		All in 12 hours, i.e. 250 g.p.c.d. rate
Painesville, O.**	1947	125	600	Includes infiltration & roof water
Rochester, N.Y.	1946	—	250	N.Y. State Bd. St'd.
Springfield, Mass.	1949	—	200	150 g.p.c.d. was used on a special project
Toledo	1946	—	160	

*Design flow determined from estimated average flow by use of charts (Figures I & II).

‡Ralph W. Horne—Jour., B.S.C.E., *XXX*, p. 69, April 1943.

**Courtesy Havens and Emerson, Consulting Engineers.

Some illustrations of the use of factors to compute maximum per capita flows, reasonable for capacity design of sewers, from estimated average per capita sewage flows are as follows:

a) Davis' — "Handbook of Applied Hydraulics" (2nd Ed., pp. 1025).

$$(1)\quad M = \frac{5}{P^{1/5}} \qquad \text{or} \qquad (2)\quad M + \frac{14}{4 + P^{1/2}}$$

where P = population in 1000's

M = ratio of max. sewage flow, for sewer capacity, to estimated average sewage flow rate

b) Jones & Henry, Consulting Engrs.

(letter Nov. 3, 1949, by T. B. Henry)

[P and M same as for equations (1) & (2)]

$$(3) \quad M = 1 + \frac{22}{4 + P^{1/2}}$$

Baltimore and Washington Suburban Sanitary District engineers have used a chart originally devised by Ezra B. Whitman, revised and published by Robert B. Morse in 1914. The Baltimore Curve (Figure I) gives ratio factors for sewer capacity design rates of flow related to computed average cfs rates of flow, while the Washington Suburban District Chart (Figure II) gives sewer capacity design rates of flow in MGD from the chart on the basis of estimated average sewage flow rates in MGD. The Borough of Richmond, New York City, has used a curve similar to the Baltimore Curve, while the Borough of Queens has used a chart similar to Figure II, excepting the sewage flow rates are computed in cfs.

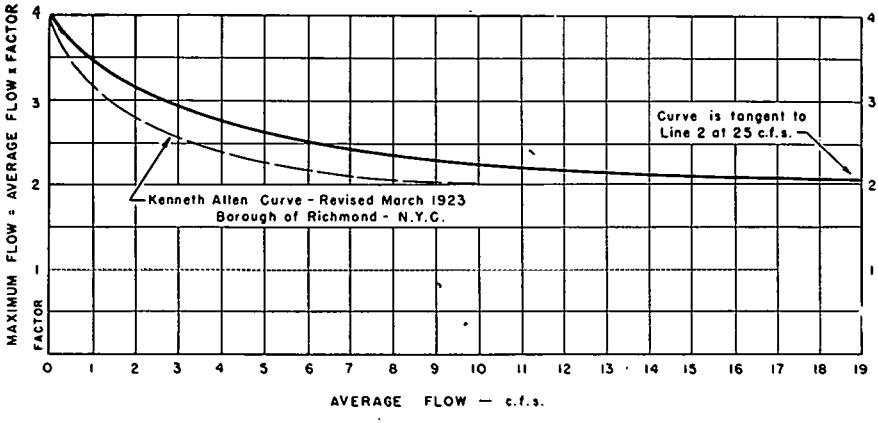


FIG. I · BALTIMORE CURVE SHOWING RELATION BETWEEN COMPUTED AVERAGE AND MAXIMUM RATES OF SEWAGE FLOW

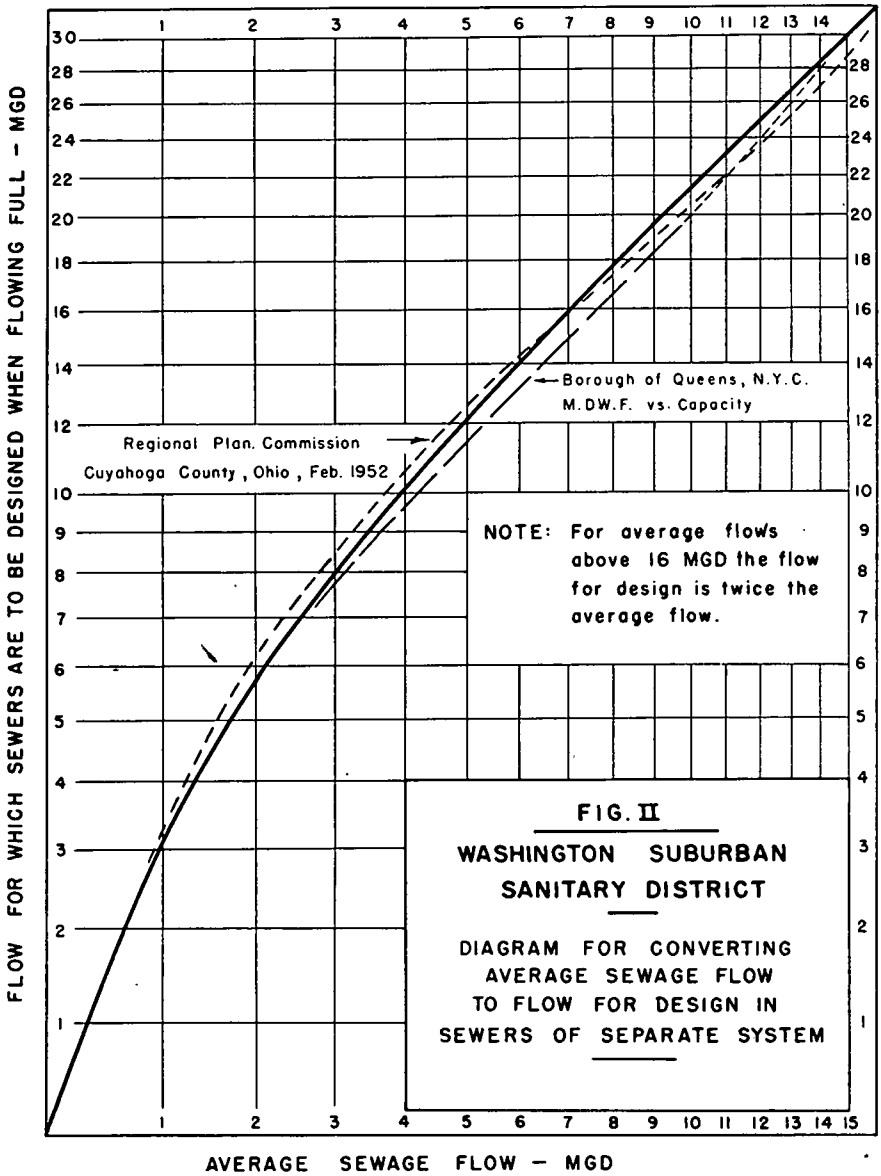
More recently (February 1952) the Regional Planning Commission, Cuyahoga County, Ohio, has issued an "Engineering Design Manual for Sanitary Sewerage and Drainage Facilities" in which the ratio between average and peak flows, in sanitary sewers, are computed by the formula:

$$r = 1.80 + (1.9 \times 0.98^{n-1})$$

where r = ratio peak/average flow

n = 10 x average 24-hour flow in mgd

Comparative results of this formula are shown graphically on Figure II.



Commercial Area Contributions

Sewer capacities for sewage flows from commercial establishments—stores, hotels, offices, excepting for relatively small areas in otherwise residential districts, commonly have been computed on the basis of an allowance in gallons per acre per day (g.a.d.) sewage flow rate additional to the domestic sewage flow rates. Data illustrative of current practice (Table 3) show sewer capacity allowances for commercial areas, ranging from 4500 to over 60,500 g.a.d.

TABLE 3

Sewer Capacity Allowances for Commercial Areas

City	Year of Data	Allowances for Commercial Areas	Remarks
Baltimore	1949	.135 gal/cap/dy (Range 6750 to 13,500 g.a.d.)	Resident population
Washington Sub. Dist.	1946	None—excepting in special cases	
Boston City	1949	No standard—each area specially studied	
Cincinnati	1949	Commercial areas not served by sanitary sewers	
Columbus, O.	1946	40,000 g.a.d.	Excess added to residential amount
Cranston, R.I.	1943	25,000 g.a.d.	
Dallas	1949	30,000 g.a.d.	Downtown area—rate added to domestic
		4,500 g.a.d.	Outlying area — rate added to domestic
Los Angeles	1948	.015 c.f.s./A	(= 9700 g.a.d. for all commercial areas
Milwaukee	1945	.0936 c.f.s./A	(= 60,500 g.a.d.)
New York City	1949	Allowances determined by special gagings	
Toledo	1946	15,000 to 30,000 g.a.d.—Average to maximum allowances	
Jones & Henry (Const. Engrs.)	1946	10,000 to 20,000 g.a.d.	Outlying areas and smaller communities
		35,000 to 50,000 g.a.d.	Congested areas in larger cities
			(Also determined by water use surveys)

Industrial Area Contributions

Two situations arise—(1) industrial areas which are occupied by industries without liquid process wastes, or areas that are set aside by zoning for future industries, and (2) industries with large flow rates of liquid process wastes which are being or may be discharged into sanitary sewers.

The second situation, of large flows of process wastes, requires special studies of the amounts and characteristics of the liquid wastes from each industry. The sewer capacity provided for the first situation has been computed on the basis of gallons per acre daily (g.a.d.) of industrial contributions. A few illustrative basic design factors are given in Table 4.

TABLE 4

Sewer Capacity Allowances for Industrial Areas

City	Date of Data	Capacity Allowance for Industrial Areas	Remarks
Baltimore	1949	7500-15,000 g.a.d.	No max. flow factor applied—No industries
Washington Sub. Dist.	1946	None in addition to domestic flow	
Boston City	1949	No standards	Each area studied specially
Columbus	1946	6000 g.a.d.	Excess above domestic flow
Cranston, R.I.	1943	20,000 g.a.d.	
Dallas	1949	4000 g.a.d.	Added to domestic flow
Los Angeles	1948	0.021 c.f.s./A (= 13,600 g.a.d.)	Light industrial—special studies for all major industries
Toledo	1946	8000 to 12,000 g.a.d.	Average to maximum allowances
Painesville, O.*	1949	35 g.p.c.d. (= 350 g.a.d.)	Part of 600 g.p.c.d. general basis for sanitary sewer design
Jones & Henry (Const. Engrs.)	1946	7000 to 15,000 g.a.d	Determined by water use surveys

*Courtesy, Havens and Emerson, Consulting Engineers.

These available data (Table 4) indicate that sewer capacity allowances for industrial areas, not including special process wastes, have ranged from 4,000 to 20,000 gallons daily per acre. The lower basic rate of flow has been added generally to the domestic flow which would be provided for from the same area.

Larger sewer capacity allowances have been provided for in-

dustrial flows by Milwaukee, Wisconsin, as determined from a chart giving rates for various areas as follows:

Area—Acres	Runoff from Ind. Areas* c.f.s. per acre	g.a.d.
5 or less	0.375	242,000
8	0.25	161,500
10	0.20	139,200
25	0.088	57,000
50	0.05	32,300
100	0.036	23,300
500	0.016	10,350
1000	0.015	9,700

*Within 1916 City limits double these rates have been used.

Infiltration Into Sanitary Sewers

The proper allowance in sanitary sewer capacity design to provide for infiltration is an uncertain factor—several design engineers in their letters stated that sewer gagings were needed to properly check on infiltration rates. This would be very desirable in extending or relieving existing sewers. However, some reasonable allowance for infiltration must be included in sanitary capacity design for new areas.

The capacity allowances for infiltration which have been provided by various sewer designers (Table 5) range from no allowance up to 5000 g.a.d. or more, additional to the capacity provided for domestic, commercial and industrial sewage flows. Several sanitary sewer designers have estimated infiltration on the basis of an allowance of gallons per day per mile. A few have used as their basis gallons per day per mile per inch diameter.

Storm Water Into Sanitary Sewers

Many sanitary sewers would be surcharged with storm water runoff from roof leaders and surface inlets if some capacity were not provided over that required for domestic, commercial, and industrial sewage flows. In some cases the capacity design of sanitary sewers has provided for roof water, as at Waukegan (Figure III). Occasionally a definite allowance for storm water has been included as at Washington Suburban Sanitary District and at Painesville, Ohio

TABLE 5
Sewer Capacity Allowances for Infiltration

City	Date of Data	Capacity for infiltration g.a.d.	Remarks
Baltimore	1949	135 gal/cap/dy.	Domestic flow considered sufficient to provide for infiltration in lateral sewers.
		500 to 1000	For trunk and interceptor sewers.
Washington)	1946	400	Ground water leakage.
Sub. Dist.)		3524	Storm water from 2 areaways per acre.
Boston City	1949		Taken care of by designing sewers on basis of flowing one-half full.
Columbus, O.	1949	3000	Original basis for capacity chart. Charts later revised to provide 50% more capacity—largely for infiltration and storm water inlets.
Cranston, R.I.	1943	9500	Based on 25,000 gal/mi/dy with 0.038 miles/acre.
Dallas	1949	4250	Incl. storm flows—areas up to 1000A.
		3050	Incl. storm flows for areas over 1000A.
Des Moines	1949	5700 to 11,400	Basis 15,000 to 30,000 gal/dy/mi with 0.038 miles per acre.
Madison, Wis.	1949	2000	Nakoma design — 1937 — Typical practice.
Milwaukee	1945	5000	Metro. Sew. Dist. allowance of 810 g.a.d. found to be too little.
New York City			
Bronx	1949	1940 to 3230	Based on an amount per mile of sewer with 0.038 miles per acre.
Queens	1949	2850 to 3800	
Richmond	1949	1520	
Painesville, O.*	1949	1125	Infiltration allowance.
		6000	Roof water allowance (400 g.p.c.d. at 15 pop./acre)
Springfield, Mass.	1949	2000	New outlying areas.
Toledo	1946	600 to 800	Most sewers are combined.
Toronto	1946	1000	Combined sewers generally used.
Jones & Henry	1946	400 to 800	For new sewers—old sewers run very high.

*Courtesy of Havens and Emerson, Consulting Engineers.

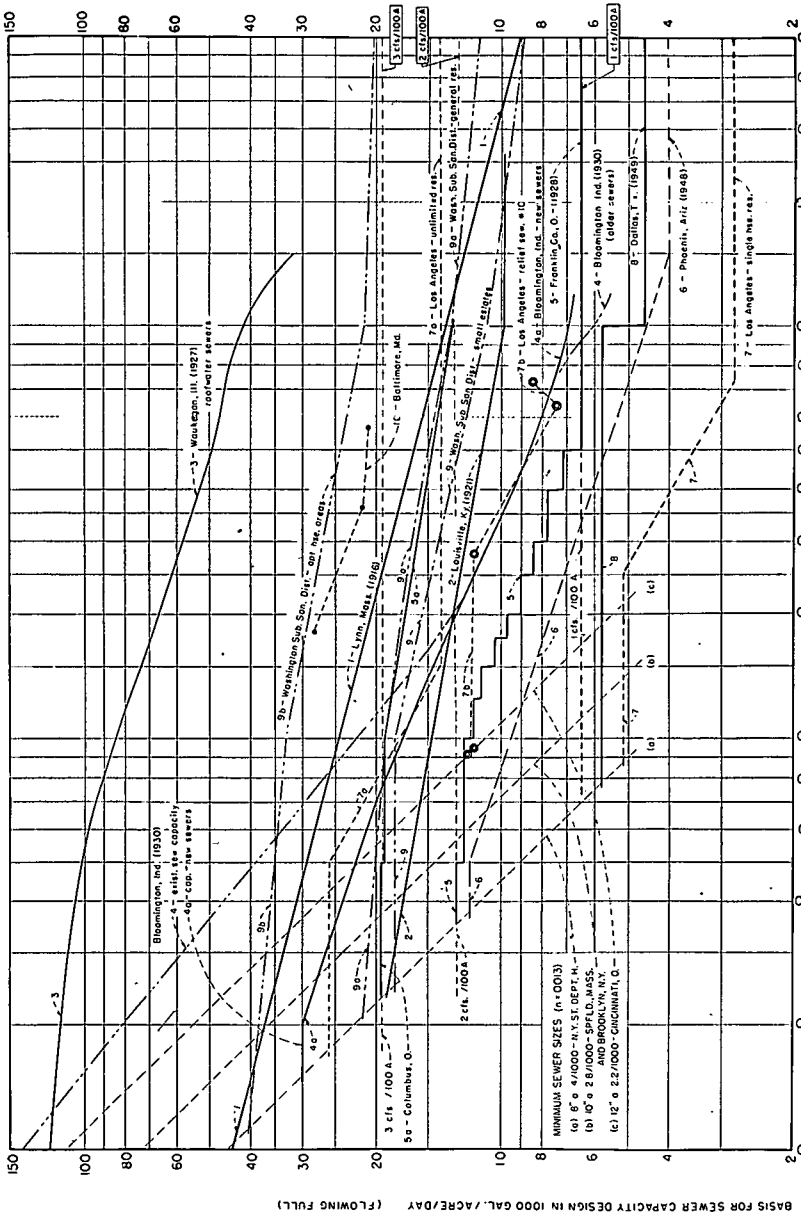


FIG. III BASIS OF SANITARY SEWER DESIGN – RELATION OF CAPACITY TO AREA

(Table 5). More generally the allowance for infiltration has been made sufficiently liberal to provide for a limited quantity of storm water from roof leaders and surface inlets.

The use of sanitary sewers to take storm water from any considerable number of surface inlets results in serious surcharging and basement flooding.

Sanitary Sewer Capacity Design—Gallons Per Acre Daily (g.a.d.)

The basic data for sanitary sewer capacities (Tables 1 to 5, inclusive) show a wide range in sewer capacities which have been provided for domestic sewage, commercial, and industrial sewage flows and for infiltration (including storm water allowance) by various sewer designers. Certain designers have simplified the problem by developing a unit rate of sewage flow per acre of tributary area as the basis for estimating sewer capacity.

One of the earlier records (1916) of this procedure was published by Metcalf and Eddy for Lynn, Mass. (M. & E. Textbook—"Sewerage and Sewage Disposal," 2nd Ed., p. 58, Fig. 10). A similar procedure was adopted by the Commissioners of Sewerage at Louisville, Ky. (Oct. 17, 1921 report). Recently this procedure has been given attention by Bennett at Phoenix, Ariz. (Engr. News-Record, October 14, 1948) and has been adopted by the Borough of Queens in New York City (letter Nov. 4, 1949, James A. Burke, President). Greeley and Stanley have presented data for sanitary sewer design in Davis' Handbook of Applied Hydraulics (2nd Ed. pp. 1052-1054).

The authors have collected a number of illustrations of the use of this procedure—using a unit rate of sewage flow per acre daily (g.a.d.)—and have translated several series of sewer capacity sewage flow rates determined by other computation procedures into equivalent unit rate of sewage flow per acre. The resulting illustrative data are shown graphically by Figure III.

This chart should be useful for sewer designers as a method for comparing the capacity design of any specific sanitary sewer project, based on local data, with the sanitary sewer capacities provided elsewhere, which should be a good guide by which to judge the reasonableness of a proposed basis for computing sewage flows for which capacities should be provided in sanitary sewers.

Summary of Capacity Design for Sanitary Sewers

Selection of proper values for the several basic factors—population density per capita flow, commercial, industrial, infiltration, and storm water allowances—requires careful evaluation of local data and the exercise of mature engineering judgment. The amounts of infiltration and storm water, particularly the latter, which might be admitted into sanitary sewers within the anticipated design period are uncertain and depend on future developments beyond the control of sanitary sewer designers. The use of unit per acre sewer capacity quantities, larger for smaller areas and decreasing in magnitude for larger areas is a very useful procedure as a basis for checking estimates of sewage flow rates for capacity design, computed from values applied to the several basic factors. Figure III illustrates the sewer capacities, in g.a.d, for several such sewer design practices.

Liberal capacities in sewers for smaller tributary areas are provided by the quite general use of an 8-inch diameter as a minimum sewer size. The more recent use of a 10-inch minimum sewer size by Springfield, Mass., and by Brooklyn Borough, New York City, and a 12-inch minimum size for sanitary sewers in Cincinnati, are indicative of a trend in sewer capacity design toward more liberal capacities in sanitary sewers for the smaller areas. Other illustrations of sanitary sewer design procedures are briefly described in Appendix I.

PART II—CAPACITY DESIGN FOR STORM SEWERS

Introductory Statements—Storm Sewers

Ideally, the determination of a proper basis of the capacity design of storm sewers should include three important forecasts:

a. A forecast of a reasonably probable future development of the area to be served.
b. A forecast of probable future civic demand for protection against inconvenience or damage from storm water runoff.
c. A forecast of probable future storm water runoff into sewers—related to partial records of past rainstorm frequencies and rainfall intensities.

Actually, storm sewer capacity design, in past routine practice, appears to have been based more generally on precedent practice rather than on any exact application of ideal basic factors or any extensive study and forecast of the probable future characteristics of the areas to be sewered.

Computation Procedures

Three "methods," or computation procedures, have been developed and used over the past years for storm sewer capacity design. These may be briefly stated as follows:

I. *The Formula Method*—formerly used extensively in several cities. This method has been extensively discussed in the engineering literature and need not be discussed herein. John C. Wenrick (Comm. of Engng., Cleveland, O.) has reported that the McMath formula—
$Q = CRS^{1/5} A^{4/5}$—had been used in Cleveland to compute runoff from large areas.
Example—In planning the Doan Brook Sewer (6/14/40) to serve 7100 acres, Richardson computed a runoff of 3700 cfs by the rational method and 3823 cfs by the McMath formula using $C = 0.60$; $H = 3''$/hour; $S = 17$; and $A = 7100$ acres.

II. *The Rational Method*—(so-called)—is presently extensively used. In this method storm sewer capacity design is based on peak rates of runoff computed on the premise that the runoff rate is a fraction of the average rainfall intensity for a rainstorm of some selected frequency of occurrence with a storm duration equal to an arbitrarily computed time of storm flow concentration.

III. *The Hydrograph Method*—an application of the synthetic hydrograph to storm sewer capacity design. A hydrograph of the storm water runoff rate is computed at various points along the sewer after making allowances for reduction in flow rates due to rain water detention, storage, and the relative timing of peak flow rates in passing from the point of rainfall to the particular section of sewer under consideration. This, a relatively new procedure in storm sewer design, is briefly explained by Mr. Kaufman hereinafter in Part III—The Hydrograph Method of Storm Sewer Design.

Data on recent sewer design practice, received from a number of storm sewer design offices, indicate that most storm sewer designers use some variant of the so-called "Rational Method." One or two engineers reported an occasional use of the Formula Method. Information on the routine use of the Hydrograph Method was received from only one city—Los Angeles, California. (See W. I. Hicks—"A Method of Computing Urban Runoff"—Trans. ASCE, Vol. 109 (1944), p. 1217.)

Note:—Recently (Feb. 1952) the Regional Planning Commission, Cleveland, Ohio, has published an "Engineering Design Manual" in which an effort has been made to include a hydrograph method designated the "Horton Method."

Also Tholin reports that extensive studies are being made of the application of the hydrograph method to replace the rational method at Chicago.

The Rational Method

Several "Rational Method" computation procedures are used by sewer designers. Two variant forms of this method are in most frequent use:

1. The form published by Metcalf and Eddy in their textbook on Sewerage and Sewage Disposal which recognizes as axiomatic a direct relation between the rainfall and the runoff, as shown by the formula $Q = CiA$, in which Q = cfs runoff flow rate; C = a runoff ratio factor; i = average rainfall intensity in inches per hour (equivalent to cfs per acre); and A = tributary drainage area in acres.
2. A second form published by Babbitt who states (Sewerage and Sewage Treatment—p. 39, 6th Ed.) ". . . the basic expression for the so-called rational method can be written $Q = AIR$," in which Q = cfs, A = tributary watershed in acres; I = percentage of impervious surfaces on the drainage area; and R = average rainfall intensity in inches per hour.

In practice, regardless of the formula used, storm sewer designers generally consider the storm water runoff rate to be the product of an area in acres multiplied by a coefficient (usually less than 1.0) times a rainfall intensity, sometimes it is the product of an equivalent area (the actual area multiplied by a coefficient—usually less than 1.0) multiplied by a rainfall intensity. In St. Louis the equivalent area is greater than the actual area, in case of smaller districts. The drainage area can usually be closely determined by surveys. So the rainfall intensities and the runoff coefficients to be used, in the capacity design of a particular storm sewer, are the two critical items to be selected by the designing engineer. He must select these by his engineering judgment or on the basis of precedent practice.

Storm Duration-Time of Concentration

The rainfall curves or formulas used in storm sewer design relate average rainfall intensities to storm duration. Computations of storm water flow rates usually make the storm duration equal to a time of concentration determined by an arbitrary "inlet time" plus a computed time of travel along the longest length of sewers above the sewer location for which the required capacity is being computed.

"Inlet time" required for storm water to reach and pass through the storm water inlet depends on many factors none of which can be accurately computed. Thus, debris on the street, clogging of the inlets by debris or by snow and ice are factors. Some designers at-

tempt to control the time of entrance by the hydraulic design of the inlet openings. However, frequently the sewer maintenance crews may have more actual control of the inlet capacities, depending on the effectiveness of their inlet cleaning efforts, especially in the fall when leaves are falling or during freezing winter periods.

Accordingly, the inlet time period selected by storm sewer designers is only an arbitrary estimate. Several storm sewer designers have reported use of inlet time values ranging from 4 minutes at San Francisco to 20 minutes at Springfield, Mass., and Toledo, Ohio. A few inlet time values used in storm sewer capacity design practice have been reported as follows:

Buffalo	15 minutes	Rochester	7 miniutes
Chicago	10 "	St. Paul	10 "
Cincinnati	5* - 10# "	San Francisco	4 "
Cleveland	8 "	Springfield, Mass.	20# "
Columbus	10 "	St. Louis	5 "
Des Moines	5 "	Toledo	20# & 15* "

*Business district. #Outlying area.

In general, drainage areas in Chicago are quite flat while ground surface slopes in San Francisco may be much steeper. Inlet time values used in design reflect this difference. The character of the area being sewered is another factor. Thus, the 20-minute inlet time values for Springfield, Mass., were for outlying fairly flat residential districts.

In Pittsburgh the sewer designers select values for the initial or "inlet time" on the basis of per cent of imperviousness and the average surface slopes as illustrated by the following tabulation:

Per Cent Impervious Area	Initial Time—Minutes Flat Districts Less than 5% Average Slopes	Hill Districts Over 5% Average Slopes	
100	3.0	3.0	Downtown
90	3.5	3.0	
80	4.0	3.5	
70	5.0	4.0	
60	6.0	4.5	
50	7.0	5.0	
40	8.0	5.5	
30	9.0	6.0	
20	10.0	7.0	
0 to 10	determine	determine	Parks

The Bureau of Engineering, City of San Francisco, has set up a standard "Storm Sewer Design Practice" (X-1481, 12/10/45) in which values of inlet time were related to type of area development and to surface slopes as follows:

Type of Area	Values of Inlet Time—Min. Under 3% Slope	Over 3% Slope
Industrial	4-6	3-5
Commercial	5	3
Apts. & Flats	5	3
Res.-Att. Houses	6	4
Res.-Det. Houses	7	5
Suburban Parks	10	6

A comprehensive, three-year experimental study of storm water inlets, sponsored by Baltimore City, Baltimore County, and Maryland State Roads Commission, carried forward at The Johns Hopkins University, was summarized by J. C. Geyer and B. C. Goodell in "Report on the Storm Drainage Research Project" dated January 1952. (Unpublished.) These tests developed many useful data on the hydraulics of inlets free of obstructions. Publication of the results would be a great service to storm sewer designers.

The time of travel in sewers or "flow time" computed from the length of sewer divided by the average velocity of flow, will be longer for flat slopes and shorter for steep slopes. Thus, steeper topography results in a shorter time of concentration, hence, with a given rainfall intensity curve, a higher average rainfall intensity and accordingly, a larger sewer capacity requirement, but not necessarily a larger sewer diameter—since sewer capacity is a function of both diameter and slope. Theoretically, time of travel in the sewer, i.e. "flow time" should be a simple computation (length divided by average velocity).

However, there is considerable uncertainty as to both sewage flow rates and internal sewer channel characteristics, i.e. resistance to flow, hence there is uncertainty as to velocity of flow for the design period of 30 or 40 years in the future.

Accordingly, some sewer designers have attempted to relate the time of concentration to the tributary area, as illustrated by Figure IV.

The time of concentration for a given type of topography should

bear some general relationship to the tributary area. John A. Rousculp, Engineer of Sewer Design—City of Columbus, Ohio, published a chart (Fig. 2, p. 1475, Trans. ASCE, Vol. 104, 1939) showing "Concentration-of-Area Curves of Various Districts," from which the author has selected a "Typical District" curve with time-area relationships as follows:

Time in minutes (based on S = 0.003)	Area Acres	Time in minutes (based on S = 0.003)	Area Acres
15	25	35	650
20	80	40	940
25	200	45	1200
30	400	50	1500

See Fig. IV for these figures graphically presented.

A. L. Tholin, Engineer of Sewer Design—City of Chicago, furnished a curve (File No. DC-23) showing "Relation Between Area Drained and Time of Concentration—" for several proposed new sewer districts in Chicago, where slopes are quite flat, from which the following illustrative data have been scaled:

Area Acres	Time of Concentration Minutes	Area Acres	Time of Concentration Minutes
50	22	700	61
100	29	1000	70
200	38	1200	75
300	45	1500	82.5
500	54	1800	89

Tholin has observed that the time of concentration in existing sewers varied widely, but he estimated they could be averaged roughly about 25% greater than indicated by the above figures (Figure IV—dashed curve).

In addition to the published Columbus Curve and the Chicago Curve (DC-23) illustrative computations were obtained from seven other cities from which illustrative tributary area-time of concentration relationships were determined. The illustrative data from the several cities, together with the Chicago and Columbus data, are plotted on

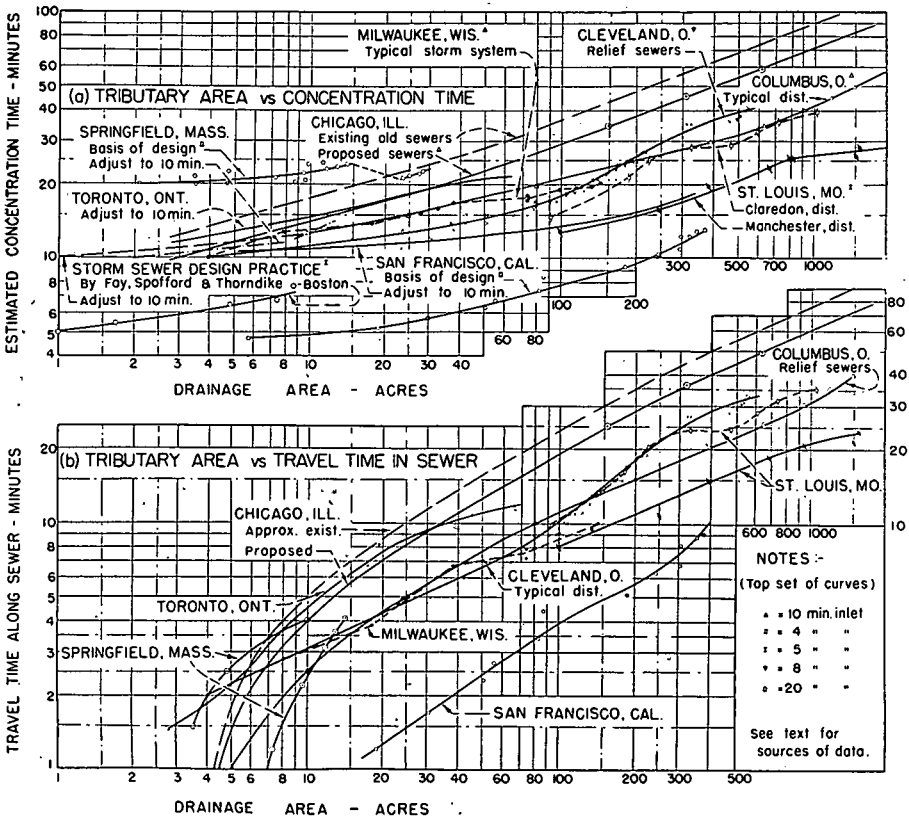


FIGURE IV - STORM SEWER DESIGN - TIME OF CONCENTRATION RELATED TO TRIBUTARY AREA

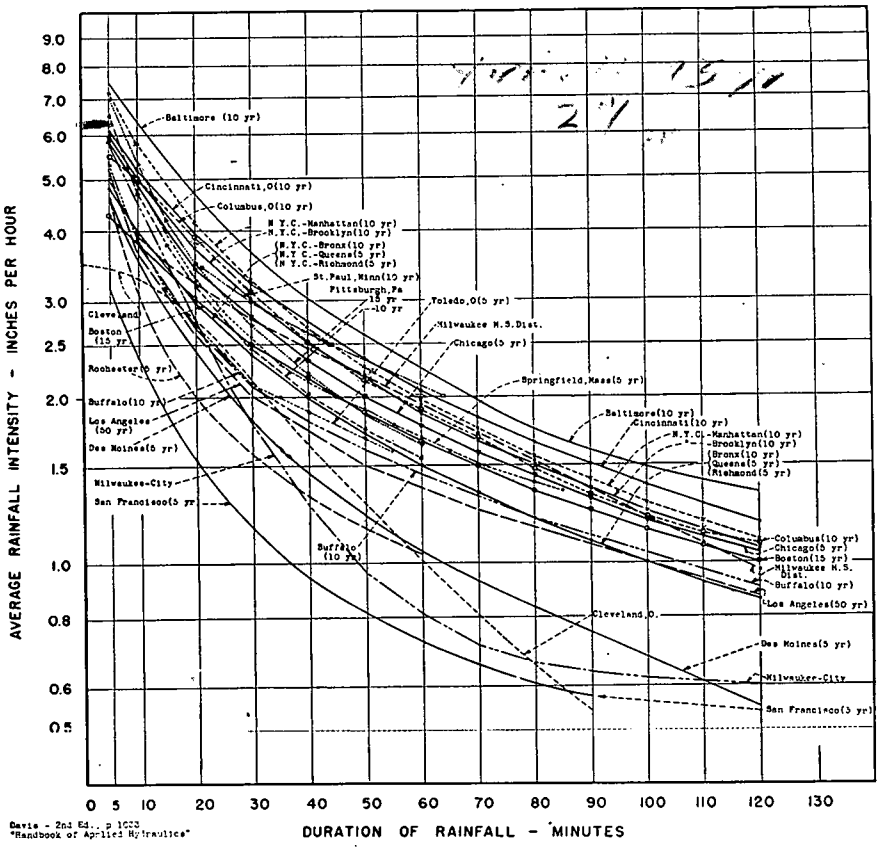
log scales in Fig. IV. (The several data were furnished through the courtesy of various Engineers as acknowledged in Appendix III.)

These Tributary Area-Time of Concentration curves illustrate graphically the spread of concentration time for flat ground slopes, as in Chicago, compared to the relatively shorter time for steeper slopes, as in San Francisco and St. Louis. The Springfield Curve of Figure IV(a) is higher than the Chicago Curve because of the 20 minutes' inlet time used at Springfield as compared to the 10 minutes' inlet time used in Chicago. The Figure IV(b)—Tributary Area vs. Travel Time-curves, in which the inlet time has been omitted, show higher time values for the Chicago and Toronto Curves.

These curves (Figure IVa and IVb) should be used by storm sewer designers only as approximate indices. The actual time of concentration time should be computed for each specific storm sewer design computation.

Rainfall Intensities-Duration Data

Most city storm sewer design offices have adopted either a rainfall intensity-duration diagram, or a rainfall formula, to represent the relationship between storm duration and the average rainfall intensity for a storm sewer of a certain frequency of occurrence—



Davis - 2nd Ed., p 1033
"Handbook of Applied Hydraulics"

FIG.V —RAINFALL INTENSITY CURVES

TABLE 6

Illustrative Rainfall Intensities Used in Storm Sewer Capacity Design by Indicated Cities

City	Storm Frequency Years	Average Rainfall Intensity-inches/hour for Storm Durations of — Minutes 10	20	30	60	120
Baltimore	10	6.3	4.8	3.7	2.2	1.4
Boston (1)	15	3.75	3.0	2.5	1.7	1.0
Buffalo	10	3.8	2.8	2.1	1.8	0.9
Chicago (2)	5	4.7	3.5	2.8	1.8	1.0
Cincinnati	10	5.2	4.0	3.2	2.1	1.2
Cleveland	—	3.3	2.7	2.1	1.0	—
Columbus	10	4.9	3.9	3.1	1.9	1.1
Des Moines	5	3.5	2.4	1.8	1.0	0.6
Lansing	10	5.6	4.0	3.1	1.9	1.0
New York City (Practice determined by individual boroughs)						
Manhattan	10	5.8	4.2	3.3	2.0	—
Brooklyn	10	5.0	3.8	3.0	1.9	1.1
Bronx	10					1.1
Queens	5	4.0	3.0	2.4	1.5	0.9
Richmond	5					
Pittsburgh (3)	15	5.3	3.5	2.7	1.6	—
	10	4.7	3.2	2.5	1.5	—
Rochester	5	3.0	2.0	1.6	1.0	—
Saint Paul (4)	(10)		3.5	3.0	2.1	—
San Francisco (5)	5	2.3	1.5	1.1	0.7	—
Springfield, Mass.	5	4.9	3.3	2.6	1.7	—
St. Louis (6)	15	6.5	4.6	3.75	2.5	1.5
	20	6.8	4.8	3.9	2.7	1.6
Toledo	5	3.8	2.7	2.2	1.5	—
Toronto	2 to 3	2.0	1.7	1.5	1.1	
Mariemont, O.	—	5.1	3.6	2.9	2.1	1.5

Notes:—(1) Dorr Formula $i = 150/(t + 30)$ generally employed in Boston (J. F. Flaherty by letter 7/20/49).

(2) Based on Eltinge formula $i = 90/(t^{0.9} + 11)$—the Schafmayer formula $i = 137/(t + 18)$ was used until about 1944, gives same intensities as Eltinge formula up to storm durations of 120 minutes.

(3) Pittsburgh has used the 15-year curve where the drainage area is built up with paved streets and there is possibility of large property damage, and the 10-year curve for trunk sewers of large investment and where overflow and surcharges might occur without much trouble.

(4) St. Paul sewer design is only incidentally related to rainfall rates.

(5) San Francisco rainfall curve has been divided into two segments at storm duration time of 38 minutes.

(6) St. Louis had used a 15-year curve for more than 35 years until September 1946 when a 20-year curve was established as the standard.

once in 5 years, or once in 10 years, or once in 15 years. R. C. Cassidy (City Sewer Engineer) of Indianapolis reported the use of storm intensities of 5 years, 10 years, and 15 years' frequency for residential and apartment house areas, and up to 30 years' average frequency for highly developed business areas. Representative rainfall intensities-duration curves (some from formulae) are shown graphically by Figure V—Rainfall Intensity Curves. Comparative average rainfall intensities in figures for several cities are given in Table 6, as taken from charts or tables received from the several cities. A number of illustrative rainfall formulae are given in Table 7.

TABLE 7

Illustrative Rainfall Formulae Used in Storm Sewer Design by Indicated Cities

City	Rainfall Intensity-Duration Formula for "i"*	Storm Frequency Years	Remarks
Boston	$i = 150/(t + 30)$	15	Dorr Formula
Chicago	$i = 90/(t^{0.9} + 11)$#	5	(See Footnote)
Cincinnati	$i = 170/(t + 23)$	10	
Cleveland	$i = 5040/(t^2 + 1440)$	—	
Des Moines	$i = 74.5/(t + 10.85)$	5	
Lansing	$i = 140/(t + 15)$	10	
New York City			
Manhattan	$i = 150/(t + 16)$	10	
Brooklyn	$i = 150/(t + 20)$	10	
Bronx	$i = 120/(t + 20)$	10	
Queens, Richmond	$i = 120/(t + 20)$	5	
Rochester	$i = 12/t^{0.6}$	5	
San Francisco	$i = 44.7/(t = 9.3)$	5	5 to 38 min. duration
	and $i = 8/t^{0.587}$	5	38 to 90 min. duration
Springfield	$i = 20.4/t^{0.61}$	—	
Toledo	$i = 12/\sqrt{t}$	—	

* The "i" in Q = CiA.

\# Eltinge formula for Chicago—(the Schafmayer formula $i = 137/(t + 18)$ gives same results up to a storm duration of 120 minutes and was used for sewer design until 1944).

The several rainfall intensity-duration curves (or formulae), represented in Figure V and Tables 6 and 7, show average rainfall

intensities for a 20-minute storm duration ranging from 1.5 inches (San Francisco) to about 4.8 inches (Baltimore) i.e., a range of about 1 to 3+; and for a 60-minute storm duration the range in average rainfall intensities is from about 0.73 inches to 2.25 inches, again a range of about 1 to 3+.

A few notations relative to peculiarities of the present day rainfall intensity-duration curves (Figure V and Table 6) may be of interest:

a. The Baltimore rainfall curve gives the highest intensities for any 10-year curve. St. Louis uses a 20-year curve (since Sept. 1946) which gives only slightly higher intensities (Table 6) than the Baltimore curve.
b. The Springfield, Mass. 5-year rainfall intensity-duration curve gives intensities higher than the Boston 15-year curve, but corresponds closely with the Chicago 5-year curve.
c. The rainfall curves for Cleveland, Des Moines, Milwaukee City, Rochester, and San Francisco give much lower intensities for a given storm duration than the other curves.
d. In New York City, the Boroughs of Bronx, Queens and Richmond use the same rainfall curve. Bronx calls it a 10-year curve. The other two Boroughs label it a 5-year curve.
e. The Cleveland rainfall intensity-duration curve has an individualistic form.

It appears obvious that the variations in average rainfall intensities related to storm duration, as determined from the various rainfall curves (or formulae), are greater than the probable actual differences in rainfall intensities from city to city. Accordingly, the use of rainfall curves or formulae, in some cases, must be only valid as determining an index number for the magnitude of rainfall and may not be the actual average rainfall intensities in the city. Modern extensive rainfall records do not justify continued use of such outdated rainfall intensity curves or formulae.

Some evidence indicates that sometimes storm sewer designers adopt a rainfall intensity, in inches per hour, which is used as a constant for all sizes of drainage areas.

Samuel M. Gray is reported to have selected (in 1884) a rainfall intensity rate of two inches, irrespective of storm duration, and a runoff factor of 50 per cent for all areas as the basis of the design of storm sewers in Providence, R. I.—which basis of design has influenced the capacity provided in most of the existing storm sewers of Providence. Referring to Figure IV (or Table 6) a 2-inch average

rainfall intensity corresponds to a 12-minute storm duration in San Francisco and to a 70-minute storm duration in Baltimore.

In recent Chicago sewer design practice (Bureau of Sewers) a modifying factor "K" is applied to the average rainfall intensity "i" to compensate for the uneven distribution of rainfall over large drainage areas. The following illustrative figures have been scaled from a Bureau of Sewers chart dated 4/23/48 entitled "Rainfall Modification Factor 'K'."

Drainage Area Acres	Values of K.= $\frac{\text{areal average rainfall intensity}}{\text{areal maximum rainfall intensity}}$			
	30 Min.*	60 Min.*	120 Min.*	180 Min.*
640	.934	.964	.985	.996
2,500	.854(.91)#	.922(.93)#	.964(.98)#	.985
5,000	.788(.87)#	.885(.90)#	.940(.96)#	.972
10,000	.682(.82)#	.828(.86)#	.906(.93)#	.952
15,000	.604	.784	.875	.935
20,000	.544(.75)#	.752(.80)#	.850(.89)#	.915

* Rainfall period — minutes.
# Pasadena data by Hicks (T. ASCE *109* (1944) p. 1222).

Frank A. Marston proposed this idea in a paper years ago (Trans. ASCE, Vol. LXXXVII (1924) p. 535). Hicks also has published (Trans. ASCE, Vol. 109 (1944) p. 1222) such rainfall distribution factors for Pasadena, California.

Runoff Coefficient

The coefficient "C" in the formula Q = CiA, or its equivalent, the imperviousness factor "I" in the formula Q = AIR, either of which represents the fraction of the rainfall estimated to produce the maximum storm sewage flow rate for which sewer hydraulic capacity should be provided. The selection of runoff coefficient or ratio is the most uncertain item in computing sewer capacity requirements by the so-called "rational method."

These coefficients, or factors, actually represent several phases of storm water runoff, including (a) infiltration loss, (b) reduction in peak flow rates by surface detention, and (c) channel storage effect on flow rates.

The origin of runoff coefficients used by some city sewer designers

appear to be historic. Examples:—(1) In Providence, R. I., Samuel M. Gray selected, many years ago, a 2-inch rainfall intensity and a runoff ratio of 0.50; (2) In Boston Mr. Flaherty advised that Bryant and Kuichling in 1909 set up runoff coefficients which are still useful, although several of the pavements and types of street surfacing are no longer used.

Some sewer design practices in several cities relative to the selecting of runoff coefficients are recorded in Appendix II—"Illustrative Runoff Coefficients Used in Various Cities."

Runoff coefficient values appear to be selected on a variety of bases, with no apparent coordination between sewer designers of various cities. Some of the bases of selection may be summarized generally as follows:

a. Values for coefficient "C" based on area classification relative to character of area development.
   Examples: Baltimore (during past), Rochester, San Francisco.
b. Values for coefficient "C" based on surface type classifications and estimates of the proportion of the several types of surfaces.
   Examples: Boston, Chicago, Cincinnati.
c. Values of coefficient computed by multiplying uniform "C" values for impervious and pervious areas by the percentages of impervious and pervious areas based on the percentage of building surfaces permitted by zoning regulations.
   Example: Buffalo, N. Y.
d. Values of coefficient based on estimated percentages of impervious area with different values for flat districts (slopes less than 5%) and for hill districts (slopes more than 5%).
   Example: Pittsburgh, Pa.
e. Values of coefficient computed by formula involving the storm duration time—a different formula for pervious and impervious area.
   Example: Lansing, Mich.

The sewer design practices, relative to the runoff coefficient, recorded in Appendix II, include some interesting procedures as follows:

In Chicago—Tholin proposes to reduce the "C" values by an "F" factor ranging from 0.46 for a storm duration of 10 minutes to 1.00 for a storm duration of 140 minutes.

In Des Moines—Jones reported "C" values computed by R = ratio of impervious area to total area multiplied by a factor "D" the values of which are less than 1.0 and set up for different types of surfaces with different sets of values for "flat" slopes (less than 3%) than for "steep" slopes (more than 3%). Also different sets of values for concentration time of 20 minutes or less and for concentration time more than 20 minutes.

Reports from smaller towns seem to indicate that values of coefficient "C" are selected more or less arbitrarily by the sewer designer on a judgment basis.

Thus, there appears to be very little, if any, fundamental principle involved in current design practice in selecting the runoff coefficient. Precedent design practice appears the most common basic factor.

Storm Sewer Capacities Per Unit Area—cfs Per Acre

The storm sewer capacity provided per unit of tributary area is perhaps the only logical basis for comparing the sewer capacity design practice in various communities. The form of rainfall curves may be quite dissimilar and the procedure for determining the runoff-rainfall ratio vastly different, yet curiously the resulting unit capacities in cfs per acre, when plotted on log log paper (Figure VI) give quite similar curves—almost straight lines.

In most cases representative computations of storm sewer capacities were furnished by city sewer designers. In some cases approximate runoff quantities were computed by the author using runoff coefficients for residential areas and time of concentration figures taken from the appropriate curve of Figure IV. In either case the resulting sewer capacities in cubic feet per second were divided by the tributary area to obtain cfs per acre, which plotted as vertical ordinates against the acres of tributary area as abscissa gave the curves shown in Figure VI—Capacities Provided in Storm Sewers by Various Cities.

Figure VI provides a good index chart by which computed runoff values for sewer capacity design may be checked for reasonableness. It requires some knowledge of the relative characteristics of the storm sewer district under consideration compared to the characteristics of the districts represented by the curves on Figure VI.

Thus, the Toronto and the Springfield, Mass., curves, which have the lowest cfs per acre values should be compared with outlying sparsely populated residential districts, while the St. Louis and Baltimore curves, which have the highest cfs per acre values, should be compared with computed runoff for highly developed heavily populated downtown districts.

The curves in Figure VI suggest a summary of unit storm sewer capacities—cfs per acre—might be as follows:

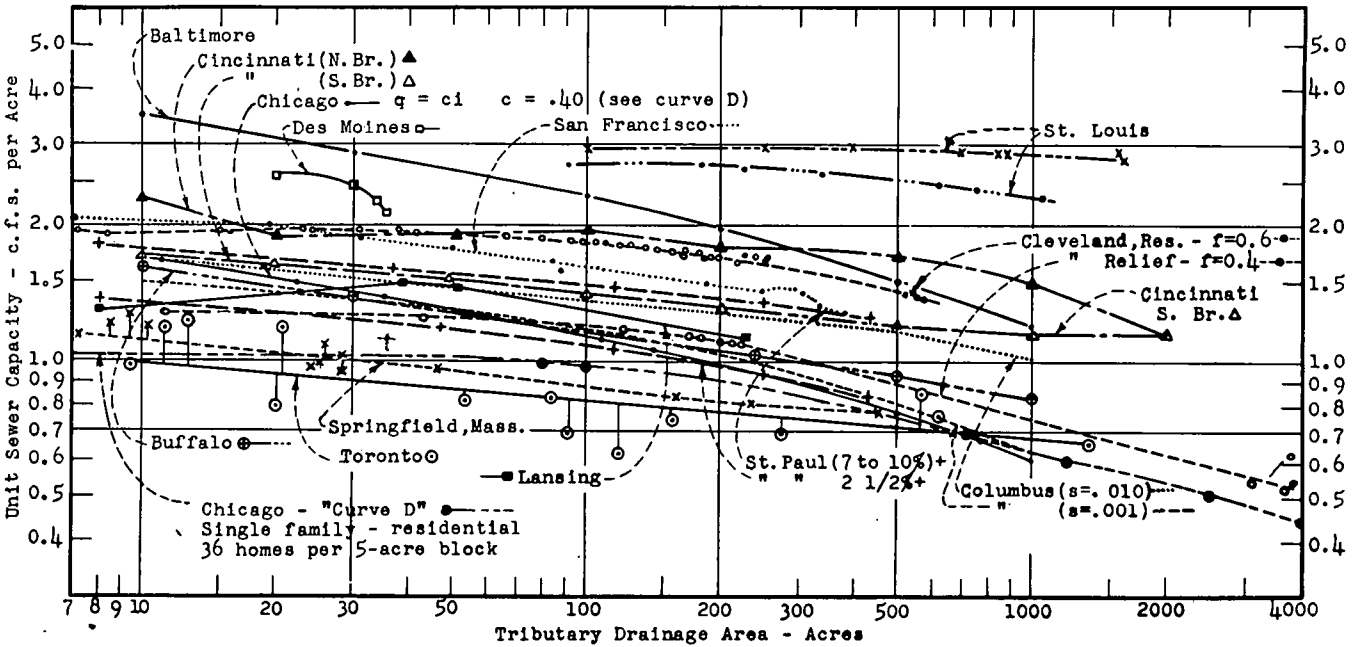


FIG. VI CAPACITIES PROVIDED in STORM SEWERS BY VARIOUS CITIES

Extent of Drainage Area	Reasonable Sewer Capacity-cfs per Acre	
	Outlying Residential Districts	Downtown or Thickly Populated Areas
10 Acres	1.0	2.0 to 3.0
100 Acres	0.8	2.0 to 2.5
1000 Acres	0.6	1.2 to 1.5

Summary

Sewer design practice data from a number of municipal sewer designers show extensive differences in the methods and design factors used for both sanitary sewer and storm sewer capacity design computations.

The data on design factors should be useful as guides to sewer capacity design. The charts of unit capacity per tributary area, including (1) gallons per acre—g.a.d.—for sanitary sewers, Figure III, and (2) cubic feet per acre—cfs per acre—for storm sewers, should be useful as bases for checking the reasonableness of computations for the design of any proposed system of sewers.

Curiously storm sewer capacities in cfs per acre have ranged from about 0.8 cfs/acre up to about 2.5 cfs/acre for areas of 100 acres, while sanitary sewer capacities have ranged from about 0.8 cfs per 100 acres up to about 3.5 cfs per 100 acres for areas of 100 acres. Thus, roughly, capacities for storm sewers have been approximately 100 times the capacities provided in sanitary sewers.

PART III—THE HYDROGRAPH METHOD OF STORM SEWER DESIGN

General Outline of the Hydrograph Method

The "Hydrograph Method" of computing storm water runoff is essentially the synthesis of the runoff hydrograph and the utilization of the maximum runoff rates from this graph for the selection of sewer sizes. The Rational Method of computing storm water runoff for storm sewer design is dependent upon the hypothesis that runoff is a simple linear function of the precipitation intensity, i.e. is some fraction of the rainfall. However, studies of considerable rainfall and runoff data from experimental watersheds indicate the rate of runoff to be equal to the rainfall intensity modified by certain meteor-

ological and terrain factors which in nature are subtractive quantities rather than rate coefficients. A theoretical analysis of surface runoff supports the contention that a more fundamentally basic procedure than the so-called "Rational Method" might well be used for the design of storm sewers.

The factors determining the runoff rate from a given area may be divided into two categories: (a) those relating to the manner in which the precipitation occurs and (b) those dependent upon the characteristics of the drainage basin. The most commonly used aid in describing precipitation characteristics, and the only one necessary in employing the "Rational Method", is the "intensity-duration" curve for the desired storm frequency and locale.

Such a diagram provides only the probable average rainfall intensities for selected storm durations. The actual instantaneous intensities may deviate considerably from these average values. Breihan, in a study of recording rain gauge records of the U. S. Weather Bureau in the Mississippi Valley, developed a chart which illustrates the extent of the deviation. He found that, for average hourly rainfall rates of one half to one inch per hour, 80 per cent of the precipitation fell at rates exceeding the average rate while 45 per cent fell at rates exceeding twice the average rate. As infiltration losses, and consequently excess rainfall and runoff, are dependent upon instantaneous precipitation intensities, a procedure employing simply the average intensity is not strictly rational.

The "storm pattern" or chronological position of peak precipitation rates is an important factor in describing the variation of infiltration rates with time as is illustrated by Fig. VII in which an average rainfall intensity of 2 inches per hour is interpreted as having three precipitation patterns. The areal distribution of intense rainfall is another factor that should be considered, particularly when the area is large. Other contributing factors related to precipitation characteristics, but of a lesser importance, include the direction and velocity of storm travel and the antecedent precipitation.

The influential basin characteristics include area, slope, infiltration capacity, a factor defining the degree of imperviousness, surface retention, vegetal interception, surface detention, and channel storage. The least understood and most discussed of these is infiltration or "infiltration-capacity" which has been defined by Horten as the

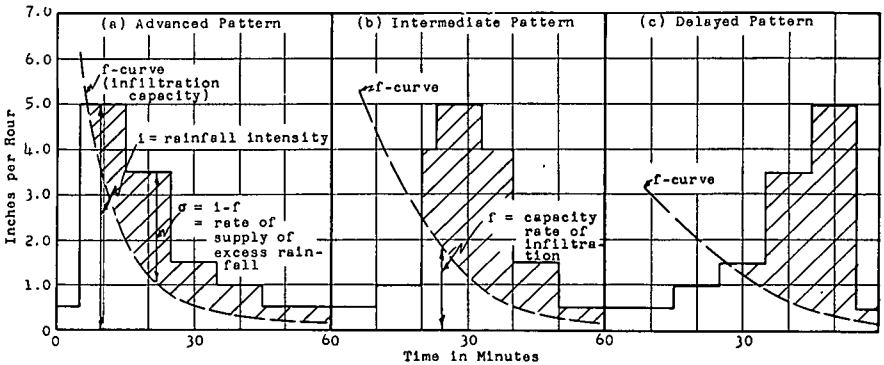


Fig. VII. PRECIPITATION PATTERNS AND CORRESPONDING INFILTRATION CAPACITY CURVES

maximum rate at which a soil, in a given condition, can absorb rain water. The term "capacity" refers to the limiting rate of entrance of water into the soil and does not indicate a measure of water volume. Infiltration is extremely important since the excess rainfall and the resulting runoff rates are dependent on the extent the storm intensity exceeds the infiltration-capacity. Figure VII. Infiltration is a function of the soil moisture content, the soil porosity, the precipitation intensity which may cause an inwashing of silt, the vegetal cover, and soil and water temperature. It is nearly independent of surface slope and of precipitation intensity providing the soil has a full vegetal cover.

"Surface retention", which is essentially "depression storage", may be defined as the quantity of water on the surface of the ground at the instant surface runoff begins. It is independent of rainfall intensity but dependent upon the general rugosity, slope, and vegetal cover of the drainage area. "Surface detention", designated (δ), which is hydraulically similar to channel storage, is defined as the average water depth on the ground surface at a given instant during runoff. It is related to the runoff by the simple power function, $q = K\ \delta^M$, M being dependent on Reynold's number and varying between limits defined by turbulent and laminar flow. Surface detention and channel storage are to be considered as runoff reservoirs which tend to delay and decrease the peak runoff rates.

The measured hydrograph of runoff represents graphically an integration of the effects of the physical characteristics of a drainage basin on the pattern of storm precipitation. The hydrograph describes

the reduction of incident rainfall intensities by interception, by surface retention, and infiltration and the modifications resulting from surface detention and channel storage. The most important hydrograph characteristics related to storm sewer design are the peak runoff and the chronological position and general configuration of the peak. These two latter factors are of importance in summing confluent hydrographs in intercepting sewers.

The only major application of the synthetic hydrograph methodology to practical municipal storm sewer design has been in the City of Los Angeles as outlined by W. I. Hicks in 1943. In a prior paper, W. W. Horner and S. W. Lens in 1941 presented the first application of the basic hydrologic hypotheses to urban drainage problems.

Hicks, in applying the hydrograph method in Los Angeles, has simplified the computations with numerous nomographs and by the delineation of a most probable precipitation pattern for any desired storm frequency. Based on gagings of the runoff from typical urban blocks, a runoff-to-inlet hydrograph was developed for a 100 per cent impervious area and a ten year rainfall curve on a 1.33 isohyetalline and employed in computing peak runoff rates for various times of concentration ranging from five to one hundred and twenty minutes.

SYNTHESIS OF THE RUNOFF HYDROGRAPH

The synthesis of the runoff hydrograph may be reduced to six basic steps:

1) The selection of a basic storm pattern and the delineation, from this pattern, of the critical or "design storm" from which the runoff is to be computed.
2) The selection of a suitable infiltration-capacity curve corrected to the particular area under consideration and modified to encompass antecedent rainfall effects.
3) The derivation of the supply curve of excess rainfall.
4) The derivation of the "inflow-to-gutter" or "overland-flow" hydrograph by integrating the influence of terrain features on the excess rainfall curve.
5) The calculation of the "inflow-to-sewer" hydrograph by routing the inflow-to-gutter hydrograph through "gutter storage".
6) The calculation of peak rates of runoff at points of concentration, the summing of confluent hydrographs, the routing of these hydrographs through "conduit storage" to determine peak conduit rates, and finally, the selection of sewer sizes.

Without doubt, the hydrograph method can be greatly simplified

in application without sacrificing any of its fundamental advantages. These simplifications should take the form of the publishing, in a readily usable manner, of certain basic hydrologic data. If storm patterns and basic infiltration curves were available a great portion of the problem would be removed from the engineer attempting to apply the synthetic hydrograph. The standardization of the runoff-to-sewer hydrographs for typical city blocks for various slope and precipitation conditions might be accomplished and would solve the problem completely.

APPENDIX I—ILLUSTRATIVE PROCEDURES FOR CAPACITY DESIGN OF SANITARY SEWERS

The following brief notes illustrate basic data which have been used in several cities for determining the sewer design capacities to be included in sanitary sewers. Illustrative sewer capacities, in g.a.d. are shown graphically by Figure III.

I-1) *Lynn, Mass.*—M. & E. textbook—"Sewerage and Sewage Disposal" (2d Ed., p. 58, Fig. 10) includes a chart for residential sewage quantities computed on the basis of 220 gallons per capita with population densities decreasing from 130 for a few acres to about 41 for 1000 acres. Changing these quantities to gallons per day per acre and plotting on log log paper with area as the horizontal ordinate gives a straight line (Figure III, Curve 1).

I-2) *Louisville, Ky.*—The October 17, 1921 report of the Commissioners of Sewerage included the capacity basis of design, for separate sewers, ranging from 22,000 gallons per acre per day for 10 acres to 9960 gallons per acre per day for 1000 acres or larger. This is also a straight line below 1000 acres when plotted on log log paper (Figure III, Curve 2).

I-3) *Waukegan, Ill.*—Roof Water Sewers (Ref. Davis—Handbook of Applied Hydraulics, p. 1052-2d Ed.). A basis of design of residential sewers was prepared (by Greeley & Hansen) in 1927 for North Sewer District in Waukegan, Illinois, with the following basic data:

a) Population density .....15 to 75 per acre, inversely with area
b) Domestic sewage flows ....175 gallons per capita daily
c) Infiltration .....1600 gallons per acre daily

d) Roof runoff .....3 to 5 houses per acre (1000 sq. ft. each) 90% connected—3-year storm frequency—with 0.40 to 0.80 runoff coefficient

These basic data provided computed sewer capacities for maximum rates of sewage flows as follows:

Acres	*Maximum Flows (g.a.d.)*
10	120,000
50	100,000
100	84,000
500	50,000
1000	42,000
1200	40,000
1500	32,000
1800	26,000

These sewer capacities show the effect of storm water runoff from a considerable number of roof connections. The unit sewer capacities per acre were substantially higher than usually provided in sanitary sewers (Figure III, Curve 3).

I-4) *Bloomington, Ind.*—(Ref. Davis—Handbook of Applied Hydraulics, 2d Ed., p. 1052). An extensive study (1930) of capacities in the sanitary sewers of Bloomington, Ind., showed unit per acre capacities ranging from 140,000 gallons per day per acre for areas of 10 acres decreasing to 6000 gallons per day per acre for areas of 1000 acres (Figure III, Curve 4). Investigation of sewer operating experience led to the conclusion that a unit sewer capacity in gallons per day per acre decreasing from 30,000 [6 c.f.s./1000 A] for areas of 20 acres or less to 6500 [1 c.f.s/100 A] for 1200 acres would provide a satisfactory basis of design for sanitary sewers (Figure III, Curve 4a).

I-5) *Franklin County, Ohio*—E. G. Bradbury published (Proc. ASCE, Sept. 1928, p. 2065) a basis of sewer capacities for sanitary sewers in Franklin County Sewer District, Columbus, Ohio, with unit capacities in c.f.s. per acre decreasing from 0.020 for 50 acres or less to 0.010 for 501 acres or more (12,920 to 6460 gallons per acre per day) (Figure III, Curve 5).

Some of the Franklin County sewered areas have since become developed sections of the City of Columbus. Mr. John A. Rousculp, Designing Engineer, reported (letter, Aug. 10, 1949) experience data on sewer operation which indicate that the capacities provided by Bradbury's basis of design have proven inadequate in some cases

for wet weather conditions—due to flows from sub-basement drains, roof drains, and infiltration.

In 1938 an investigation resulted in a sanitary sewer design curve (Acc. No. 383—Mar. 29, 1938) with sanitary sewer capacities in c.f.s. per 100 acres decreasing from 2.94 (19,000 g.a.d.) for 100 acres and less to 2.01 (12,900 g.a.d.) for areas of 1000 acres (Figure III, Curve 5a). This basis of design provided 50% greater sewer capacities than the Bradbury curve.

Mr. Rousculp also reported that certain studies of relief sanitary sewers indicated desirable sewer capacities in the magnitude of 4.0 c.f.s. per 100 acres (25,840 g.a.d.).

I-6) *Phoenix, Arizona*—Bennett recently published (ENR, Oct. 14, 1948) a curve for sanitary sewer capacities in gallons per acre per day decreasing from 12,000 for areas of 50 acres and less to 4000 for areas of 1500 acres and larger (Figure III, Curve 6) based on measurements of maximum flows in Phoenix sewers at some 40 locations.

The foregoing six illustrations include published design practice factors in which the sewer capacity quantities were related directly to the tributary sewered area. The following items are examples of sewer design practices in which the unit sewer capacities per acre were not related so directly to the sewered areas by the local sewer designers, but have been translated, by the senior author, into g.a.d. capacity units.

I-7) *Los Angeles, Calif.*—The capacities of sanitary sewers for Los Angeles have been based on coefficients, in terms of c.f.s. per acre. Mr. Lloyd Aldrich, City Engineer, has stated (letter, Nov. 8, 1948) their basis of design of sanitary sewers, recently adjusted to more accurately conform with property zoning, as follows:

R-1	Single-family residential zone	0.004 c.f.s./A
R-2	Two-family residential zone	0.008 c.f.s./A
R-3	Three-family residential zone	0.012 c.f.s./A
R-4	Four-family residential zone	0.016 c.f.s./A
R-5	Unlimited residential zone	0.020 c.f.s./A
	All commercial zones	0.015 c.f.s./A
M-1	& M-2 light industrial districts	0.021 c.f.s./A
M-3	Major industrial districts	Determined for each case

a) Sewers 18-inch or larger are designed to flow at 3/4 depth.
b) Sewers 15-inch or smaller are designed to flow at 1/2 depth.*
c) Kutter's roughness coefficient = 0.013 is used.

* Remainder of depth reserved for ventilation.

On the basis of sewers flowing full on grades to give 2.5 feet per second velocities the following sewer capacities have been computed for residential zones:

a) *R-1 Single-Family Residential Zone*

1) Areas of 250 acres or less, unit capacity	0.80 c.f.s./100 acres or 5170 g.a.d.
2) Areas of 725 acres or larger, unit capacity	0.44 c.f.s./100 acres or 2810 g.a.d.

b) *R-5 Unlimited Residential Zone*

1) Areas of 50 acres or less, unit capacity	4.0 c.f.s./100 acres or 25,840 g.a.d.
2) Areas of 150 acres or larger, unit capacity	2.18 c.f.s./100 acres or 14,000 g.a.d.

The Los Angeles basic data for sanitary sewers in single-house areas provide the lowest per acre sewer capacities of any major city (Figure III, Curve 7). The curve (Figure III, Curve 7a) for unlimited residential districts compares favorably with capacities found desirable in less arid regions.

Recently (reported by letter Sept. 22, 1949) the Los Angeles city engineers completed an analysis of sanitary sewer capacity in a sewer district known as Relief Sewer District No. 10." The capacities found desirable for this district of 732 acres were as follows (Table I-1):

TABLE I-1

Los Angeles Relief Sanitary Sewer Capacity

(Relief Sewer District No. 10)

	Sewer Location	Area Acres	Est. Sewage Flow cfs	Av. Slope %	Req'd Pipe Diam. inch	St'd Design Capacity cfs	Sewer Capacity Flowing Full cfs	Sewer Capacity Flowing Full g.a.d.
a.	Sierra & Rolle	95	0.50	2.5	8*	0.86**	1.72	11,700
b.	Bway & Gates	279	1.98	2.25	12	2.50**	5.00	11,600
c.	Griffin S/O Ave 28	91	0.80	2.5	8*	0.86**	1.72	12,200
d.	Workman S/O Bway	539	5.22	3.0	15	5.35**	10.7	12,800
e.	Mozart & Pasadena	639	6.54	0.5	18	6.62#	7.28	7,400
f.	" & end of area 6	732	7.94	0.35	21	8.45#	9.30	8,200

* Min. pipe size 8-in. for main sewers.

**Standard L. A. design—Flowing 1/2 full.

Standard L. A. design—Flowing 3/4 full.

These proposed sewer capacities, g.a.d. flowing full, follow closely the Bloomington, Ind., curve for areas greater than 250 acres (Figure III, Curves 7b and 4a). The unit per acre sewer capacities (g.a.d.) for 91 and 95 acres are lower than the Bloomington Curve 4a, but compare closely to the 1928 Bradbury curve for Franklin County, Ohio (Fig. III, Curve 5).

I-8) *Dallas, Tex.*—Mr. R. E. Morris, Jr., Engineer of Design, has furnished (by letter, Sept. 12, 1949) the following design practice data for Dallas' sanitary sewers:

a) *Residential areas of 10 to 1000 acres*

Domestic flow	150 g.p.c.d.	
Infiltration	25 g.p.c.d.	
Storm flow	400 g.p.c.d.	Total 575 g.p.c.d.

Average population density 10/A ∴ design capacity = *5750 g.a.d.*

b) *Residential areas of 1000 to 10,000 acres*

Same, except storm flow reduced to 280 g.p.c.d.
So design capacity = *4550 g.a.d.*

These sanitary sewer design capacities (Figure III, Curve 8) compare closely with the Los Angeles single-house residential curve for smaller areas and with the Phoenix curve for larger areas.

Additional capacity is provided for commercial and industrial areas as follows:

Commercial Areas—	Downtown	30,000 g.a.d.
	Outlying	4,500 g.a.d.
Industrial Areas—		4,000 g.a.d.

I-9) *Washington Suburban Sanitary District*—Incorporated by General Assembly of Maryland in 1918—lies adjacent to Washington, D. C. and includes relatively little industrial or commercial development. Mr. Harry R. Hall, Chief Engineer, reports (letter, Jan. 23, 1946) their sewer design practice as follows:

a) *Population Density*

5 per acre for small estates
18 per acre for general suburban development
62 per acre for apartment areas

b) *Average per Capita Sewage Flows* at 100 gallons per day.

c) *Infiltration*—at 400 g.a.d. ground water leakage plus 3524 g.a.d. storm water (for two uncovered 30 square feet areaways per acre) in residential areas.

d) *Sewer Capacities* are determined from the above computed average sewage flows by means of a chart (Figure II) which provides a sewer capacity decreasing from 4.00 times the computed average sewage flows less than 0.2 M.G.D. to 2.00 times the computed average sewage flows greater than 16.0 M.G.D.

The resulting unit sewer capacity quantities per acre for the light and general residential districts (Figure III, Curves 9 and 9a) compare quite well with Columbus, Ohio, and Louisville, Ky., experience. The apartment house sewer capacities (Figure III, Curve 9b) are higher than the Lynn curve, but compare closely with capacities recently provided in sewers at Baltimore (Figure III, Curve 10).

I-10) *Baltimore, Md.*—Mr. John J. Hunt, Sewerage Engineer, (and his predecessor) has furnished (letters, 25 July 1949 and Jan. 18, 1946) liberal data on their sanitary sewer design practice. Some basic items are as follows:

a) *Population Densities* (based on net area—generally 70% of total area)

Suburban cottage property	20 to 50 per acre
Suburban property—groups	50 to 70 per acre
Closely built—small houses	100 to 135 per acre

b) *Unit Average Daily Sewage Flows*

Residential areas	135 g.p.c.d.
Mercantile areas	135 g.p.c.d.
Industrial areas (residents)	135 g.p.c.d.
Industrial wastes	10,000 g.a.d.
Infiltration (trunk sewers)	500 to 1,000 g.a.d.

c) *Sanitary Sewer Capacity*—computed from average sewer flows by multiplying by a maximum flow factor from a chart (Figure I). Computed industrial flows are not multiplied by a maximum flow factor.

d) *Sewer Characteristics*

Minimum lateral sewer size = 8-inch minimum grade at 0.8 per cent (8/1000).

Kutter's coefficient "n" used for design.

Vitrified pipe drains	0.013
Concrete pipe and reinforced concrete with brick inverts	0.015
Reinforced concrete with clay tile inverts	0.014

e) *A typical computation*

Data:

Area—Residential 60 acres; Industrial 20 acres; Total 80 acres.

*Population/*Acre 8 dwellings at 5 = 40 people per acre.

Runoff

$$\frac{60x40x135}{646,300} = 0.50 \text{ c.f.s. average flow—residential}$$

$$\frac{20x10,000}{646,300} = 0.31 \text{ c.f.s. average flow—industrial}$$

0.50x3.65 (Max. flow factor) - - - = 1.83 c.f.s. Residential
+0.31 c.f.s. Industrial
2.14 c.f.s. Total (80A.)
= 2.67 c.f.s. per 100 acres
or 17,250 g.a.d.

f) *Illustrative Actual Sewer Capacities per Acre*

Mr. Hunt was good enough to furnish actual sewer capacities serving areas of varying size from 181 acres up to 70,310 acres which data are summarized in Table I-2.

TABLE I-2

Baltimore Sewer Capacities—Existing Sanitary Sewers
(Illustrative Examples—based on sewer size and slope)

Sewer	Drainage Area Type	Drainage Area Acres	Sewer Capacity cfs/100A	Sewer Capacity g.a.d
Hoffman Trunk	Residential	181	4.3	28,000*
Reed-Bird Interceptor	"	361	3.3	21,500*
Reed-Bird Interceptor at T.P.	"	561	3.2	20,700*
Low Level Main	Res. & Industrial	11,600	1.65	10.650
High Level Interceptor	" " "	39.064	0.53	3.460
Outfall at H. L. Interceptor	" " "	69,035	0.58	3,750
" " Pul. Hwy.	" " "	70,310	0.6	3,870

*Figure III, Curve 10.

I-11) *Madison, Wisconsin*—Mr. H. O. Lord, Chief Engineer, Madison Metropolitan Sewerage District, reports (letter, Nov. 1, 1949) that basis of design used for Nakoma district is fairly representative of Madison sewer design practice for heavily developed areas. This basis of design was as follows:

a) Population density ..... 30 per acre
b) Max. hourly sewage flow ..... 300 g.p.c.p.d.
c) Max. rate of sewage flow .....9,000 g.a.d.
d) Infiltration .....2,000 g.a.d.
e) Sewer capacity flow .....11,000 g.a.d.
or 1.7 c.f.s. per 100 acres.

Madison experience is of particular interest as all sewage has been pumped one or more times and an effort has been made to keep infiltration down. The hilly terrain also assists in keeping storm water out of sanitary sewers, as compared to Columbus where, in certain cases, Bradbury's rates proved to be low. Earlier design data are given in Davis' "Handbook of Applied Hydraulics" p. 879, Ed. 1.

I-12) *City of Milwaukee, Wis.*—Mr. Lloyd D. Knapp, City Engineer (Formerly Supt. of Sewers), has reported (letter, Dec. 26, 1945) that the earlier specified basis of sanitary sewer capacity published in Davis' "Handbook of Applied Hydraulics," p. 879, has not allowed enough capacity for ground water infiltration into sanitary sewers, which he reported to be as much as 5000 g.a.d. at times. He has computed sanitary sewer capacity flows as follows:

a) Population density .....20 per acre
b) Per capita flow .....125 gal per day
c) Assuming flow occurs in 12 hrs. .....2,500 gal/Ac. for 12 hrs
d) Add ½ infiltration rate .....2,500 gal/Ac. for 12 hrs

Total .....5,000 gal/Ac. for 12 hrs

Equivalent to .01543 c.f.s./Ac.

e) Adding a small safety factor
use for design .....02 c.f.s./Ac.
Equivalent to 2.0 c.f.s./100Ac. or 12,920 g.a.d.
f) In commercial areas 0.0936 c.f.s./A (60,500 g.a.d.) has been used.

This more recent basis for capacity design of sanitary sewers corresponds closely to the Los Angeles relief sewers for District #10 (Figure III, Curve 7b).

The Rules and Regulations adopted by the Milwaukee Sewerage Commissioners in 1923 provided for not less than 20 population per acre at 125 gallons per capita per day plus a maximum infiltration of 810 g.a.d. which basis amounted to 3310 g.a.d. More recent experience has indicated this to be too low.

Capacity allowances for industrial areas have been determined

from a curve prepared by the Sewerage Commission of the City of Milwaukee with sewage flows per acre decreasing from 0.375 c.f.s./A (242,000 g.a.d.) for 5 acres or less down to 0.015 c.f.s./A (9700 g.a.d.) for 1000 acres (see Table 4). Double these quantities are used within 1916 city limits.

I-13) *Rochester, N. Y.*—Mr. Kenneth J. Knapp, City Engineer, reported (letter, Aug. 8, 1949) that their sanitary sewer capacities were determined on basis of New York State Department of Health rules. No sewers less than 8-inch diameter with minimum velocity of 2.5 feet per second (Rochester prefers 3.0 f.p.s.), and with sewer capacities determined as follows:

a) Population density 4.3 persons/lot and 5.5 lots/A ..... 23.65 per acre
b) Sewage flow (max.) Lateral sewers..... 400 gay/cap/dy
Trunk & outfall..... 250 gal/cap/dy
c) Sewer capacity flows (lateral) ..... 9460 g.a.d. (or 1.46 c.f.s./100A)
d) Sewer capacity flows (trunk & outfall)..... 5912 g.a.d. (or 0.92 c.f.s./100A)

These capacity rates are considerably lower than have been found desirable at Columbus. The lateral sewer capacity basis compares closely with the Dallas, Tex., capacity for smaller areas (Figure III, Curve 8). Also see Painesville sewer capacities in the following paragraph.

I-14) *Painesville, Ohio*—Wm. L. Havens (Havens & Emerson) reports (letter, Oct. 26, 1949) the basis of capacity design for new sanitary sewers (May 1947) as follows:

a) Population density ..... 15 per acre

	Average Rate	*Maximum Rate*
b) Domestic and commerical	60 g.p.c.d.	90 g.p.c.d.
c) Railroad and industrial	15 g.p.c.d.	35 g.p.c.d.
d) Ground water infiltration	750 g.a.d.	1125 g.a.d.
e) Roof water allowance.	—	400 g.p.c.d.
Totals (Pop. 15/A)	125 g.p.c.d.	600 g.p.c.l.
Sewer capacity for design		9000 g.a.d.
Equivalent to ..		1.39 c.f.s./100A

I-15) *Saint Paul, Minn.*—Mr. Charles M. Colestock, Sewer Designer, has reported (letter, Aug. 30, 1949) that St. Paul Sewer

System is 95-98% combined, and that sanitary sewers, as are built, must provide capacity for roof drainage (Building Code required roof leaders connect to house sewers) and some storm water inlets. The routine capacity basis of design is 1/4 c.f.s./A (25 c.f.s./100A or 161,500 g.a.d.). This capacity basis compares, for small drainage areas, with Waukegan, Ill., roof water sewers (Figure III, Curve 3).

I-16) *Springfield, Mass.*—Mr. Richard E. Dudley, Deputy Superintendent, Engineering Division, has reported (letter, Oct. 31, 1949) their general sanitary sewer capacity design practice as follows:

Population density—75/A for 10 A or less to
30/A for 1000 A (following a straight line change on log log paper)
Per capita (max.) flow ..... 200 gal/dy
Infiltration .....2,000 g.a.d.

This basis gives unit per acre capacities decreasing from 17,000 g.a.d. for 10 A or less to 8000 g.a.d. for 1000 A (or 2.63 to 1.24 c.f.s. per 100 A) which corresponds closely to Bradbury's Curve for Franklin County, Ohio (Figure III, Curve 5).

A minimum sewer size of 10 inches diameter at 0.40% grade has been adopted. A coefficient "n" of 0.015 is used.

A recent sewer design, now under construction for a large (4330 acres) partially developed residential area, zoned for single family residences with minimum lots of 8250 square feet, was based on the following:

a) Population density 15/A
b) Sewage flow .....150 g.p.c.d.
c) Infiltration .....2,000 g.a.d.

Total sewer capacity .....4,250 g.a.d.
(or 0.66 c.f.s./100Ac.)

Figure III shows this unit per acre capacity (4250 g.a.d.) to be relatively low; comparing closely with sewers for drier areas of Dallas and Phoenix. It compares with the lower basis of sewer capacity design in some residential Boston Suburban Towns.

I-17) *Cranston, R. I.*—Mr. Ralph Horne has published (Jour., B.S.C.E. *XXX*, p. 66, April 1943) the basis of the capacity design for an entirely new sanitary sewer system; the summary capacity

design data are repeated here, for comparison with the other data, in the following:

a) Population density ..... 30 per acre
b) Max. per capita per day flows 167
c) Commercial areas flows ..... 25,000 g.a.d.
d) Industrial areas flows ..... 20,000 g.a.d.
e) Infiltration, per mile of sewer ..... 25,000 gal/dy
600 g.a.d.

Indicated per acre sewer capacity for residential areas ..... 5010 g.a.d.

Illustrative computations published by Horne give actual per acre sanitary sewer capacities as follows:

Acres	Per Acre Sewer Capacities g.a.d.	c.f.s./100A
49	10,500	1.63
86	10,700	1.65
115	10,900	1.68
178	10,300	1.59

I-18) *New York City*—Mr. Richard H. Gould has advised (letter July 29, 1949) that the five Engineering Offices of the Borough Presidents have jurisdiction over sewer design. Mr. Gould has summarized the practice of the 5 Boroughs and the City Department of Public Works, with reference to capacity design of sanitary sewers, as follows:

a) Chezy-Kutter formula used by all Boroughs and the City Department of Public Works with Kutter's "n"
= 0.013 for vitr. pipe sewers
= 0.015 for conc. pipe and monolithic sewers

b) Sewers designed to flow part full:

Manhattan ..... 0.92 full

Brooklyn ..... 0.5 full for 8 to 15-inch
0.7 full for 18-inch to 60-inch
& full for sewers 66-inch & over

Bronx ..... 0.5 full for sanitary sewers

Queens, Richmond & Public Works ..... 0.8 full

c) Domestic sewage flows

Borough	Population per acre	Water cons. g.a.c.d.	Infiltration allowance	Maximum d.w.f.
Manhattan	*	100 to 125	‡	2 m.d.w.f. pumped 3 m.d.w.f. gravity
Brooklyn	100	100	‡	
Bronx	125	140	1940 to 3240 g.a.d. or 0.3 to 0.5 cfs/100A.	2 to 4 m.d.w.f.
Queens	75	100	750,000 to 1,000,000 gal/sq mi/dy = 1170 to 1560 g.a.d.	do
Public Works	*	100 Min.	‡	do

*All sewer bureaus, particularly Public Works, make extensive studies of each outlet area, including sewage flow gagings.

‡Brooklyn uses 20,000 gal/mi/dy for inverts 0 to 5′ below water and 40,000 gal/mi/dy for inverts 5 to 10′ below water.

Manhattan and Public Works make no special allowance for infiltration since this is included in the gagings.

The Bronx Borough—Arthur V. Sheridan, Commissioner of Borough Works, has reported (letter Nov. 18, 1949) that combined sewers are used for all except about 2% of the Borough. The total area of about 26,500 acres (41 sq. mi.) has population in excess of 1,500,000. Population densities used for sewer design in the past were 125 to 200 per acre. Recently in small areas like Parkchester (about 2% of the total area) densities of 400 persons per acre have been required.

Formerly 140 gallons per capita per day was considered adequate. Now it is possible larger per capita rates would be used. Infiltration allowance of 0.0033 c.f.s. per acre (2132 g.a.d.) was used in the Parkchester area.

Sanitary Sewers have been designed to flow one-half full with a maximum dry weather flow taken at 2 times the average. These basic procedural data would indicate the following possible per acre capacities (sewer flowing full):

1. *Formerly*—at 125 persons and 140 gallons per capita = 72,000 g.a.d.
   [(125 x 140 x 2 x 2) = 70,000 + infiltration (2100 g.a.d.)]
2. *Parkchester* (400 x 150 x 2 x 2) + 2100 g.a.d. = 242,100 g.a.d.

Queens Borough—President James A. Burke (letter Nov. 4, 1949) has advised):

1. That 42.6% of the area is served by Sanitary Sewers with 86.5% of the dwellings of one- and two-family type.
2. That infiltration of 750,000 gal. per sq. mi. (1170 g.a.d.) is used for sewers above water table, and 1,000,000 gal. per sq. mi. (1560 g.a.d.) for sewers below water table.
3. That average sewage flows have been computed on basis of 75 persons per acre, 100 gal/cap/dy equal to 0.0134 c.f.s. per acre. This factor times acres tributary gives a mean dry weather flow (M.D.W.F.) which is taken to a chart (Figure II) to obtain the proper sewer capacity in c.f.s. This Queens Borough curve for ratio of sewer capacity to M.D.W.F., dated February 1925, corresponds closely with the Washington Suburban Sanitary District curve (Figure II). Mr. Burke also presented a log log curve for Borough of Queens showing per acre sewer capacities decreasing from 34,000 g.a.d. for 20 acres, or less, to 17,500 g.a.d. for 1500 acres and up to 3000 acres. This is somewhat less than used for apartment house areas in the Washington Suburban Sanitary District (Figure III, Curve 9b).

Brooklyn Borough—Mr. John J. Lynch, Commissioner of Borough Works (letter Nov. 7, 1949) has furnished a memorandum by Mr. Charles A. Riedel, Chief Engineer of Sewers, which stated:

1. Sanitary Sewers served 1/3 Borough (Brooklyn) area with population densities 100 to 350.
2. Maximum per capita sewage flow 150 gallons.
3. Smallest sewer size 10-inch which drains about 25 acres at minimum slope of 0.38% giving a 2.25 f.p.s. velocity.
4. Illustrative sanitary sewer capacities—based on minimum slopes to give 2.25 f.p.s. velocity, the sewer sizes would be required as follows:

Acres	Diam. inches	Unit Capacity—per Acre c.f.s./100A	g.a.d.
100	24	7.0	45,000
300	36	5.3	34,000
500	42	4.3	28,000
700	48	4.0	26,000
1000	60	4.3	28,000

Comparing with Figure III these per acre capacities are about 35% higher than Washington Suburban Sanitary District Apartment house areas (Figure III, Curve 9b).

Richmond Borough—President Cornelius A. Hall (letter, Jan. 9, 1950) has advised that the basis of sanitary sewer capacity is:

a. Population density, 60 persons per acre.

b. Max. per capita per day—115 gallons.

c. Infiltration—gallons per mile daily.
   (1) 40,000—where sewer invert is more than 5 feet below ground water table.
   (2) 20,000—where sewer is 0 to 5 feet below ground water table.
   (3) Nil—where sewer is above water table.

d. *Sewer Capacity*—The computed rate of sewage flow is 1.5 times average rate of water supply with ground water infiltration added. When the computed rate of sewage flow is greater than 6½ mgd, the sewer capacity is taken twice the computed flow. When the computed flow is less than 6½ mgd. or 10 c.f.s., the ratio of sewer capacity to computed sewage flow ranges from 2 to 4, based on a curve signed by Kenneth Allen dated March 1923 which is quite similar to the Baltimore, Washington Suburban Sanitary District, and Borough of Queens curves (Figures I and II).

e. Illustrative sewer capacity quantities—omitting infiltration:

Area Acres	Unit Sewer Capacity — per Acre c.f.s./100A*	g.a.d.*
10	4.24	27,400
100	3.32	21,400
300	2.73	17,600
500	2.41	15,600
700	2.23	14,400
1000	2.04	13,200

*Infiltration must be added, ranging from 20,000 to 40,000 gallons per mile daily.

I-19) *Boston Suburban Towns*—Messrs. George F. Brousseau, Joseph W. Kales, Walter A. Devine and Ashley Q. Robinson, Engineers of Braintree, Belmont, Brooklyn and Newton (city), have very kindly furnished data on their sewer design practice relative to sanitary sewer capacities. These have been summarized in Table I-3.

TABLE I-3

Sanitary Sewer Capacities—Boston Suburban Towns

Item	Braintree	Belmont	Brookline	Newton City High level Sewer 1932	Newton City Other Areas# Lower	Newton City Other Areas# Upper
Pop. density	10-20	30-50	100	13.5	10	20
Max. p.c.p.d.	200	150	300	419	150	150
Allow for Com. Area	None	None	Included†	—	—	—
Allow for Com. Area	None	None	Included	—	—	—
Infiltration g.a.d.	2000	*	Included	—	3500	350
Sewer cap, g.a.d.	4000-6000	5000 up	30,000	5660	5000	6500
Sewer cap. cfs/100Ac.	0.62-0.93	0.77	4.65	0.88	0.77	1.0
Min. Sewer Size	8″	8″	10″	—	—	—
Min. Sewer Grade—	6/1000	5/1000	5/1000	—	—	—
Absolute Min.	3.5/1000					

*600 to 800 gallons per day per mile per inch diameter of sewer.

†Included in per capita flow—sewer capacity determined by a chart giving c.f.s./A for various population per acre.

#Newton capacity basis—g.a.d.—as follows:

	Single Houses	Double Houses	Mfg. Areas g.a.d.
Domestic Sewage	1500	3000	1,500
Ground Water	2000	2000	2,000
Storm Water	1500	1500	1,500
Industrial Flows	—	—	10,000
	5000	6500	15,000

APPENDIX II—ILLUSTRATIVE RUN-OFF COEFFICIENTS USED IN VARIOUS CITIES FOR STORM SEWER DESIGN

The runoff-rainfall ratio factors or runoff coefficients used by several city storm designers are briefly reported herein.

Baltimore, Md.—(John J. Hunt, Sewerage Engineer).

Area Classification	"C" Values
Unoccupied land	40
Cottage development—large lots	50
Cottage development—50-ft lots	60
Cottages and row houses interspersed	70
Row houses in groups	75
Row houses with no separation	80 to 90

Note:—Baltimore, the Baltimore County Metropolitan District, and the Maryland State Roads Commission have initiated a storm water research.

Boston, Mass.—(John F. Flaherty, Senior Civil Engineer).

Bryant and Kuichling's 1909 report gives following generally used runoff coefficients:

Surface Classification	"C" Values
Watertight roof surfaces	0.70 to 0.95
Asphalt pavements in good order	0.85 to 0.90
Block pavements with cemented joints	0.75 to 0.85
Block pavements with uncemented joints	0.50 to 0.70
Block pavements with open joints	0.40 to 0.50
Macadamized roadways	0.25 to 0.60
Gravel roadways and walks	0.15 to 0.30
Unpaved surfaces, vacant lots, R.R. yards	0.10 to 0.30
Parks, lawns, gardens, etc., depending on surface, slope, and soil	0.05 to 0.25
Wooded areas, dep'g on surface, slope, and soil characteristics	0.01 to 0.20

Buffalo, N. Y.—(Cecil F. Seitz, Chief Engineer, Buffalo Sewer Authority).

"C" values vary from 0.30 to 0.95 increasing from suburban area to the downtown district. A Report on Comprehensive Plan for Relief Sewers (1938), by Greeley & Hansen, proposed:

C = 0.80 for impervious areas
C = 0.30 for pervious areas

Related to specific zoned areas the proposed resulting coefficients were as follows:

Zoned as to Per Cent of Area to be Used		Runoff Coef. Related to Both % and Type of Use			
Type of Area	Allowable Bldg. % of Area	Residential	Apartments	Commercial	Industrial*
A	90%	—	0.65	0.70	0.60
B	55%	0.58	0.60	0.60	0.60
C	40%	0.53	—	—	0.55
D	25%	0.48	—	—	—

*Industrial areas are often special cases. The indicated coefficients are minimums.

Chicago, Ill. (A. L. Tholin, Eng. of Sewer Design)—A brief list of runoff coefficients includes:

a) Roofs of major structures (omitting small detached buildings not connected to sewers) 0.95

b) Street and alley pavements 0.85

c) Public walks (continuously attached to pavements) 0.85

d) Large parking lots—Paved 0.75
Unpaved 0.40

e) Large public parks 0.15*

f) Large railroad yards 0.15*

g) Forest Preserves 0.10*

h) All other areas (detached sidewalks, parkways, lawns, storage yards, vacant lots and minor roofs not connected to sewers) 0.20*

*Pervious area coefficients based on flat terrain, typical of most of Chicago. Coefficients for rolling terrain, such as Beverly and Morgan Park, given special attention.

Tholin furnished data also on factors "F" introduced as modifying factors applied to "C" coefficients, which he says were suggested years ago by Gregory because rainfall intensities for small areas are so great that sewer sizes become unnecessarily large. A chart (File No. DC-3) records the 17 curves from several sources. The curve (No. 17) selected for Chicago indicates the following values of "F" (scaled from DC-3) related to storm duration:

Storm Duration Minutes	Value of "F" in Q = FCiA
0	0.23
10	0.46
20	0.64
30	0.745
40	0.81
50	0.85
60	0.875
80	0.92
100	0.96
140	1.00

Cincinnati, O. (H. H. Kranz, City Engr.)—Coefficients primarily predicated upon type of development permitted by Zoning and Use regulations, using 0.9 for roof areas; 0.7 for sidewalks, driveways and paved areas; and 0.2 for unpaved areas, yards, and lawns. These computations result in over-all runoff coefficients of:

0.35 —single family houses on ordinary size lots.
0.40 —single family, duplex, and scattered apartment houses.
0.5 to 0.65—apartment house areas.
0.7 to 0.9 —retail business and downtown areas.

Cleveland, O. (John C. Wenrick, Comm. of Engng.)—The coefficients of runoff being used were as follows:

"f" = 0.60 Residential areas
f = 0.25 Park areas or railroad yards
f = 1.00 Congested areas at center of city

Recently (February 1952), the Regional Planning Commission, Cuyahoga County, Ohio, published an "Engineering Design Manual for Sewerage and Drainage Facilities" in which, after precautionary "assumptions" appears a table of "C" values for various "typical" areas during storms of *various duration* as follows:

"C" in Per Cent of Rainfall Intensity Rate

Frequency in Years	Pervious Areas* 1.0	3.0	5.0	10	Impervious Areas
Duration in Minutes					
10	51	62	67	75	75
20	47	60	66	72	80
30	43	58	65	70	85
40	40	56	63	69	90
50	35	54	61	68	95
60	32	52	60	67	95
90	31	49	58	66	95
120	27	47	56	64	95

*The decrease in runoff percentages with increase in storm duration is not explained in the "Design Manual."

Also,

Proportions: Pervious and Impervious Areas

	Commercial Industrial	Urban Residential	Suburban Residential	Rural Areas
Pervious	10	65	75	90
Impervious	90	35	25	10

These tables are followed by a note and seven more "assumptions from which the values in the above article were determined. . . ."

Columbus, O. (John A. Rousculp, Engr. Sewer Design)—Rousculp has published a study ("Relation of Rainfall and Run-off to Cost of Sewers," Trans. ASCE, Vol. 104 (1939), p. 1485) in which he shows runoff coefficients and relative costs of sewers from which the following values have been scaled:

Runoff Coefficient	Relative Cost
30	1.00
40	1.15
50	1.28

Des Moines, Ia. (A. S. Jones, Design Engr.)—In the rational formula $Q = CiA$ the coefficient $C = \Sigma (R \times D)$ where factors "R" and "D" are determined from type of district development and type of

surface. Ground slopes also affect values of "D". The following are illustrative values:

I—Values of "R" = Ratio of Impervious to Total Area

Type of Development in District		Values "R"
a) Commercial		1.00
b) Heavy Industrial		0.80
c) Suburban Bus. & Apart. Houses		
Pop. Density	= 30 + /Ac	0.80
d) Residences—Closely Spaced Pop.	= 25 — 30/A	0.65
e) Residences—Suburban Pop/A	= 20 — 25	0.46
" "	= 15 — 20	0.38
" "	= 10 — 15	0.32
" "	= 5 — 10	0.20
f) Parks & Large Estates Pop/a	= 0 — 5	0.10

II—Values of Factor "D" in Equation— "C" = Σ(RxD)

	"D" Values for			
	Flat Slopes* Conc. Time		Steep Slopes* Conc. Time	
Type of Surface	20 or less min.	more than 20 min.	20 or less min.	more than 20 min.
1—Roofs and streets paved with concrete	.70	.80	.85	.95
2—Brick and asphale pavement. Br. walks	.65	.70	.75	.85
3—Macadamized roads and drives	.60	.70	.70	.80
4—Gravel roads and walks	.50	.60	.65	.70
5—Unpaved str. and playgrounds	.45	.50	.60	.70
6—Loose bare soil—gardens and fields	.25	.35	.40	.50
7—Lawns & parks—heavy clay soil	.05	.08	.10	.15
8—Lawns & parks—sandy soil	.03	.07	.08	.12
9—Yards—average residences	.10	.15	.15	.20

*Flat slopes less than 3% — Steep slopes more than 3%.

Lansing, Mich. (K. B. Fishbeck, San. Engr.)—Runoff coefficients determined by formulas:

Pervious areas — $C = 0.3t/(20+t)$
Impervious areas — $C = t/(8+t)$

Percentage of impervious areas about 37.5 in residential sections and about 100% in commercial and industrial sections.

Milwaukee County Metro. District (Jas. L. Ferebee, Chief Engr. (formerly)).

Type of District	Coefficient "C"	Inlet Time	Runoff Factor cfs/Acre*
a) Most densely built up (commer.)	0.75	5	2.62
b) Adjoining densely built up	0.65	6	2.27
c) Residential—well built up	0.53	7	1.92
d) Adjoining built up residential	0.45	9	1.57
e) Suburban areas	0.30	10	1.05

*Flow rates, uniform cfs/Acre, for initial 15 minutes.

The above reduced runoff quantities for the first 15 minutes time of concentration have similar effect as the "K" factors proposed in Chicago, excepting they apply only for initial 15 minutes' storm duration time. Whereas the Chicago "K" factors apply to storm durations (= time of concentration) of 0 to 140 minutes.

Lloyd D. Knapp (Supt., Bur. of Sewers, City of Milwaukee, Dec. 1945) stated the City of Milwaukee applied the Metropolitan factors to a steeper rainfall curve (see Figure V) and added the computed increment of flow to the computed flow already in the sewer. This procedure results in lower "i" values, but higher sewer flows related to the "i" values.

Pittsburgh, Pa. (July 1949) had a standard set up for runoff coefficients "C" related to per cent impervious area as follows:

Per Cent Impervious Area	Runoff Coefficient Flat Dist. Slopes less than 5%	Hill Dist. Slopes more than 5%	
100	.9	.9	——Downtown Triangle
90	.82	.83	
80	.74	.77	
70	.66	.70	
60	.58	.64	
50	.50	.57	
40	.42	.51	
30	.34	.44	
20	.27	.38	
0 to 10	.20	.30	——Parks

Rochester, N. Y. (K. J. Knapp, City Engr.)—Some years ago city zoned by ordinance. "C" values worked out for different types of uses as follows:

Type of District	"C"
a) Central traffic area	90%
b) A.B.C.D. light industrial	60
c) D commercial	60
d) A.B.C. commercial	50
e) Heavy commercial	70
f) A.B.C. residential	40
g) D residential	33 1/3
h) E residential	25

Knapp stated "The trouble (problem) with the design of storm water sewers in this city has been the selection of the proper coefficient to serve partially developed and undeveloped territory."

San Francisco, Cal. (R. G. Wadsworth, City Engr.)—Mimeograph Form X-1481, 12/10/45.

Type of Area	Runoff Coef. "C" (Proposed for 1946)
a) Industrial	60-90
b) Commercial	80-90
c) Apts. and Flats	60-75
d) Residential—Attach. houses	45-60
e) Residential—Detach. houses	40-50
f) Suburban	25-35
g) Parks	10-20

Springfield, Mass. (F. C. Collins, Asst. Deputy Engineer).

Runoff Coefficients Adopted

Duration of Rainfall Min.	Impervious Areas Paved & Roofed Areas Nonabsorptive	Pervious Areas Absorptive or Grassed and Sandy Areas
5	.50	.01
10	.60	.03
15	.70	.05
20	.80	.07
30	.85	.10
45	.90	.15
60	.95	.20
75	.95	.25

City of St. Louis, Mo. (E. J. A. Gain, Engineer in Charge of Sewer Design).

$$P = \text{ratio } \frac{\text{Runoff}}{\text{Rainfall}} \quad \text{i.e. same as C in } Q = CiA$$

P = Factor for Runoff

Per Cent Impervious	Duration of Rain — Minutes					
	15	20	30	60	90	120
0	.30	.35	.41	.51	.56	.60
10	.34	.39	.455	.555	.60	.635
20	.38	.435	.50	.595	.635	.67
30	.42	.475	.54	.64	.675	.705
40	.46	.52	.585	.68	.715	.74
50	.50	.56	.63	.725	.75	.775
60	.54	.60	.675	.77	.79	.81
70	.58	.645	.72	.81	.83	.845
80	.62	.685	.76	.855	.87	.88
90	.66	.73	.805	.90	.905	.915
100	.70	.77	.85	.94	.945	.95

Q = PI = runoff cfs/acre (P as given above — I is rainfall intensity)

Toledo, Ohio (H. A. Nelson, Division Engineer—Engineering and Construction).

Runoff coefficients—0.3 for residential areas
0.9 for congested business districts

Topeka, Kans. (W. E. Baldry, City Engineer).

Q = AIR I = C = percentage runoff
= 30% for parking and ground surfaces
= 90% for pavements and roof areas

Summary

The foregoing items are representative practice in a few cities. Many variations are found in practice, in the effort of engineers to relate runoff to rainfall intensities. In some cases the intensities are adjusted with reference to area as well as duration of storm. These data are not complete but are presented as illustrations only.

APPENDIX III—STORM SEWERS—AREA-TIME OF CONCENTRATION CURVES

Data for the curves of Figure IV were obtained as follows:

(1) Chicago—"Approx. Mean Curve"—Bureau of Sewers
Courtesy—A. L. Tholin, Engr. of Sewer Design (1950)

(2) Cleveland—Relief Sewers Computations
Courtesy—E. C. Richardson, Engr. of Sewer Design (1949)

(3) Columbus—"Relation of Rainfall and Run-off to Cost of Sewers" by John A. Rousculp—T.A.S.C.E. *104,* p. 1475 (1939)

(4) Milwaukee—"Typical Storm System Computations"
Courtesy—Chief Engr. Sewerage Comm. City of Milwaukee (1949)

(5) St. Louis—Design Values—Clarendon and Manchester Rd. Distr.
Courtesy—E. J. A. Gains, Engr. of Sewer Design (1949)

(6) San Francisco—Sample computations—Lake St. Sewer
Courtesy—Ralph G. Wadsworth, City Engr. (1949)

(7) Springfield—Sample computations—Alden St. and Wyckoff Ave. in Res. areas
Courtesy—F. C. Collins, Asst. Deputy Engr. (1946)

(8) Toronto, Ont.—Check Computations—Old Crescent Rd. Sewer
Courtesy—M. A. Stewart, Comm. of Works (1949)

(9) Storm Sewer Design Practice by Fay, Spofford & Thorndike, Boston—Illustrative office methods
Courtesy—R. W. Horne, Partner (1946)

THE FIELD OF ENDEAVOR OF THE CONSULTING ENGINEER

By Thomas R. Camp,* *Member*

(Presented at New England Conference of Student Chapters ASCE on May 2, 1953.)

The sponsors of your conference have graciously invited me to present to you a long-range employment picture in the field of the consulting engineer. I am honored to have been selected for this task, but I hope that you will not expect too much from me as a prophet of what the future holds for you. As I look back over my own career, I find that most of the things which have happened were unpredictable.

Change we have with us always. Chance will influence our careers as much as any other single factor. Nevertheless, successful careers are those with a purpose. More than anything else, the world needs leadership. Successful leadership in any field requires definite aims and purposes. The most successful engineers are usually those who have selected their specialized field wisely and at an early age, and who have stuck by their selections steadfastly.

As most of you know, the earliest engineers were military engineers. The term "civil engineer" was adopted more than 100 years ago to differentiate between engineers whose activities were primarily with civil works from the older military engineers. The first consulting engineers were civil engineers. In the early days most civil engineers were in private practice, and the term "consulting" was reserved for a select small group at the top of their profession whose principal activity was consultation and advice. Nowadays nearly all engineers in private practice are called consulting engineers to differentiate them from engineers in the employ of others.

Strictly speaking, a consulting engineer is a specialist in some branch of engineering who is in practice for himself serving clients on specific projects or assignments for which services he receives a fee arrived at by agreement with the client. To assist him in his work the consulting engineer employs other engineers, draftsmen, field engineers and office assistants. The consulting engineer, himself,

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however, is fully responsible for his acts and those of his employees. Most state registration laws for engineers accept this definition of the individual responsibility of the engineer.

During my lifetime engineering projects have become so vast in scope that no single consulting engineer has sufficient knowledge to be expert in all the complexities of large projects. To overcome this difficulty and to permit larger organizations, engineering partnerships have been formed. In many cases each partner of a firm is a specialist in his own right with a specialty differing somewhat from that of the other partners. To maintain the personal professional relationship with the client, one of the partners usually acts as principal on a project.

The work of the consulting engineer is usually divided into three phases: first, the preliminary investigation and report to determine the need, feasibility and cost of a project; second, the preparation of detailed contract drawings, specifications and contract documents for the project, and third, supervision of construction of the project. In all three phases the consulting engineer is the agent of a client who pays him and also pays for the cost of the project. Since the award of the construction contracts is usually made on the basis of competitive bids submitted by contractors and manufacturers of equipment and materials, it is of the utmost importance that the consulting engineer have no interest in any of the bidding firms. In order to protect their unbiased professional standing, most consulting engineers have no interest in any construction activity or manufacturer of equipment or materials which enter into the type of work which he designs.

You have doubtless been taught that engineering is one of the learned professions like medicine, the law and the ministry. Most of us who have been in practice a long time are continuously made aware of the difficulties many engineers have in maintaining their professional status. In order to maintain such a status, one must have a pretty clear idea of what constitutes a learned profession. Technical competence of a high order is a first requisite to membership in any learned profession. This is not enough, however, to differentiate the learned professions from other professions. If we are to aspire to public esteem, the public interest must take precedence over all else.

In this connection I would like to quote from a paper entitled, "Standards of Professional Relations and Conduct", by the late Daniel W. Mead, past-president and honorary member of the American Society of Civil Engineers. Dr. Mead stated:

"It is the duty of the engineer to satisfy himself to the best of his ability that the enterprises with which he becomes identified are of legitimate character. If, after becoming associated with an enterprise, he finds it is of unsound or questionable character, he should sever his connection with it as soon as practicable. The engineer should engage in no occupation nor undertake any project that is contrary to law or which is inimical to the public welfare."

It is evident that the consulting engineer, in contrast with the engineer who is employed on a salary, is in the best position to give leadership in matters affecting the public interest. The consulting engineer, therefore, occupies the top position in the engineering profession and it is his responsibility to lead in the advancement of the profession.

During the past 20 years or so there have been more and more corporations organized for the purpose of practicing engineering. In many ways the corporate structure is advantageous in the development of large organizations such as are required for very large engineering projects. This form of organization, however, poses a threat to the professional status of the engineer. The corporate structure is a means of escape from responsibility. If the corporation is allowed to become too large, it will become a business where profit is the principal motive. The engineers who formerly were the principals of the organization, may find themselves relegated to positions of lesser power and responsibility. Many such corporations which were started in the private practice of engineering have been expanded to include construction and management. These organizations are business firms and not professional firms. Their existence constitutes a threat to the professional status of engineering.

There has been a considerable change in the field of opportunity for the private consulting engineer during my lifetime. When I first entered practice, the automobile age was in its infancy, and there was a great demand for highways and bridges. Very few state highway departments were in operation at that time, however, and most of the work was handled by private consulting engineers. Today,

most states have large and competent highway and bridge departments which do most of the work. Just before World War II, the field of opportunity for the private consulting engineer in highway and bridge engineering was severely limited. Since the end of the war, however, there has been a revival of private engineering activity in highways and bridges because of the demand for toll roads.

During the early years of my career, practically all flood-control work was handled by private consulting engineers or by local districts formed for the purpose. During the depression years, the U. S. Army Engineers entered this field, so that today practically all flood-control work is handled by the U. S. Engineers. The field of opportunity in flood-control work for private engineers is at present severely limited in the United States.

In the early years of my career, irrigation work was also carried on by private consulting engineers and local districts were formed especially for the purpose. Here, also, the Federal government has entered, through the Reclamation Service, to take over practically the whole of the work. The opportunity for private consulting engineers in irrigation work is very limited in this country.

The development of the hydro-electric power industry in America has for the most part been in the hands of private consulting engineers and engineers in the employ of electric utilities. During the past decade or so, however, there has been an effort on the part of the Federal government through the Tennessee Valley Authority, the Federal Power Commission, the Reclamation Service, and the U. S. Army Engineers to enter the field of power production through multiple-purpose dams. This trend has, to a considerable extent, limited the field of opportunity for private consulting engineers in waterpower engineering. There are signs at hand that the present administration in Washington will attempt to reverse this trend. The future may offer greater opportunities to the private consulting engineer in the field of waterpower engineering.

In the field of my specialty, sanitary engineering, which embraces waterworks, sewage works and stream pollution abatement, there has been little or no interference by public agencies with the opportunities of the private consulting engineer. Since the depression years, the field of opportunity for private consulting engineers in sanitary engineering has been growing tremendously. The problems to be solved

have become more numerous, have become larger, and have become more complex. It is an extremely interesting field with a great variety of problems waiting to be solved.

In all of the foregoing fields, much of the work of the engineer is structural. The consulting engineer's value in the field of his specialty, however, is determined in most cases by his grasp of the functional and economic aspects of projects rather than his ability to design structures. Structural engineering is common to all the specialized fields of civil engineering, but in most of the fields it is only a necessary process like drafting. Nevertheless, there are many consulting engineers who make structural engineering their specialty. One field of opportunity is on large bridge projects, but relatively few consultants are specialists in this field. Most consulting structural engineers work with architects on building design. In some cases, they are principals in so-called architect-engineer firms; but in most cases the architect deals directly with the client and the structural engineer is in the employ of the architect. There is not much professional satisfaction in the latter arrangement.

Doubtless you are aware that at the present time there is a great dearth of engineers of all types. You will have no difficulty finding a job, and most of you will be able to choose from several opportunities. Your salaries will probably be good in the immediate future, and many of you will undoubtedly be paid more than you are worth. Let me caution you not to let the salary be the prime consideration in your selection of a job.

In 1927 the American Society of Civil Engineers published its first manual of engineering practice, which was entitled, "Code of Practice". In the preamble it is stated that, "Any code of ethics is founded on the golden rule. A code of practice is an application of a code of ethics." I urge you to study this manual. Its tenets are as good today as they were when the manual was published 26 years ago.

The ASCE Code of Practice is a guide to the private consulting engineer and to other engineers employed by public agencies, utilities and private industry who have to design works and deal with contractors. The contents of the code include, (1) the relations of engineers among themselves, (2) the relations of the engineer with the owner, (3) relations concerning contractors, (4) relations concerning subcontractors and material men, and (5) relations with the public.

The fifth item in the Code of Practice is of such importance today that I am going to end this address by quoting it to you. It reads as follows:

"1. National and local prosperity and development affect the employment of the engineer. Public duty as well as personal interest justify the engineer in advancing worthy public enterprises.

"2. The engineer shall maintain a dignified interest in the welfare of the community and should join and support the local civic associations to the end that the influence of his profession may be felt in all public matters in which his special training, knowledge and experience qualify him to advise.

"3. The engineer shall endeavor to assist the public to arrive at a correct general understanding of the technical phases of public questions. He shall discourage and challenge untrue, unfair and exaggerated statements on technical subjects, especially when such statements may lead to unworthy or uneconomic public enterprises.

"4. The engineer shall accept personally his responsibility as a citizen, assume his share of gratuitous public work for the general good, support public officials in the enforcement of technical regulations, and take an active interest in the formulation and improvement of such regulations.

"5. He shall give consideration to the effect upon the community of every proposed project with which he is connected.

"6. In his attitude toward public technical questions, the engineer shall bear in mind his responsibility, both to his profession and to the public.

"7. Public appreciation of the profession will be increased by the advancement of the technical sciences. Engineers shall promote such advancement by their own efforts and by their encouragement of sound technical training and research.

"8. In addition to the safeguards required of them, engineers shall recognize their broader obligation to provide amply for the safety, health and comfort of the public insofar as affected by their operations."

Gentlemen, it has been a pleasure to address you. I wish you every success in your profession.

NEW DEVELOPMENTS IN CONSTRUCTION

By Col. Jack Singleton*

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on February 11, 1953.)

In speaking to you I am going to do so as an engineer, rather than as a representative of the structural steel fabricating industry. Aside from time out in which to engage in a couple of wars, I have spent my entire business life in engineering, and even in those wars I was an officer of the Corps of Engineers. I expect to devote the remainder of my years to some phase of engineering endeavor. I have a pretty fair idea of what it will be. Certainly I will not be a public speaker who is, as near as I can find out, a professional author who has lost his amateur standing.

I can't quite arrive at a true definition of "New Developments", as such. Isn't it a matter of relativity?

When a lot of us went to school we were bold indeed if we even considered designing a structure according to the principles of continuity. If we did so, about the only tool we had was the Theorem of Three Moments. Slope Deflection and Moment Distribution were as yet undreamed of. Any building over 8 or 10 stories in height was hailed as a skyscraper. Even the theory and application of reinforced concrete in this country is only about 50 years old.

Now, if you measure 50 years as against 3 or 4 thousand years, why, what we did in the last 50 years is *New*.

For scores of thousands of years machinery and machine-made products were virtually unknown. We have no actual knowledge of the centuries that man lived in trees or caves, before building his first crude artificial shelter. We do know that wood, brick and stone were about the only materials used in building construction up to the present era. We know that all during this period constituting such a great proportion of recorded history man kept himself warm with fires of wood and coal, used horses or other animals as the sole means of land transportation and wooden ships on the waters, and that it was

*Chief Engineer, American Institute of Steel Construction.

an extravagance far beyond the reach of the average human being to have even the crudest illumination at night. The people of the earth lived largely by and for themselves. Only in the fine arts such as painting and sculpture, architecture and the likes, had there been notable advancement. But it must have been cold comfort, indeed, to have strolled before the Parthenon, to have worshipped at the shrine of Pallas Athene, to have lived amidst the glories that were Greece—and had no better plumbing facilities afforded than the distant ancestors of our Chic Sales bungalow.

Imagine, if you can, the structural difficulties alone that would be encountered in building a city such as yours without the materials which we now accept so casually. With the average business building limited in height to five or six stories, as it would be if built of brick or stone, a city of half a million souls would occupy an area far greater than that of a present New York or a London. Ten-story structures would be built, yes, and would probably be exceeded in height if necessary—at a sacrifice of from twenty-five to fifty percent of the most valuable rental area, starting with the first floor. Without steel we should be largely compelled to utilize horse-drawn vehicles to traverse such a city; true, we probably would not encounter the same traffic problems that now confront us, but most assuredly we should find our present streetcleaning department wholly inadequate.

Yes, they built the Pyramids without using steel or concrete and they will continue to endure for countless ages. As an example of enduring construction, well and good; from the standpoint of first cost and income return they weren't so hot. As I recall, the builders were not troubled with labor difficulties, strikes or lockouts, and the sit-down certainly would not prove popular when some chap with a bull-whip served as Resident Engineer, Inspector or local representative of the Secretary of Labor. Labor meant just what it implied in those days. As for income, based upon available rental area, a hot-dog stand on Beacon Street, here in Boston, would most certainly prove a better investment.

The truly efficient building, as regarded from every angle, has not as yet been built. Real Estate values, fluctuating from year to year, determine the economical height of structure. This height is also affected by the means and amount of vertical transportation, elevator speeds, variables such as occupancy density. Clearly it is

foolish to construct a ten-story building upon land so valuable that the return from rental floor space cannot possibly provide an adequate income. Similarly it is at once apparent that there is a limit as to height, or area, or size which must not be passed save at a sacrifice in economic values. In the book, "The Skyscraper," published by the American Institute of Steel Construction, detailed analyses of buildings of from eight to seventy-five stories for a given site, reveal most interesting details not only as to cost and income, but in operating charges as well, and clearly reveal the absolute necessity of making such studies.

The engineer, in the very nature of things, is a Trust Officer in everything that the name implies. His is a very responsible duty. I suppose almost anyone could build a structure strong enough, stable enough, to carry given loads or withstand given stresses, if no regard for economy was concerned. But that isn't the problem—and it isn't engineering. And, although they are perhaps engaged in slightly different fields, the architect and engineer have one thing in common, if they are worthy of their professions. That is the desire and ability to give to the owner of any given structure the very greatest amount of actual and potential value for the least possible expenditure. This in no wise implies that *cheapness* of any item enters into the matter. Indeed, in this latter phase it is often particularly true that it is not the first cost, alone, that affords true economy.

I am sure that cost—cost of the frame, the floors, footings, etc., resolved into dollars and cents per square foot or per cubic foot—is not the entire answer to any building study. There are too many indeterminates. But very often too much stress is placed upon cost of the frame, the very backbone of the building investment, while little heed may be given to excessive architectural ornamentation such as bronze work, elevator doors, marble trim—why, in many cases the cost of rugs and carpets exceeds that of the structural floor.

But I am equally certain that no responsible engineer can consciously, or conscientiously, avoid a study of true costs.

As an Army man in a couple of wars, I've had to forget about costs on many occasions; that is understandable because of the absolute necessity of getting the job done in a minimum of time, regardless of cost. If 2 x 6 timbers would do a job, but only 4 x 12's were on hand, why, you used the 4 x 12's. It's a gruesome thought, but if

you needed crushed stone to repair a road and the only stone available was in a nearby cemetery, why—you followed orders to get that road in shape to carry traffic at a certain hour, on a carefully pre-selected day, as part of a vital overall plan.

Let's forget Relativity, and consider what has actually happened in the last few years.

About 1900 the theory of reinforced concrete was introduced in this country. First used as a material for floors in steel frame buildings, it later was incorporated into the entire structural frame. For a long time it was also about the only fireproofing material used in protecting the members of a steel structure; since the steel designer could not utilize the concrete in carrying stress it placed the steel under a decided economic disadvantage, since as much concrete was used for fireproofing, of no design significance, as the concrete designer used to carry stress.

Concrete was not too efficient as a fireproofing material; few or no tests were made on loaded concrete floor slabs, yet even up to the present time many building codes rate 4″ reinforced concrete slabs as 4 hour construction. However, exhaustive tests, recently completed, conducted in accordance with "Standard Methods of Fire Tests of Building Construction and Materials (ASTM E 119-50)" on not one but many specimens of 4 3/4″ slabs, made with calcareous or silicious aggregates, give a maximum rating of 2 hours. This supports bulletin BMS #134 of the Bureau of Standards, issued December 26, 1952.

In searching for more efficient, yet lighter types of fireproofing, many tests have been made at Underwriters Laboratories and at Bureau of Standards. As a result, as approved by Underwriters Laboratories, a 2 1/2″ concrete slab on steel joist with a 3/4″ vermiculite-gypsum plaster ceiling, or 1″ gypsum plaster ceiling, both on metal lath, is now given the same official 3 hour rating that is given a 6″ solid concrete slab.

Of course it is at once apparent that great savings in dead weight are thus possible. The 2 1/2″ slab with ceiling and dead weight joists will weigh in the neighborhood of 40 lbs. sq. ft. as against 75 lbs. for the 6″ slab. Similarly, 1 3/8″ vermiculite plaster on metal lath fireproofing on a 12″ Wide Flange Column will weigh about 38 lbs. per foot of height; stone concrete encasement for the same column about 260 lbs. per foot. The difference in weight of fireproofing for Wide

Flange Beams is even more marked; with 1″ plaster on lath, the protection for a 36″ @ 300# beam will weigh about 45 lbs. per lin. foot, while stone concrete encasement for the same beam would weigh approximately 800 lbs. per foot.

These savings in dead weight are reflected throughout the building—in footings, joist, columns and beams. In the 30 story Mercantile Bank Building, Dallas, Texas, it has been publicly stated that use of vermiculite gypsum plaster saved 15,500 tons of dead load resulting in a saving of 1,880 tons of structural steel. The 13 story General Petroleum Building in Los Angeles cut 13,100 tons of dead weight by using light-type construction, with a saving of 1,200 tons of structural steel; the 12 story Prudential Life Building, same city, saved similar amounts.

For many years the commonly used exterior wall for tier buildings was a 13″ brick or masonry wall weighing about 120# per sq. ft. In a desire to save weight this was changed to 4″ of masonry with tile backup, weighing some 80# per sq. ft.

We now have 4″ wall, faced with steel or aluminum, and correctly insulated, doing a better job than 18″ of brick. The saving in weight is tremendous. Weight savings mean reduced costs, since they indicate that a member has inherently a greater capacity for, or ability to perform, more useful work within its load-carrying capacity.

I am quite sure that you are familiar with "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings", of the AISC. Before this specification was nationally accepted there were any number of individual specifications, no two of which were in entire agreement. In fact, within my lifetime, I seem to recall a time when I knew of 57 different Column Formulae, alone.

We did evolve an acceptable Specification, applicable alike as an authority by a designing engineer in New York or in Los Angeles and similarly capable of intelligent fabrication by a steel company in either city, without argument.

It's well enough to properly design a structure, and to erect it properly. But to protect it from the elements, or any unusual exposure, it must be painted. Clearly, what is required in the dry heat of Arizona, is entirely different than that of humid Florida. Too, no two paint manufacturers entirely agree on what is required for a given exposure.

Recognizing that a Paint Specification, almost universally applicable, was needed, the Institute initiated the necessary research. As a result a Paint Specification is forthcoming which is felt will be of utmost value to you, not as the recommendation of any individual paint manufacturer, but which that manufacturer can intelligently meet.

There are five primary methods of joining steel members of a structure. Chronologically they are:

1—Unfinished bolts
2—Rivets
3—Special bolts
4—Welding
5—High strength bolts

In the early days of steel fabrication the ordinary machine bolt was widely used; indeed, about the only other way in which a joint could be developed at that time was by the use of hand-driven rivets, which was relatively a costly operation. Unfinished bolts still have a definite place in steel construction, as covered in Section 7 (e) of our Specifications.

In the Stuyvesant Town and Peter Cooper Village apartment developments in New York City, approximately 60,000 tons of structural steel were erected by means of unfinished bolts.

With the development of the air driven rivet hammer, and the hydraulic and compressed air riveting machines, both stationary and movable, almost all structural work became hot riveted, with some attention being given to cold rivet use.

Certain types of work were encountered wherein the use of hot rivets became difficult or inadvisable, yet for which ordinary unfinished bolts were not deemed satisfactory. Accordingly, the relatively costly turned bolt was used for certain important connections, as was the rib-bolt with lock nuts. I recall using the latter by the scores of thousands in military bridge construction, where it was impossible to get air to the job, and where experienced riveting crews were not available.

The introduction of gas and electric arc welding was not immediately accepted as you may recall; this was largely due to the early, unduly extreme enthusiasm of its proponents. As it became better understood it came into general use and is now accepted as a

most valuable tool. Indeed, it offers unique possibilities for the development of continuity in steel structures.

A little known application of resistance welding is now in limited use, whereby welds the equivalent in size of a 1 1/4″ rivet may be made through 3″ to 6″ of steel.

Much has been written of late concerning the high strength bolt. Certain railroads found that under repeated reversals of stress in railroad bridges the rivets often became loose. Replaced by high strength bolts, torqued to the desired amount, the bolt withstood millions of cycles of stress reversals. Considerable research was initiated, leading to the adoption of a procedure whereby, utilizing hardened steel washers, sufficient clamping action was developed in the bolt to obviate any slipping or loosening.

Many research specialists in this country and abroad have long been intrigued with the subject of stresses in steel above the elastic limit of the material, or within the plastic range. To those who have held that Hooke's Law must govern, this may seem revolutionary, until it is realized that steel, when stressed beyond its yield point, then acts as do other materials under their normal design stress. The investigators are proceeding satisfactorily and seem to offer distinct future economies.

The question is often asked, "Are our iron ore supplies running out?"

It is true that the great Mesabi Range is being depleted; we shot away millions of tons in the two world wars, in addition to our other uses. But tremendous deposits of high grade ore are available in Venezuela, Liberia and Labrador and we are importing that ore. Perhaps of greater importance to us, since it is not subject to foreign attack by sea, is the almost endless supply of low grade ore—the taconites—which we possess. It averages but approximately 25% iron, while the Mesabi ore ranges about 50% iron. For years it was thought that processing would be too expensive, but it is now becoming economically feasible. We still have plenty of limestone for steel making—and alas (from the steel man's viewpoint) for making a lot of concrete as well.

Our other metals, with the possible exception of copper ore, are in good supply. While we are importing a great deal of aluminum ore, we have tremendous supplies of clays and rocks containing the metal.

It is a relatively costly process to reduce it, but the cost of the finished product is now expressed in cents per pound while only a few years ago it was similarly expressed in dollars. When made in the form of duraluminum, by the use of alloys, it does a remarkably able job for many purposes. During the war we even learned to weld it successfully.

A few years ago magnesium metal was virtually non-existent; today we have a capacity of some 130,000 tons annually. And that means a lot of magnesium, since it weighs only a little over a fourth as much as steel. Since it is largely extracted from sea-water the supply of magnesium is limitless.

Aluminum, with an average modulus of elasticity of 10,000,000, weighs about a third as much as steel.

A hitherto little known metal, titanium, offers many possibilities. Six years ago there was no commercial production; today it is at the rate of 5,000 tons a year. It has been predicted that in 20 years the rate will be 2,000,000 tons annually. It has been extremely expensive; as was the case with aluminum, increased production and more efficient methods of manufacture will greatly reduce the cost.

Titanium offers distinct possibilities since it is highly corrosion-resistant, is almost as strong as steel, and is about 42 percent lighter. As with aluminum and magnesium its principal use at present is in airplane construction.

The Construction Industry, nor any of its elements, is not frozen. We are in constant search of means of doing a job not only better but more economically. In this day when all those elements which enter into Job Cost are, by past standards, inordinately high; when labor rates and material costs seem out of reason—we still need Construction, with a capital C, because after all Construction is our largest Industry.

THE DELAWARE — WATER FOR MILLIONS

By John Boardman*

(Presented at a joint meeting of the Boston Society of Civil Engineers and the Hydraulics Section, B.S.C.E., held on February 18, 1953.)

It is a pleasure to come to Boston to tell you about the activities and programs of the Interstate Commission on the Delaware River Basin, generally known as Incodel. I particularly want to explain our plans for the utilization of the waters of the Delaware River Basin by the four states for their mutual benefit.

Before explaining the water project, it might be well if I give you some background information relating to the organization of Incodel and some of the things which have been accomplished by the Commission since it was organized.

It was established in 1936 for the specific purpose of formulating programs for the wise control, utilization and development of the natural resources of the Delaware River Basin, an interstate region occupying a part of each of the four states of New York, New Jersey, Pennsylvania and Delaware.

The Commission derives its authority from laws reciprocally enacted by the Legislatures of the four states creating individual state "Commissions on Interstate Cooperation." The New Jersey statute is Chapter 21, P.L. 1936. The other authorizing measures are: Pennsylvania, Act No. 37, Laws of 1937; Delaware, Chapter 202, Laws of 1939; New York, Annual Concurrent Resolutions.

Incodel directly represents and is responsible to the "Commissions on Interstate Cooperation" of the four states. At the present time, consideration is being given to establishing Incodel by a formal compact among the states.

The membership of the Commission is composed of five officials representing state government from the four participating states. Incodel's Articles of Organization provide that of each group of five, one shall be a member of the state Senate, one a member of the House of Representatives, one a member of the State Planning

*Engineer, The Interstate Commission on the Delaware River Basin, Philadelphia, Penna.

Board or agency, and one an administrative official directly responsible to the Governor. The fifth member is a member-at-large, selected by the other four members of the respective state delegations.

The Commission operates through a small staff financed by proportionate appropriations from each of the states and utilizes, as fully as feasible, the services of various departments of the states which it serves, as well as the services of federal agencies working in the region.

The Commission is essentially a service organization. Its powers are entirely advisory. Whenever the Commission has a problem to tackle, it follows the principle of appointing an advisory committee to assist in the investigation. The Commission is very careful in the selection of the members of such committees to see that they are fully qualified specialists in the subject under consideration.

All programs which the Commission develops are referred to the state governments affected thereby for their approval and adoption. The administration of such programs remains with the established agencies of the respective state governments.

The Commission has followed the policy of tackling first and urgent things first. In keeping with this principle, it has met with great success in the development of unified programs in the field of pollution abatement, water supply, flood control, recreation, and soil and forest conservation.

Incodel's initial activity was to devise a procedure by which the states could successfully cope with the problem of stream pollution. Before Incodel's creation, the states, municipalities and industries were going about stream pollution abatement in a rather haphazard manner. One state would not take action against a violator within its borders waiting for an adjoining state to take action against a municipality or industry at a corresponding location across the river. This led to an impasse resulting in very little stream pollution abatement and a very polluted river.

Following its policy of interstate cooperation, Incodel called together the chief engineers of each of the states' health departments and formed a water quality committee. This committee adopted a program sponsored by Incodel whereby the river and its tributary drainage area was divided into four zones and appropriate standards for the treatment of sewage, industrial wastes and other polluting materials

were devised for each of these zones. This program was adopted by each of the four states as a legislative measure and is now being vigorously prosecuted. In contrast to the situation that existed before Incodel came into existence, the states, municipalities and industries are now cooperating in correcting and preventing the undesirable, unwarranted degradation of the waters of the Delaware and its tributaries. Practically every municipality and a majority of the industries which contributed to stream pollution have constructed treatment works or taken other steps to correct their problem. Well over $100,000,000 has been spent in the past few years or is now being spent so that the Delaware River Basin leads the country in the drive for clean waters for America.

But, getting back to our subject—"The Delaware—Water For Millions"—let us look at Incodel's latest project.

During the 1949 legislative sessions, the states of New York, New Jersey and Pennsylvania, by the enactment of reciprocal legislation, directed our Commission to be responsible for the conduct of a survey to determine the feasibility and advisability of the future construction of an integrated water project in the upper Delaware Basin to meet estimated water requirements in the metropolitan areas and political subdivisions of the three participating states. Incodel was instructed to give careful consideration to other incidental benefits of such a project. The authorizing statutes are: New Jersey, Chapter 105, Laws of 1949; New York, Chapter 610, Laws of 1949; and Pennsylvania, Act No. 475 of 1949.

On behalf and with the approval of the three states, Incodel engaged the services of consultants of outstanding ability and integrity to assist in carrying out its assignment.

Malcolm Pirnie Engineers, of New York City, and Albright and Friel, of Philadelphia, were Incodel's consulting engineers.

Mr. Edward Hopkinson, senior partner of Drexel and Company, chairman of the Philadelphia City Planning Commission and a member of the Pennsylvania State Planning Board, served as chairman of Incodel's financial advisory committee.

The legal aspects were handled by a committee composed of a deputy attorney general from each of the participating states, the chairman of which was M. Vashti Burr, of Pennsylvania.

All of these experts, as well as the entire membership of Incodel,

sincerely believe that the project is sound and equitable; that it should be approved and adopted by the Delaware Basin states and prosecuted by an administrative agency jointly created by and responsible to those states.

Our first stage program consists of a system of four reservoirs with a total storage capacity of over 500 billion gallons. Two of these reservoirs—one at Cannonsville, the other at Godeffroy—are located entirely within the state of New York. These two reservoirs provide more than 75 per cent of the total storage capacity of the entire project.

The third reservoir in the Incodel project straddles the Delaware between New York and Pennsylvania at Barryville. It provides less than two per cent of the total capacity.

The fourth reservoir, created by the dam across the Delaware between Pennsylvania and New Jersey at Wallpack Bend, provides about 23 per cent of the total capacity of the project.

This first stage program will produce a dependable supply of approximately 1,600 million gallons of water daily during as serious a period of drought as has ever been encountered. It is estimated that this will be enough water to take care of the needs of areas in the Basin states for approximately the next 35 years.

Of this quantity, approximately 240 million gallons a day, or only 15 per cent, will be required in New York City.

Northeastern New Jersey will need about 225 million gallons a day, or 14 per cent.

The remaining 1,135 million gallons a day, or 71 per cent of the total safe yield, has been earmarked for the exclusive purpose of increasing the flow in the Delaware River throughout its entire length during periods of depleted stream flows. This is a sufficient quantity to assure the maintenance of a flow of at least 3,000 million gallons of water a day into the tidal section of the Delaware River at Trenton during the worst conceivable period of drought.

River Regulation

In order better to understand this phase of the project, it is necessary to emphasize a fact which everyone knows but is inclined to overlook. It is that rivers do not flow at a uniform rate. To the contrary, they are subject to violent fluctuations daily, weekly, monthly and yearly—a one-time-too-much, another-time-too-little, situation.

For example, the lowest daily flow of record in the Delaware River at Trenton was only 800 million gallons. This is substantially less than the present total demand for water for municipal and industrial needs in the Trenton to Philadelphia section of the river. If such a flow persisted for many days, this stretch of the tidal stream would soon become a stinking, brackish mess, totally unfit for domestic and industrial use regardless of the extent and efficiency of pollution abatement works. This is not likely to, but could happen under present conditions.

On the other side of the scale, the highest daily flow of record was almost two hundred times the minimum flow.

Similarly, during the driest month of record, an average of only 1,200 million gallons of water a day flowed in the Delaware at Trenton as compared with 42,000 million gallons daily for the wettest month. It was during this driest month of record that salt water from the ocean was pushed upstream as far as Allegheny Avenue near the upper boundary of Philadelphia, because there was an insufficient fresh water flow to counteract the forces of tidal action.

Fortunately, floods along the Delaware are not a serious problem. The proposed Incodel project, however, will be beneficial in this respect.

Periods of drought, on the other hand, have been and will continue to be damaging and disastrous until a program for their abatement has been consummated. It is generally agreed that the Delaware River is in "drought" whenever its flow falls below a rate equivalent to about 2,600 million gallons of water a day at Trenton, New Jersey. Under present conditions, based upon a 37 year period of record, the flow in the river gets below this rate, on the average, about 25 per cent of the time. In other words, a drought occurs almost every summer and fall and lasts approximately three months. One year, 1930, it extended over a period of seven months. In 1949, it lasted 133 days.

Droughts in the summer and fall spoil the river for recreation. Droughts in the summer and fall give the water works and sewage treatment plant operators their greatest headaches. Droughts in the summer and fall cause salty ocean water to invade the intensely industrialized section of the tidal estuary between Philadelphia and the Pennsylvania-Delaware boundary line. Droughts in the summer and

fall forced the City of Chester, Pennsylvania, to abandon the Delaware because of salty water and go to the hinterlands for a new source of municipal water supply. Droughts in the summer and fall permit the drill, the young oyster's greatest enemy, to invade oyster farms and kill off as much as fifty per cent of the potential production in a single growing season. Droughts in the summer and fall will be the greatest deterrent to the restoration of the shad fishery after the basinwide pollution program has been completed in 1953 or 1954. Droughts in the summer and fall raise serious questions in the minds of representatives of new industries desiring to locate along the Delaware River between Trenton and Philadelphia.

All of these conditions would either be entirely overcome or largely alleviated when the proposed Incodel program becomes a reality. Serious droughts in the form of depleted stream flows could then no longer occur. This is because a total volume of over 300 billion gallons of flood waters would be held back in the four proposed reservoirs above Water Gap to meet just such a contingency. This is a sufficient quantity to add 1,500 million gallons of water every day for 200 days, or almost seven months, to the natural flow of the river. It would be enough to maintain a minimum flow at Trenton of over 3,000 million gallons daily during such a period. A drought of 200 days' duration only occurred once in this region, in 1930, as far as is known and is about the worst condition ever likely to be experienced. For the average drought period of 90 days, over 3,000 million gallons of water per day could be added to the natural flow of the river, raising the total flow at Trenton to over 4,500 million gallons daily.

Value of River Regulation Benefits

It is impossible, of course, to make a completely accurate appraisal of the value of the elimination of droughts in the Delaware. However, a reasonably reliable approximation has been made. It indicates that it would be worth approximately $7,000,000 a year to be able to maintain, at all times, a minimum flow in the Delaware River equivalent to 3,000 million gallons a day at Trenton. I will briefly discuss the items which comprise this total. In the section of the Delaware River between Philadelphia and the Pennsylvania-Delaware boundary, the estimated loss to existing manufacturers, particularly in the Chester-Marcus Hook area, in coping with problems caused by salt water invasions amounts to close to $1,000,000

a year, and in some years has exceeded $2,000,000, based upon present price levels.

This estimate does not take into account the enhanced value of a substantially salt-free Delaware River water to the potentially great industrial area in Pennsylvania and New Jersey in this region. A figure of $1,000,000 a year would seem to be conservative.

It is believed that the existing oyster industry along the New Jersey and the Delaware shores of the lower river and bay is damaged by adverse salinity conditions to the tune of about another $1,000,000 per year.

Municipalities and industries which depend upon the section of the Delaware River between Trenton and Philadelphia for water supply and for the disposal of treated wastes will be benefited greatly by the "drought" control features of the proposed water project. Uppermost in this category, municipality-wise, will be the City of Philadelphia. The raising of summertime low flows at Trenton by about 100 per cent will give Philadelphia a better quality of Delaware River water for water supply, and the larger quantity of flow will aid substantially the dissipation of the effluent from its new $70,000,000 sewage treatment plants. Among other municipalities that will benefit in either or both of these respects are Trenton, Bordentown, Burlington, Riverton, Riverside, Palmyra, Camden and Gloucester in New Jersey, and Morrisville, Tullytown and Bristol in Pennsylvania.

Abatement of droughts should result in equal, if not greater benefits in the development of this region industrially. The new United States Steel Corporation plant being constructed at Morrisville and the many allied industries which are springing up along both sides of the river as the result, as well as existing industries, will be able to count upon the river for an adequate and satisfactory source of water supply all year round. Under present conditions, this is not possible.

Certainly, the value of the drought control aspects of the proposed Delaware Basin project to municipalities and industries in the Trenton to Philadelphia region would amount to well over $1,000,000 annually.

Based upon the above analysis, the value of eliminating droughts would be worth in the neighborhood of $4,000,000 a year to the lower Delaware River Basin area below Trenton alone. This does not include an evaluation of the benefits which will accrue to the proposed deepening and widening of the river channel from Philadelphia to Trenton.

In the section of the Basin above Trenton, benefits which would result from the drought control feature of the proposed project would include the enrichment of recreational facilities, the restoration of the shad fishery, the reduction of flood damages and appreciation of property values, as well as those which accrue to operators of water supply and waste disposal systems in the area.

The total value of these additional project benefits should certainly be worth at least $3,000,000 per year.

None of the above estimates places any price tags on the value inherent in the preservation and immediate availability of the reservoir system for new or additional sources of municipal water supply for Philadelphia, southeastern Pennsylvania and southern New Jersey, if and when needed.

Water Supply Aspects

In addition to the 316 billion gallon water bank for a stream flow augmentation, there is another 100 billion gallons of storage capacity in the four reservoir system which has been earmarked for the purpose of providing additional sources of water supply to New York City and northeastern New Jersey. It will be recalled that these areas had to resort to bathless Fridays and other stringent emergencies to ride the drought of 1949. As I stated before, the storage provided for water supply will be sufficient to assure a dependable yield of 465 million gallons of water a day. Of this amount, it is estimated that, by 1980, New York City will need an additional 240,000,000 gallons of water daily over and above the capacity of their existing sources of supply and sources under construction, and northern New Jersey will need 225,000,000 gallons a day.

Cost of Project

The project is estimated to cost about $550,000,000. Of that total, $400,000,000 has been allocated to the water supply aspects of the project and would be self-liquidating from revenues received from the sale of water to New York City, northeastern New Jersey and other areas of need, if any. The Delaware Basin states are being asked to underwrite the balance of about $150,000,000 allocated to "drought" control. It has been recommended that this be apportioned among the states in the following manner: $60,000,000 each to Pennsylvania and New Jersey, $22,500,000 to New York, and $7,500,000 to Delaware.

It is believed that the annual value of the benefits received from the expenditure for "drought" control is considerably in excess of the annual cost. It is interesting to note that the four states are currently paying considerably more than this amount, not as a single sum investment, but every year, for the construction of federal water projects in other sections of the country, particularly the South and the West. The states could go a long way in removing this inequity by their approval and adoption of the Incodel water project and their participation with other states in the northeastern section of the country in a campaign for the adoption of a national water policy which would call upon all regions to bear a substantial part of the cost of any water project constructed primarily for their benefit.

Delaware Basin Water Commission

In order to give life to the Incodel water project, it will be necessary for the legislatures of the states of New Jersey, New York, Pennsylvania and Delaware to approve the proposed program. The legislatures will also be required to ratify an interstate compact creating an administrative agency, to be known as the Delaware River Basin Water Commission, to which power would be given to prepare construction plans and specifications and to finance, construct and operate the proposed project.

New Jersey and Delaware approved the project and passed compact legislation during their 1951 legislative sessions, and New York followed suit in 1952.

In Pennsylvania, Governor John S. Fine appointed a special committee to study the project to determine if Pennsylvania's interests were being adequately protected. The Pennsylvania Water Resources Committee has been busily engaged in carrying out the governor's directive and as yet have not disclosed their findings.

The project offers a brave challenge to the four Delaware Basin states. It is a public necessity, and eventually will be built by some agency.

Incodel is firmly convinced the states will seize this glorious opportunity to demonstrate that they have the ability and capacity to provide and build for their own needs. If they do this, they will be providing a tremendous service to their own citizens, while at the same time setting an example which the whole country would do well to follow.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

SEPTEMBER 23, 1953.—A Joint Meeting of the Boston Society of Civil Engineers with the Transportation Section, BSCE was held this evening at the American Academy of Arts & Sciences, 28 Newbury Street, Boston, Mass., and was called to order by President Chester J. Ginder, at 7:10 P.M.

President Ginder stated that the Minutes of the May 20, 1953 meeting would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes therefore be waived unless there was objection.

The President announced the death of the following members:—

J. Stuart Crandall, who was elected a member July 2, 1920 and who died July 20, 1953. (Elected President March 18, 1953.)

Edward Hutchins, who was elected a member March 20, 1907 and who died May 30, 1953.

Clinton C. Barker, who was elected a member November 17, 1937 and who died June 5, 1953.

Arthur C. Grover, who was elected a member December 18, 1895 and who died March 17, 1953.

Stuart E. Coburn, who was elected a member April 18, 1917 and who died August 21, 1953.

Scott Keith, who was elected a member January 26, 1921 and who died July 4, 1953.

Paul U. Nickerson, who was elected a member November 17, 1952 and who died May 3, 1953.

The Secretary announced the names of applicants for membership in the B.S.C.E.

The Secretary announced that the following had been elected to membership on June 15, 1953:—

Grade of Member.—Charles J. Christy, Joseph B. Kelly.

President Ginder announced that this was a Joint Meeting with the Transportation Section, BSCE and called upon Chairman Hugh P. Duffill to conduct any necessary business for that Section at this time.

The President announced that the Annual Student Night Meeting would be held on October 14th at Tufts College, Medford, Mass.

President Ginder then introduced the speaker of the evening, Mr. John D. M. Luttman-Johnson, Senior Engineer, Fay, Spofford & Thorndike, Boston and New York, and consultant on Transportation to the International Bank for Reconstruction and Development, Mission to Ceylon, 1951-52, who gave a most interesting illustrated talk on "Transportation Problems of Ceylon's Ports".

A short discussion period followed after which the President announced that a collation would be served in the Lounge on the floor above.

Thirty-seven members and guests attended this meeting.

The meeting adjourned at 9:20 P.M.

ROBERT W. MOIR, *Secretary*

ADDITIONS

Members

David R. Campbell, 207 Marked Tree Road, Needham, Mass.

DEATHS

Stuart Coburn, August 21, 1953
J. Stuart Crandall, July 20, 1953
Arthur C. Grover, March 17, 1953
Scott Keith, July 4, 1953
Paul U. Nickerson, May 3, 1953
Edward Hutchins, May 30, 1953
Clinton C. Barker, June 5, 1953

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PROFESSIONAL SERVICES AND ADVERTISEMENTS

The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

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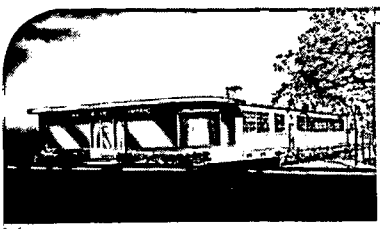

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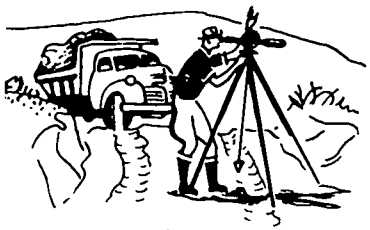

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