CHAPTER C12
WASTEWATER AND STORMWATER PIPING SYSTEMS

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This chapter presents information on the analysis and design of wastewater and stormwater piping systems. Wastewater piping systems convey waterborne domestic, commercial, or industrial wastes to a point of discharge and/or treatment. Such systems are also known as sanitary sewer collection systems. Stormwater piping systems convey captured stormwater runoff to points of discharge. Such systems are also known as storm sewer systems. There are systems, chiefly in older cities, that convey domestic, commercial, and industrial wastewater and stormwater runoff in a single piping system. Such systems are called combined sewer systems.

The design of both wastewater and stormwater collection systems must comply with standards of city, county, and state as well as federal regulation agencies. Permits must be obtained for the disposal of domestic, commercial, and industrial waste into receiving streams or water bodies. Stormwater discharges to receiving streams or water bodies may also require permits.

This chapter focuses on public and/or private wastewater and stormwater collection systems generally serving a number of buildings. For information about in-building plumbing, which is not covered in this chapter, see Chap. C13 of this book. The scope of this chapter does not permit examination of appropriate piping materials for handling exotic wastes such as oils and acids. Refer to the index of this book for specific information on piping systems to convey such wastes.

DEFINITIONS

A selection of terms used in wastewater and stormwater technology follows. Additional terms are defined at the place of use.

Sewer: A pipe or conduit that carries wastewater or stormwater.

Wastewater: The spent water of a community. From the standpoint of the source, it may be a combination of liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with any groundwater, surface water, and stormwater that may be present. The term *wastewater* is often used instead of the less inclusive *sewage*.

Stormwater: The portion of water that runs off the ground during and immediately following a rainstorm, snowmelt, or other flooding event.

Sanitary sewer: A sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with varying quantities of storm waters, surface waters, and groundwaters that are not admitted intentionally.

Storm sewer: A sewer that carries stormwater and surface water, street wash, and other waters or drainage, but excludes domestic or industrial wastewater. It is also called a storm drain.

Combined sewer: A sewer intended to receive both wastewater and stormwater or surface water.

Building sewer: In plumbing, the extension from the building drain to the public sewer or other place of disposal; also called *house connection*.

Lateral sewer: A sewer that receives wastewater from buildings and discharges into a branch or other sewer and has no other common sewer tributary to it.

Branch sewer: A sewer that receives wastewater from a relatively small area and discharges into a main sewer serving more than one branch sewer area.

Main sewer: The principal sewer to which branch sewers are tributary; also called *trunk sewer*.

Trunk sewer: A sewer that receives dry-weather flow from a number of branch or main sewers or outlets and frequently additional predetermined quantities of stormwater (in combined sewer systems) and conducts such waters to a point for treatment or disposal. Trunk sewer is also commonly referred to as an *interceptor sewer*.

Outfall sewer: A sewer that receives wastewater from a collection system or from a treatment plant and carries it to a point of final discharge.

Sanitary sewer system: A sewer system comprised exclusively of sanitary sewers which carry only wastewater and to which stormwater, surface water, and groundwater are not intentionally admitted; also referred to as *separate system* or *separate sanitary system*.

Storm sewer system: A system composed only of sewers carrying stormwater, surface water, street wash, and other wash waters or drainage from which domestic wastewater and industrial wastes are excluded.

Combined sewer system: A system of sewers receiving both surface runoff and wastewater.

Infiltration: The water entering a sanitary sewer system and service connections from the ground, through such means as, but not limited to, defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include, and is distinguished from, inflow.

Inflow: The water discharged into a sanitary sewer, including service connections, from such sources as, but not limited to, roof leaders; cellar, yard, and area drains; foundation drains; cooling water discharges; drains from springs and swampy areas; manhole covers; cross-connections from storm sewers and combined sewers; catch basins; storm sewers; surface runoff; street wash waters; or drainage. Inflow does not include, and is distinguished from, infiltration.

Exfiltration: Exfiltration is the process of wastewater inside the sewer flowing out of the sewer through joints, pipe cracks, etc. into the surrounding bedding.

Inflow/infiltration: The total quantity of water entering a sanitary sewer system from both infiltration and inflow without distinguishing the source.

UNITS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
cfs	cubic feet per second
cm	centimeter
d	depth of flow
D	pipe diameter
f	friction factor
fps	feet per second
ft	foot
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
h, H	hydraulic head
hr	hour
in	inch
kg	kilogram
km	kilometer
lb	pound
lpcd	liters per capita per day
lpd	liters per day
m	meter
m^3	cubic meter
mgd	million gallons per day
min	minute
mm	millimeter

n	dimensionless roughness coefficient
q, Q	quantity of flow
r	hydraulic radius
s, S	slope of the energy gradient
s	seconds
v, V	velocity
WPCF	Water Pollution Control Federation (now called Water Environment Federation)

QUANTITY OF FLOW

Quantity of Wastewater Flow

One of the first critical steps in the design of sanitary sewers is the estimation of design flows of wastewater tributary to them. Sanitary sewers must be designed to provide capacity for the present and estimated future quantities of domestic sewage, commercial and industrial wastewaters, and infiltration/inflow.

Design Period and Tributary Flow. Lateral and branch sewers of a sanitary sewer system should be designed for the ultimate population density to be expected in the area served. Main and trunk sewers are commonly designed to handle the flows to be expected from a population 25 to 50 years in the future and in some cases the ultimate buildup of the service area. The estimation of future flows can be arrived at only after a detailed study of the land use, population growth trends, water consumption rates, commercial and industrial growth, etc. Forecasts of population and the commercial, industrial, and institutional development expected to be in place at the end of the design period should be obtained from local and regional planning organizations and adjusted when appropriate by site-specific knowledge.

Quantities of Flow—Overview. Wastewater tributary to sanitary sewers emanates from residential, commercial, and industrial land uses. The most important single index in estimating flows to the sanitary sewer from each of these land-use sources is the rate of water consumption. Flows entering the sanitary sewers generally can be estimated as the water consumption, less an allowance for uses outside the building that typically do not return to the sewer (landscaping, car washing, etc.), plus an allowance for infiltration/inflow of clear water into the sewers. Note that water consumption means water delivered to the user. The quantity of water actually pumped into the water distribution system includes water that will be lost as leakage en route to users. If the designer has pumping records available, as opposed to consumption (or water billing) records to estimate water consumption, the pumping records should be adjusted downward by 5 percent to as much as 30 percent to account for leakage in the potable water system.

The quantities of water consumption depend to some degree on the type of plumbing fixtures in residential and commercial buildings. Low-flow plumbing fixtures, if prevalent in an area, will decrease water consumption and consequent sewage flows. The presence of such fixtures should be accounted for in estimates.

When estimates of sanitary sewage flow from small areas are being prepared, the most accurate procedure is to make separate estimates of the various classifications of flow which make up the total. The classifications which are commonly used are domestic, commercial, and industrial sewage flows and infiltration/inflow.

Quantity of Domestic Sewage. Domestic sewage flow, excluding allowances for infiltration from residential land uses, is generally equivalent to water consumption minus an accounting for outside-the-home water use that does not flow back to the sanitary sewer (landscaping, car washing, etc.). The amount of water used for such outside uses varies depending on climate of the area and landscaping practices. In temperate climates, such as the midwest and eastern United States, there is a distinct seasonal fluctuation of domestic water consumption due to significant outside water uses for landscaping, etc. Peak water consumption occurs during summer seasons, and minimum consumption occurs during winter months when outside uses are negligible. In such areas the wastewater flow rate, excluding infiltration, is best gauged by observing water consumption records during winter months. In warmer, drier climates where landscaping water use continues year-round, an estimated allowance can be made during lowest-water-consumption months. The ratio of annual domestic sewage flow, excluding infiltration, to annual water consumption (water billing, if available) ranges from 70 percent in arid areas with extensive landscaping water use to 95 percent in areas where little or no landscaping water use is practiced.

The average per capita domestic water consumption varies from about 40 to 120 gpd (150 to 450 lpd), depending upon the character of the area and the economic status of the population. If water is supplied through meters, accurate estimates of average per capita consumption can be made. If water is supplied unmetered, estimates have to be based on the water consumption rates or measured sewer flow rates which are known to prevail in other areas of similar character. Table C12.1 presents typical wastewater flow rates from residential sources.[1]

TABLE C12.1 Typical Wastewater Flow Rates from Residential Sources

		Flow, gal/(unit·d)	
Source	Unit	Range	Typical
Apartment			
High-rise	Person	35–75	50
Low-rise	Person	50–80	65
Hotel	Guest	30–55	45
Individual residence			
Typical home	Person	45–90	70
Better home	Person	60–100	80
Luxury home	Person	75–150	95
Older home	Person	30–60	45
Summer cottage	Person	25–50	40
Motel			
With kitchen	Unit	90–180	100
Without kitchen	Unit	75–150	95
Trailer park	Person	30–50	40

Note: gal/(unit · d) × 3.785 = l/(unit · d).
Source: Adapted in part from Ref. 1.

Quantity of Commercial Sewage. The quantity of sewage flow from commercial areas varies widely depending upon the nature of the commercial activity. Allowances made for the quantity of sewage from commercial areas in large sewer districts are commonly in terms of gallons per day per acre, gallons per day per square foot of floor space, or gallons per day per capita or per employee. Allowances made by wastewater utilities for average wastewater flow from commercial areas vary from 2000 to 60,000 gpd per acre (18 to 560 m^3/d per hectare) and from 15 to 500 gpd per capita (60 to 1900 lpd per capita). Table C12.2 shows typical average commercial

TABLE C12.2 Average Commercial Wastewater Flow[a]

Type of establishment	Avg. flow (gpd per capita)
Stores, offices, and small businesses	12–25
Hotels	50–150
Motels	50–125
Drive-in theaters (three persons per car)	8–10
Schools, no showers, 8-h period	8–35
Schools with showers, 8-h period	17–25
Tourist and trailer camps	80–120
Recreational and summer camps	20–25

Note: gpd per capita × 3.785 = lpd per capita.
Source: Adapted in part from Ref. 2.

flows for various commercial categories.[2] It is evident from the wide range of estimates shown in Table C12.2 that for any area in which commercial activity is an important factor, the estimate of sewage flow should be based on a special study of the area.

Quantity of Industrial Sewage. The flow of sewage from industrial establishments may be purely sanitary sewage, or it may also include waterborne industrial wastes. Estimates of the sanitary sewage are made by the procedures already considered. The quantity of industrial wastes can be determined only by special studies of the individual industrial activities.

Quantity of Infiltration. The rate of infiltration of groundwater into sewers is influenced by the size, age, and condition of the sewers; the position of the sewers with respect to the groundwater table; the character of the soil; and the amount of precipitation. The infiltration rate for any one system will vary from season to season. Extensive work has been done to quantify and remove excessive infiltration in existing systems in the United States since 1975. Therefore, sewer monitoring results of infiltration are often available for existing sewer systems and should be consulted.

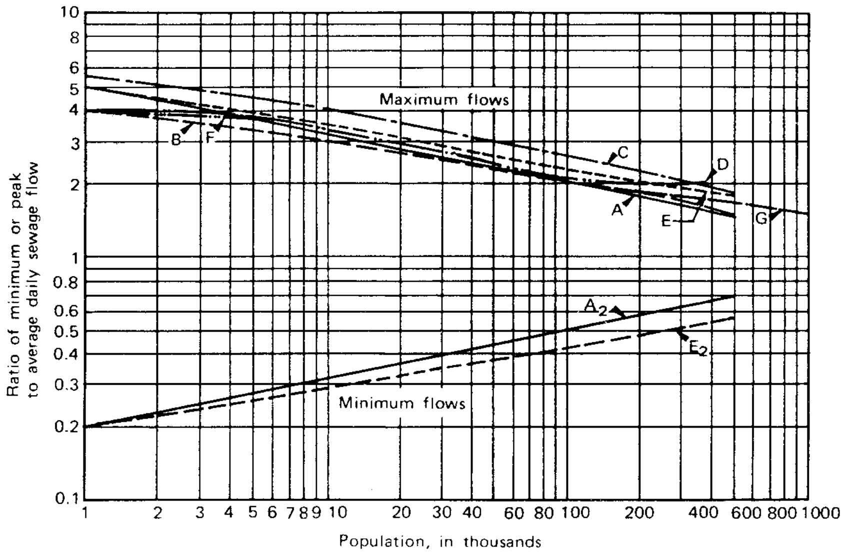
Specifications for installation of new sewers limit infiltration to between 100 and 500 gpd/in-diameter/mi (90 and 460 lpd/cm-diameter/km) of sewer at the time of testing. Allowances for infiltration at the end of the design period are higher than the initial test allowances. Design allowances for infiltration for new sanitary sewers at the end of the design period range from 3000 to 10,000 gpd/in-diameter/mi (2800 to 9200 lpd/cm-diameter/km) of sewer for a well-maintained sewer system and as high as 40,000 gpd/in-diameter/mi (37,000 lpd/cm-diameter/km).

Quantity of Inflow. Inflow is clear water which enters the sanitary sewer from sources such as roof leaders, yard and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross-connections between storm and sanitary sewers, or surface water runoff. Inflow is generally associated with periods of rainfall, snowmelt, and surface water runoff. In many existing systems, inflow is the single largest flow component on rainfall days and is often responsible for the backup of wastewater into basements and homes, or the bypassing of untreated wastewater to nearby streams and water courses. Inflow rates have been found to vary greatly from system to system and even within a given system owing to the age and condition of the sewer mains and laterals and the type of plumbing practices which have been employed. Typical inflow rates experienced range from 50 gpd/ft (600 lpd/m) of sewer length to over 1000 gpd/ft (12,000 lpd/m) of sewer length on a maximum-hour basis. In some systems, inflow results in total peak flows 7 to 10 times the average dry-weather flow. Owing to the high variability of inflow and the dramatic impact it can have on wastewater conveyance system sizing, engineering studies are generally recommended to establish the existing inflow conditions and the appropriate inflow allowance for future service extension.

Quantity of Exfiltration. Exfiltration is the process by which wastewater inside the sewer flows out of the sewer through joints, pipe cracks, etc., into the surrounding bedding. Exfiltration is generally not a concern with respect to estimating wastewater flows because sanitary sewers generally do not function under pressurized conditions. However, exfiltration is important to consider in conjunction with protection of groundwater in pressurized tunnels. In near-surface open-channel sewers, significant exfiltration can occur only when the sewer is in gross disrepair in granular soils or is cross-connected to a storm sewer or other outlet. In cases where flow monitoring indicates exfiltration, investigation of the situation is warranted to determine the flow path of sewage.

Flow Variations. Sewers should be designed to handle peak flow rates that are expected to occur at the end of the design period. It is also desirable to design sewers so as to minimize the problem of solids deposition during the early years of use, when the flows may be much lower than the future flows. In most cases the two design objectives cannot be achieved simultaneously (especially when the sewers are circular). Priority of the design objective should be established on a case-by-case basis. The flows vary from day to day and from hour to hour within each day. The ratio of the absolute maximum future flow rate to the initial minimum rate may vary from about 3 to 1 for large sewers serving highly developed areas to more than 20 to 1 for small sewers serving areas still under development. Figure C12.1 presents commonly used formulas for predicting ratios of maximum to average flows and minimum to average flows as a function of connected population.[2] These formulas assume dry-weather conditions (without excessive inflow) and no unusual industrial use patterns. Where nondomestic flows make up a significant amount of flow, the flow patterns should be considered separately.

Fixture-Unit Basis of Design. For very small tributary populations and for institutions such as schools, hospitals, hotels, and factories, the required capacities of sanitary sewers may be estimated from the *fixture-unit flow rates* defined by various local and national plumbing codes. The designer should consult local applicable plumbing codes for estimating such flows.



Curve A source: Babbitt, H.E., "Sewerage and SewageTreatment." 7th Ed., John Wiley & Sons, Inc. New York (1953).

Curve A_2 source: Babbitt, H.E., and Baumann, E.R., "Sewerage and Sewage Treatment." 8th Ed., John Wiley & Sons, Inc., New York (1958).

Curve B source: Harman. W.G., "Forecasting Sewage at Toledo under Dry Weather Conditions," Eng. News-Rec. 80, 1233 (1918)

Curve C Source: Youngstown Ohio, report

Curve D source: Maryland State Department of Health curve prepared in 1914. In "Handbook of Applied Hydraulics." 2nd Ed., Mcgraw-Hill Book Co., New York (1952)

Curve E source: Gifft, H.M., "Estimating Variations in Domestic Sewage Flows." Waterworks and Sewerage, 92,175 (1945).

Curve F source: "Manual of Military Construction." Corps of Engineers, United States Army, Washington, D.C.

Curve G source: Fair, G.M. and Geyer, J.C., "Water Supply and Waste-Water Disposal." 1st Ed., John Wiley & Sons, Inc., New York (1954)

Curves A_2, B, and G were constructed as follows:

Curve A_2, $\dfrac{5}{P^{0.167}}$

Curve B, $\dfrac{14}{4+\sqrt{P}}+1$

Curve G, $\dfrac{18+\sqrt{P}}{4+\sqrt{P}}$

in which P equals population in thousands.

FIGURE C12.1 Ratio of Extreme Flows to Average Daily Flow Compiled from Various Sources. *(Adapted from Ref 2.)*

Requirements of Regulatory Agencies. Some state regulatory agencies have established definite per capita flow rates to be used when detailed studies and estimates of expected flows have not been made. The designer should look into municipal and state regulations to obtain such regulatory requirements.

Typical of such regulations in the United States are the recommended standards of the Great Lakes–Upper Mississippi River Board of State Sanitary Engineers.

These standards, used widely by state agencies in the United States in regard to the design of sanitary sewers, are as follows.[3] The latest edition should be consulted.

Design Period. In general, sewer systems should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. Similarly, consideration should be given to the maximum anticipated capacity of institutions, industrial parks, etc.

Design Factors. In determining the required capacities of sanitary sewers, the following factors should be considered:

1. Maximum hourly rate of sewage flow
2. Additional maximum rate of sewage or wastewater from industrial plants
3. Maximum rate of groundwater infiltration

Design Basis. Two methods are commonly used:

- *Per capita flow:* New sewer systems should be designed on the basis of an average daily per capita flow of sewage of not less than 100 gallons per capita per day (gpcd) [380 liters per capita per day (lpcd)]. This figure is assumed to cover normal infiltration, but an additional allowance should be made where ground conditions are unfavorable. Generally the sewers should be designed to carry, when running full, not less than the following daily per capita contributions of sewage, exclusive of sewage or other waste from industrial plants.
- *Laterals and submain sewers:* 400 gpcd (1500 lpcd).
- *Main, trunk, and outfall sewers:* 250 gpcd (950 lpcd).
- *Interceptors:* Intercepting sewers, in the case of combined sewer systems, should fulfill the above requirements for trunk sewers and have sufficient additional capacity to care for the necessary increment of stormwater. Normally no interceptor should be designed for less than 250 percent of the gauged or estimated average dry-weather flow.
- *Alternate method:* When deviations from the foregoing per capita rates are demonstrated, a brief description of the procedure used for sewer design must be included in the design report.

Summary of Considerations for Determination of Sanitary Sewer Capacity. The following summary of the considerations necessary for the determination of sanitary sewer capacity is given in the American Society of Civil Engineers (ASCE) *Manual of Engineering Practice,* No. 37 [Water Pollution Control Federation (WPCF) Manual of Practice No. 9], ''Design and Construction of Sanitary and Storm Sewers.''

The quantity of wastewater which must be transported is based on full consideration of the following:

1. The design period during which the predicted maximum flow will not be exceeded, usually 25 to 50 years in the future.
2. Domestic sewage contributions based on future population and future per capita water consumption; or if a more satisfactory parameter than water consumption is available, that parameter should be used. Careful analysis should be made of population distributions and the relationship of maximum and minimum to average per capita sewage flows. The fixture-unit method of estimating peak rates should be employed for small populations, giving due care to the forecasting of the probable number of fixture units and water use per capita. When large

areas are to be considered, the peak rate of flow per capita or per acre sometimes is decreased as area and population increase.

3. In some instances, maximum flow rates may be determined almost entirely by extraneous flows, the source of which may be foundation, basement, roof, or areaway drains; storm runoff entering through manhole covers; or infiltration. Foundation, roof, and areaway drain connections to sanitary sewers should be prohibited. Proper construction and yard-grading practices should be mandatory. Nevertheless, there may be times when strict prohibition may not be feasible or even practicable. In any event, some stormwater and surface water will get into separate sanitary sewers, and a judgment allowance, therefore, must be made.
4. Commercial area contributions are sometimes assumed to be adequately provided for in the peak allowance for per capita sewage flows in small communities. A per acre allowance for comparable commercial areas based on records is the most reasonable approach for larger communities.
5. Industrial waste flows should include the estimated employee contribution, estimated or gaged allowances per acre for industry as a whole, and estimated or actual flow rates from plants with process wastes which may be permitted to enter the sanitary sewer.
6. Institutional wastewaters are usually domestic in nature although some industrial wastewater may be generated by manufacturing operations at prisons, rehabilitation centers, etc.
7. Air-conditioning and industrial cooling waters, if permitted to enter sewers, may amount to 1.5 to 2.0 gpm (0.1 to 0.13 l/s) per ton of non-water-conserving cooling units. Unpolluted cooling water should be kept out of separate sanitary sewers.
8. Infiltration may occur through defective pipe, pipe joints, and structures. The probable amount should be evaluated carefully. Design allowances should be larger (under some circumstances very much larger) than those stipulated in construction specifications for which acceptance tests are performed very soon after construction. Underevaluation of infiltration is one reason why some sewers have become overloaded.
9. The relative emphasis given to each of the foregoing factors varies among engineers. Some have set up single values of peak design flow rates for the various classifications of tributary area, thereby integrating all contributory items. It is recommended, however, that maximum and minimum peak flows used for design purposes be developed step by step, giving appropriate consideration to each factor which may influence design.

Example C12.1: Design Example for Calculating Wastewater Flows. Determine the average and peak wastewater design flows for the downstream end of a sanitary sewer designed to serve an area of 340 acres. Current residential population is 1000 (primarily single-family homes). Projected ultimate population in the future is 3500 (3000 people in single-family homes and 500 people in apartments). Additionally, commercial and institutional employment in the service area currently is 150 employees, primarily in office and non-water-intensive businesses. Future commercial and institutional employment in the service area is projected to be 900, again in similar types of businesses. There is negligible existing industrial development, but an 80-acre industrial park is planned. Future number of employees working in the industrial park is estimated to be 800.

Solution. The solution proceeds stepwise through the parameters to be considered:

1. *Select the design period of the sewer.* For this sewer, because the area and population is small, the design condition is the ultimate future condition.
2. *Estimate the average unit rate of flow of domestic sewage from residential areas.* Based on water billing records of existing residential single-family accounts, average water consumption during winter months (November to January) is 80 gpcd and during summer months (June to August) is 110 gpcd.

   Apartments in adjacent communities have winter water unit consumption rates of 65 gpcd. Average wastewater flow from the residential sector is therefore 80 gpcd (assumed to be same as water consumption during winter months) times 1000 people, or 80,000 gpd for existing conditions. For future conditions, the average flow is 3000 people times 80 gpcd for single-family residential plus 500 people times 65 gpcd for apartments, which totals to 272,500 gpd.
3. *Estimate commercial and institutional wastewater flows.* By using flow rates from similar establishments in adjacent communities, it was found that an average per employee flow rate of 25 gpd was appropriate. Current flow from this sector is estimated to be 3800 gpd, and future average flow would be 900 employees times 25 gpcd, or 22,500 gpd.
4. *Estimate the industrial wastewater flows.* Current industrial flow is negligible. Projection of future flow requires a thorough analysis of the planned industrial park. It was determined in consultation with the developer, municipal officials, and the zoning board that the ultimate future industrial park would have 800 employees and one significant water-intensive industry. The water-intensive industry will discharge process water to the sewer at the rates of 40,000 gpd (average) and 80,000 gpd (peak hour). Total average future flow is estimated to be 800 employees times an allowance of 25 gpd per employee (20,000 gpd) plus 40,000 gpd for process water. These two factors total 60,000 gpd of average future flow.
5. *Estimate an allowance for the peak quantity of future infiltration.* Because the sewers are of new construction, the future allowance for infiltration will be 15,000 gpd/mi of sewer. Estimate sewer length roughly at 1 mi of sewer per 25 acres of service area. Estimated sewer length for the 340-acre area is 13.6 mi. Allowance for infiltration will be 15,000 gpd/mi times 13.6 mi of sewer, or 204,000 gpd.
6. *Determine the peaking factor.* The peaking factor is taken from Fig. C12.1 using curve *G*. Where significant nonresidential sources are present, it is common to substitute population equivalent for population in the equation. Population equivalents of commercial and industrial flow are estimated by dividing the per employee water rate by the per resident water rate and multiplying by the number of employees. For the future condition, the population equivalent of the commercial industrial component is 25 gpcd divided by 78 gpcd (weighted-average rate of single-family and apartment use) times 1700 employees (900 commercial and 800 industrial), or 545. The total population equivalents in the future will be 3500 population plus 545 population equivalents for commercial and industrial sector, or 4045. The peaking factor from curve *G* would be 3.33.
7. *Estimate peak design flows.* Peak design flows are calculated by first assessing the average flow rate from residential, commercial, and industrial sectors excluding infiltration. This would be 272,500 gpd (residential) plus 22,500 gpd (commercial) plus 20,000 gpd (industrial), for a total wastewater flow of 315,000 gpd. Multiply

this sum by the peaking factor of 3.33 to obtain a peak hourly flow of 1,050,000 gpd. Add to this the peak allowance for infiltration of 204,000 gpd and peak flow of process water from the water-intensive industry of 80,000 gpd. The total peak hour design flow would be 1,334,000 gpd.

QUANTITY OF STORMWATER FLOWS

Stormwater runoff is the portion of precipitation which flows over the ground surface during and after a precipitation event. The design of storm sewers and combined sewers requires the estimation of design stormwater flows. There are a number of methods to develop peak stormwater flows. Among these are the rational formula, the hydrograph method, the inlet method, and the unit hydrograph method. These basic methodologies have been incorporated into a number of proprietary and public-domain computer programs. Among the public-domain programs, Illinois Urban Drainage Area Simulator (ILLUDAS) and Stormwater Management Model (SWMM) are in common use in the profession in the United States.

Discussion of each of the methods is beyond the scope of this chapter. As such, discussion here is limited to a description of the rational method, the simplest and most basic analytic procedure in common use.

Design Flows

The development of design flows for storm sewers is based on the area-specific political, economic, regulatory, and meteorological conditions. Typical practice is to design the system to accommodate the peak flows of a rainfall event that has a probability of recurring every 3 to 10 years. For commercial and high-value districts, the design recurrence interval could be as high as 30 to 50 years. The duration and intensity of that rainfall, established from the area's meteorological history, are used to estimate the peak design flows.

The Rational Method

The rational method is the most basic procedure in common use for the computation of rates of stormwater runoff for storm sewer design. The rate of runoff Q in cubic feet per second (m^3/hr) is given by the equation

$$Q = CiA \tag{C12.1}$$

$$Q = 10CiA \qquad \text{SI units} \tag{C12.1M}$$

in which A is the size of the drainage area in acres (hectares), i is the average intensity of rainfall during the design event in inches per hour (mm/h) for a duration equal to the time of concentration of the area, and C is a runoff coefficient whose value depends principally upon the character of the area.

The assumption behind the rational method is that the runoff rate at a point in a drainage basin for a given rainfall intensity will increase and reach its maximum when the duration of the rainfall is equal to the time of concentration of the area (the time required for the runoff to flow from the remotest point of the contributory area to the point where Q is being measured). By this assumption, the maximum runoff rate Q which can be expected to occur with any given frequency will be produced by a storm having the average rainfall intensity corresponding to the given frequency and duration which equal the time of concentration. The application of the rational method requires knowledge of the rainfall intensity-duration-frequency characteristics for the locality. Although the assumptions are not strictly in accord with the mechanics of the runoff process, the rational method has proved to be a practical procedure for storm sewer design because the accumulated experience has resulted in practicable values for the runoff coefficient. Reported practice generally limits the use of the rational method to urban areas less than 5 mi^2 (15 km^2). For larger areas, storm sewers designed using the rational method will result in an overdesign of the storm sewer sizes. Therefore, for larger areas, application of hydrograph methods is usually warranted to avoid overdesign of the storm sewers.

Time of Concentration

The time of concentration is the time required for the runoff to flow from the remotest point of the drainage area to the point under design. This is the minimum time necessary to permit the drainage area to contribute to the flow and generate the maximum flow rate at the point. The time of concentration consists of the inlet time, or time required for the runoff at the upper end of the drainage area to reach the nearest inlet to a sewer, plus the time of flow in the sewer from this inlet to the point being considered.

Inlet time will vary with the nature of the surface (e.g., grassy, paved), the slopes, the nature of the established drainage channels such as street gutters, and the antecedent conditions. Because the inlet time is small, it is commonly chosen on the basis of experience. In densely developed areas with a high percentage of paved surfaces and closely spaced inlets, an inlet time as low as 5 min may be considered reasonable. In moderately developed urban areas with flat slopes, the inlet time may vary from 10 to 20 min. In flat residential areas with a relatively low percentage of paved surface, the inlet time may be as high as 30 min. It is possible to make estimates of the inlet time by calculating the time of flow over the various types of surfaces, but such estimates can rarely be made with a high degree of accuracy.

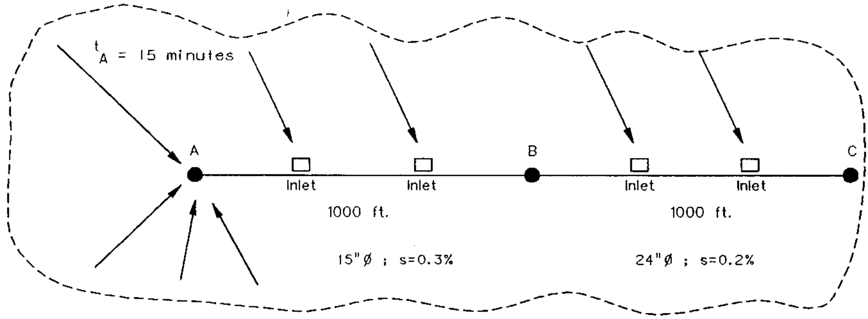
The time of flow in the sewer is computed from the hydraulic properties of the sewer, the common practice being to use the average full-flow velocity for the sewer computed for the prevailing ground slope. Alternatively a velocity of 3 to 5 ft/s (1 to 1.5 m/s) can be assumed for initial calculations. The use of full-flow velocity in the calculation of the time of flow in the sewer is a convenient and reasonable method of making the initial determination of time of concentration for hand calculations. The design can be checked to account for variance in travel times for partly full or surcharged sewers and rainfall patterns during the design storm.

The calculation of the time of concentration is illustrated by example C12.2.

Example C12.2. Determine the time of concentration for the drainage system shown in Fig. C12.2. Assume Manning's roughness coefficient for the pipe sections to be 0.013.

Solution. The following information is derived from Fig. C12.2:

Pipe section AB: Length, 1000 ft; diameter of the pipe, 15 in (1.25 ft); slope of pipe s, 0.003; Mannings roughness coefficient n, 0.013.



Time of Concentration for Design Point C =

$$
\begin{aligned}
& t_A = 15.0 \text{ minutes} \\
+ \; & t_{A-B} = 5.7 \text{ minutes} \\
+ \; & t_{B-C} = 5.2 \text{ minutes} \\
\hline
& t_C = 25.9 \text{ minutes}
\end{aligned}
$$

FIGURE C12.2 Calculation of time of concentration.

Pipe section BC: Length, 1000 ft; diameter of the pipe, 24 in (2 ft); slope of pipe *s*, 0.002; Mannings roughness coefficient *n*, 0.013.

First, estimate the time for conveyance of runoff from the remotest catchment area to the remotest inlet to the sewer. In a moderately developed urban area, use 15 min for flow from the catchment area to travel to point *A*. Next calculate the time for flow to travel from *A* to *B*, using Manning's equation, (C12.2). For metric units, Eq. (C12.2M) must be used. For pipe section *AB*, the parameters *n* and *s* are 0.013 and 0.003, respectively. Parameter *r* (hydraulic radius) must be calculated. For pipe running, the full value of *r* is $\pi D^2/4/\pi \cdot D$ or $D/4$ where *D* is the diameter of the pipe in feet (for SI, *D* must be expressed in meters). By substituting the values of the parameters in Eq. (C12.2), the velocity in the *A*-to-*B* section at full flow is 2.9 fps. The time to travel from *A* to *B* is 1000 ft divided by 2.9 ft/s, or 5.7 min. Similarly, the time to travel from *B* to *C* at a full-flow velocity of 3.2 ft/s is 5.2 min. The time of concentration, therefore, to point *A* is 15 min, to point *B* is 20.7 min, and to point *C* is 25.9 min. For drainage systems described in metric units, use Manning's equation (C12.2M).

Rainfall Frequency

It is usually prohibitive, on the basis of cost, to construct storm sewers capable of handling the largest conceivable storm. Current practice is to use storm rainfalls having an average expected frequency of once every 3 to 10 years for the design of storm sewers in residential areas and storms of 10 to 30 years for commercial and high-value districts.

Rainfall Intensity-Duration-Frequency Relationships

The rainfall characteristics which must be known for stormsewer design are presented in a concise manner by the intensity-duration-frequency curves, which can be prepared from a long record of precipitation at a given station. Figure C12.3 shows such a set of curves prepared by the Illinois State Water Survey[4] for the northeastern area of Illinois for the years 1901 to 1983[4]. This figure shows, e.g., that for a system to be designed with a time of concentration of 30 min and a design recurrence interval of 5 years, the total precipitation is 1.3 in. This means that the total rainfall of 1.3 in during a 30-min period will be equaled or exceeded in the area for which the curves apply, on average, once every 5 years. The rainfall intensity term in the rational method equation [Eq. (C12.1)] is expressed in units of inches per hour. For the 30-min-duration, 5-year recurrent storm, the rainfall intensity would be 1.3 in divided by 0.5 h, or 2.6 in/h. Similar data to those shown in Fig. C12.3 for many other localities have been compiled and published by the U.S. Weather Bureau. State agencies in charge of stormwater management should be consulted to obtain rainfall intensity-duration-frequency data accepted for use in local areas, as such data vary widely by geographical region.

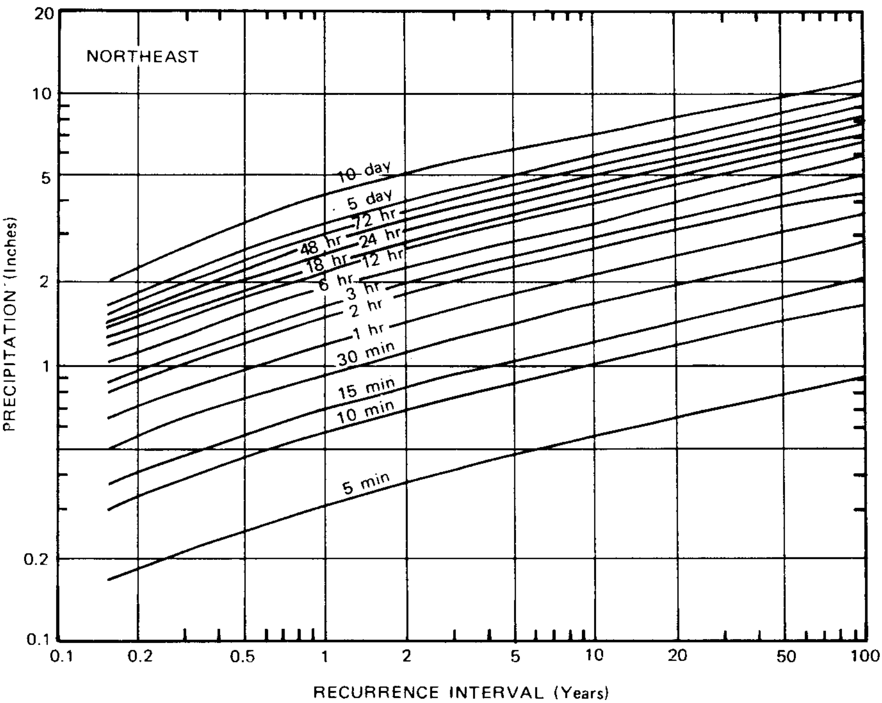


FIGURE C12.3 Frequency distributions of rainfall for Northeast Illinois climatic section for storm periods of 5 min to 10 days and recurrence intervals of 2 months to 100 years. *(From F. A. Huff and J. R. Angel, "Frequency Distributions and Hydroclimatic Characteristics of Heavy Rainstorms in Illinois," Illinois State Water Survey, 1989.)*

Runoff Coefficient

The runoff coefficient C in the rational method formula is the variable which is least susceptible to precise determination. Whereas the use of the runoff coefficient implies that there is a constant ratio of runoff to rainfall, the actual ratio for a given area will depend upon the condition of the area at the time of occurrence of the storm and will increase with the duration of the storm. A more logical procedure than the rational method for storm sewer design would be to subtract the rainfall losses due to infiltration and retention in surface depressions and to distribute the remainder as an actual hydrograph of runoff. Computer programs attempt to account for such parameters. However, because of the great variability in the time distribution of the rainfall itself as well as the difficulty in estimating the quantities of infiltration and surface depression storage, the use of the rational formula simply involves an estimate of the value of the runoff coefficient C. A common practice is to use average coefficients for various types of districts, the coefficients being assumed to be constant throughout the storm duration. The range of values for the runoff coefficient reported to be in common use is shown in Table C12.3.

TABLE C12.3 Runoff Coefficients by Land Use

Type of area	Runoff coefficient C
Business:	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroads, yard areas	0.20–0.40
Unimproved areas	0.10–0.30

Source: Adapted from Ref. 2.

For specific small areas it is more logical to relate the value of C to the actual type of surface. Coefficients commonly used are shown in Table C12.4. When an area is made up of different types of surfaces, a common procedure is to use a weighted-average coefficient.

The coefficients in Tables C12.3 and C12.4 are designed for use for storms of 5- to 10-yr frequencies. For less-frequent, higher-intensity storms, the coefficients should be higher, because the infiltration and surface retention will be smaller proportions of the total precipitation. Likewise, for more-frequent, lower-intensity storms, the coefficients should be lower than indicated in the tables.

TABLE C12.4 Runoff Coefficients by Surface Characteristics

Surface	Runoff coefficient
Streets	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95
Lawns, sandy soil:	
Flat, 2%	0.05–0.10
Average, 2 to 7%	0.10–0.15
Steep, 7%	0.15–0.20
Lawns, heavy soil:	
Flat, 2%	0.13–0.17
Average, 2 to 7%	0.18–0.22
Steep, 7%	0.25–0.35

Note: Percentages given for lawns are average ground surface slopes in the catchment area. Runoff coefficients for streets, drives, and roofs are little affected by the slope of the surfaces.

Source: Adapted from Ref. 2.

HYDRAULICS OF SEWERS

Where possible, it is common practice to design sewers to flow with a free water surface. Hydraulically, this condition is termed *open-channel flow*. The advantages of open-channel flow are twofold: (1) The free water surface will allow ventilation of the sewers, and (2) the velocities at lower flows can be kept reasonably high to facilitate self-cleansing of the sewers. In areas where the depth of excavation for a sewer becomes uneconomically large, a lift (pumping) station and force main are commonly installed to convey wastewater to a location where gravity open-channel flow can resume. The force main is termed such because sewage is ‘‘forced’’ by pumping and the flow is pressurized flow as opposed to open-channel(gravity) flow.

It is common practice to design sanitary sewers with slopes sufficient to provide for velocities of 2 ft/s (0.6 m/s) when flowing full. Experience shows that with such slopes, trouble from deposits is seldom encountered. Storm sewers are commonly designed for a minimum full-flow velocity of 3 ft/s (1 m/s) in order to resuspend sediment deposited from intermittent storm events.

Although the flow in sewers is seldom steady or uniform, it is impracticable in most cases to take this into account, and each section of the sewer is usually designed with the assumption that the flow is steady and uniform. In certain conditions it is important to check the impacts of nonuniform flow. These conditions include the drawdown, backwater, and hydraulic jump conditions. These specialized conditions are briefly discussed later in this chapter. Specialized computer programs that dynamically route flows can analyze such conditions. Such programs, currently proprietary, should be entering the public domain in coming years.

UNIFORM-FLOW FORMULAS AND CALCULATIONS

The most widely used formula for calculating open-channel uniform flow in sewers is the *Manning equation:*

$$V = \frac{1.486}{n} r^{2/3} S^{1/2} \tag{C12.2}$$

$$V = \frac{1}{n} r^{2/3} S^{1/2} \qquad \text{(SI units)} \tag{C12.2M}$$

where V = mean velocity, ft/s (m/s); n = dimensionless roughness coefficient; r = hydraulic radius, ft (m), which is the wetted cross-sectional area divided by the wetted perimeter; and S is the slope of the energy gradient.

The roughness coefficient n varies from 0.010 for smooth surfaces to as high as 0.10 for rough natural channels. It is common practice to use values of Manning's n of 0.013 for sewer design. This value makes some allowance for the future condition of the pipe as well as for disturbances in the flow resulting from rough joints and interior coatings of grease or other matter. Figure C12.4 is a diagram for the solution of the Manning equation applied to circular pipes flowing full, with n equal to 0.013.

Pipes Flowing Full

The Manning equation can be transformed to conveniently determine the quantity of flow of a circular pipe *running full* at a particular slope:

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{C12.3}$$

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2} \qquad \text{SI units} \tag{C12.3M}$$

where Q is the quantity of flow at full flow, cfs (m^3/s); D is diameter of pipe, ft (m); and n and S are defined as above.

Similarly, other convenient transformations of the full-flow pipe Manning formula are as follows:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \tag{C12.4}$$

$$V = \frac{0.397}{n} D^{2/3} S^{1/2} \qquad \text{(SI units)} \tag{C12.4M}$$

$$S = \frac{4.66}{D^{16/3}} n^2 Q^2 \tag{C12.5}$$

$$S = \frac{10.27}{d^{16/3}} n^2 Q^2 \qquad \text{(SI units)} \tag{C12.5M}$$

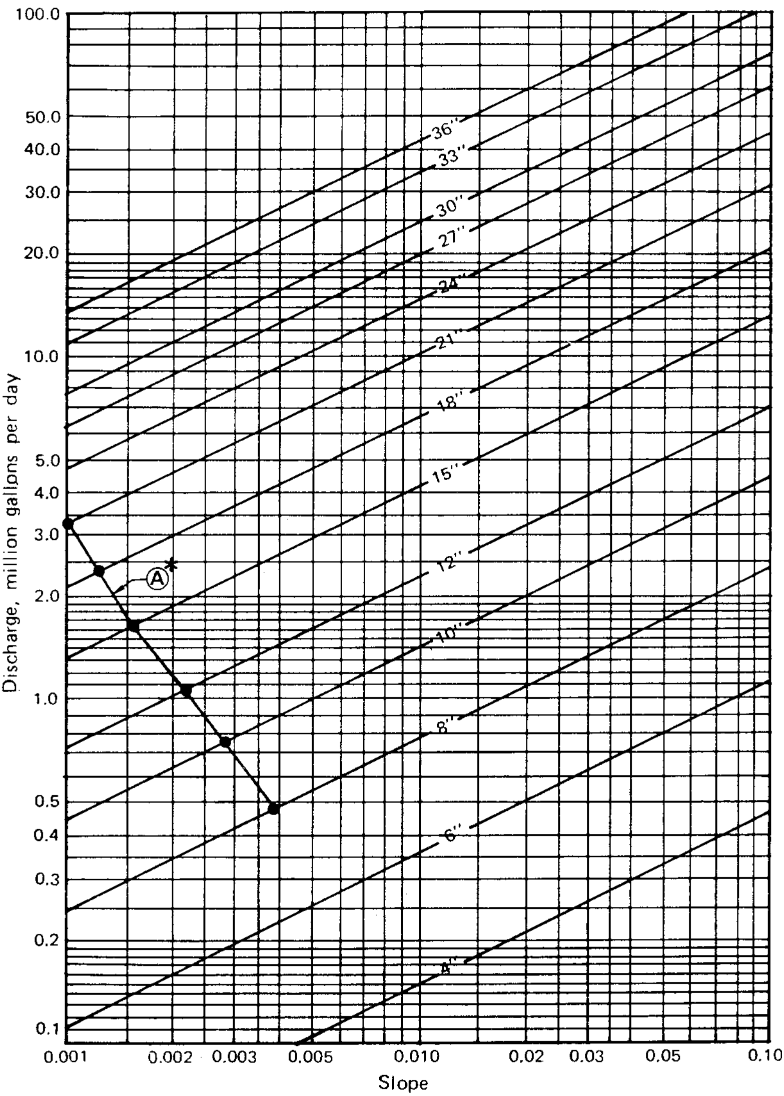


FIGURE C12.4 Discharge of circular pipes (running full) based on Manning formula:

$$Q = A\left(\frac{1.486}{n}\right)r^{2/3}S^{1/2}$$

where Q = flow rate, A = cross-sectional area, r = hydraulic radius, S = slope of energy gradient, and n = coefficient of roughness = 0.013. Curve Ⓐ—Minimum Slopes per "Recommended Standards for Sewage Works," Great Lakes—Upper Mississippi River Board of State Sanitary Engineers. *(Adapted from Ref. 3.)*

$$D = \frac{1.33}{S^{3/16}} Q^{3/8} n^{3/8} \qquad \text{(C12.6)}$$

$$D = \frac{1.548}{S^{3/16}} Q^{3/8} n^{3/8} \qquad \text{(SI units)} \qquad \text{(C12.6M)}$$

where V is velocity of flow, ft/s (m/s); and D, S, Q, and n are defined as above.

Pipes Flowing Partly Full

Manning's equation and Fig. C12.4 are convenient for determining flows and velocities for pipes running full. In determining velocities and depths of flow when pipes are flowing under partially full conditions, diagrams like Fig. C12.5 are used. Figure C12.5 gives the hydraulic elements of circular pipes flowing partly full. This figure shows the ratios of the values of the various elements to the values for the flowing-full condition. The cross-sectional area and the hydraulic radius are purely geometric functions and hence are independent of n. The velocity and discharge for any particular ratio of depth to diameter depend upon whether n is assumed to be constant or variable with the depth. Velocity and discharge curves computed from both assumptions are shown.

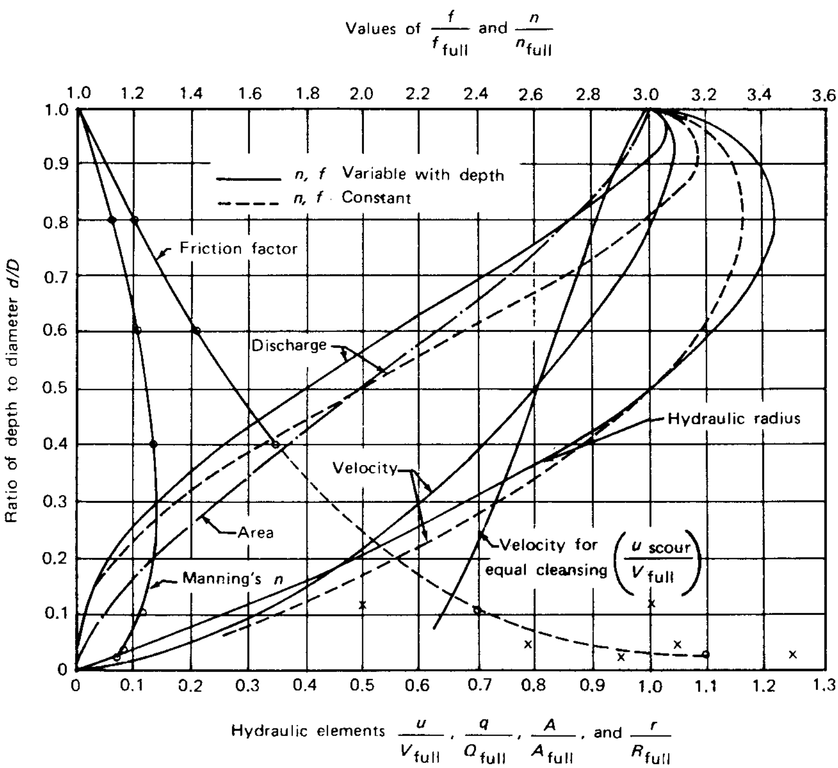


FIGURE C12.5 Hydraulic elements of circular sewers. *(Adapted from Ref. 8.)*

The use of Manning's equation, Fig. C12.4, and Fig. C12.5 is illustrated by example C12.3.

Example C12.3. A 36-in sewer pipe ($n = 0.013$) is laid on a slope of 2.00 ft per 1000 ft. Find the discharge capacity and velocity of flow of the sewer flowing full, and find the depth of flow and the velocity when the flow rate is 20.0 cfs. Assume that n varies with the depth of flow.

Solution. Using the transformed Manning equation for full flow

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2}$$

the full-flow discharge capacity is 29.8 cfs ($n = 0.013$, $D = 3.0$ ft, $S = 0.002$). This checks against Fig. C12.4 which yields a reading of 20.0 mgd (1.0 mgd $\approx$ 1.5 cfs). Velocity at full flow from Mannings's equation is 4.22 ft/s, as calculated from Eq. (C12.4).

At 20 cfs the sewer is partially full. Referring to Fig. C12.5, we see that q/Q full $= 20.0/29.8 = 0.67$. The term q is the flow in cubic feet per second for the partially full condition being analyzed. Start at the bottom axis at $q/Q = 0.67$, and read up to a point on the solid-line discharge curve (n, f variable with depth). Refer to the left axis to determine that the ratio of the actual depth to the full depth (d/D full) is 0.67. Then, reading across to the velocity curve, we see that the mean velocity at $d/D = 0.67$ is 0.94 of the full-flow velocity. Consequently, for a flow of 20 cfs, the depth of flow will be 0.67 times 36 in, or 24 in, and the mean velocity will be 0.94 times 4.22 ft/s, or 3.97 ft/s.

Figure C12.5 also shows the relative velocities required to obtain equal cleansing of the pipe at all depths of flow. This is based on T. R. Camp's analyses of the movement of granular materials in open channels.[5] The diagram indicates that if a sewer has self-cleansing velocities under flowing-full conditions, the velocity will also be self-cleansing for all flow conditions at depths greater than one-half the diameter. This concept is important for designing sewers that initially will experience relatively small flows, but are installed to accommodate large flow in the future when population growth occurs. In the early years the sewer may never experience full-flow conditions. The use of the velocity for the equal cleansing curve on Fig. C12.5 is best illustrated by example.

Example C12.4. A 36-in-diameter sewer is being installed to accommodate a future flow of 14.0 cfs. Design flow in the first 10 years of installation will be 2.5 cfs. The target full-flow velocity for self-cleansing is 2.0 ft/s. Determine the necessary velocity for equivalent scouring at 2.5 cfs flow.

Solution. First, determine d/D for 2.5 cfs. This is done by first calculating q/Q (2.5 cfs/14.0 cfs $= 0.18$). Then read up from the bottom axis to the discharge curve, and then read d/D from the left axis as 0.33. Next read to the right to the velocity for an equal cleansing curve to determine the v_{scour}/V_{full} ratio. The term v_{scour} is that velocity required at a given depth (d) that will provide equivalent scouring to that provided at full depth ($d/D = 1.0$). The v_{scour}/V_{full} ratio for $d/D = 0.33$ is 0.74. If the target self-cleansing velocity at full flow is 2.0 ft/s, then an equivalent cleansing velocity of $d/D = 0.33$ is 0.74×2.0 ft/s, or 1.48 ft/s. Determine what slope is needed to provide 1.48 ft/s velocity at $d/D = 0.33$.

Velocity at $d/D = 0.33$ is determined by reading from the left axis to the velocity curve and then down to determine v/V_{full}. The v/V_{full} ratio is 0.64. If the self-cleansing velocity of full flow is targeted at 2.0 ft/s, the actual velocity at $d/D = 0.33$ would be $2.0 \times 0.64 = 1.28$ ft/s. Because the actual velocity at $d/D = 0.33$ of 1.28 ft/s is less than the required self-cleansing velocity of 1.48 ft/s, there is a probability of deposition of solids in the first 10 years. To remedy this situation, it will be necessary to steepen the slope of the sewer to be installed to provide a full-flow velocity greater than 2.0 ft/s. However, other ramifications of steepening the slope must be evaluated to determine if this is practical.

Surcharged Flow

Although sewers are not usually designed to flow in a pressurized condition, sometimes storm sewers are designed to surcharge slightly under peak flow. The Manning equation can be used to conduct design analysis of slightly surcharged sewers. The analysis of flow under pressurized conditions in the water distribution systems and wastewater force mains has been traditionally carried out using the Hazen-Williams formula. Information on the use of the Hazen-Williams formula can be found in Chap. C2.

Example C12.5. A 36-in sewer pipe ($n = 0.013$) is laid on a slope of 2.00 ft per 1000 ft. The length of the sewer is 1000 ft. Find the level of water in the upstream manhole with respect to the sewer invert when 50 cfs is conveyed. Assume that the downstream discharge water level is equal to the crown of the sewer at the downstream end.

Solution. By solving for S, the Manning equation under full-flow conditions becomes

$$S = \frac{4.66\, Q^2 n^2}{D^{5.33}}$$

At $Q = 50$ cfs, $n = 0.013$, and $D = 3.0$ ft, S becomes 0.0056. Note that under surcharged conditions s in the Manning formula is taken to be the slope of the hydraulic gradient rather than the actual slope of the physical pipe. Therefore, for 1000 ft the required gradient would be 5.6 ft to convey 50 cfs of flow. The drop in elevation of the pipe over the 1000 ft at 0.002 slope is 2.0 ft. Therefore, the surface of the water at the upstream manhole would be surcharged 3.6 ft above the crown of the pipe.

Hydraulic Impact of Bends and Manholes

In extensive sewer systems (except in hilly areas), most of the pipes will have mild slopes, and the flows will be subcritical. Extra energy losses occur at all transitions where changes occur in size, slope, or direction of the pipe and at junctions where several pipes come together. If the transitions are properly designed to allow for these energy losses, the condition of uniform flow may be approximated in the individual lines and the flows will never be at depths greater than the depths computed on the assumption of uniform flow. However, if the transitions are not properly designed, then pipes may at times flow at depths greater than the computed depths and surcharge may occur under peak flow conditions. The hydraulic principles involved in transition design are illustrated in Fig. C12.6, which shows a transition where a pipe flows into a larger pipe laid on a flatter grade. Owing to the turbulence created by the flow expansion, there will be a head loss h_t. To prevent the upstream pipe from flowing at a depth greater than its normal depth d_1, the

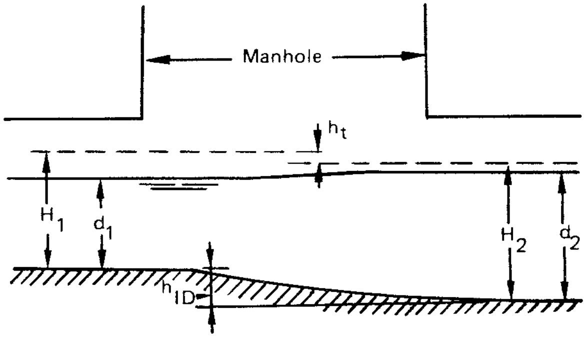


FIGURE C12.6 Flow profile at junction.

relative elevations of the bottom of the pipes must be such that the energy gradient of the downstream pipe is lower than that of the upstream pipe by the amount of h_t. This requires that the invert of the downstream be placed below that of the upstream pipe by the amount h_{ID}. The relationship between the various vertical dimensions is given by

$$h_{ID} = H_2 - H_1 + h_t \tag{C12.7}$$

The equation assumes that the loss is concentrated at the center of the transition. As actually constructed, the transition will take place within a manhole, and the channel section within the manhole is made to provide for a gradual transition between the two pipes. If the computed invert drop h_{ID} is negative, it is usually taken as zero and the pipe inverts are placed at the same elevation.

The energy loss h_t is usually small, but it can assume fairly high values when high velocities are involved. Data on the magnitudes of h_t are scarce, but such data as are available indicate that h_t can be represented as a fraction of the change in velocity heads in accordance with the equation

$$h_t = K\Delta\left(\frac{V^2}{2g}\right) \tag{C12.8}$$

where h_t is the energy loss in feet (meters) caused by the transition, K is a constant characterizing the degree of the hydraulic disturbance of the transition, V is the velocity in feet per second, and g is 32.2 ft/s per second, or ft/s^2 (9.81 m/s^2).

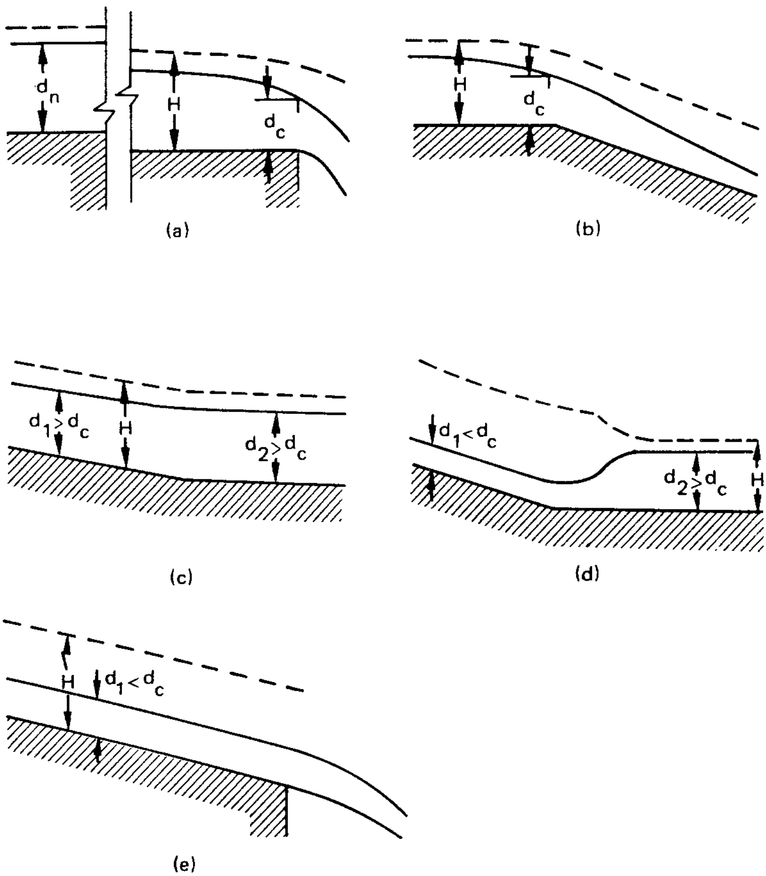
Values of K for smooth transitions are as low as 0.10 for increasing velocity transitions and 0.20 for decreasing velocity transitions. Increased transition losses occur when a sewer line changes direction and at junctions where one or more branch sewers join a main sewer. Reliable information on the transition head losses in such cases is almost entirely lacking. The hydraulic design of junctions may be considered as the design of two or more transitions, one for each path of flow. The exit sewer is common to all paths, and its invert must be placed at the lowest computed elevation. Because of the lack of information on the transition losses, allowances are usually made in accordance with the judgment of the designer. An arbitrary procedure which is commonly adopted is to allow about twice as much loss along flow paths in junctions as the allowances for simple transitions involving the same velocities.

ASPECTS OF NONUNIFORM FLOW

As stated previously, routine design procedures for sewers assume uniform-flow conditions. This assumption, although not precisely correct, is adequate in the majority of situations. Note that some situations require a more thorough analysis of nonuniform flow conditions.

The significance of some nonuniform flow conditions is illustrated by reference to Fig. C12.7. The flow conditions for each of the cases shown are explained briefly below:

1. A channel with a mild slope discharges freely into the atmosphere (Fig. C12.7*a*). The critical depth will occur at the outlet. The depth will increase at successive sections upstream until the normal depth is reached, beyond which the flow will



Note:
d_n = normal depth, d_c = critical depth, $d_{1,2}$ = actual flow depth, H = energy head

FIGURE C12.7 Examples of nonuniform flow profiles.

be uniform. In many cases the length of the channel will be much less than the distance required to develop normal depth. The significance of this fact is that for a short stretch of pipe, its actual capacity to carry flow without surcharge may be much greater than would be calculated by assuming uniform flow.

2. A channel changes slope from mild to steep (Fig. C12.7*b*). The conditions upstream of the junction will be the same as for case 1. The depth downstream of the junction will decrease and approach normal depth.

3. A channel changes slope from a mild slope to another mild slope (both slopes steeper than the uniform-flow slope) (Fig. C12.7*c*). There will be a gradual decrease in the depth from section to section.

4. A channel changes slope from steep to mild (Fig. C12.7*d*). In this case, the change from the upstream supercritical flow to the downstream subcritical flow will take place suddenly in a ''hydraulic jump.'' The position of the jump may be either upstream or downstream of the junction, depending upon the relative values of the various parameters which control the flow pattern.

5. A steep slope discharges into the atmosphere (Fig. C12.7*e*). In this case the flow will be at the normal supercritical flow, provided that the upstream control has permitted normal depth to be developed.

The foregoing discussion on nonuniform flow is presented to show the reader the importance of understanding these principles. For detailed presentation of nonuniform flow, the reader is referred to textbooks on the subject.[6,7]

DESIGN ASPECTS OF SEWERS

Sewer designs are governed by minimum design standards in many areas. In the United States, various city, county, or state standards may apply. Various countries also have national standards. Many of these standards repeat similar design criteria.

Commonly Used Design Criteria

Commonly used design criteria include the following:

- *Minimum size for gravity sewers:* 8-in (200-mm) diameter.
- *Minimum depth:* Sufficient to receive sewage from basements and prevent freezing.
- *Slope:*

  As required to produce velocities no less than 2.0 ft/s (0.6 m/s) when flowing full at $n = 0.013$.

  Uniform slope between manholes.

  Sewers with velocities greater than 15 ft/s (4.5 m/s) should be properly protected.
- *Alignment:* Sewers 24 in (600 mm) or less should be straight between manholes.
- *Changes in pipe size:* When smaller sewer joins a larger pipe, the invert of the larger sewer should be lowered to maintain the energy gradient. This criterion is approximately met by matching the 0.8 depth point of both sewers, or by matching crowns in smaller pipes.
- *Manholes:* Manholes should be placed at the end of each line; at all changes in grade, size, or alignment; at all intersections of sewers; and at intervals of not

more than approximately 400 ft (120 m) in smaller sewers and 600 ft (180 m) in larger sewers.

- *Relation to water mains:* Sewers shall be at least 10 ft horizontally from water mains, and where they cross, an 18-in (450-mm) minimum separation shall be maintained.
- *Design pressure:* Sewers convey wastewater under atmospheric pressure. In some instances, where the wastewater must be pumped, the pipe must be designed for maximum expected internal pressure. For additional information refer to Chap. C1.
- *Design temperature:* Wastewater and stormwater are generally at ambient temperatures. The temperature of the stormwater may vary significantly depending on the season, such as winter and summer; however, no special consideration is warranted for the design of underground sewer pipes.

DESIGN EXAMPLES

Design examples are presented for a sanitary sewer system and a storm sewer system in the following section. These examples are adapted from Ref. 8.

Design Example of Sanitary Sewer

Example C12.6. Design a sanitary sewer system for the residential district shown in Fig. C12.8. The district is two-thirds developed; therefore the probable future population density can be estimated without making a detailed population study.

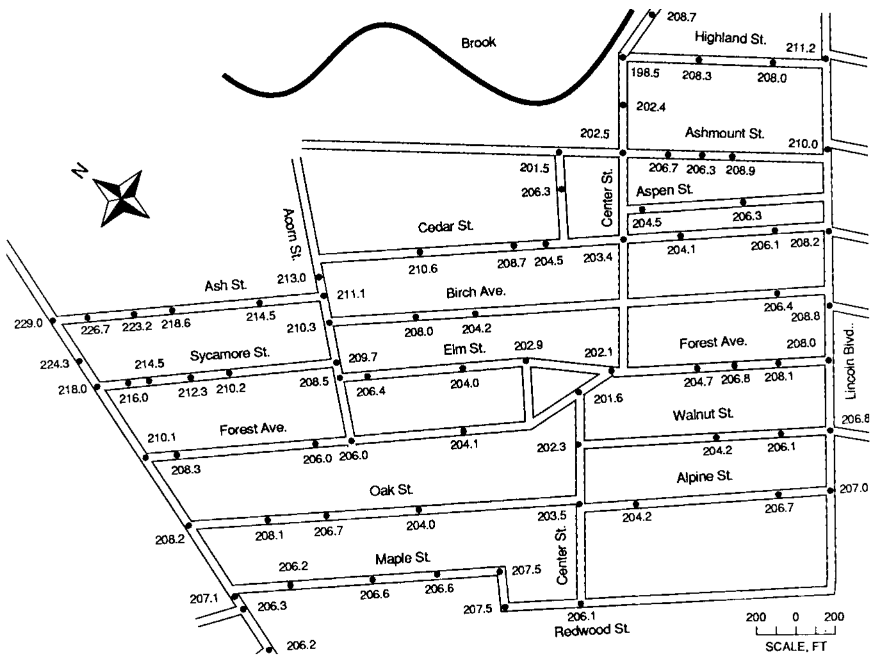


FIGURE C12.8 Typical map for the design of sanitary sewers. *(Adapted from Ref. 8.)*

It is estimated that the future average saturation population density will be 65 persons per acre. The maximum hour rate of flow of sewage is estimated at 250 gpcd. The maximum rate of groundwater infiltration to the sewers, to be provided for, is 2000 gpd per acre of the service area.

The minimum size of sewer is to be 8 in. The minimum velocity of flow in the sewer when full is to be 2.0 ft/s. The capacity of the sewers will be determined by using Manning's equation with a recommended n value of 0.013.

Since the homes in this area have basements, the minimum depth below the street surface to the top of the sewers will be 7.0 ft. (In areas where basements are not normally constructed, the depth of cover to the top of the sewer may be as little as 3.0 ft.)

Solution

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

The lines representing the system will often resemble a tree and its branches. In general, the laterals connect with the submains; and these, in turn, connect with the main or trunk sewer, which leads to the point of discharge.

2. Locate the manholes, giving each an identification number.

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

4. Measure the acreage of several service areas. At this point, the design may be represented as shown in the plan view in Fig. C12.9.

5. Prepare a tabulation, such as that shown in Table C12.5, with columns for the different steps in the computation and a line for each section of sewer between manholes. This tabulation is a concise, time-saving method and shows both the data and the results in orderly sequence for subsequent use.

Use column 1 for numbering the lines of the table, for ready reference. Determine by inspection the manhole that is farthest from the point of discharge, and enter its identification number in the first line of column 2, and the number corresponding to the manhole next on the line downstream toward the trunk sewer in column 3. Enter the name of the street or alley in column 4, the length between manholes in column 5, and the area in acres to be served by the sewer at a point just upstream up the lower manhole in column 6.

On the next line enter the corresponding data for the next stretch of sewer, and in column 7 enter the sum of the areas listed in column 6. The area in column 7 is the basis for computing the required capacity of the sewer. Enter the data for each section of sewer in the above manner, following the line to the point of discharge, including the trunk or main sewer.

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

Enter in column 9 the rate of allowance for groundwater infiltration, which is equal to the rate per acre to be provided for, multiplied by the area in column 7.

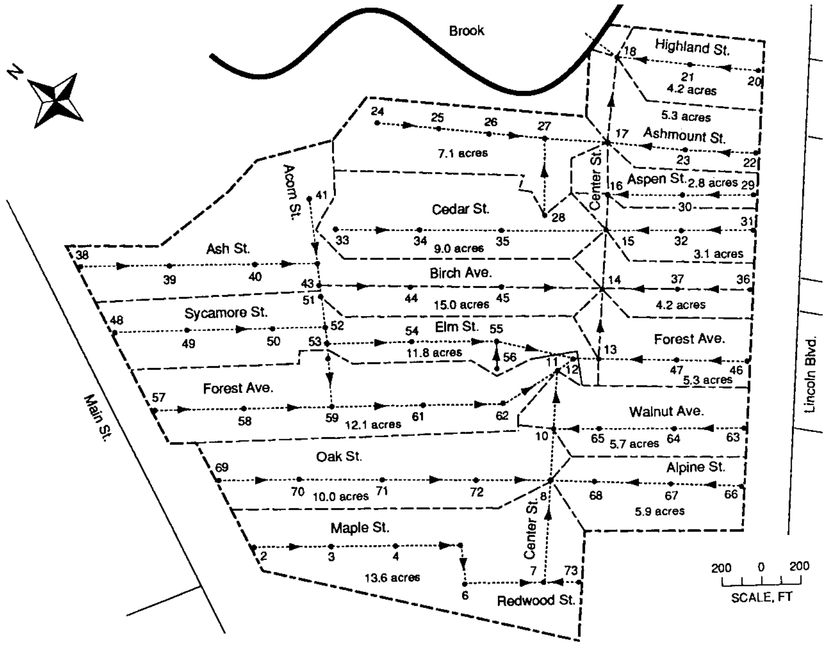


FIGURE C12.9 Map showing manholes, sewer lines, and subareas for sanitary sewer design example. *(Adapted from Ref. 8.)*

Column 10 contains the sums of the figures in columns 8 and 9, in million gallons per day. In column 11 this rate is converted to cubic feet per second, which is the more convenient way of expressing the capacity of sewers, since most diagrams and tables indicate the capacity of circular pipes in cubic feet per second (1.0 mgd = 1.54 cfs).

Column 12 contains the required sewer sizes; column 13, the slope, column 14, the velocity when the sewer is full; and column 15, the capacity. Column 16 contains the elevations of the street surface at the manhole corresponding to the identification number in column 2. Columns 17 and 18 contain the invert elevations of the upper and lower ends, respectively, of each reach of sewer.

In selecting the sewer sizes and slopes, the designer makes use of profiles, such as the one shown in Fig. C12.10. This allows the designer to select a minimum sewer size and slope that will carry the computed flow and that also will meet minimum-depth criteria.

Design of Storm Sewer System

Example C12.7. Design a storm sewer system for the area shown in Fig. C12.11. The location of the proposed main storm sewer that is to receive the stormwater from the district is shown on the map, and the invert elevation is known at the point where the proposed branch storm sewer is to be connected and for which provision has been made in the design of the main storm sewer. The required lowest elevation of the invert of the branch storm sewer is therefore known at the proposed point of discharge into the main storm sewer.

TABLE C12.5 Computations for a Sanitary Sewer

					Area, acres				Total max. flow, sewage and groundwater							Invert elevation, ft	
Line (1)	From manhole no. (2)	To manhole no. (3)	Location (4)	Length, ft (5)	Increment (6)	Total (7)	Sewage, mgd* (8)	Groundwater at 2000 gpad (9)	mgd (10)	ft^3/s (11)	Size of sewer, in (12)	Slope, ft/ft (13)	Velocity, ft/s (14)	Capacity, ft^3/s (15)	Surface elevation, upper end, ft (16)	Upper end (17)	Lower end (18)
1	57	58	Forest Ave.	380							8	0.004	2.2	0.77	208.2	200.40	198.89
2	58	59	Forest Ave.	370							8	0.004	2.2	0.77	206.4	198.89	197.40
3	59	61	Forest Ave.	365							8	0.004	2.2	0.77	205.2	197.40	195.94
4	61	62	Forest Ave.	370							8	0.004	2.2	0.77	204.3	195.94	194.46
5	62	11	Forest Ave.	240		12.1	0.196	0.024	0.220	0.34	8	0.004	2.2	0.77	202.0	194.46	193.50
6	11	12	Forest Ave.	130	35.2	47.3	0.767	0.095	0.862	1.34	12	0.0023	2.2	1.70	201.6	189.30	189.00
7	12	13	Forest Ave.	82	11.8	59.1	0.960	0.118	1.078	1.67	12	0.0023	2.2	1.70	202.1	189.00	188.61
8	13	14	Center St.	280	5.3	64.4	1.046	0.129	1.175	1.82	15	0.0017	2.3	2.70	202.8	188.56	188.08
9	14	15	Center St.	275	19.2	83.6	1.360	0.167	1.527	2.37	15	0.0017	2.3	2.70	203.2	188.08	187.61
10	15	16	Center St.	113	12.1	95.7	1.555	0.191	1.746	2.70	15	0.0020	2.4	2.90	203.6	187.61	187.38
11	16	17	Center St.	245	2.8	98.5	1.600	0.197	1.797	2.78	15	0.0024	2.6	3.20	203.7	187.38	186.79
12	17	18	Center St.	375	12.4	110.9	1.800	0.222	2.022	3.13	15	0.0024	2.6	3.20	203.3	186.79	185.89
13	18	19	Right of way	130	4.2	115.1	1.871	0.230	2.101	3.26	15	0.0025	2.7	3.27	196.0	185.89	185.56

* Based upon a maximum rate of 250 gpcd and 65 persons per acre.

Note: ft × 0.3048 = m; acre × 0.4048 = hectare; mgd × 3785 = m^3/d; ft^3/s × 28.3 = l/s; in × 25.4 = mm; ft/s × 0.3048 = m/s; mgd = million gallons per day; gpad = gallons per acre per day; gpcd = gallons per capita per day.

Source: Adapted from Ref. 8.

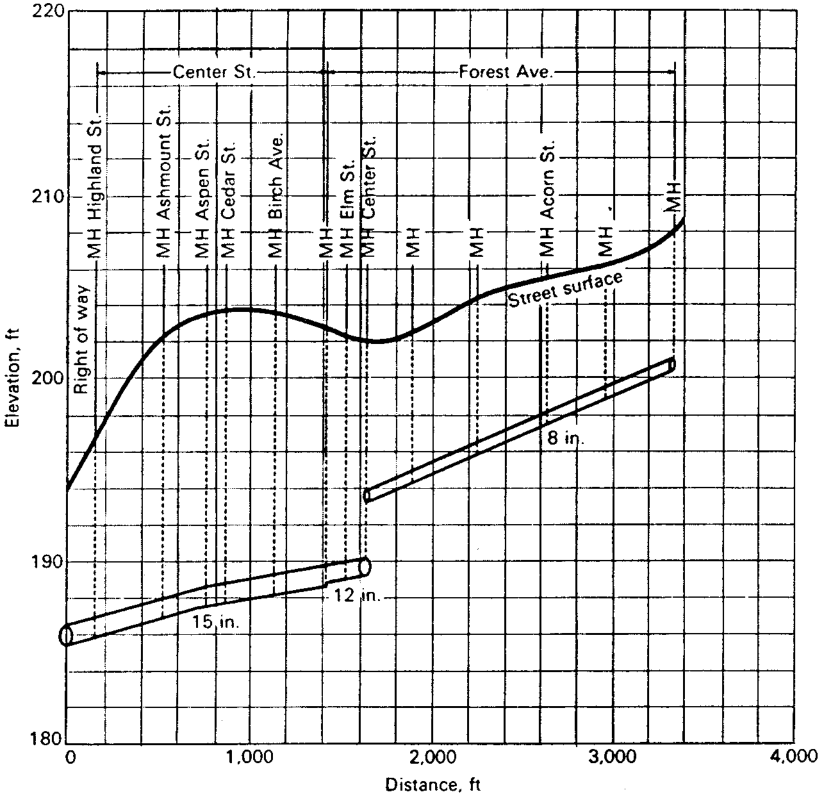


FIGURE C12.10 Typical profile for sanitary sewer design example. *(Adapted from Ref. 8.)*

A careful study of local conditions, including the present and probable future development of the district, indicates that 70 percent of the surfaces in the district are expected to be impervious. The inlet time has been assumed to be 20 min. The storm sewers are to be designed for the 5-year level of protection of the area.

For this design example, the rate of rainfall is taken from the assumed curve of intensity of precipitation represented by the formula $i = 20.4/t^{0.61}$, in which i is rainfall intensity in inches per hour and t is rainfall duration in minutes. This formula represents the average rate of rainfall for a duration of t min which may be expected to be equaled or exceeded on the average once in a 5-year period. The rainfall and runoff curves are shown on Fig. C12.12.

Note that the intensity of precipitation curve in Fig. C12.12 is site-specific for the geographical area under consideration. For different areas different curves may exist, and they can be obtained normally from state agencies. Note also that the intensity of precipitation in the example is for the 5-year recurrent-interval storm. For the design of systems for greater or lesser levels of protection, different curves would be used. The runoff curves are provided for various coefficients of runoff C. The curves plot out the Ci terms in the rational method equation $Q = CiA$. Units for runoff are in inches of runoff per hour.

To illustrate the use of the chart, assume that a particular area has a time of

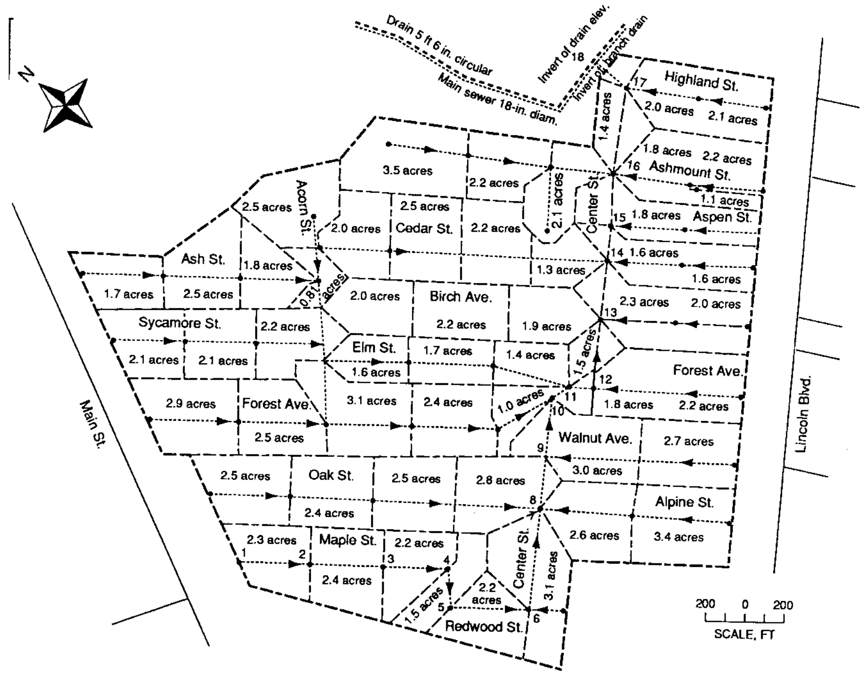


FIGURE C12.11 Map for storm sewer design example. *(Adapted from Ref. 8.)*

concentration of 60 min and that the area is 70 percent impervious. The runoff would be read directly from the *Ci* curve for 70 percent imperviousness at 1.02 in/h of runoff. Multiplying this number by the drainage area in acres yields the runoff flow in cubic feet per minute (1 cfs is almost exactly equivalent to 1 acre-in/h). Note that a 70 percent impervious area yields a coefficient of runoff C of about 0.60 for a 60-min. duration storm. The weighted-average coefficient of runoff C of an area composed of 70 percent impervious area at a runoff coefficient of 0.80 and 30 percent pervious area with a runoff coefficient of 0.15 is 0.605. Runoff from a 60-acre area with a composite C of 0.605 for a storm with an intensity of precipitation of 1.68 in/h would be 61 cfs. This result can also be obtained by reading 1.02 in/h off the 70 percent impervious curve on Fig. C12.12 for a 60-min duration and multiplying by the area of 60 acres, to yield 61 cfs.

While it was recognized that storm sewers designed on this basis might be overtaxed on the average of once in about 5 years, it was not considered reasonable to provide for storms of greater intensity, because of the greater cost. During the earlier years of the life of storm sewers, they will be able to carry the runoff from higher rates of rainfall than they will be able to carry later, because the assumed coefficients of runoff are based upon future (with larger fraction of impervious areas) rather than present conditions, which may include a larger fraction of pervious areas. A progressive increase in impervious surface and associated runoff will be caused by the gradual substitution of roofs and paved areas for presently unimproved areas.

Figure C12.11 shows the drainage area. Street elevations are shown in Fig. C12.8. The limits of this area are influenced not only by the surface contours but also by

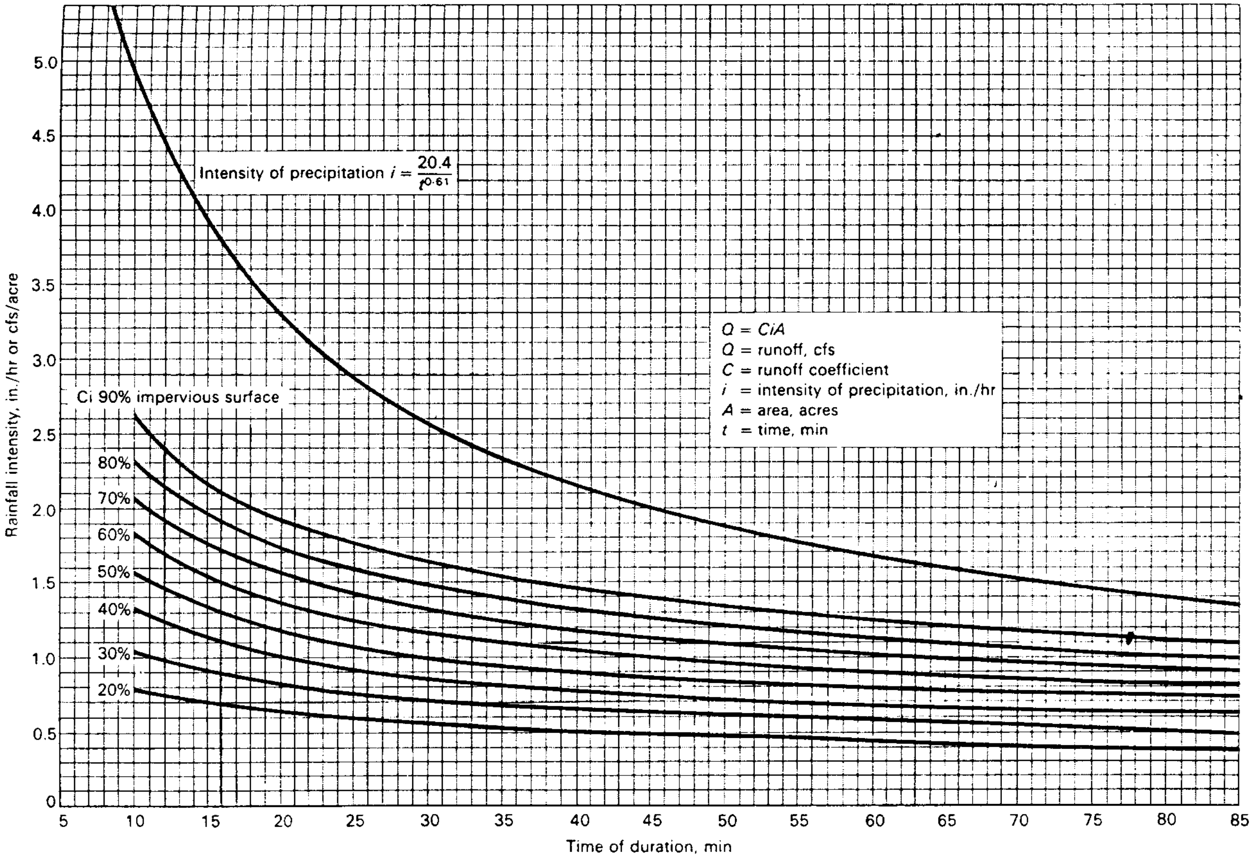


FIGURE C12.12 Precipitation-intensity curves for various degrees of imperviousness. *(Adapted from Ref. 8.)*

the service areas of existing storm sewers. In a district where the surface slopes are moderate and generally uniform, contour maps may not be required. Instead, surface elevations may be adequate if they are obtained for street intersections, for high and low points, and at locations of change of surface slope.

The storm sewers are to be designed, in general, with the crown at a depth of at least 5 ft (1.5 m) below the surface of the street. The minimum size of sewer is to be 12 in (300 mm). The assumed minimum average velocity is 3.0 ft/s when flow is at full depth.

The capacities of the sewers are to be determined by using a value of $n = 0.013$. Velocities for flow at design conditions may be higher for storm sewers than for sanitary sewers because the design storm flow is many times greater than the peak sewage flow in sanitary sewers. Because of high velocities, it is important to provide for additional head to compensate for losses, such as those due to bends, manholes, transitions, and velocity changes.

Solution

1. Draw a line to represent the storm sewer in each street or alley to be served. Place an arrow near each sewer to show the direction of flow. The sewers should, in general, slope with the street surface. It will usually prove to be more economical, however, to lay out the system so that the water will reach the main storm sewer by the most direct route.

2. Locate the manholes tentatively, giving to each an identification number. In this example, a manhole is to be placed at each bend or angle, at all junctions of storm sewers, at all points of change in size or slope, and at intermediate points where the distance exceeds 400 ft (120 m) on 12- to 24-in (300- to 600-mm) sections. Where a good velocity would be maintained during practically all conditions of flow, and the sewer is large enough for workers to walk without stooping, intervals between manholes up to 600 ft (180 m) may be used. Sufficient manholes should be built to allow access for inspection and cleaning. Later, when the profiles are drawn and the final slopes are fixed, it may be desirable to change the locations for some manholes so that the sewers would be at the most advantageous depth, particularly where the slope of the street surface was not substantially uniform. Other considerations, such as obstacles underground, may require the installation of additional manholes, due to change in alignment or special forms of construction involved in junctions or connections with other sewers.

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

4. Measure each individual area by planimeter or other methods that will give equally satisfactory results. Now use of computer programs makes it much easier to obtain very accurate results.

5. Prepare a tabulation to record the data and steps in the computations of each section of sewer between manholes.

The computations for a selected line of this section are shown in Table C12.6. Computations are carried out as follows: Columns 1, 2, 3, 4, 5, and 6 are identified from the layout map of the system in Fig. C12.11. Column 7 is the travel time to the upstream manhole considered. For line 1, 20.0 min is the inlet time for overland flow to reach the farthest inlet. For line 2 it is 20.0 min plus the travel time (1.7 min) in the sewer from points 1 to 2. The travel time in section (column 8) is calculated by dividing column 4 (length) by column 13 (velocity) and then dividing

TABLE C12.6 Computations for Storm Sewers*

							Travel time										Invert elevation, ft	
	From (1)	To (2)	Location (3)	Length, ft (4)	Increment, acres (5)	Total acres (6)	To upper end, min (7)	In section, min (8)	Rate of runoff, ft^3/s/acre (9)	Required capacity, ft^3/s (10)	Pipe size, in (11)	Slope, ft/ft (12)	Velocity, ft/s (13)	Capacity, ft^3/s (14)	Surface elevation, ft (15)	Fall, ft (16)	Upper end (17)	Lower end (18)
1	1	2	Maple St.	300	2.3	2.3	20.0	1.7	1.56	3.6	15	0.003	2.9	3.6	206.2	0.90	199.90	199.00
2	2	3	Maple St.	300	2.4	4.7	21.7	1.7	1.51	7.1	21	0.002	3.0	7.2	206.5	0.60	198.50	197.90
3	3	4	Maple St.	300	2.2	6.9	23.4	1.6	1.47	10.1	24	0.002	3.2	10.2	206.6	0.60	197.65	197.05
4	4	5	Maple St.	165	1.5	8.4	25.0	0.7	1.42	11.9	24	0.0027	3.8	11.9	207.1	0.45	197.05	196.60
5	5	6	Redwood St.	325	2.2	10.6	25.7	1.4	1.40	14.9	27	0.0023	3.8	15.0	207.4	0.75	196.35	195.60
6	6	7	Center St.	400	3.1	13.7	27.1	1.7	1.38	18.9	30	0.0021	3.9	18.8	206.1	0.84	195.35	194.51
7	7	8	Center St.	35	6.0	19.7	28.8	0.2	1.34	26.4	30	0.004	5.2	26.0	203.2	0.14	194.51	194.37
8	8	9	Center St.	230	10.2	29.9	29.0	1.0	1.34	40.0	42	0.0016	4.1	41.0	203.2	0.37	193.37	193.00
9	9	10	Center St.	240	5.7	35.6	30.0	0.9	1.32	47.0	42	0.0022	4.9	47.0	201.9	0.53	193.00	192.47
10	10	11	Forest Ave.	110	11.9	47.5	30.9	0.4	1.31	62.3	48	0.0018	4.9	62.0	201.6	0.20	191.97	191.77
11	11	12	Forest Ave.	95	11.1	58.6	31.3	0.3	1.30	76.3	54	0.0015	4.8	76.0	202.1	0.14	191.27	191.13
12	12	13	Center St.	295	5.6	64.2	31.6	0.9	1.30	83.3	54	0.0018	5.2	84.0	202.8	0.53	191.13	190.60
13	13	14	Center St.	260	17.2	81.4	32.5	0.8	1.28	104.0	60	0.0015	5.3	103.0	203.2	0.39	190.10	189.71
14	14	15	Center St.	145	13.7	95.1	33.3	0.5	1.27	121.0	66	0.0013	5.1	121.0	203.6	0.19	189.21	189.02
15	15	16	Center St.	225	2.9	98.0	33.8	0.7	1.26	124.0	66	0.0014	5.2	126.0	203.7	0.32	189.02	188.70
16	16	17	Center St.	380	13.2	111.2	34.5	1.1	1.25	139.0	66	0.0017	5.8	129.0	202.5	0.65	188.70	188.05†
17	17	18	Private land	165	4.1	115.3	35.6	—	1.24	143.0	48	0.0096	11.4	143.0	196.0	1.59	187.32	185.73

* Figures in column 8 are obtained by dividing those in column 4 by 60 and by the figures in column 13. Figures in column 10 are obtained by multiplying those in column 6 by the figures in column 9. Figures in column 16 are obtained by multiplying those in column 4 by the figures in column 12.

† The difference in the elevations of the 66-in sewer at point 17 and the 48-in outlet sewer would allow for velocity head increase and bend losses. ft × 0.3048 = m; in × 25.4 = mm; acre × 0.4048 = hectare; ft^3/s × 28.3 = l/s.

Source: Adapted from Ref. 8.

the result by 60 to convert the time to minutes. Column 9 (runoff rate) is read from Fig. C12.12, using the 70 percent impervious curve for the time listed in column 7. Column 10 (required capacity in cubic feet per second) is calculated by multiplying column 9 by column 6.

Columns 11 and 12 involve designing the pipe segment. A particular size and slope are determined from the Manning equation. Slope are determined by the constraints of (1) maintaining adequate cover of 5.0 ft over the crown of the pipe at upstream and downstream manholes, (2) maintaining a minimum full-flow capacity of 3.0 ft/s, and (3) avoiding as much as possible excessive excavation depths. Column 13 (velocity) is determined from the Manning equation and from Fig. C12.5 for determining velocities in partially filled sewers. Column 14 is the full-flow capacity of the pipe as determined from the size and slope. The capacity should equal or exceed the required capacity listed in column 10.

Column 15 (surface elevation) is taken from street maps, survey information, or topographical maps in preliminary design. Column 16 (fall) is calculated by multiplying column 12 (slope) by column 4 (length). Column 17 (invert at upstream end) presents the invert at the upstream manhole. At the farthest manhole the required depth of cover of 5.0 ft normally governs as a starting point. Note that in line 1 the invert of 199.90 plus the pipe size of 1.30 ft (15-in pipe with allowance for pipe thickness) gives an elevation of the top of the pipe as 201.20. This is 5.0 ft below the surface.

Column 18 (invert elevation at downstream end) is calculated as column 17 (upstream invert) minus column 16 (fall). The lower or downstream elevation should be checked to ensure that a minimum of 5.0 ft of cover is maintained. Note that when pipe sizes change, the crowns of the sewers are generally matched. Therefore column 17 on line 2 is 0.50 ft lower than column 18 for line 1. This is accounted for by the increase in pipe size from 15 to 21 in.

Each lateral is then designed in a similar way. If necessary, the first design of the submain is subsequently modified so as to serve the laterals properly. It is possible in some cases to omit some manholes on lateral storm sewers, using the inlet substructures at which junctions and changes in size, direction, or slope may be made.

In relatively flat areas, the design is influenced by the total available head (elevation difference) between the outlet of the system and the upstream area. In such cases, the designer needs to be cognizant of the overall limitation when starting the analysis. In such areas, larger pipe sizes at flatter slopes will be required.

Finally, the outlet conditions at the downstream end of the system may be variable, especially if the storm sewer discharges to a stream or river. In such cases, the designer must determine an outlet design condition (level of the river) to form the basis for the hydraulic computations. The outlet condition, which may be the 5-year recurrent stage level in the river, is determined in conjunction with policymakers. The outlet condition selected will, especially in flat areas, have a large impact on the design and the cost of the project.

SEWER PIPE

Available Pipe Materials

The materials of which street sewer pipes are most commonly constructed are vitrified clay pipe, concrete, plastic, and ductile iron pipe. Suggested specifications and other pertinent information about sewer pipes are presented in Table C12.7.

TABLE C12.7 General Information and Suggested Specifications for Piping and Joints

Pipe material specifications	Commercial size, in	Commercial length, ft	Joint type & material specifications	Max. buried depth, ft	Installation specifications	Remarks
Vitrified clay pipe						
ASTM C700 ASTM C700	4–12 15–24	4,6 7	ASTM C425 ASTM C425 bell-and-spigot joint & rubber gaskets	Buried depth shall be calculated based on bedding conditions. For calculation refer to clay pipe engineering manual	ASTM C12 for installation ASTM C828 for air testing ASTM C1091 for infiltration and exfiltration testing	
Concrete pipe						
ASTM C14	6–24	3½, 4, 6, 7¼	Mortar ASTM C443 bell & spigot joint, rubber gaskets	Buried depth for 6 in Sewer up to 25 ft; for 24-in sewer up to 10 ft.	ASTM C924 for air testing ASTM C969 for infiltration and exfiltration testing	Classification, 1–3 for gravity sewer only
Reinforced-concrete pipe						
ASTM C76	12–120	6, 7½, 12	ASTM C443 bell & spigot joint, rubber gaskets	35	ASTM C924 for air testing ASTM C969 for infiltration and exfiltration testing ASTM C1103 for joint acceptance testing	Classification, I–V for both pressure and gravity. For the strength of each class, see latest revision of ASTM C76.
ASTM C361	12–42 48–108	8 7½	ASTM C433 bell & spigot joint, rubber gaskets	20	ASTM C924 for air testing ASTM C969 for infiltration and exfiltration testing ASTM C1103 for joint acceptance testing	Classification A, B, C, D. For low head 25, 50, 75, 100, 125 ft only.
Reinforced-concrete elliptical culvert						
ASTM C507	14 × 23 to 63 × 93	7½	ASTM C877 external sealing bands, bell & spigot	15 ft for horizontal elliptical pipe	ASTM C924 for air testing ASTM C969 for infiltration and exfiltration testing ASTM C1103 for joint acceptance testing	Classification, HE-A, HE-I to HE-IV, VE-II to VE-VI

TABLE C12.7 General Information and Suggested Specifications for Piping and Joints *(Continued)*

Pipe material specifications	Commercial size, in	Commercial length, ft	Joint type & material specifications	Max. buried depth, ft	Installation specifications	Remarks
Reinforced-concrete box culvert						
ASTM C789	36 × 36 to 144 × 144	7½	ASTM C877 bell & spigot	60		For buried depth for specific culvert, consult with concrete products manufacturer
ASTM C850	36 × 36 to 144 × 144	7½	ASTM C877 bell and spigot	1		
PVC pipe						
ASTM D1785	⅛–12 15–27	20 20	ASTM D2672 socket joint ASTM D2564 solvent cement, bell and spigot	Buried depth shall be calculated based on bedding conditions and sewer strength	ASTM D2321 for installation ASTM D2774 for installation ASTM C969 for infiltration and exfiltration testing ASTM C924 for air testing	Schedule 40, 80, 120. For up to 6-in pipe For both pressure and gravity sewers
ASTM D2241			ASTM D2672 socket joint ASTM D2564 solvent cement ASTM D3139 for pipe joints ASTM F477 rubber gasket, bell & spigot		ASTM D2321 for installation ASTM C924 for air testing ASTM C969 for infiltration and exfiltration testing	Classification SDR from 13.5–64 For both pressure and gravity sewer
ASTM D2729	2–6	20	ASTM D2672 socket joint		ASTM D2321 for installation ASTM C924 for air testing ASTM C969 for infiltration and exfiltration testing	For gravity sewer
ASTM F679	18–36	20	ASTM D3139 for pipe joint ASTM F477 rubber gasket, bell & spigot		ASTM D2321 for installation ASTM C924 for air testing ASTM C1103 for infiltration and exfiltration	For gravity sewer
ASTM F794	4–48	20	ASTM D3139 ASTM F477 bell & spigot		ASTM D2321 for installation ASTM C924 for infiltration and exfiltration testing ASTM C1103 for joint acceptance testing	For gravity sewer
Ductile iron pipe						
AWWA C151/ ANSI A21.51	4–36	18	AWWA C111 ANSI A21.11 bell & spigot mechanical, push-on, flange, rubber gaskets	32		Classification, 50–56 laying condition and buried depth governs pipe classifications

Vitrified Clay Pipe. Vitrified clay pipe is manufactured in standard and extra-strength classifications. It is widely used, especially in smaller sizes because of its resistance to corrosive wastewaters. It is generally available in sizes from 4 in to 36 in (100 mm to 900 mm), in laying lengths from 1 to 10 ft. Larger-diameter (48 in) pipes may be available by special order.

Concrete Pipe. Concrete pipe is available in either plain or reinforced classifications in various strength categories. Concrete pipe is widely used especially in larger sizes. Unreinforced concrete pipe is available in sizes from 4 in to 36 in (100 mm to 900 mm), and reinforced concrete pipe is available in sizes from 12 in to 144 in (300 mm to 3600 mm) in five strength classifications. For details of classification refer to ASTM Standard C76. Joints are normally made with rubber gaskets in grooves formed in the tongue. Other jointing systems are also available.

Advantages of concrete pipe are its wide range of sizes, laying lengths, and strengths. A disadvantage of concrete pipe for sewers is that it is subject to corrosion under acidic conditions. If flow velocities are insufficient to prevent the deposition of organic solids, septic conditions may result. Hydrogen sulfide gas produced by anaerobic decomposition of organic matter becomes oxidized to produce sulfuric acid, which damages the pipe. This condition can usually be prevented by designing the sewers so that self-cleansing velocities will occur most of the time. Protective linings including coal tar, coal-tar epoxy, vinyl, and epoxy mortars can be used to prevent corrosion where wastewater is expected to be highly acidic or where deposition of solids is anticipated. Additional information can be obtained from the American Concrete Pipe Association.

Plastic Pipe. Plastic pipe used in sewage systems includes PVC (polyvinyl chloride), ABS (acrylonitrile-butadiene-styrene), and PE (polyethylene). All offer advantages of corrosion resistance, low friction characteristics, and light weight. Plastic pipes are generally less rigid. They require proper bedding and lateral support. Polyethylene and ABS pipes are also available in a variety of standard and proprietary products. The reader is referred to manufacturers for more information on these pipes. Available size ranges from 4 in to 15 in (100 mm to 375 mm).

Ductile Iron Pipe. Ductile iron pipe (DIP) is employed in sewerage primarily for force mains and for piping in and around buildings. It is generally not used for gravity sewer applications.

APPURTENANCES AND SPECIAL STRUCTURES

Essential to all sewerage systems are the appurtenant structures such as service connections, manholes, junction chambers, stormwater inlets, and diversion chambers. The design of such structures is not covered in detail in this chapter, but typical designs for the most commonly used appurtenances are presented briefly.

Building Service Connections

Figure C12.13 shows typical details of service connections to a sanitary sewer laid in a relatively shallow trench; Fig. C12.14 shows a typical connection to a deep sewer. Note that the connection shown in Fig. C12.13 makes use of either a wye branch or a tee branch in the main sewer line.

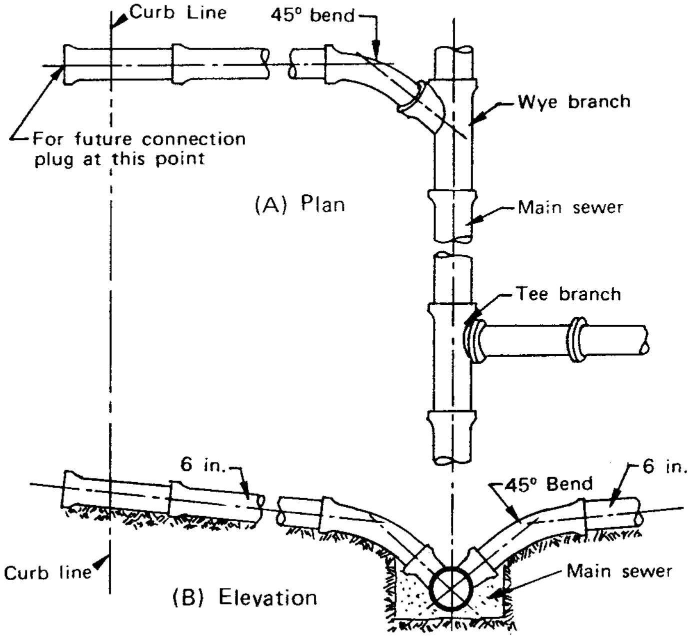


FIGURE C12.13 Typical service connections to a shallow sewer.

Junction Chambers and Manholes

Figure C12.15 shows a typical design for a junction chamber and manhole for relatively small sewers. For manholes on large sewers, a special underground structure will ordinarily be required, and the entrance to it will be provided for by a manhole located at one side. Such chambers and manholes are required at every sewer junction and at every point where the sewer changes in size, slope, direction, or elevation. It is general practice to install sewers in straight lines between manholes, except that for the larger sizes [36 in (900 mm) and above] which may be laid on curves. Manholes are usually installed at the upper end of every lateral sewer and in straight-line sewers so that the spacing will not exceed about 400 ft (120 m) to 600 ft (180 m). Figure C12.16 shows typical details of a ‘‘drop manhole’’ at a point where a sewer takes an abrupt drop in grade.

Stormwater Inlets

Stormwater inlets which carry stormwater from the streets to the storm sewers are located upstream of the crosswalks at street intersections and at low points. The designs vary considerably, and most cities have adopted their own standard design details. There are three general types of inlets: (1) curb inlets, which have a vertical opening in the curb; (2) gutter inlets, in which a horizontal opening in the gutter is covered by a cast-iron grating; and (3) combination inlets, which combine both the above features. Many types and sizes of standard castings are available for the construction of inlets.

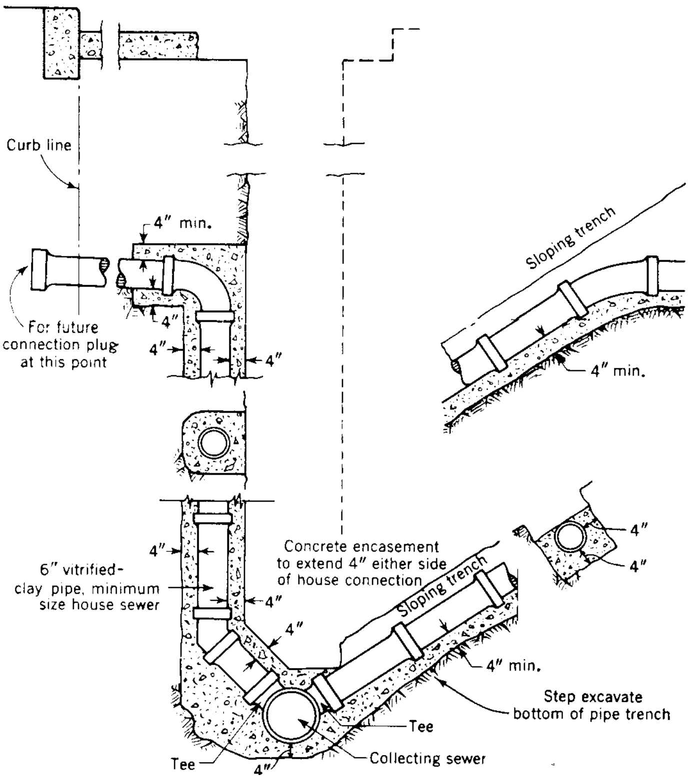


FIGURE C12.14 Typical service connection to a deep sewer.

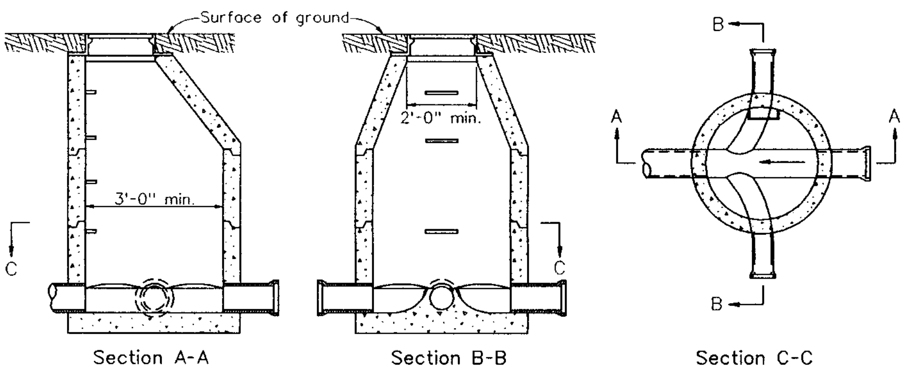


FIGURE C12.15 Junction chamber and manhole for small sewers.

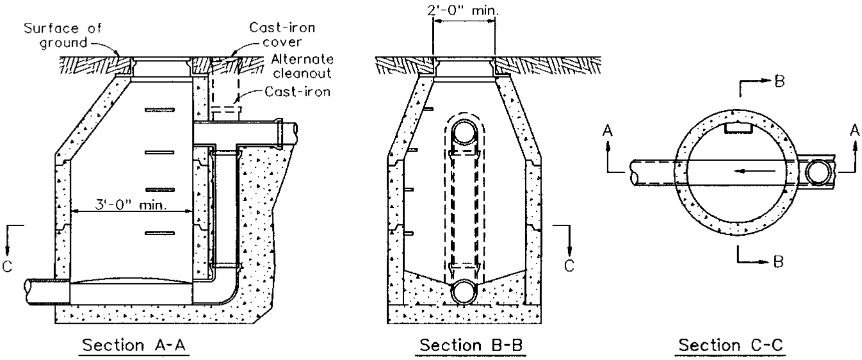


FIGURE C12.16 Drop manhole.

Sewage Pumping

In many sewer system layouts, it is necessary to provide for pumping at one or more points. The required pumping capacity will vary from a few gallons per minute for stations serving only a few laterals to many millions of gallons per day for stations serving large districts. The smaller stations are frequently built underground, either as built-in-place or as complete factory-assembled units. Pumping is usually done with nonclog centrifugal pumps, although pneumatic ejectors are sometimes used for the smaller installations. For detailed discussion of the design of sewage pumping stations, see the ASCE-WPCF manual.[2]

Structural Requirements

Sewers must be installed so as to be able to withstand the loads imposed upon them by the weight of the earth and any superimposed loads. The supporting strength of a buried pipe depends upon the installation conditions as well as the structural properties of the pipe itself. Sewer pipes are classified as rigid pipes, which cannot deform materially without cracking. For rigid pipes in trenches, the load can be represented by the equation

$$W = CwB^2 \tag{C12.9}$$

where W = load, lb/ft (kg/m) of length
w = weight of soil, lb/ft^3 (kg/m^3)
B = width of trench at the top of pipe, ft (m)
C = dimensionless coefficient whose value depends upon type of soil and ratio of depth of cover to trench width

Table C12.8 gives values for C, and Table C12.9 gives values of w to be used in the equation.

If a pipe is placed on undisturbed ground and covered with a fill, the load can be estimated from

$$W = CwD^2 \tag{C12.10}$$

TABLE C12.8 Values of C for Use in Formula $W = CwB^2$

Ratio of depth to trench width	Sand and damp topsoil	Saturated topsoil	Damp clay	Saturated clay
0.5	0.46	0.46	0.47	0.47
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.24	1.28
2.0	1.46	1.50	1.56	1.62
2.5	1.70	1.76	1.84	1.92
3.0	1.90	1.98	2.08	2.20
3.5	2.08	2.17	2.30	2.44
4.0	2.22	2.33	2.49	2.66
4.5	2.34	2.47	2.65	2.87
5.0	2.45	2.59	2.80	3.03
5.5	2.54	2.69	2.93	3.19
6.0	2.61	2.78	3.04	3.33
6.5	2.68	2.86	3.14	3.46
7.0	2.73	2.93	3.22	3.57
7.5	2.78	2.98	3.30	3.67
8.0	2.81	3.03	3.37	3.76
8.5	2.85	3.07	3.42	3.85
9.0	2.88	3.11	3.48	3.92
9.5	2.90	3.14	3.52	3.98
10.0	2.92	3.17	3.56	4.04
11.0	2.95	3.21	3.63	4.14
12.0	2.97	3.24	3.68	4.22
13.0	2.99	3.27	3.72	4.29
14.0	3.00	3.28	3.75	4.34
15.0	3.01	3.30	3.77	4.38
Very great	3.03	3.33	3.85	4.55

Source: Iowa State Univ. Eng. Expt. Sta. Bull. 47.

TABLE C12.9 Weights of Ditch-Filling Materials

Material	lb/ft^3
Dry sand	100
Ordinary (damp) sand	115
Wet sand	120
Damp clay	120
Saturated clay	130
Saturated topsoil	115
Sand and damp topsoil	100

Note: lb/ft^3 × 16.02 = kg/m^3.

TABLE C12.10 Values of *C* for Projection Conditions

Ratio of depth of cover to pipe diameter	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
C	0.6	1.2	2.0	3.0	4.2	5.6	7.5	10.0

where *D* is the pipe diameter in feet. Table C12.10 gives values of *C* for the latter condition, which is known as the *projection condition*. The load on a pipe placed in a trench will increase with the trench width until it equals the load for the projection condition. If there is a doubt as to whether the ditch condition or the projection condition controls, the load should be calculated by both formulas and the maximum value used.

TABLE C12.11 Percentage of Wheel Loads Transmitted to Underground Pipes for Unpaved Roadway or Berm Areas*

(Tabulated figures show percentage of wheel load applied to 1 lin. ft of pipe)

Depth of backfill over top of pipe, ft	Trench width at top of pipe, ft						
	1	2	3	4	5	6	7
1	17.0	26.0	28.6	29.7	29.9	30.2	30.3
2	8.3	14.2	18.3	20.7	21.8	22.7	23.0
3	4.3	8.3	11.3	13.5	14.8	15.8	16.7
4	2.5	5.2	7.2	9.0	10.3	11.5	12.3
5	1.7	3.3	5.0	6.3	7.3	8.3	9.0
6	1.0	2.3	3.7	4.7	5.5	6.2	7.0

Live loads transmitted are practically negligible below 6 ft.

* These percentages include both live load and impact transmitted to the pipe.

In addition to the load of the backfill, some allowance should be made for the superimposed loads caused by vehicles. It is usually safe to assume that H-20 wheel loads will be the greatest live loads to be supported. H-20 loads refer to trucks having a gross weight of 20 tons, 80 percent of which is on the rear axle, each rear wheel carrying 8 tons. Table C12.11 gives the percentage of wheel loads that can be assumed to be transmitted to buried pipe.[9]

Pipe Bedding Conditions

The supporting strength of a rigid pipe depends upon the type of bedding used in the installation of the pipe. Four general types of bedding conditions have been defined for ditch conduits:

Type 1: Impermissible bedding. Little or no care is taken to shape the foundation to fit the lower part of the pipe or to fill and tamp around the pipe.

Type 2: Ordinary bedding. The soil at the bottom of the trench is shaped to fit the lower part of the pipe with reasonable closeness for a width of at least 50 percent of the pipe diameter; and the remainder of the pipe is covered to a height of at least 6 in (15 cm) above its top by granular material which is hand-placed and -tamped.

Type 3: First-class bedding. The pipe is carefully bedded on fine granular material in an earth foundation carefully shaped to fit the bottom part of the pipe for a width at least 60 percent of the diameter; the remainder of the pipe is entirely surrounded to a height at least 1.0 ft (30 cm) above the top by granular materials placed by hand in layers not exceeding 6 in (15 cm) and thoroughly tamped.

Type 4: Concrete cradle bedding. The lower part of the pipe is embedded in concrete.

The load factors, or the ratios of the supporting strength to the crushing load, as determined by the three-edge bearing method (ASTM Methods C497) for the various types of bedding are generally taken as follows:

Impermissible bedding	1.1
Ordinary bedding	1.5
First-class bedding	1.9
Concrete cradle bedding	2.2–3.4

The factors for the concrete cradle bedding depend upon the amount and quality of the concrete used. The value of 2.2 will generally apply when the concrete extends from about one-quarter of the pipe diameter [with minimum of 6 in (15 cm)] below the pipe to the height where the lower 120° sector radii intersect the outside of the pipe. If the concrete is carried up to cover the entire bottom half of the pipe, the load factor may be as high as 3.4. If the entire pipe is encased in concrete with a minimum of $0.25D$ (4 in, or 10 cm) both above and below, the load factor may be as high as 4.5.

Safety Factor

The specified minimum strength by the three-edge bearing method for a rigid pipe should be divided by an appropriate safety factor in order to obtain the working strength. Some engineers use safety factors as low as 1.0 to 1.2 for reinforced-concrete pipe culverts. For street sewers a safety factor of 1.5 is recommended by the ASCE-WPCF Manual.

OPERATION AND MAINTENANCE

Protecting the investment in sewer systems requires attention to installation, testing, operations, maintenance, and cleaning.

Start-up, Operation, and Maintenance

Sanitary and storm sewer systems should be cleaned and tested prior to being put into service. For both types of sewer systems, tests should be carried out to confirm the following items:

- Alignment
- Elevations and pipe grades
- Watertightness of joints
- Location and accessibility of manholes

Relevant data should be used to prepare a sewer atlas which will include locations and elevations of all sewers. This atlas is generally crucial to good operation and maintenance of the sewer system.

After the sewer system is put into use, adequate care and maintenance must be provided to ensure continued good operation. Preventive maintenance will include

- Spot inspections of sewers for damage at manholes
- Identification and correction of system misuse such as illegal connections
- Identification of missing or damaged manhole covers
- Inspections to identify accumulations of grit and grease and the intrusion of roots into the system

Testing

Pipe joints should be tested for watertightness using an air test or a water test. Air tests should be carried out in accordance with ASTM C924. Water tests should be infiltration tests where the depth of groundwater is sufficient or exfiltration tests where groundwater is insufficient to submerge the section of the sewer to be tested. No standard testing methods are available for infiltration/exfiltration tests, but local agencies generally have maximum allowable rates.

Inspections

Periodically, the insides of sewers should also be inspected. For sewers too small to enter, inspections are done by using TV cameras which are pulled through the sewers between manholes. Modern televising systems can enter sewer laterals as small as 4 in (100 mm) in diameter and televise mains in excess of 60 in (1500 mm) diameter. The lateral inspection camera can gain access through a sewer cleanout or can be inserted from the main with the use of special lateral launching equipment. For larger sewers, inspections can also be carried out by wading or floating through them in a boat. The frequency of such inspections will depend on the type of system (sanitary or storm); the pipe material and joint types; and the local experience with the system.

In all sewer inspections, safety is a major concern, especially with sanitary sewers where there is a danger of septicity. Precautions should be taken to avoid exposure to dangerous gases or to the absence of oxygen. Adequate ventilation must be provided, and rules must be established to indicate the appropriate course of action in emergencies. In addition, all local safety requirements must be satisfied, and personnel should be given adequate training and equipment.

Cleaning

Based on results of inspections, sewers and appurtenances such as manholes and catch basins must be cleaned. Sewers can be cleaned by pulling, thrusting, or dragging some form of instrument through them. Cleaning can also be accomplished by flushing the sewer, using a sudden rush of water down the sewer at high velocity. The most common sewer cleaning systems are the hydraulic jet, sewer rodder, and bucket machines. Sewer cleaning programs are undertaken to provide cleaning of all sewers generally once every 3 to 5 years. More frequent cleaning may be required in sewers where low flows or flat slopes do not provide self-cleaning velocities.

On storm sewer systems, it is important to clean catch basins which are designed to intercept settleable solids before they enter the sewers. Catch basins are cleaned by hand, suction pumps, or grab buckets.

DISPOSAL OF WASTEWATER AND STORMWATER

Stormwater can ordinarily be disposed of by discharge into any natural drainage channel. However, increased emphasis is being placed on water quality impacts of stormwaters on the receiving streams. Storm sewer systems may require inclusion of sediment traps, wet ponds, and other features which reduce the pollution loadings to receiving streams. Sanitary sewage and industrial wastewaters containing objectionable constituents must be disposed of in accordance with the requirements of the local health and environmental authorities. The most satisfactory method of disposal of sanitary sewage is to convey it to an adequate public sewerage system. In areas which do not have public sewerage systems, individual disposal systems must be provided. These will vary in size from septic tank systems for private residences to large treatment plants handling the wastewaters from large institutions and industries. The design of such systems is beyond the scope of this handbook. See Ref. 1 for further information.

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