CHAPTER 24

WASTEWATER COLLECTION

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OVERVIEW

Proper management of wastewater is one of the most important factors for ensuring the general health of a community and its surface water quality. Much has been written about the early sewer systems, such as the underground drains installed in ancient Rome, but these early sewer systems were generally storm drains. In most urban areas, it was unlawful to discharge sanitary sewage into these drains prior to the mid-1800s. Thus, the history of sanitary sewers only covers the period from the mid-1880s to the present.

Major construction of sewer systems in many communities occurred during the 1930s period of economic depression as a part of the federal government's Works Progress Administration (WPA) program. Many smaller communities obtained their first central sewer system and sewage treatment facilities as a part of this program. This was the first significant period of sanitary sewer construction in this country. This period was also noted for the significant increase in the popularity of indoor plumbing.

World War II was a period of both urban and industrial growth throughout the United States. At the end of the war, many segments of surface streams were excessively polluted. Two of the reasons were the substantial increase in the discharge of untreated industrial wastes and the untreated overflow from combined sewers. Many older urban areas installed storm drains prior to the time of widespread adoption of indoor plumbing. When indoor plumbing became fashionable, the building sewers were connected into the existing storm drains. These combined sewer systems generally drained directly to the nearest watercourse without treatment of the flow. Also, the wastewater collected in many sanitary sewer systems received little or no treatment prior to its discharge into surface waters at that time.

The U.S. Public Health Service was the federal agency responsible for water quality–related matters prior to the establishment of the Federal Water Quality Administration in 1968. This agency later became the Environmental Protection Agency. It should also be noted that at this time federal agencies had little control over intrastate matters related to the environment. The U.S. Public Health Service, private organizations and universities in the United States, Great Britain, and Europe were conducting research and studies related to stream water quality and waterborne diseases as early as the last half of the nineteenth century. One of the most significant contributions to water quality knowledge in the first half of this century was the work of two U.S. Public Health Service officers, published in 1925, on the analysis of the oxygen balance in streams. This classical work of Streeter and Phelps remains the basis for modeling stream quality today.

It was only after World War II that the individual states began to establish agencies and enforce regulations controlling surface water pollution. The Water Quality Act of 1972 (P.L. 92-500) was the first legislation giving the federal government authority to set requirements and enforce standards on a national basis. This legislation, together with the federal funding for water quality programs and facilities, has changed the quality of sewer system design and construction.

A primary limitation to the effectiveness of any water pollution control program prior to the 1970s was the poor quality of sewage collection systems. Most community sewer systems were in a state of disrepair because of the lack of suitable materials for use in construction, poor construction practices, and poor maintenance. From 1950 to 1970, while significant progress was made in the construction of sewage treatment facilities, most sewer systems, pumping stations, and treatment facilities were still subject to flooding, overflow, and bypassing of flow, particularly during wet periods.

Segments of some sewer systems, primarily in older, larger communities, are still combined sewers. Thus, during wet periods, overflow and bypassing may occur with these systems. Federal and state programs are attempting to correct these conditions, as funds become available.

P.L. 92-500 established regulations and timetables for eliminating pollution on a uniform national basis. To accomplish these new goals it became necessary to design proper reliability into new sewer systems and to rehabilitate existing sewers. This meant using suitable construction materials and methods. The engineers of today have suitable materials and technology to design and construct sewer systems that will eliminate problems with both infiltration and exfiltration. New sewers, when designed and constructed to today's standards, should provide reliable service for many years.

The design engineer now has several choices of suitable pipe material for use in sewer construction that will provide long service and near 100 percent exclusion of both infiltration and exfiltration. Proven design procedures are available to the engineer for sizing and routing of sewers to ensure that the completed project will properly serve the contributing area. The design engineer must be thoroughly familiar with federal, state, and local laws and regulations before proceeding with a design.

This chapter presents material that, when properly applied, will result in a reliable and effective sewer system design.

Glossary of Terms

This glossary includes common terms used in regard to sewer systems. Many other terms are defined in the text as they are used.

Sewer: A conduit for conveying wastewater. Sewers may be designated as sanitary, industrial, or storm depending on the type of flow being conveyed. Sanitary sewers convey sanitary sewage that originates in a community. All the wastewater flow from a community—the flow from residential areas, commercial areas, and normal community industry such as laundries, restaurants, and service stations—is designated as domestic wastewater. Sanitary sewers are generally closed circular conduits. They are designed such that the hydraulic gradient is below the inside crown of the conduit (see section entitled "Hydraulics of Sewers").

Sewage (Wastewater): The liquid conveyed by a sewer. *Wastewater* is now the more common term. Sewage may be designated as domestic, industrial, or storm to indicate the source. Domestic sewage is also called *sanitary sewage* or *wastewater.*

House (building) spur or lateral: The conduit between the building foundation and the sewer owned and maintained by the local utility. The uppermost segments of a sewer system may also be designated as a lateral (where no other public sewer discharges into the segment). It is common practice for the local utilities to maintain the building lateral segment between the public sewer and the property line.

Submain, Main, and Trunk sewers: These terms are used subjectively in defining the type of sewer. Trunk sewers are generally the main conduits receiving the flow from a large area and would include the flow from several submain and main sewers.

Interceptor sewers: A conduit that intercepts other sewers, such as the sewer routed along a stream and intercepting sewers in the collecting system.

Outfall sewer: The conduit between the treatment facility and the receiving body of water. Also, the trunk sewer between the collection sewers and the treatment facility is often designated as an outfall sewer.

Combined sewers: A combined sewer refers to the older systems where the initial drainage was storm drainage. Before the mid-1900s it was not uncommon for sanitary sewers to be connected to existing storm sewers. Thus, the sewers conveyed sanitary and storm wastewater and were designated as combined sewers. These combined sewers generally discharged directly to a nearby body of water without treatment. Since all wastewater is now treated prior to discharge, combined sewers are no longer constructed.

Manhole: An opening to a sewer for inspection and maintenance. Manholes are generally placed at all sewer intersections, at all changes in alignment and grade of a sewer system, and at least every 400 feet. The manhole structure usually consists of a barrel section with a vertical 4-foot diameter, a top cone section reducing the circular opening to 2 feet, and a cast-iron frame and cover with a 2-foot opening.

Infiltration and Inflow: Infiltration is ground water that enters the sewer system. An infiltration problem results from broken and cracked sewers, from poor pipe material and poor joints, and from poorly constructed manholes. Additionally, concrete and clay pipes are semipervious and thus susceptible to *barrel weeping,* a principal source of infiltration in sewer systems employing these materials. Local design standards generally limit infiltration to less than 100 gallons per day per inch-mile (length of sewer line in miles multiplied by the diameter in inches). However, the engineer now has several excellent sewer pipe materials available that should provide a completely waterproof line when properly installed.

Inflow is surface water that enters the sewer system. The principal sources of inflow include inundation of manholes and unauthorized connections. Unauthorized connections can include the building of downspouts, area drains, and other connections of storm sewer to the sewer system. Ideally, all storm or surface water should be excluded from sanitary sewers.

Exfiltration: The seepage of sewage from the sewer system into the surrounding soil. Exfiltration may result from the same physical conditions as listed previously for infiltration. Infiltration may occur when the water table is above the sewer elevation, and exfiltration may occur when the water table is below it.

Sewerage: Sewage works, including the facilities for collection and conveyance of sewage.

Pump Station and Forcemain: A mechanical means of conveying collected sewage, usually over an area that cannot utilize gravity flow. A pump station normally consists of a structure to collect and hold the sewage, pumps to add energy to sewage and lift it over the land obstruction, controls to work the pumps, and a forcemain or pressure line to use for the conveyance of the sewage to a gravity system or wastewater treatment works.

Parties Involved in the Planning and Design of Sewer Systems

The design of any sewer system always involves several parties. Input from all impacted parties should be organized early in the design. The design engineer must be familiar with the local, state, and federal laws and regulations as they apply to the proposed design. The local government and often a state agency must approve the design and issue a permit for construction. Also, it is important to determine whether the downstream segments of the sewer system and the treatment facilities that receive the flow have adequate capacity. If not, a treatment plant and disposal of the treated effluent must be a part of the planning phase. The general responsibilities of the principal parties are discussed next.

Sewer system planning and design are normally a part of the overall project plans. The overall project plans should show all infrastructure, and the construction contract should provide for coordination of all utilities to be constructed.

Owner: The owner is the party that initiates the project. The owner may be a local government or utility or it may be a landowner or developer.

Legal Counsel: Legal counsel is generally needed by the private owner early in the planning stage of a project. Legal counsel provides direction to the project with respect to property rights, zoning requirements, and so on. In most urban areas, attorneys who specialize in legal requirements and procedures for land development can be found.

Engineer: The engineering firm offering services in sewer design is generally staffed to provide complete professional services, including both surveying and engineering. Thus, the term *engineer* as used herein refers to an organization that offers consulting services capable of handling the project from the beginning through the completion of construction and acceptance by the owner.

Regulatory Agencies: The engineer must be knowledgeable in the laws and regulations related to the project. It is common for sewerage projects to be under both state and local regulations. One or both levels of government may review the plans and specifications and issue a permit for construction. Generally, these agencies inspect the completed project before it is placed in operation. Most regulatory agencies have minimum design standards that represent good design practice and also ensure uniformity throughout the sewer system, which is important to the maintenance staff. It is critical that the engineer be well informed on all local design standards and procedures.

Contractor: The contractor is responsible for constructing the project in accordance with the plans and specifications.

DESIGN OF SEWER SYSTEMS

Characteristics of Sewage

Sewage is a water transport means of conveying wastes from the source. Domestic sewage conveys wastes from dwellings and the normal business, commercial, and industrial activities associated with any community, such as restaurants, laundries, and service stations. Sewage also contains the dissolved inert chemicals from the water supply as well as dissolved and suspended organic matter from the sources. The primary source of the material in sewage is the bathroom, kitchen, and laundry. Sewage also contains very high concentrations of bacteria and viruses. Sewage should always be considered as being a source of enteric pathogens.

Since sewage behaves hydraulically as water, the design engineer is only concerned with the composition of sewage when it contains substances that may be damaging to the sewer system or maintenance personnel. Principal substances of concern include those that are corrosive, present potential fire or explosive hazards, emit toxic fumes, or interfere with downstream wastewater treatment. Normal domestic sewage does not contain hazardous constituents other than pathogens. However, poor design or construction can create hazards. The accumulation of solids can result in the creation of odors as well as corrosive and toxic gases from the decomposition of the solids.

Domestic sewage is more than 99.9 percent water and generally between 0.05 to 0.075 percent dissolved and suspended solids. The level of organic matter in sewage is indicated by varying degrees of strength or weakness. A weak sewage generally results from high infiltration to the sewer system. A strong sewage indicates the presence of industrial wastes. There is little difference in sewage strength that is related to living standards.

The two most important parameters for defining the strength of sewage are the biochemical oxygen demand (BOD) and suspended solids concentration. BOD is a measure of the amount of organic matter present, expressed in terms of the oxygen required to biologically oxidize the material to a stable form. A 5-day period is generally used for the test. Thus, the five-day BOD is an expression for the strength of sewage. The use of a broad test of this type is necessary because sewage is a mixture of many organic compounds, making it nearly impossible to conduct a complete chemical analysis. Furthermore, many of the compounds present are partly refractory to biological oxidation, making a chemical analysis meaningless. The BOD test measures the strength in a manner that most nearly represents treatment accomplishment and potential impact of the flow on the receiving body of water.

Suspended solids concentration is also an important characteristic of domestic sewage. The sewer system must be designed to transport the solids. Suspended solids also represent a load to the treatment plant. Solids not removed in the treatment process may accumulate on the bottom of the receiving stream and create a pollution problem. The suspended solids in domestic sewage are generally more than 80 percent organic matter. These solids originate from fecal material, laundry, and kitchen wastes. Sewage will contain some organic material from laundry and kitchen wastes. Inorganic matter also originates from washing vegetables and from laundry wastes. Inorganic suspended material is known as *grit* or *fixed solids*.

Table 24.1 presents information on the range of strength-related parameters for domestic sewage that may be of interest to the sewer design engineer. Textbooks and other sources present more detailed analyses of sewage that would be of interest in the design of treatment works.

Quantity of Sewage

All wastewater management facilities should be designed to serve the needs of the contributing area for some time into the future. The design engineer for a wastewater treatment facility may use several methods for determining future design populations. These methods will always include population projections for the design period. In sewer design, the engineer

TABLE 24.1 Strength of Sewage in mg/L

	CONSTITUENT		
	WEAK	NORMAL	STRONG
Total solids*	400	700	1000
Suspended solids	100	200	350
5-day BOD	100	200	400

*Includes the dissolved inorganic material from the water supply

needs to know the area to be sewered and the developed density. The developed density will depend on the planned or zoned activity. The area to be served can be established from available maps or from field surveys. Most areas subject to being sewered should be included in a comprehensive plan or have been zoned for a specific land use. The use of this information permits the design engineer to establish sewage flow for an area at complete development. This information can then be used to establish sewage flow for each segment of the sewer to be designed. Sewers are normally designed to sewer the tributary land area at complete development. Specific conditions, such as political boundaries or land use planning, may also dictate the design area. It is important to design a sewer with sufficient capacity to serve the area for at least 25 years. Regulations of the local utility may control the area to be included in the design and the design requirements.

Most states and many local utilities have established minimum design standards for sewers. The engineer must be familiar with the requirements for the area where the sewer will be installed. Tables 24.2 and 24.3 present data that may be used in the absence of any local requirement. Table 24.2 presents the requirements of Fairfax County, Virginia. Table 24.3 is taken from the sewage regulations of the Virginia Department of Health. No separate allowance is needed for infiltration if the design and construction keep this source of flow below 100 gpd/in-mile of pipe.

Variation in Sewage Flow

Sewage flow from commercial, business, and industrial establishments are generated during the operating hours. Very little flow is generated from many of these sources between late evening and early morning, as well as on weekends and holidays.

Sewage flow from residential areas is associated with activities in the homes, with peak flows occurring generally from 7 to 10 A.M. and in the evening between 6 and 10 P.M. Peak flow hours will vary somewhat for different sections of the country, as they depend on the living habits of the community. Figure 24.1 shows a typical daily variation in sewage flow for a home. The sewage flow from a home will be very close to the rate of water use. However, sewage flow in a sewer system will differ from the rate of water use in two distinct ways. The peaks in sewage flow are less pronounced because the time of concentration to a measuring point will be different for different segments of the system. For example, flow from a section of the community that is 0.25 miles away will reach the measuring point after the flow from 0.10 miles away has passed. The peak sewage flow at a treatment plant will occur some time after the peak demand on the water supply because of the flow time in the sewer. The water supply is a pressure system and a demand is placed on the source at the instant water is drawn at a faucet. Thus, peaks in flow in a sewer system occur later than what is shown in Figure 24.1 and will be less pronounced. The actual conditions depend on the size and configuration of the sewer system. Note from Figure 24.1 that water use

TABLE 24.2 Recommended Average Design Flows from the Requirements of Fairfax County, Virginia

TYPE OF DEVELOPMENT		DESIGN FLOW (GPD)
Residential:	General	100/person
	Single-family	370/residence
Townhouse		300/unit
Apartment unit		300/unit
Commercial:	General	2000/acre
	Motel	130/unit
	Office	30/employee
		0.20/net square feet
Industrial:	General	10,000/acre
	Warehouse*	600/acre
School Site:	General	16/student

*Varies with type of industry

drops to near zero from about midnight until about 6 A.M. Flow in a sewer system during these hours primarily will be infiltration except for some industrial and business areas.

The design engineer must design sewers to accommodate peak flows. Tables 24.2 and 24.3 provide information on daily rates of sewage flow generation. A peak factor is used to account for the differences in the average daily flow and the instantaneous flow. Nominal infiltration and inflow is accounted for by the daily flow quantities shown in Tables 24.2 and 24.3. The table values are considered to be high for properly constructed sewers. No additional allowance needs to be made for infiltration and inflow. The average daily flows shown in Tables 24.2 and 24.3 must be adjusted for peak flow rates for use in designing a sewer. The periods of minimum flow are generally not of concern in sewer design.

Some local utilities have developed or adopted from other sources regulations for establishing peak flow. Most, if not all, states also have minimum standards. Most equations for calculating a peak factor are based on population. Two such equations are given below:

$$\text{Peak Factor} = \frac{5}{p^{0.2}} \tag{24.1}$$

$$\text{Peak Factor} = 1 + \frac{14}{4 + p^{0.5}} \tag{24.2}$$

where p is the contributing population in thousands. The peak factor should be limited to a maximum value of 5 and a minimum value of 2.5.

A more useful approach in sewer design is the use of curves to establish the peak factor. This type of curve is shown in Figures 24.2 and 24.3. The average daily flow that has been calculated by using the tributary area and appropriate densities is then multiplied by the peak factor to establish the design flow for use in sewer design.

Hydraulics of Sewers

Sewage is considered to have the same hydraulic characteristics as water. It is an incompressible fluid having viscous properties similar to water. Both gravity sewers and forcemains are generally designed as circular pipes flowing full. Design practice also assumes that the flow is steady and uniform for each segment of pipe. In instances where building connections are adding flow to a segment of line, the flow at the downstream end of the segment is used for the design of the entire segment. A manhole should be installed where sewers other than building laterals connect to the line being designed. Design segments always run from manhole to manhole. Uniform flow applies when the cross-sectional area and slope of the segment are constant throughout the length. Manholes are located at all changes in slope and pipe diameter.

The flow rate is equal to the cross-sectional area of flow times the velocity at the section. This is expressed by the continuity equation,

$$Q = AV \tag{24.3}$$

where Q is the quantity of flow passing the reference point in cubic feet per second (cfs), A is the cross-sectional area of

TABLE 24.3 Recommended Design Flows as Required by the Regulations of Virginia*

Source of Flow	Design Units	Flow (gpd)
Dwellings	Per person	100
Schools with showers and cafeteria	Per person	16
Schools without showers and cafeteria	Per person	10
Boarding schools	Per person	75
Motels (rooms only)	Per room	130
Trailer courts	Per trailer	300
Restaurants	Per seat	50
Interstate or through restaurant	Per seat	180
Factories	Per person	15–35
Hospitals	Per bed	300
Nursing homes	Per bed	200
Doctors' offices	Per 1000 square feet	500
Laundromats	Per machine	500
Community colleges	Per person	15
Swimming pools	Per swimmer	10
Theaters	Per seat	5
Picnic/park areas	Per person	5
Camps with flush toilets	Per camp site	100

The above flows are considered to be adequate to include limited infiltration.
State and local utility regulations are generally considered as being minimum design conditions.
*Waterworks Regulations, Virginia Dept. of Health (1993).

flow in square feet (sq ft or ft^2), and V is the velocity of flow at the section in feet per second (fps).

If the flow is steady and uniform as generally assumed, the continuity equation is applicable and Equation 24.4 applies.

$$Q = A_1V_1 = A_2V_2 = A_3V_3 \cdots A_nV_n \tag{24.4}$$

According to the principle of conservation of mass, mass can be neither created nor destroyed. Since Q remains constant along a segment of sewer having a constant slope and diameter, the velocity remains constant.

The energy of a particle of mass is the sum of the position of the particle plus the momentum of the particle. In a homogeneous fluid, all particles in the fluid at a reference section have the same position energy (a particle at any depth has the same energy as any particle at the surface). The position energy for open channel flow is the water surface. The position energy for a closed conduit under pressure, such as a sewage forcemain, is the surface to which the liquid would rise if a vertical open tube were inserted in the top of the conduit. The pressure is indicated by the height of the water column above the center of the conduit and is known as *pressure head.* This is shown in Figure 24.4.

The momentum of a fluid particle is called *velocity head.* When a fluid moves from a stationary state to a moving state, some position energy is converted to momentum energy. This velocity head is defined as

$$H = \text{Velocity Head} = \frac{V^2}{2g} \tag{24.5}$$

where velocity head H in feet = position energy equivalent of the momentum, V is the velocity of flow in ft/sec, and g is the acceleration of gravity in ft/sec^2.

A line that connects the values of position energy at various points along a segment of sewer is called the *hydraulic*

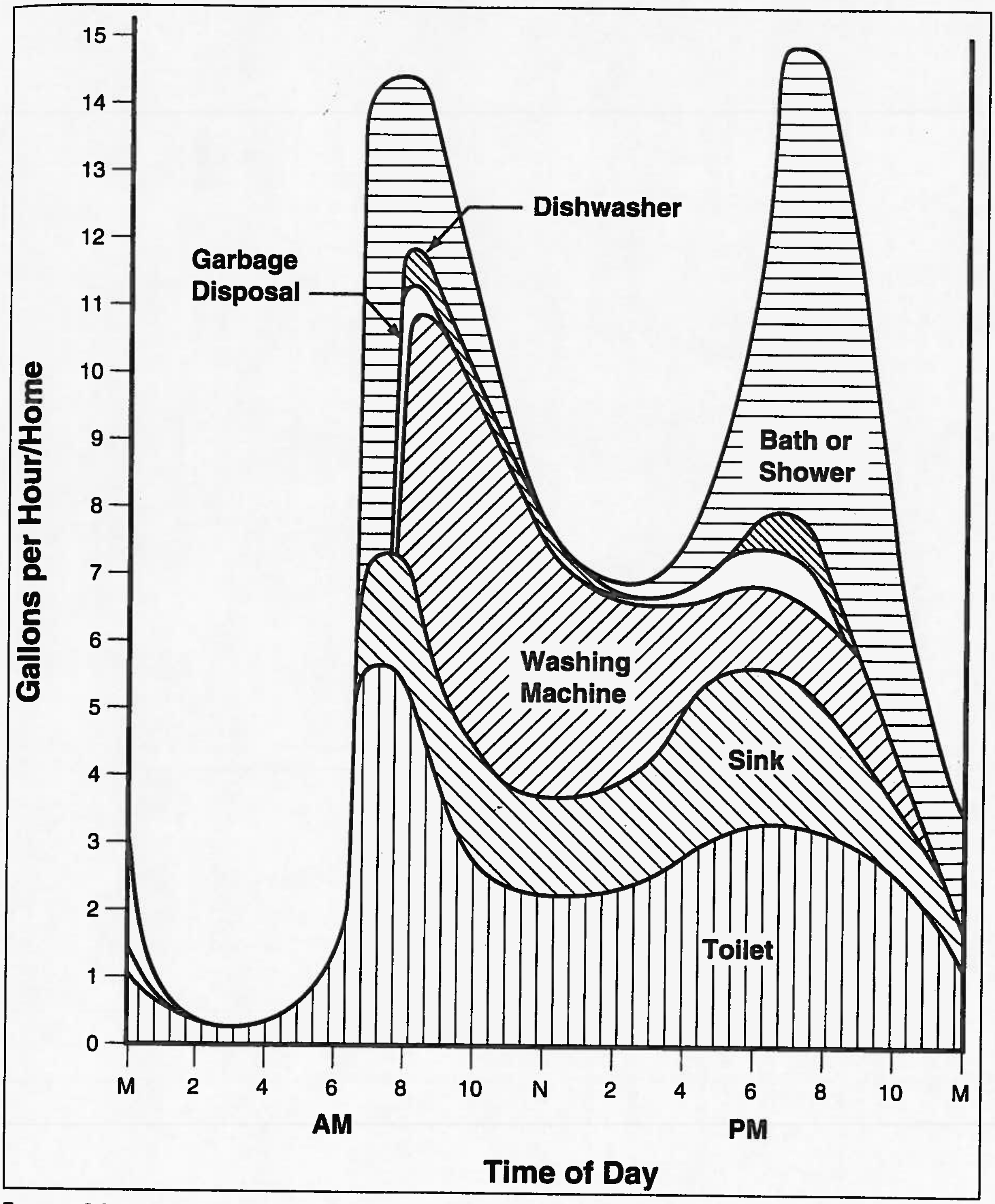


FIGURE 24.1 Hourly variation of sewage flow.

grade line (HGL). The line connecting the total energy values, which are the sum of the momentum (velocity head) plus position energy, is called the *energy grade line* (EGL). Under steady uniform flow conditions the two lines are parallel and separated by a distance equal to the velocity head of the flow. This is shown in Figure 24.4*a* and *b*. The slope of the two lines is the rate of energy loss or energy gradient and the drop in the lines over a length *L* is the energy loss, also known as head loss or friction loss. Under gravity flow conditions, the HGL and EGL are parallel to the sewer invert (lowest line of flow). Sewers are therefore designed and constructed on the basis of invert elevation.

A pump is a means of imparting energy to a flow. This added energy is generally expressed in feet and is the height that the energy grade line is raised by the pump. This is shown in Figure 24.5.

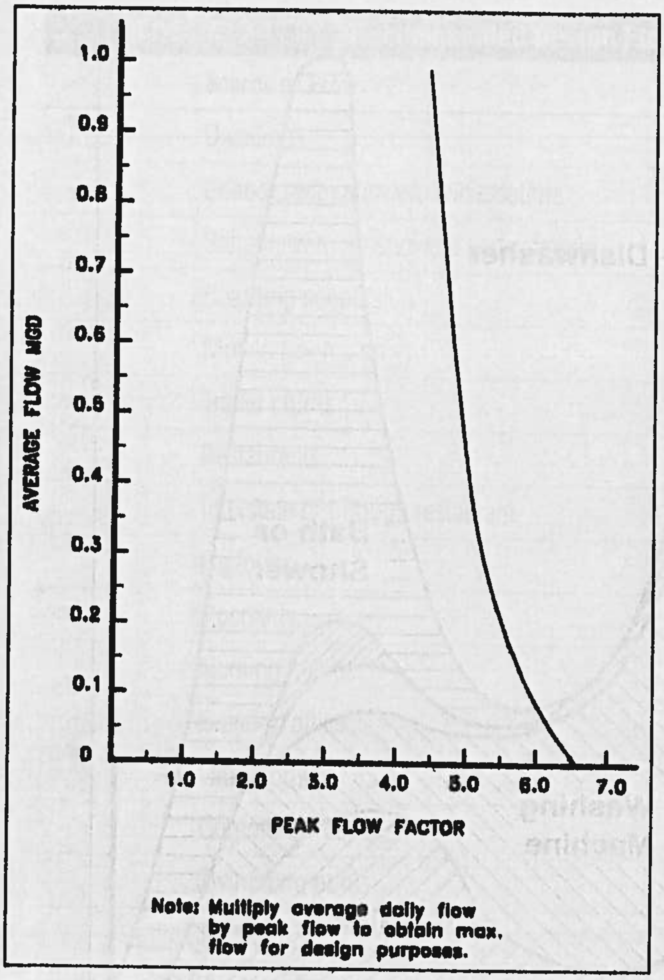


FIGURE 24.2 Peak flow curve 0–1.0 MGD.

Flow of fluid in a conduit may be defined as being laminar or turbulent. Laminar flow is flow of molecular layers of fluid where one layer is moving with respect to the adjacent layer. There is zero movement at the conduit wall under laminar flow conditions. The resistance to movement is the viscosity of the fluid. Viscosity is due to cohesion and interaction between adjacent fluid molecules. The resistance to movement is an irrecoverable energy loss that is dissipated as heat. For laminar flow, the rate of energy loss is a function of the fluid and the velocity of the fluid and not related to type of pipe material. This resistance to movement is represented by the hydraulic gradient.

The type of flow present is defined by the Reynolds number, N_R:

$$N_R = \frac{DV\rho}{\mu} \tag{24.6}$$

where D = diameter of conduit in ft, V = velocity of flow in ft/sec, ρ = fluid mass density in lb-sec^2/ft^4, and μ = shear between fluid layers in lb-sec/ft^2.

For laminar flow, the friction factor is related directly to the Reynolds number. The upper limit of the Reynolds number for laminar flow of water is 4000. As a frame of reference, water in an 8-inch pipe, which flows at a velocity of 2 fps and has a temperature of 68°F, has a Reynolds number of 50,000. This indicates that the flow is well within a turbulent flow regime, as is always the case for water and sewage flow in conduits. In sewers, the fluid particles move in a heterogeneous manner that causes a complete mixing of the fluid. This movement or turbulence is primarily created at the conduit wall by the roughness of the surface and by shear forces between the wall and the moving fluid. The conduit wall roughness and the shear forces generate turbulence vortices that move out from the wall, where they are subsequently dissipated by viscous attrition with other fluid particles. This generation and dissipation of turbulence consumes energy that is converted to heat. The energy dissipated within the flow is lost as heat. This loss is known as head loss or friction loss. As was noted for laminar flow, the slope of the HGL and EGL shows the rate of energy loss along a length of conduit. The drop in the HGL over a length of conduit is the amount of friction loss.

The roughness of the conduit interior wall is of significant importance in any hydraulic design. Pipe manufacturers

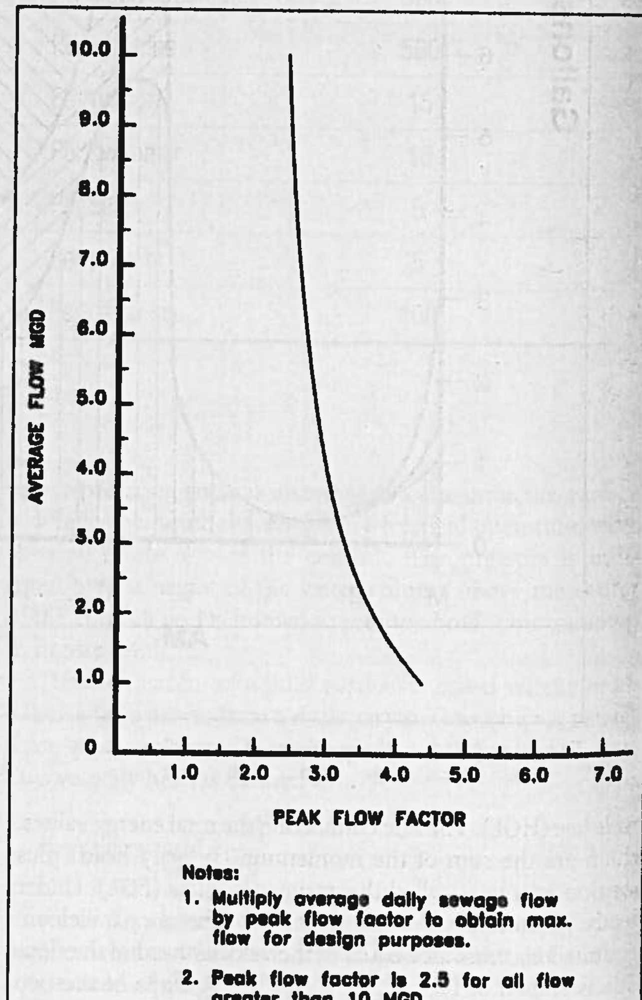


FIGURE 24.3 Peak flow curve 1.0 and greater MGD.

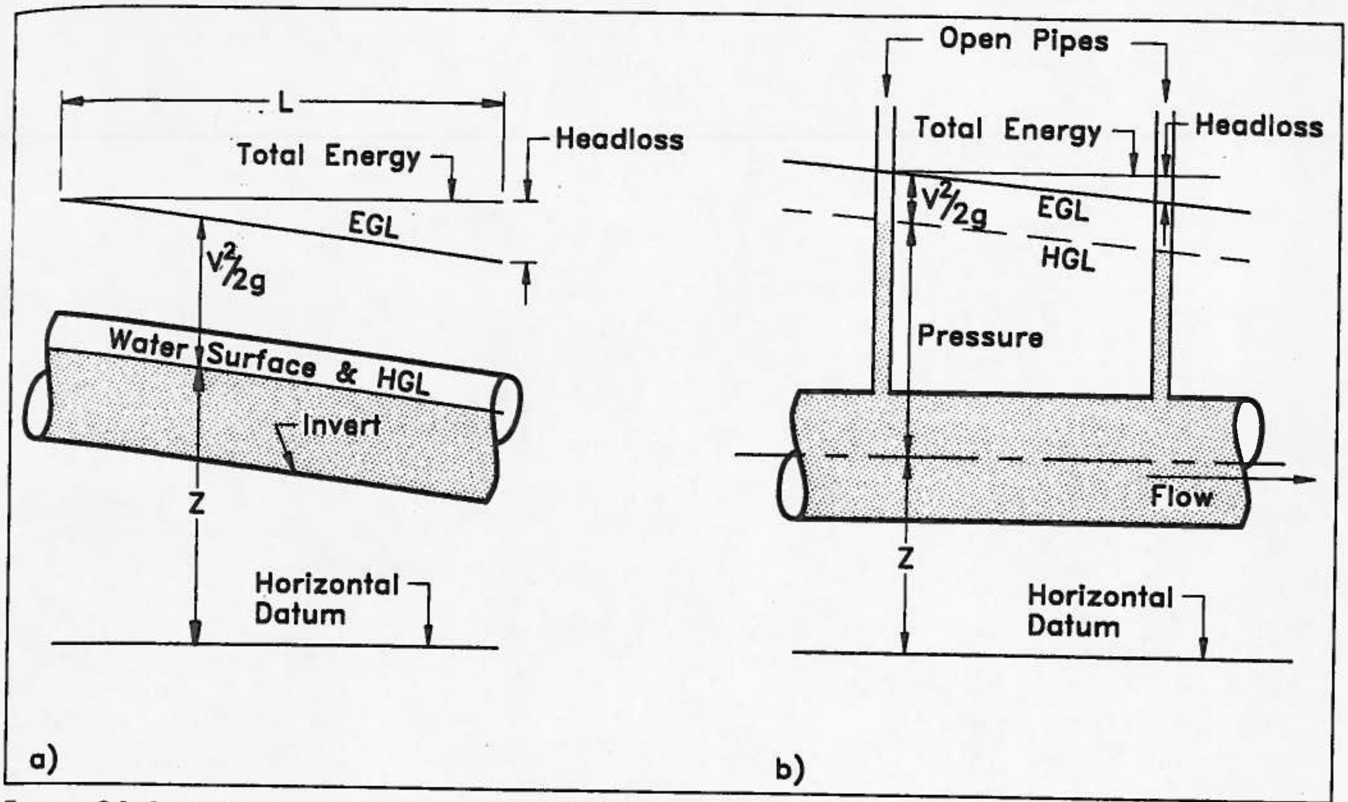


FIGURE 24.4 (*a*) HGL and EGL for gravity flow; (*b*) HGL and EGL for pressure flow.

strive to produce a pipe that has a very smooth wall surface, as this is an important item in marketing the product. A smoother pipe wall results in less turbulence generated and, hence, less energy loss in the flow. Most pipe marketed today is classified as smooth pipe.

The hydraulic design of a sewer system requires a procedure for determining the rate of energy loss. A number of equations have been developed for this purpose. Most of the equations were developed empirically from field or laboratory data, but some have been developed by dimensional analysis. Since the head loss depends on the level of turbulent vortices generated at the conduit wall, all calculations of head loss must relate to the type, height, and spacing of roughness protrusions on the pipe surface. These conduit characteristics depend on the type of pipe material selected. Pipe manufacturing is sufficiently developed to where the engineer can expect all pipe of a given material to be of uniform quality.

Furthermore, since most, if not all, pipe presently available can be considered as being smooth wall, the engineer needs to consider other characteristics when selecting the pipe material to be included in the construction. These include resistance to deterioration, ease of handling and laying, cost, and availability. The characteristics of new pipe should not be used in design because all pipe material deteriorates with age and use. There will be erosion of the surface, growth of slimes on the surface, and accumulation of solids with use and aging. Sewers should be designed to function properly at the end of the design life.

There are several equations available for determining friction loss in sewer conduits. The Manning equation is by far the most widely used. This empirical equation is accepted by all reviewing agencies. It is generally the only equation used by pipe suppliers in marketing material. Velocity is defined in open channel flow by the Manning equation as

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{24.7}$$

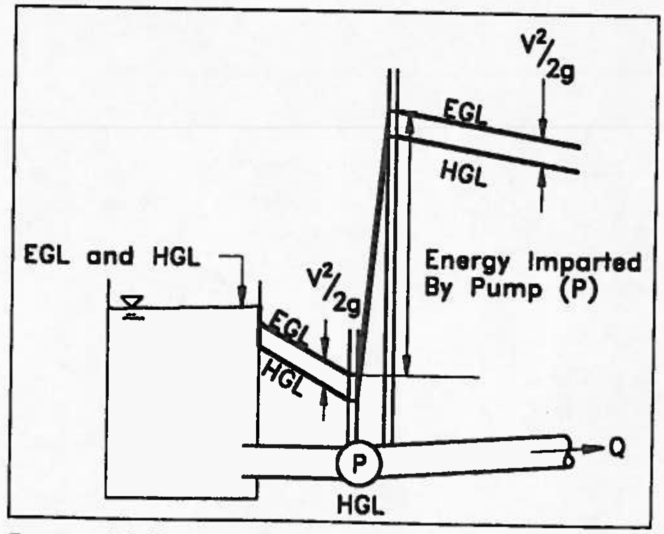


FIGURE 24.5 Change in HGL and EGL by a pump.

where V = the velocity of flow in ft/sec, n = the Manning coefficient, R = hydraulic radius = area/wetted perimeter; S = the slope of the HGL (hydraulic gradient) in ft/ft = HL/L, HL = head loss = the drop in the HGL in feet per length of sewer L.

The Manning coefficient n must be selected by the engineer. Selection is based on the type of pipe being employed in the design, giving consideration to condition of the pipe at the end of the design period.

Since $Q = AV$,

$$Q = A\left(\frac{1.486}{n} R^{2/3} S^{1/2}\right) \tag{24.8}$$

In turbulent flow, the conduit wall roughness is the principal condition that determines the intensity of turbulent vortices generated and, hence, the energy loss associated therewith. The Manning coefficient or *friction factor* is found in catalog material supplied by pipe manufacturers as well as in texts on hydraulics and related subjects. A value of 0.013 is generally used in sewer design. The Manning equation may be arranged to calculate the head loss for a length of sewer L as follows:

$$H_L = \left[\frac{VnL^{1/2}}{1.486R^{2/3}}\right] \tag{24.9}$$

Information available from pipe suppliers may indicate that a Manning n value as low as 0.009 is appropriate for their product. This would indicate that for a required velocity and capacity, the required slope of the HGL would be reduced 48 percent from that for an n value of 0.013:

$$\left[\frac{.009}{.013}\right]^2 \times 100 = 48\% \tag{24.10}$$

Or for a constant hydraulic gradient (HGL slope), the velocity and capacity would be increased 44.4 percent (0.013/0.009 = 1.444). A Manning coefficient of 0.013 is conservative for smooth wall pipe available for sewer construction. This value of n is recommended and generally used because its use allows for erosion of the pipe wall, slime buildup, and other minor obstructions to flow. These forms of pipe deterioration, resulting in reduction of capacity, occur over time. A sewer pipe material should give reliable service for at least 50 years. Other useful relationships for the Manning equation for a given diameter of conduit include:

$$Q_1/Q_2 = n_2/n_1 \tag{24.11}$$

where n is a constant.

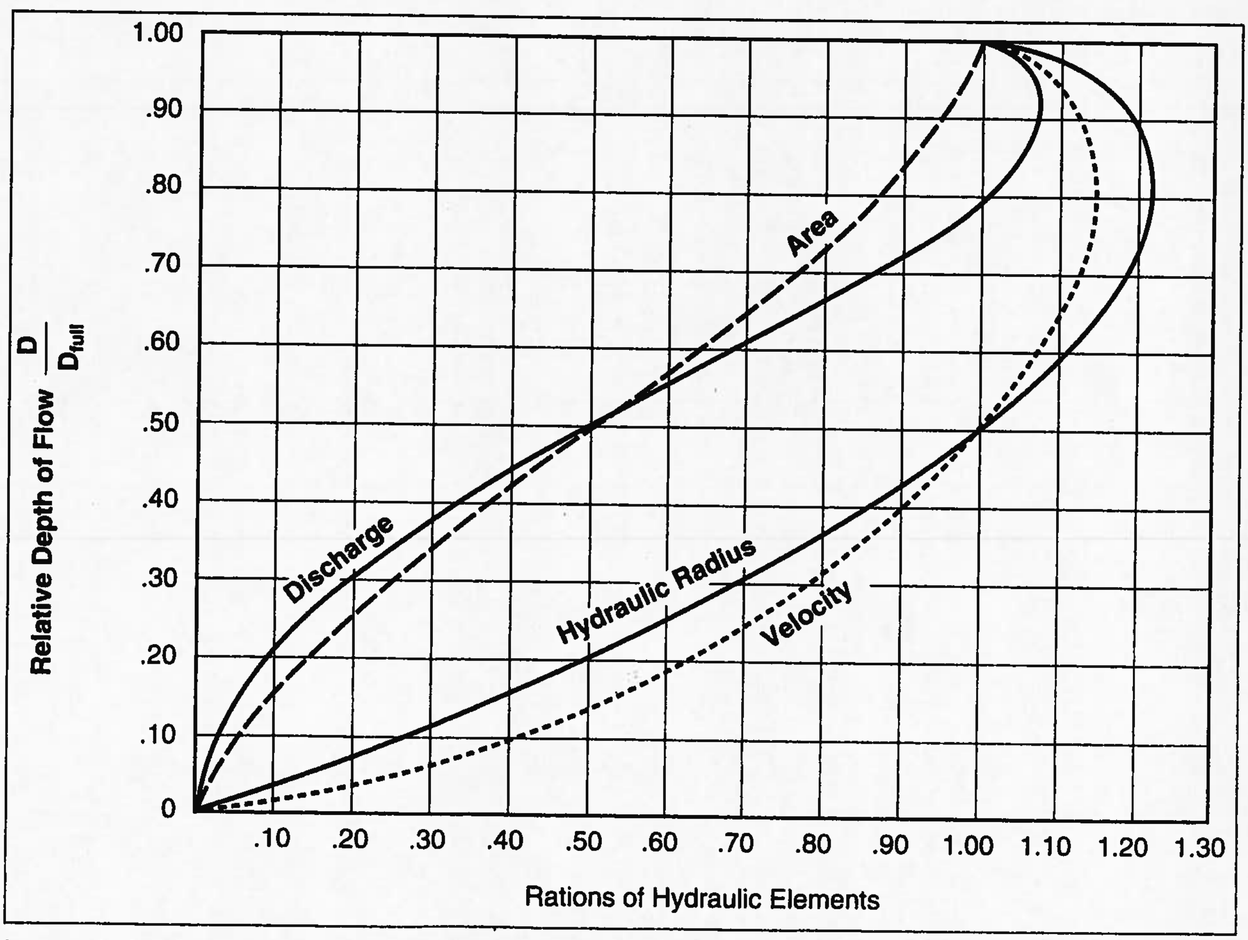


FIGURE 24.6 Nomograph of selected hydraulic elements for circular pipe flowing partially full.

$$\frac{V_1}{V_2} = \left[\frac{H_1}{H_2}\right]^2 \tag{24.12}$$

Since all pipe material used in sewer construction is essentially smooth wall, there will be little difference in the pipe roughness and, hence, in the value of n to be used in a design for different pipe material. The selection of the type of pipe to be used in a design and included in the contract specifications as being acceptable for use in construction should be based on the long-term serviceability of the pipe material. Serviceability factors should include expected useful life, resistance to problems with infiltration, ease of installation, resistance to corrosion and erosion, and maintenance requirements. Initial cost should not be the only determining factor.

Gravity sewers are designed to flow full at the design peak flow. Figure 24.6 presents the hydraulic properties of circular sewers for all depths of flow. As shown by the Manning equation, one of the factors influencing the velocity or capacity is the hydraulic radius R (area/wetted perimeter). The area of flow and the corresponding wetted perimeter do not vary uniformly with depth of flow. As shown by Fig-

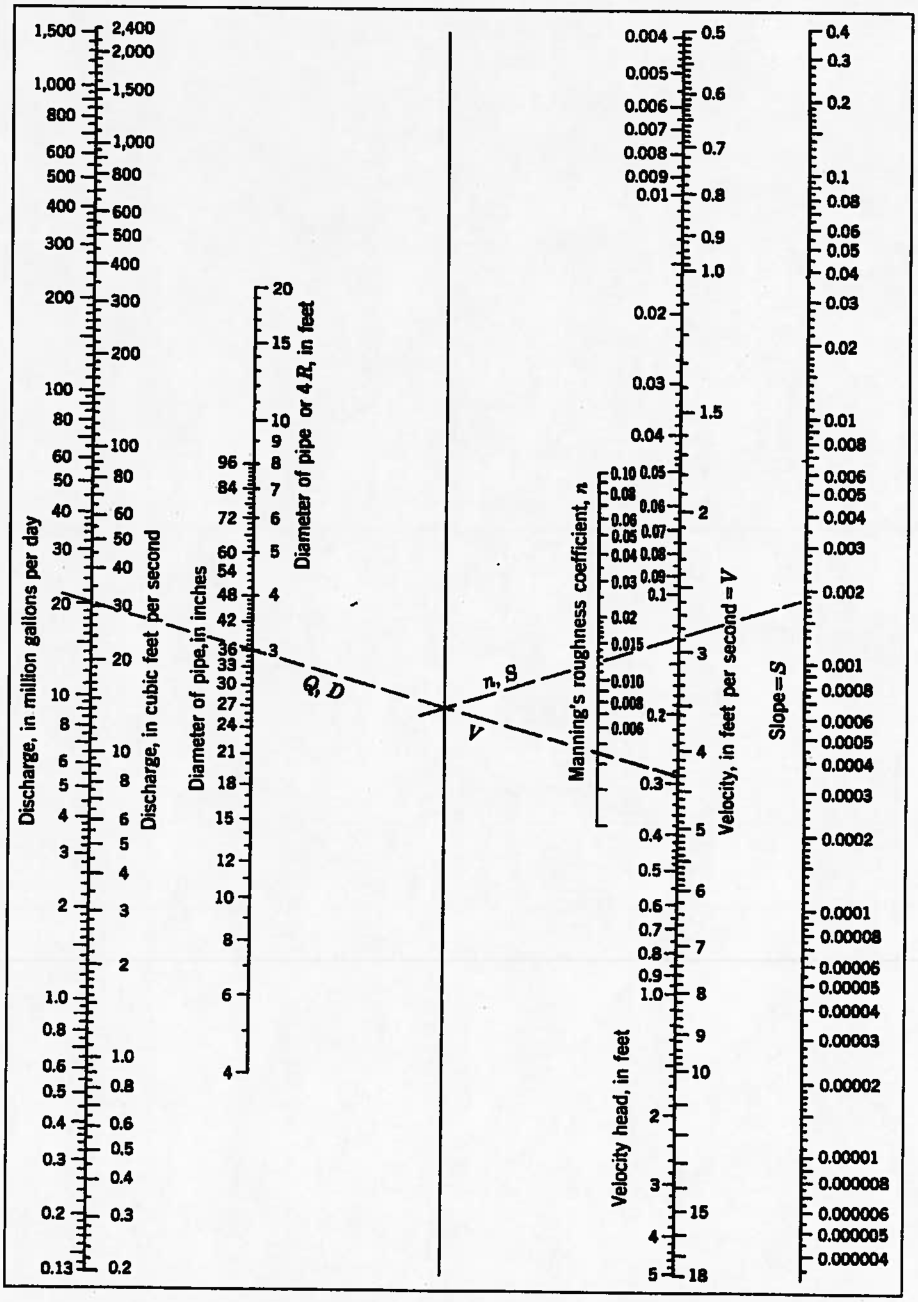


FIGURE 24.7 Nomograph for solving the Manning equation. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York)

TABLE 24.4 Common Conversion Factors

1 cfs	= 449 gpm
1 cu ft	= 7.48 gallons
1 MGD	= 1.55 cfs = 695 gpm
1 gal	= 231 cu in = 8.34 lbs
1 cu ft	= 62.4 lbs
Pressure = 62.4/144 = .433 psi per foot of water	

ure 24.6, the hydraulic radius is greatest at 0.8 of full flow depth. The velocity will also be the greatest at 0.8 of full depth. While the cross-section area of flow is maximum at full depth, flow $Q = AV$ and the maximum product of A and V is at 0.94 depth as shown by Figure 24.6. The capacity at the 0.8 depth is the same as for the full depth. Therefore, a sewer under gravity flow never flows full because the flow by natural law flows at the lower energy (energy of flow = surface elevation plus momentum). Any sewer flowing at greater than 0.8 depth is being surcharged by a downstream obstruction. Since the velocity of flow at the 0.8 depth is about 14 percent greater than at full flow, the hydraulic gradient required to achieve the equivalent full depth capacity is reduced. However, it is not common to design for the 0.8 depth conditions.

Sewers flow at less than 50 percent of full capacity most of the time. At 50 percent capacity, the depth of flow and velocity are also 50 percent of full flow conditions. Sewers should be designed to provide a minimum velocity of 2.0 to 2.25 fps at full flow. A full flow velocity of 2.25 fps ensures a scouring velocity of 1.0 fps at about 11 percent of full flow (see Figure 24.6). A scouring velocity of 1.0 fps is considered adequate to prevent deposition of solids.

Various forms of nomographs and slide rules are available for solving the Manning equation. These devices are normally used in sewer design to facilitate solving the equation. A typical nomograph is shown in Figure 24.7. The nomograph includes lines for five hydraulic elements and a pivot line. The continuity equation ($Q = AV$) may be solved by knowing two of three terms and finding the third from the nomograph without using the pivot line. The right side of the pivot line includes n, V, and S. Knowing two of the three establishes the pivot point from which knowing Q or D permits finding the other. Likewise, Q and D can be used to establish the pivot point, and knowing either n or S permits solving for the other.

EXAMPLE PROBLEM

a. What is the slope and velocity of an 8-inch pipe carrying 1 cfs?

Using Figure 24.7, knowing Q and D, find $V = 2.9$ fps.

Knowing D, Q gives pivot point, with $n = 0.013$, $S = 0.0019$.

b. What is the depth and velocity for the pipe in part (a) when the flow is 0.4 cfs?

Knowing Q/Q-full = 0.4/1.0 = 0.4 and using Figure 24.6

Depth of flow = 43% of 8 in = 3.44 in

Velocity = 95% of V-full = 2.75 fps.

There are several types of design aids in addition to the nomographs shown as Figures 24.7 and 24.29 for use in solving both the Manning and the Hazen-Williams equations. These include special nomographs and various forms of slide rules. The conventional type of nomograph shown in Figure 24.7 and the slide rule types are generally preferred by design engineers. Some pipe suppliers may provide these design aids. Some common conversion relationships for water are shown in Table 24.4. Some common design relationships from using the Manning equation are shown in Table 24.5.

TABLE 24.5 Hydraulic Elements Determined by Using Figure 24.7 and Equations 24.11 and 24.12

DIAMETER (IN.)	n	v (FPS)	s (FT/FT)	$r^{2/3}$	q (GPM)
8	0.013	2.0	0.0033	0.303	314
8	0.013	2.25	0.0042	0.303	354
8	0.010	2.0	0.0020	0.303	314
8	0.012	2.0	0.0028	0.303	314
10	0.013	2.0	0.0025	0.351	489
10	0.013	2.25	0.0031	0.351	551
12	0.013	2.0	0.0019	0.397	705
12	0.013	2.25	0.0025	0.397	793

TABLE 24.6 Class and Pressure Characteristics of Pressure PVC Pipe

Pressure	Class*	DR†	Bursting Pressure	Sustained
lb/in²	100	25	535 lb/in²	350
lb/in²	150	18	755 lb/in²	500
lb/in²	200	14	985 lb/in²	650

*The class of pipe is the limit of internal pressure to be used in design.
†DR is the ratio of the external diameter to the pipe wall thickness. For example, an 8-in DR 25 pipe has an external diameter of 9.050 in and a wall thickness of 0.362 in. Dimension ratios are controlled by manufacturing standards.

The data in Table 24.5 shows that an 8-inch sewer installed to provide a minimum velocity of 2.0 fps has a capacity of 314 gpm (0.45 mgd). This flow would be subject to a peak factor of 5 (see Figure 24.2), giving an average design flow of 63 gpm or 90,700 gpd. At a daily flow of 370 gallons per dwelling unit (see Table 24.2), this 8-inch sewer will serve 243 housing units. Likewise, a 10-inch sewer will serve 389 single-family units (note that the peak flow factor for the 10-inch line capacity of 489 gpm [0.7 mgd] is 4.8). A 12-inch sewer will serve 633 single-family units. Velocities greater than 2.0 fps and smaller n values will provide still greater capacity. Many developments are sewered by 8-inch sewers, and line sizes greater than 12 inches are seldom needed except for main or trunk sewers. An 8-inch-diameter sewer is considered the minimum size line in public sewer design.

Pipe Materials and Loads on Buried Conduits

The engineer now has several excellent types of pipe material available for use in sewer construction. It is important that the proper selection be made for the specific installation conditions. Pipe materials are classified as being either flexible or rigid. Pipe manufactured from materials such as concrete and vitrified clay are classified as rigid wall pipe. A deflection of the wall of a rigid wall pipe will result in pipe failure. The most common type of flexible pipe is polyvinyl chloride (PVC), which is widely used in both sewer and water line construction. A limited wall deflection of a flexible wall pipe does not reduce the serviceability of the conduit. Ductile iron and high-density polyethylene are also flexible pipe materials.

Rigid pipe is classified on the basis of bursting strength, and for pressure pipe the internal pressure rating is also included. The bursting strength is established by standard tests and is provided by the manufacturer in pounds of external vertical force per foot of pipe. It is listed as the three-edge bearing strength that reflects the type of standard test procedure. Pipe is designed to withstand both the internal pressure and the external loading simultaneously. The manufactured wall thickness is adjusted to provide the required strength. The engineer should have library catalog data available for the types of pipe material being considered for incorporation into a design. The strength of rigid pressure pipe will be shown by pipe class such as class 150, 200, and so on.

Gravity flow sewer pipe is designed to withstand external pressure loadings. Rigid wall pipe is manufactured in several wall strengths as may be required for design conditions. For example, pipe classes are designated as 1500, 2400, 3300,

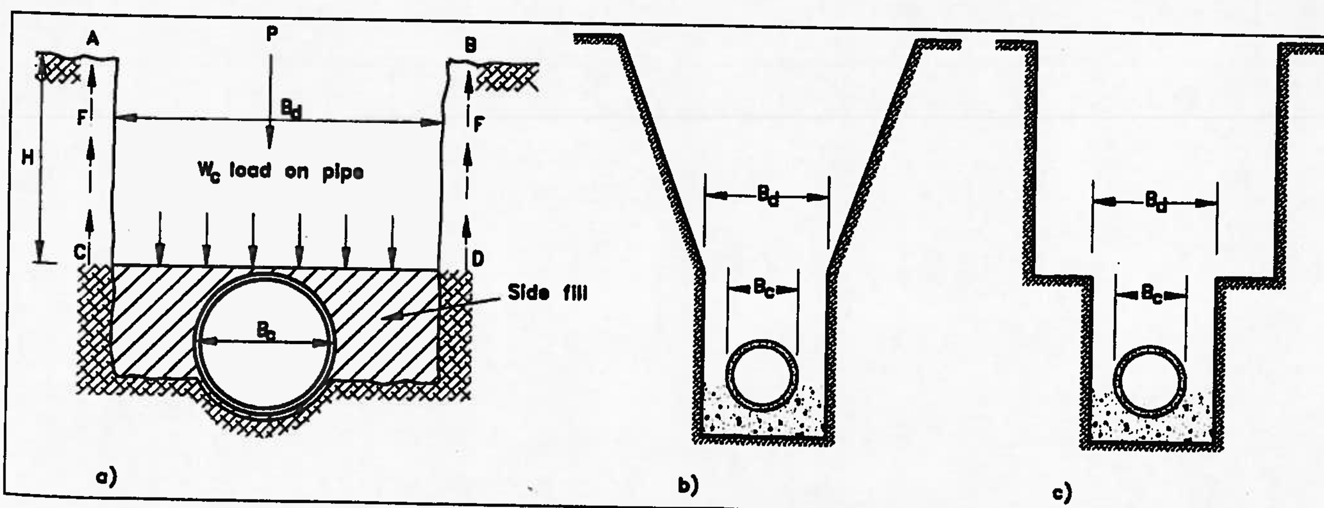
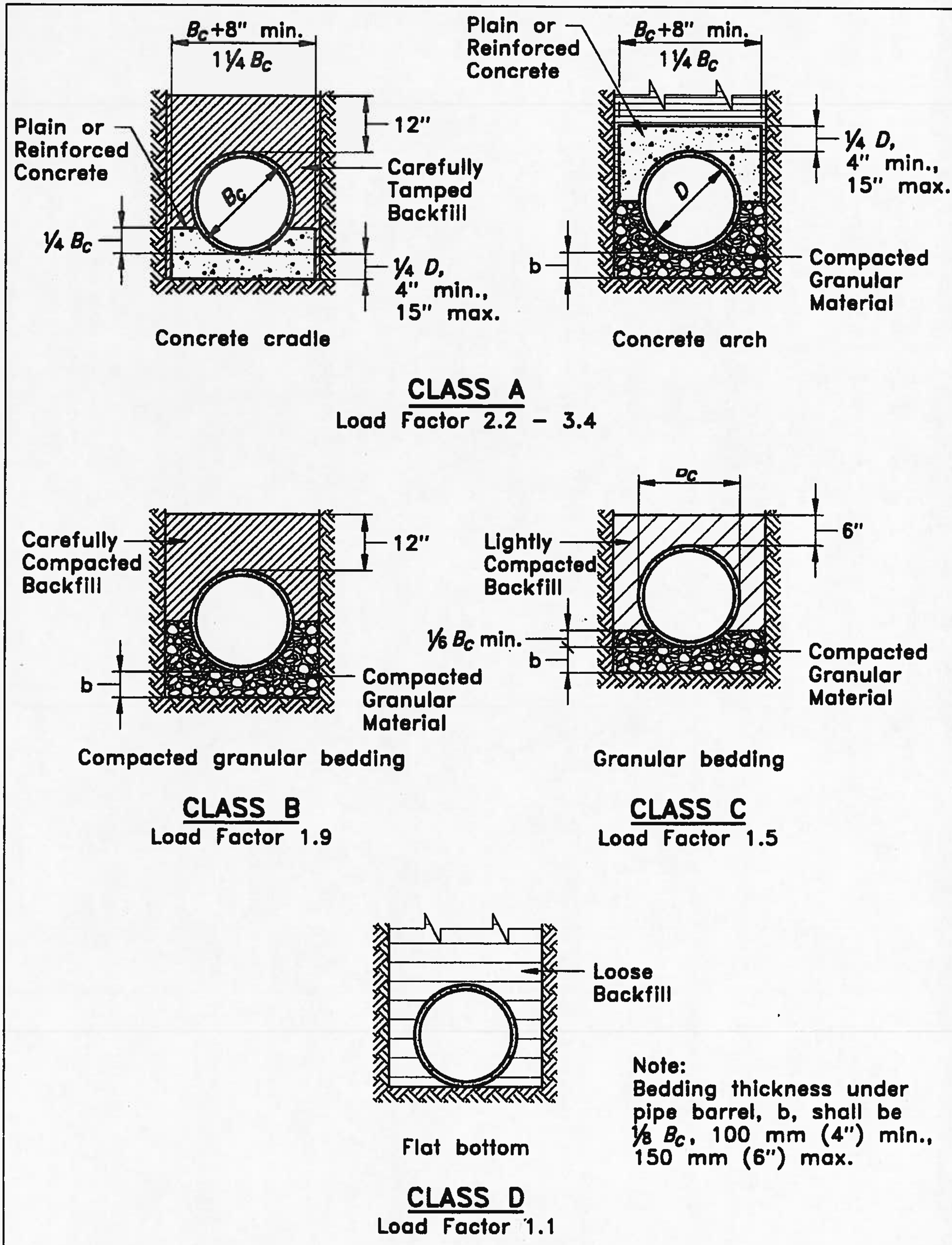


Figure 24.8 Typical trench conditions for a rigid conduit. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York)



Classes of bedding for rigid sewer pipes in trench. NOTE: In rock trench, excavate at least 15 cm (6 in.) below bell of pipe where concrete cradle is used (in. x 25.4 = mm).

FIGURE 24.9 Bedding methods for trench conduits. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York)

TABLE 24.7 Embedment Materials Classification System

Class	Type	Soil Group Symbol D 2487	Description	Percentage Passing Sieve Sizes: 1½ in. (40 mm)	No. 4 (4.75 mm)	No. 200 (0.075 mm)	Atterberg Limits: LL	PL	Coefficients: Uniformity C_u	Curvature C_c
IA	Manufactured aggregates: open-graded, clean	None	Angular, crushed stone or rock, crushed gravel, broken coral, crushed slag, cinders or shells; large void content, contain little or no fines	100%	≤10%	<5%	Nonplastic			
IB	Manufactured, processed aggregates; dense-graded,	None clean	Angular, crushed stone (or other Class 1A materials) and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; contain little or no fines	100%	≤50%	<5%	Nonplastic			
II	Coarse-grained soils, clean	GW	Well-graded gravels and gravel-sand mixtures; little or no fines	100%	<50% of "Coarse Fraction"	<5%	Nonplastic		>4	1–3
		GP	Poorly graded gravels and gravel-sand mixtures; little or no fines						<4	<1 or >3
		SW	Well-graded sands and gravelly sands; little or no fines		>50% of "Coarse Fraction"				>6	1–3
		SP	Poorly graded sands and gravelly sands; little or no fines						<6	<1 or >3
	Coarse-grained soils, borderline clean to w/fines	e.g., GW-GC, SP-SM	Sands and gravels which are borderline between clean and with fines	100%	Varies	5%–12%	Nonplastic		Same as for GW, GP, SW, and SP	

(Continued)

TABLE 24.7 (Continued)

Class	Type	Soil Group Symbol D 2487	Description	Percentage Passing Sieve Sizes: 1½ in. (40 mm)	No. 4 (4.75 mm)	No. 200 (0.075 mm)	Atterberg Limits: LL	PL	Coefficients: Uniformity C_u	Curvature C_c
III	Coarse-grained soils with fines	GM	Silty gravels, gravel-sand-silt mixtures	100%	<50% of "Coarse Fraction"	12% to 50%		<4 or <"A" Line		
		GC	Clayey gravels, gravel-sand-clay mixtures					<7 and >"A" Line		
		SM	Silty sands, sand-silt mixtures		>50% of "Coarse Fraction"			>4 or <"A" Line		
		SC	Clayey sands, sand-clay mixtures					>7 and >"A" Line		
IVA[A]	Fine-grained soils (inorganic)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity	100%	100%	>50%	<50%	<4 or <"A" Line		
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays					>7 and >"A" Line		
IVB	Fine-grained soils (inorganic)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	100%	100%	>50%	>50	<"A" Line		
		CH	Inorganic clays of high plasticity, fat clays					>"A" Line		
V	Organic soils	OL	Organic silts and organic silty clays of low plasticity	100%	100%	>50%	<50	<4 or <"A" Line		
		OH	Organic clays of medium to high plasticity, organic silts							
	Highly organic	PT	Peat and other high organic soils				>50	<"A" Line		

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TABLE 24.8 Characteristics of the Most Commonly Used Sizes of SDR-35 Pipe

Nominal Size Laying Length (in)	External Diameter (in)	Wall Thickness (in)	Weight (lb/ft)	Laying Length
8	8.4	0.24	4.42	20
10	10.5	0.300	6.93	20
12	12.5	0.36	9.91	20
15	15.3	0.43	14.90	12.5

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4000, and 5000. The class strengths represent the design crushing strength in pounds per linear foot of pipe.

Flexible wall pipe is classified by the ratio of the external diameter to the wall thickness. This is expressed as dimension ratio (DR) or standard dimension ratio (SDR). Since limited deflection of the pipe can be tolerated, the strength is expressed in terms of the force required to cause a defined deflection. Pressure flexible pipe is designated by pressure class. A PVC pressure pipe manufactured by Johns-Manville under the name Blue Brute is available in the three classes shown in Table 24.6.

In Table 24.6, a DR-25 pipe has a design pressure rating of 100 lb/in^2, while the sustained pressure strength is 350 lb/in^2. This difference provides for surges in pressure, known as water hammer, and a safety factor against failure. For example, a design pressure condition of 100 lb/in^2 plus a surge maximum of 100 lb/in^2 above the design pressure provide a safety factor against failure of 1.75 (350/200 = 1.75).

Buried Conduits

The theory for loads on buried conduits states that the load on a buried conduit is equal to the weight of the prism of earth directly over it, plus or minus the frictional shearing forces transferred to that prism by adjacent prisms of earth—the magnitude and direction of these frictional forces being a function of the relative settlement between the prisms. The equation for calculating the load on a rigid buried conduit as developed by Marston at the Iowa Engineering Experiment Station in 1913 states:

$$W_c = C_D w B_D^2 \qquad (24.13)$$

where W_c (lbs/ft) is the load acting on the conduit after settlement has taken place; C_D is a dimensionless coefficient related to the diameter of the conduit, the width of the trench, and the characteristics of the fill material and the pipe bedding conditions (C_D is obtained from tables or charts); w is the unit weight of the fill material in lb/ft^3 (normal earth material may be taken as 130 lbs/ft^3), and B_D is the width of the trench in feet (note that load increases as the square of B).

The engineering design can mitigate the load on the conduit by keeping the trench as narrow as practical (the load varies as the square of the width) and by requiring good bedding and backfill conditions. Figure 24.8 shows the type of forces affecting the load on the conduit.

In Figure 24.8*a* the load on the conduit is related to the weight of the soil in the area ABDC. The upward shearing forces F reduce the load on the conduit. If the fill below the top of the conduit is not well compacted, the load on the conduit is increased as the material settles. It is sometimes necessary to slope the ditch walls for construction safety and convenience. Figures 24.8*b* and *c* show two construction means of widening the trench without increasing the earth

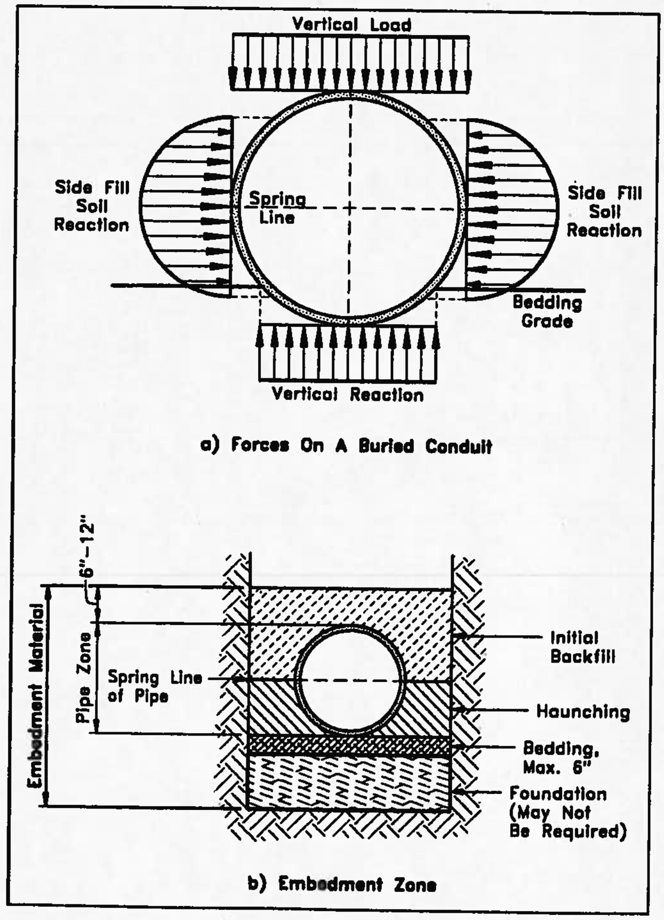


FIGURE 24.10 Forces on a buried conduit (a); Embedment zones (b). (Reprinted with permission from College of Engineering, Utah State University)

load on the conduit. It is important to keep the trench as narrow as practical from the top of the pipe to the bottom of the trench because the load on the conduit increases as the square of the width B. B is taken as the width of the trench at the top of the conduit.

There are four classes of bedding, as shown in Figure 24.9. The engineer is required to select a bedding type based on field conditions. Classes B or C are suitable except where unusual loading conditions exist. Class D bedding is used only for small-diameter pipe, not for sewer line construction.

Class A–Concrete Cradle: Bedding in which the lower part of the exterior of the conduit is set in a plain or reinforced concrete foundation of suitable thickness extending upward on each side for not less than 25 percent of the height of the conduit.

Class B–First Class: Bedding where the conduit is set on fine granular materials in an earth foundation shaped to conform to the lower part of the conduit exterior for a width of at least 60 percent of its exterior diameter. The trench is then backfilled with granular materials, hand placed, and tamped in 6-in layers to fill completely all spaces under and adjacent to the conduit for a distance of at least 1 ft above the top of the conduit.

Class C–Ordinary: Bedding in which the conduit is placed with ordinary care in an earth foundation shaped to fit the lower part of the conduit to the spring line. The ditch is then backfilled with granular materials, shovel placed, and tamped to fill all spaces to a height of at least ½ ft above the top of the conduit.

Class D–Impermissible: Bedding in which little or no care is exercised to shape the foundation to fit the pipe.

The engineer also should specify the embedment material to be used with the installation of the conduit. This information should be included in the specifications and also shown on the plans. Table 24.7 provides information on the standard material classification system. The soil types are standard under the United Soil Classification System (ASTM D2487). The materials are grouped into categories that are considered to be most suitable for embedment use.

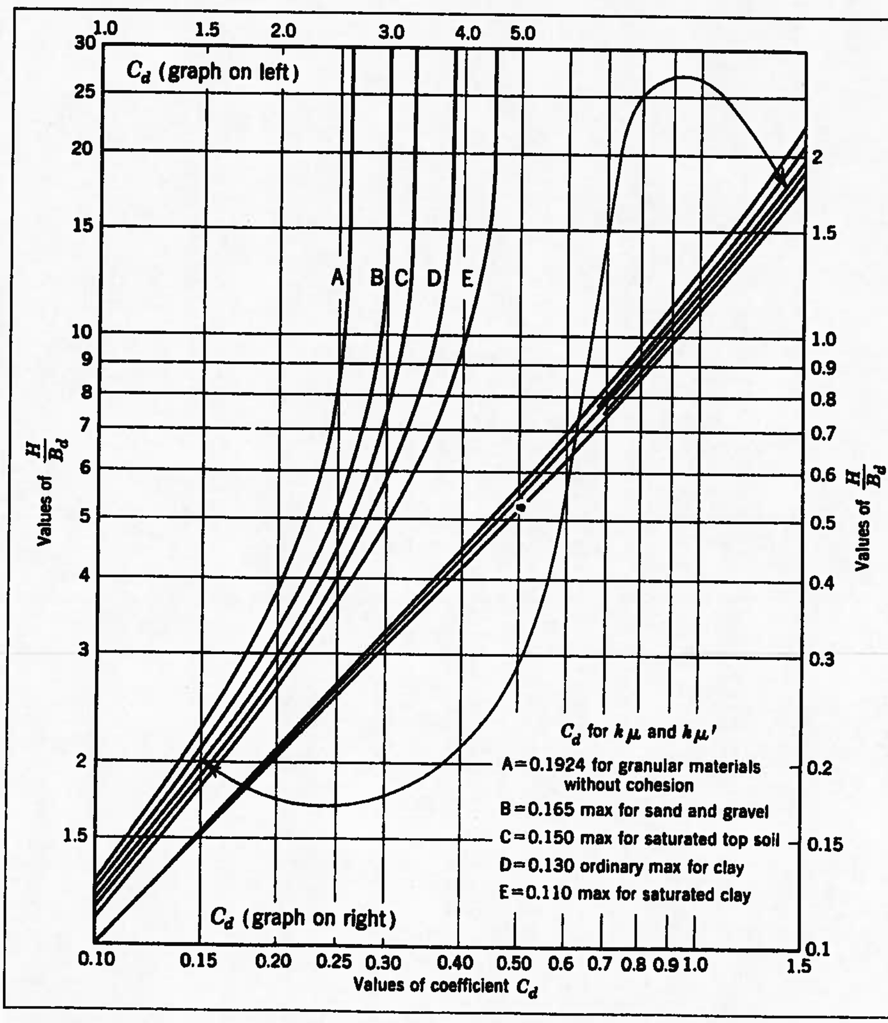


FIGURE 24.11 Values of C_d for trench conditions. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York.

Flexible Conduits

Flexible sewer pipe, such as PVC, is designated by the standard dimension ratio as noted for flexible pressure pipe. The most often used sewer flexible pipe is SDR-35. Heavier-wall-thickness pipe, such as DR-21, are available for use where required to meet laying conditions. Data for the more common sizes of SDR-35 pipe is given in Table 24.8.

A flexible pipe having a DR greater than 35 should not normally be considered for sewer construction. The smaller wall thickness associated with the greater DR values may not provide long-term reliability.

A flexible pipe yields under an external load by deflecting. This deflection results in a shortening of the vertical diameter because of the load on the pipe from the fill material on top of the pipe. Excessive deflection results in failure through openings at the joints and a reduction of internal area for flow even though the pipe has not collapsed. In conservative design the allowable deflection is limited to 5 percent, although deflections up to 7.5 percent are used in design.

Figure 24.10 gives an illustration of the embedment zones and the forces acting on a buried flexible pipe. As shown in Figure 24.10*a*, the vertical load on the pipe is distributed evenly over the width of the pipe. The vertical reaction acting on the bottom of the pipe is distributed evenly over the width of the pipe subtended by the bedding angle. The parabolically distributed side forces act through the middle 100 degrees of the pipe. The maximum pressure, found at the pipe's springline, is passive resistance of the backfill around the pipe. Using suitable backfill material and proper compaction is very important in establishing the field strength of a flexible conduit, as the vertical diameter cannot shorten, or deflect, without a corresponding lengthening on

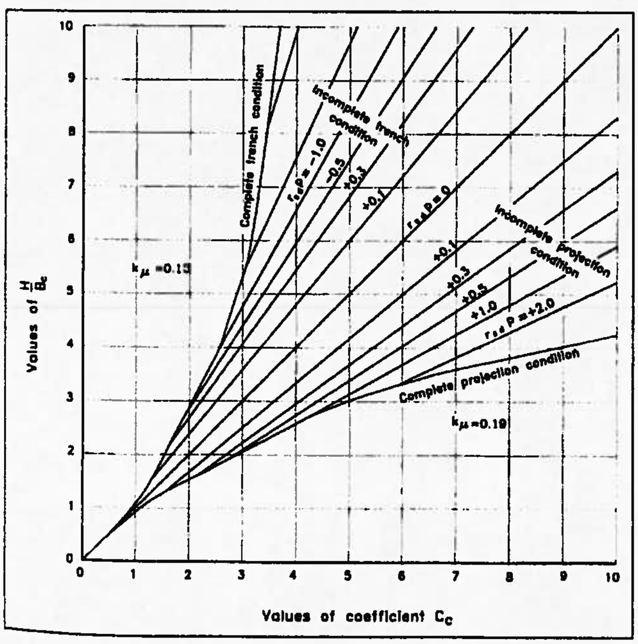


FIGURE 24.12 Values of C_c for positive projection conduits. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York)

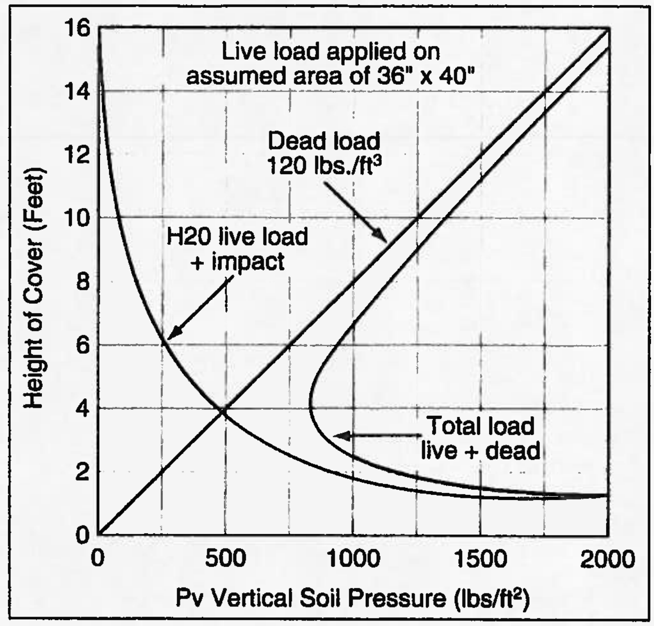


FIGURE 24.13 Total pipe load—H20 live load plus dead load. (Source: *Handbook of Steel Drainage and Highway Construction Products*, reprinted with permission from McGraw-Hill)

the horizontal diameter. The embedment zones, as shown in Figure 24.10*b*, illustrate the proper bedding conditions and the common nomenclature associated with pipe installation.

Zone Terminology

Foundation: A foundation is required when the trench bottom is unstable. Any foundation that will provide continuous support without causing loss of grade or flexural deformation is suitable.

Bedding: The bedding directly underneath the pipe is required to bring the trench to grade. The bedding should be shaped to fit the conduit with a depression formed to receive the bell of the pipe so the pipe rests on the pipe barrel for its entire length. The labor to manually shape the bottom of the trench is expensive, so it is common practice to overexcavate a couple of inches and bring the trench bedding to grade with granular material such as crushed stone. The bedding should be firm to support the pipe.

Haunching: The haunching area is most important in providing good pipe support. The contractor must insure that the haunch area under the pipe is completely filled and that the area is compacted to the specified density. Most of the pipe support comes from this area.

Initial Backfill: The initial backfill begins at the springline and continues to 6 in or 12 in above the top of the conduit. When the conduit is properly installed to this level, problems related to pipe bedding should not develop. If proper compaction is not provided above this point, there may be problems with surface settling for some time.

TABLE 24.9 Maximum Long-term Deflections* in % of SDR 35 Pipe

ASTM Embedment Material Classification		Density Proctor AASHTO T-99	Height of Cover (Feet) 3	5	8	10	12	14	16	18	20	22	24	26	28	30
Manufactured granular angular	CLASS I		0.2	0.3	0.4	0.5	0.6	0.7	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6
Clean sand and gravel	CLASS II	90%	0.2	0.3	0.5	0.7	0.8	0.9	1.1	1.2	1.3	1.4	1.6	1.7	1.8	2.0
		80%	0.9	1.4	2.3	3.2	3.6	4.1	5.0	5.5	6.0	6.4	7.3	7.7	8.2	9.1
Sand and gravel with fines	CLASS III	90%	0.2	0.4	0.6	0.8	0.9	1.1	1.2	1.4	1.6	1.7	1.9	2.1	2.2	2.3
		85%	0.7	0.9	1.7	2.2	2.6	3.0	3.5	3.9	4.3	4.8	5.2	5.6	6.0	6.5
		75%	1.1	1.8	2.9	3.8	4.5	5.5	6.8	8.5	9.9	11.3	12.7	14.1	15.5	16.8
		65%	1.3	2.4	3.6	4.7	5.5	6.8	8.5	9.6	11.4	13.0	14.5	16.0	17.3	18.0
Silt and clay	CLASS IV	85%	0.65	0.9	1.7	2.2	2.6	3.0	3.5	3.9	4.3	4.8	5.2	5.6	6.0	6.5
		75%	1.3	2.3	3.3	4.3	5.0	6.5	7.8	9.5	10.6	12.2	13.5	15.0	16.3	17.0
		65%	1.3	2.4	3.6	4.7	5.5	8.0	10.5	12.5	15.0	17.6	20.0	22.0	24.0	26.0
Organic soils and peat	CLASS V		THIS SOIL CLASS NOT RECOMMENDED													

*Deflection is the percent reduction of the vertical diameter.
(Reprinted with permission from College of Engineering, Utah State University.)

The equation for determining the load on flexible conduits installed under a ditch condition, as developed by Marston, is:

$$W_c = C_D w B_D B_C \tag{24.14}$$

where W_c = external earth load on the conduit in pounds per linear foot, C_D = load coefficient for conduits installed in a trench condition (suitable charts are available for determining C_D, as shown in Figure 24.11), w = unit weight of backfill material in pounds per ft^3, B_D = width of the trench at the top of the conduit in feet, and B_C = external horizontal diameter of the conduit in feet.

The ratio of the external load for rigid versus flexible conduits is given by the ratio of Equation 24.13 to Equation 24.14, and is equal to $W_D/W_C = B_D/B_C$. This ratio shows that the load on a flexible conduit is significantly less than that for a rigid conduit installed under the same conditions (C_D = constant). This is because the load on a buried conduit is modified by the response of the conduit. A minor deflection of the flexible conduit can materially reduce the load if the conduit has been properly installed, which means the conduit has been placed in a relatively narrow trench and properly bedded and the backfill has been properly compacted above the conduit. Where the trench is sufficiently narrow, a part of the backfill load is transferred to the sides of the trench by shear action. Where the backfill is not properly placed and compacted to the top of the conduit, the consolidation of the fill material alongside the conduit results in an additional load being placed on the conduit as the fill settles. The specifications and field inspection must ensure that the trench width is limited, the pipe is properly bedded, and the backfill is properly placed and compacted to 6 inches above the top of the conduit. This will mitigate problems with failing conduits and infiltration.

While it is desirable to keep the trench width as narrow as possible, it is necessary to provide space for the workers to install the pipe. Also, room must be allowed for a trench box. Normally a 42-inch trench is suitable for sewer diameters up to 16 inches. Above 16 inches, the trench width should be the pipe diameter plus 24 inches, allowing 12 inches on each side of the pipe for the trench box and the workers. Figure 24.11 provides a graphical means for determining C_d for use in Equations 24.13 and 24.14. H is the

TABLE 24.10 Air Test Table

	PIPE DIAMETER												
LENGTH OF LINE IN FEET	4"*	6"	8"	10"	12"	15"	18"	21"	24"	27"	30"	33"	36"
25	2.9	4.2	5.7	7.1	8.5	10.6	12.7	14.8	17.0	19.2	21.2	23.3	25.5
50												23.3	25.5
75										19.2	21.2	24.3	28.9
100								14.8	17.0	21.6	26.8	32.2	38.5
125							12.7	16.3	21.2	27.0	33.3	40.1	48.2
150						10.6	14.3	19.6	25.5	32.6	40.1	48.3	57.6
175						11.6	16.7	22.8	29.7	37.9	46.7	56.2	67.3
200					8.5	13.3	19.1	26.1	34.0	43.3	53.5	64.4	77.0
225				7.1	9.5	15.0	21.5	29.4	38.2	48.7	60.1	72.3	86.7
250				7.4	10.6	16.7	24.0	32.6	42.5	54.0	66.9	80.5	96.1
275				8.1	11.7	18.3	26.3	35.9	46.7	59.6	73.5	88.4	105.8
300			5.7	8.9	12.7	20.0	28.7	39.1	51.0	65.0	80.3	96.6	115.5
350			6.6	10.4	14.9	23.4	33.4	45.7	59.5	75.7	93.7	112.7	134.6
400		4.2	7.6	11.9	17.0	26.7	38.2	52.2	68.0	86.6	107.1	128.8	154.0
450		4.8	8.5	13.4	19.1	30.0	43.0	58.7	76.5	97.4	120.5	144.9	173.1
500	2.8	5.3	9.5	14.9	21.2	33.3	47.8	65.3	85.0	108.3	133.9	161.0	192.5

*When testing 4-in house laterals with sewer main, add 2.8 minutes to test time.

height of the fill from the top of the conduit to the surface. Other parameters are as defined previously.

An embankment condition, as opposed to a trench condition, exists when a conduit is installed in a trench that is so wide that the sides of the trench have no effect on the load to the conduit. A transition width is where there is no additional load imposed on the conduit, as the ditch becomes wider. This transition width depends on the depth and diameter of the conduit. Conduits installed in trenches wider than the transition widths are said to be installed in a *projection condition*. Conduits should not be installed in this manner. Even in fill, the fill should be placed and a trench excavated for the conduit. However, where it may be found necessary to install a conduit under a projection condition, Marston has provided the following equation for determining the resulting load to the conduit:

$$W_c = C_c w B_c^2 \qquad (24.15)$$

where C_c is the coefficient for a projection condition (replaces C_d as shown for trench condition) and w and B_c are defined above for Equations 24.13 and 24.14. A suitable chart has been developed for use in determining C_c as shown by Figure 24.12.

Superimposed Loads

Superimposed loads and live loads may come from vehicle traffic where the conduit is located in the street or from storing of material on the surface, such as in a lumber yard. Figure 24.13 shows the general effect of the combined trench earth load plus the superimposed load. It is important to note that for pipe cover greater than 8 ft, the superimposed load is generally of little significance. In general, sewers located in a roadway pavement area should have at least 6 ft of cover over the top of the pipe. Sewers not located in roadways should be installed below the freeze depth or a minimum of 3 ft. Figure 24.13 is for an H20 loading, which is what is generally used in practice (rear axle carries 16 tons). The impact of the superimposed load must also be considered, but impact is not considered significant where the conduit has at least 3 ft of cover. The superimposed load is added to the dead load (trench backfill load). Tables and equations for calculating superim-

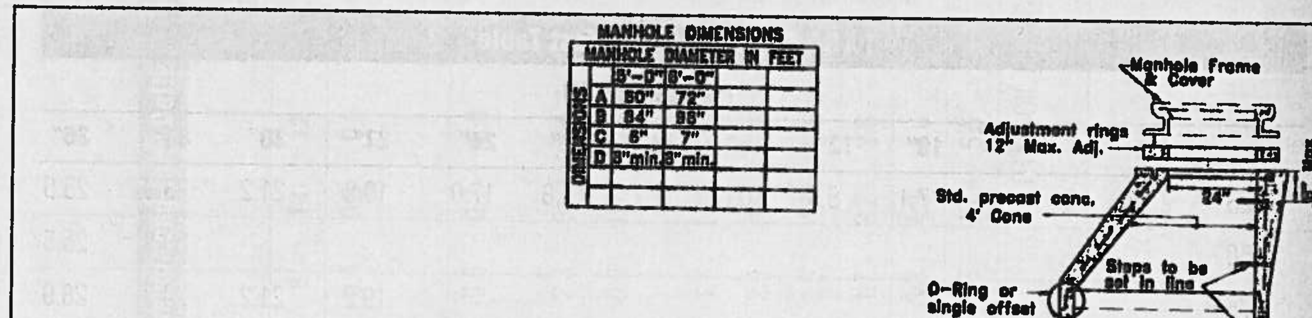


FIGURE 24.14 Typical precast manhole design. (*Handbook of Steel Drainage and Highway Construction Products*, reprinted with permission from McGraw-Hill)

posed loads are found in reference books and pipe supplier catalogs.

While it is important for the engineer to have an understanding of the principles related to loads on buried conduits, it is not always practical to attempt to calculate the load by Equation 24.13. The end result from calculating loads will be no more reliable than the assumptions about field conditions. Soil conditions vary from location to location, and even an experienced soil engineer would find it necessary to conduct extensive field studies to make the input parameters more reliable than engineering judgment and the data available from tables.

Pipe is manufactured to standards that include safety factors for the broad classes of embedment conditions presented before. Pipe suppliers provide tables or curves for use in design such as shown in Table 24.9. This table shows the deflection of SDR pipe under the indicated installed condition. As shown by the data, pipe deflections can be held to less than 5 percent by providing proper embedment.

Data similar to that provided in Table 24.9 for SDR 35 pipe is available from all suppliers of both sewer and pressure pipe. Standard design go/no go devices as well as electronic deflectometers are available for measuring the deflection of sewer pipe after installation. The engineer should include a requirement in the specifications for this measurement to be made after the backfill load has stabilized.

Air Testing

The contract specifications should require that a leakage test be performed to verify the watertightness of the sewer. An air test is generally required for this purpose, although an older water test may be used. The air test is easier to conduct and gives good results. The section of line to be tested is plugged and pumped up to a pressure of 4 lb/in^2 plus 0.433 lb/in^2 for each foot of ground water over the conduit (pressure should not exceed 9 lb/in^2). The pressure is allowed to stabilize to the sewer temperature. Pressure drop is then timed from 3.5 lb/in^2 to 3.0 lb/in^2 (0.433 lb/in^2 times the feet of ground water over the pipe must be added to these numbers). Acceptable time limits for the 0.5 lb/in^2 pressure drop are shown in Table 24.10.

Pipe Joints

Manufactured rubber compression joints should be used in sewer construction unless there are unusual conditions. The joint should be free of debris and the rubber gasket lubricated prior to installing the pipe.

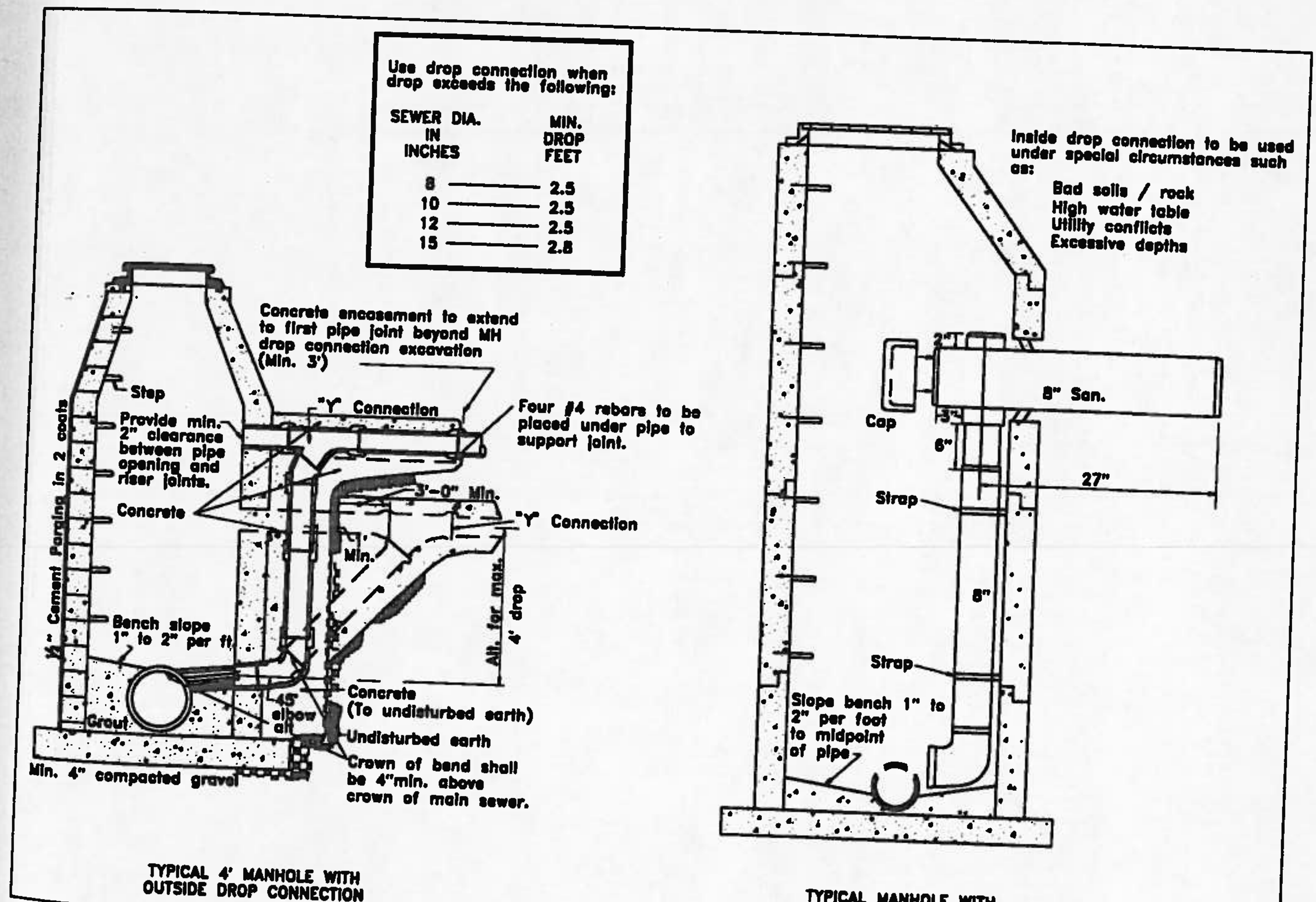


FIGURE 24.15 Typical drop manhole design.

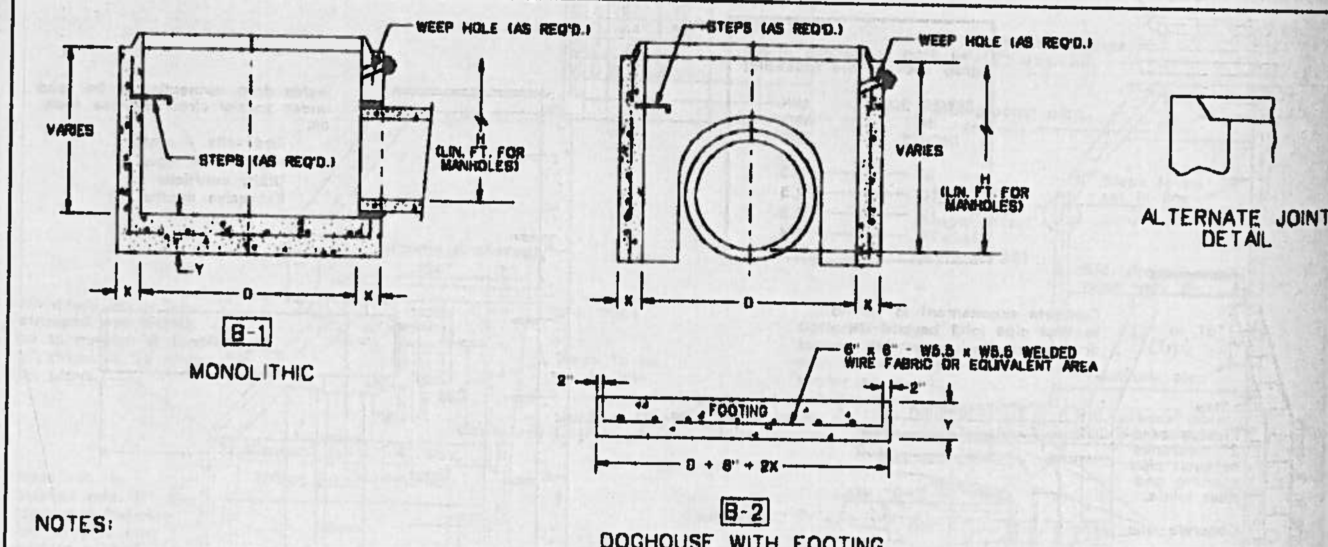
Manholes

Manholes are a required appurtenance to any sewer system. They provide access to the sewer for inspection and maintenance. Manholes should be placed at every change in grade or horizontal alignment, at every change in sewer size, at every sewer intersection, and at least every 400 ft. For larger sewers, which are readily accessible to workers, they may be spaced at greater intervals.

Only precast reinforced concrete manholes should be used except where special field conditions make it necessary to field construct the manhole with brick or cast-in-place concrete. Figure 24.14 shows two typical types of manholes. The standard manhole includes a base that is generally cast as a part of the first barrel section. The base shown is an extended base, which spreads the manhole weight over a larger area. Additional barrel sections are used as required to bring the manhole to where the eccentric cone is added. A manhole frame and cover are placed at the surface. The manhole supplier should adjust the height of the sections to comply with installation requirements. The precast supplier tailors each manhole for the specific site. Rubber boots are available that can be cast into the manhole at the locations where the sewers will pass through the manhole wall. A stainless steel clamp is used to secure the sewer to the boot making a watertight connection. The inside of the connection is grouted along with the placement of the bench (only shrink-proof grout is used). Corrosion-resistant steps are placed inside the manhole on a 12-in or 16-in vertical spacing. The exterior of the manhole is waterproofed using bituminous mastic on the surface and in the joints.

Figure 24.15 shows two types of drop manholes. A drop manhole is used to reduce slopes of incoming sewers or to permit connecting a sewer entering the manhole at a higher elevation than the main sewer. Generally, if the difference in elevation is less than 5 ft and ground water or obstacles are not a problem, the slope of the higher sewer is increased to lower the sewer to where the drop manhole will not be needed.

Doghouse manholes are typically used when a new gravity sewer will intersect an existing gravity sewer, and an existing manhole is not present. The doghouse manhole is located at the intersection of the two lines and includes the construction of the manhole base in the field through forms and cast-in-place concrete around the existing gravity sewer line (see Figure 24.16). Additional barrel sections and a cone section are added to the constructed base section until



NOTES:

1. SEE GENERAL NOTES FOR ADDITIONAL INFORMATION ON WEEP HOLES, STEP REQUIREMENTS, "H" (LIN. FT. FOR MANHOLES) DIMENSIONS, ETC.
2. ALL BASE UNITS ARE TO BE IN ACCORDANCE WITH THE REQUIREMENTS OF AASHTO M199.
3. CONCRETE SHALL BE 4000 PSI.
4. WHERE OPENINGS ARE REQUIRED FOR PIPE, THEY SHALL BE FORMED, DRILLED, OR NEATLY CUT AS APPROVED BY THE ENGINEER. THE CONTRACTOR SHALL FURNISH THE FABRICATOR WITH THE ANGLES BETWEEN CENTER LINES, THE INVERT ELEVATIONS, AND THE SIZE OF ALL PIPES TO ENTER THE MANHOLE. HOLES ARE TO BE A MINIMUM OF 4" TO A MAXIMUM OF 8" LARGER THAN THE OUTSIDE DIAMETER OF THE PROPOSED PIPE.
5. DIMENSIONS SHOWN ARE MINIMUM. ACTUAL DIMENSIONS MAY VARY WITH MANUFACTURER.
6. "D" IS NOMINAL DIAMETER.
7. IN THE EVENT THE INVERT OF THE OUTFALL PIPE IS HIGHER THAN THE BOTTOM OF THE STRUCTURE, THE INVERT OF THE STRUCTURE SHALL BE SHAPED WITH CEMENT MORTAR TO PREVENT STANDING OR PONDING OF WATER IN THE STRUCTURE.
8. TONGUE AND GROOVE JOINT ARE TO BE OF FABRICATOR'S DESIGN MEETING VDOT APPROVAL. JOINTS ARE TO BE SEALED WITH MORTAR, O-RING GASKETS, OR BUTYL RUBBER.

DIMENSIONS

D	X MINIMUM	Y MINIMUM	SUGGESTED MAX. PIPE SIZE	ABSOLUTE MAXIMUM ☆
* 36"	4"	8"	18"	21"
48"	5"	8"	24"	27"
60"	5"	8"	36"	42"
72"	6"	8"	48"	54"
84"	7"	8"	60"	66"
96"	8"	8"	66"	72"

* DEPTH "H" OF 36" DIAMETER BASE UNIT RESTRICTED TO 4'-0" MAXIMUM.

☆ ONE THROUGH PIPE ONLY. (ONE PIPE ENTERING AND ONE PIPE EXITING STRUCTURE)

FIGURE 24.16 Typical doghouse manhole base.

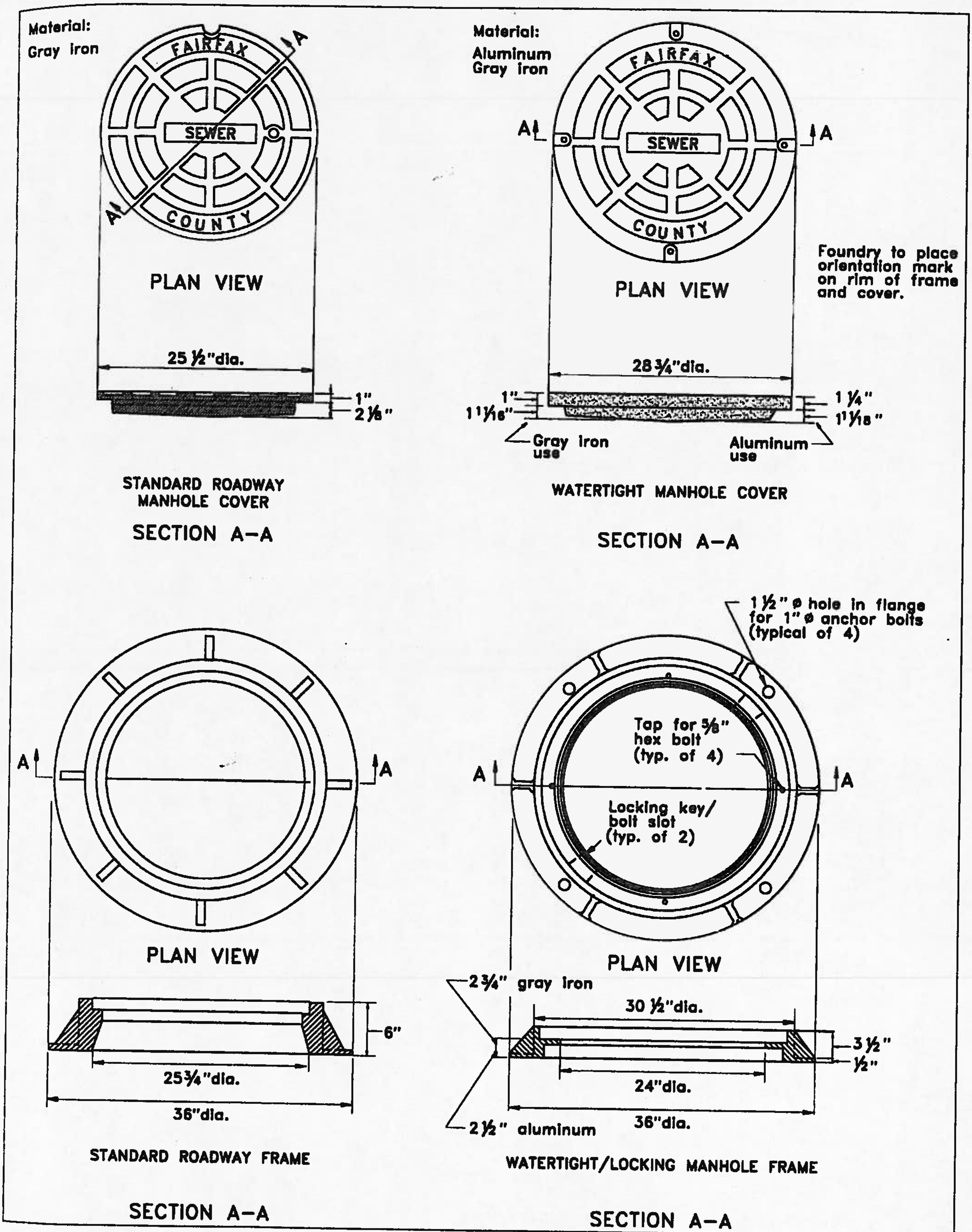


FIGURE 24.17 Typical manhole frames and covers. (*Fairfax County Public Facilities Manual*, 1993)

ground elevation is reached. Once the manhole is constructed, the top is cut out of the existing gravity sewer pipe to allow the sewage from the new line to enter the pipe, joining the collection and conveyance system.

An epoxy coating or interior liner may also be used with manhole construction if, in the opinion of the engineer, highly corrosive situations may be present that would lead to the early deterioration of the concrete manhole. Typically, these conditions could exist at manholes that receive sewage discharge from a sanitary forcemain. There is a wide variety of coatings and linings, which include epoxies applied as paint and precast high-density polyethylene linings that are formed with the original construction of the concrete barrel sections. These products can also be used in the rehabilitation of existing manholes that have not reached their useful life expectancy or are cost prohibitive to replace.

The bench is constructed to provide a smooth section through the manhole so as to reduce energy losses, to prevent the accumulation of solids in the manhole, and to provide a place for the maintenance person to stand when working in the sewer. The bench should extend to at least the springline (see Figure 24.10) of the sewer. For sewers larger than 10 inches in diameter, the bench should extend to two-thirds of the sewer diameter. The specifications should require that the base section be supplied with an extended base, that a boot be cast in the section for all sewer connections, and that the channels and bench be cast as a part of the section. All sewers entering a manhole should be provided a smooth channel into the main channel. When there is a change in sewer diameter at a manhole, the 0.8-ft depths of the sewers should be placed at the same elevation. This prevents any upstream surcharging at full flow. The change in flow direction at a manhole should not exceed 90 degrees. Where a greater change in direction is required, use two manholes with a segment of sewer between them.

In the past it has been common practice to allow for a 0.1-ft drop in sewer invert through a manhole where there is no change in pipe diameter. This is no longer necessary; in fact, the change in slope and, hence, velocity is more likely to cause problems. A well-constructed channel will not require this drop. Many manhole suppliers now provide the base section with the channel and bench cast in place when the manhole is formed. The channel invert must be smooth and have the same shape as the sewer. In all instances, the

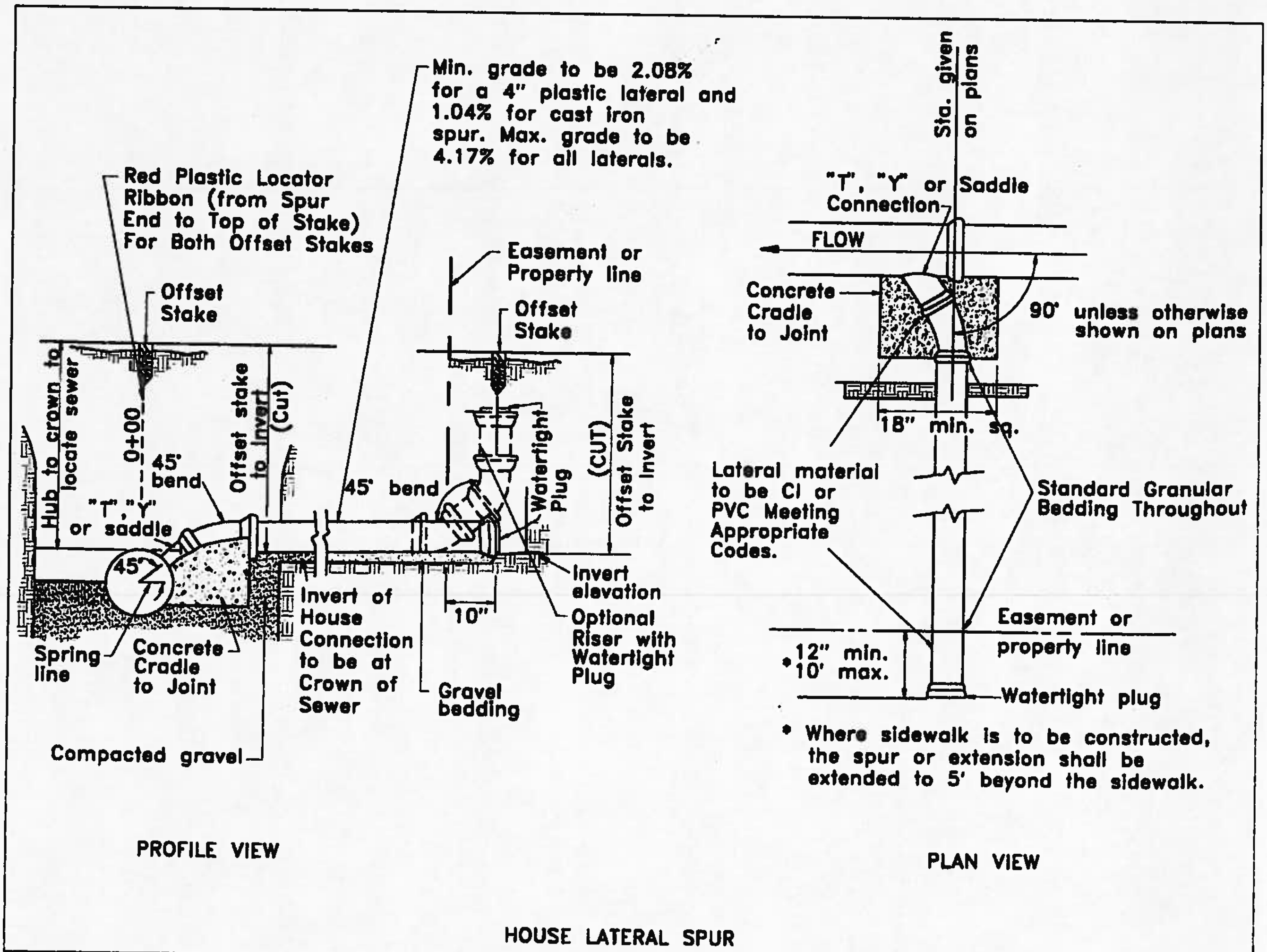


FIGURE 24.18 Service connections showing building spurs or laterals.

penetrations (openings in manhole wall for sewers) should be cast in the manhole or core drilled where an additional penetration is needed. A penetration made with a jackhammer should never be permitted. A short section of sewer is used at the manhole to provide a joint in the sewer not more than 3 ft from the manhole. This allows some flexibility for any difference in settlement between the manhole and the sewer. An air or water test is used to check the manhole for watertightness, and this requirement should also be included in the specifications.

The manhole frame and cover should be of gray cast iron. A typical frame and cover are shown in Figure 24.17. A standard frame and cover as well as a watertight/locking frame and cover are also shown in the figure. The engineer's library should include a catalog on standard manhole castings. The weight of the frame and cover must be selected to carry the expected loading. For example, a traffic-bearing frame and cover are required when the manhole is located in a street, whereas a lighter one is suitable for off-street locations. Locking frame and covers are available for use where required. Waterproof frames and covers should be used at locations where the area is subject to flooding. Ventilation to the sewer is provided through the manhole cover. When waterproof frames and covers are used, alternate ventilation should be provided for at least every 1000 feet of sewer.

Many utilities have a specific design on the manhole covers such as the name of the utility. These utilities stock the covers and sell them to the contractor.

Building Spurs

When manholes are placed in the street, the building spur (often referred to as building lateral) should be installed from the sanitary sewer to a minimum of 1 ft inside the property line. A separate spur should be connected for each lot or building site. Where sidewalks are to be constructed, the building spur should be constructed to 5 ft beyond the back of the sidewalk. It is important that each building spur be shown on the sewer plan with the station of the connection being confirmed as a part of the as-built drawings. SDR-35 or heavier pipe should be used for building spurs. The spur should enter the main sewer through a manufactured wye or tee. An approved saddle may be used when connecting to existing sewers. Some localities require that the connecting wye or ell be of ductile iron because mechanical rodding equipment will bore through a PVC connection if care is not used. Building spurs should be laid to a grade of at least 0.5 percent slope, with a minimum slope of 1 percent being provided where possible.

A wye may be installed and the extended line capped at the surface at the lot line for access in the future as needed. Where the main sewer is excessively deep, the spur should be brought to a reasonable depth prior to reaching the lot line, but the spur must be kept deep enough to serve the building. Figure 24.18 shows typical connections for building spurs (Service Connections).

DESIGN EXAMPLE

Preliminary Investigation

This design example has been structured to include principal considerations the engineer may encounter with a sewer design project. A hypothetical example, such as presented here, is considered more appropriate for this purpose than an actual design because a hypothetical case can be structured to incorporate the circumstances needed to cover many aspects of sewer design. The example assumes that a Mr. John Jenn, the owner of 40 acres of land fronting on West End Road (see Figure 24.19) has commissioned an engineering firm to design the infrastructure and obtain the required approvals so that construction permits for a single-family development on the entire tract can be obtained. This example covers the course of action the engineer may take to complete the sewer design and obtain the required approvals. Note that the engineering firm is responsible for providing the owner with complete plans and specifications that have been approved by all reviewing agencies, both local and state. When this has been properly accomplished, the contractor should not have any problems in obtaining a permit for the construction.

The first stage of the project is to conduct a preliminary investigation and prepare a report on the findings. The preliminary investigation should include:

1. Securing topographic mapping of the drainage shed that includes the 40-acre tract
2. Preparing a drawing showing the location of the 40-acre tract, the drainage shed, and environs
3. Meeting with the staff of the sewer utility (Department of Public Works or a separate authority) to advise them of the project and to obtain information on the availability of public sewer to serve the area

This early meeting with the staff of the sewer utility is also necessary to obtain information on how the utility plans to sewer the area, as shown by the Master Utility Plan for the area where the 40-acre tract is located. Several additional pieces of information are obtained from this meeting:

- The parcel of land is in an area approved to receive public sewer and that the nearest public sewer is approximately 16,000 feet to the west.
- Capacity is available in a trunk sewer located to the west, but it may be 5 to 10 years before service will be available to the property under the utility's sewer extension program.
- The utility does permit property owners to extend sewer service into areas approved for service at the property owner's expense.
- Any design for sewering the parcel will have to consider the needs of the entire sewer shed, and the comprehensive sewer plan provides for the construc-

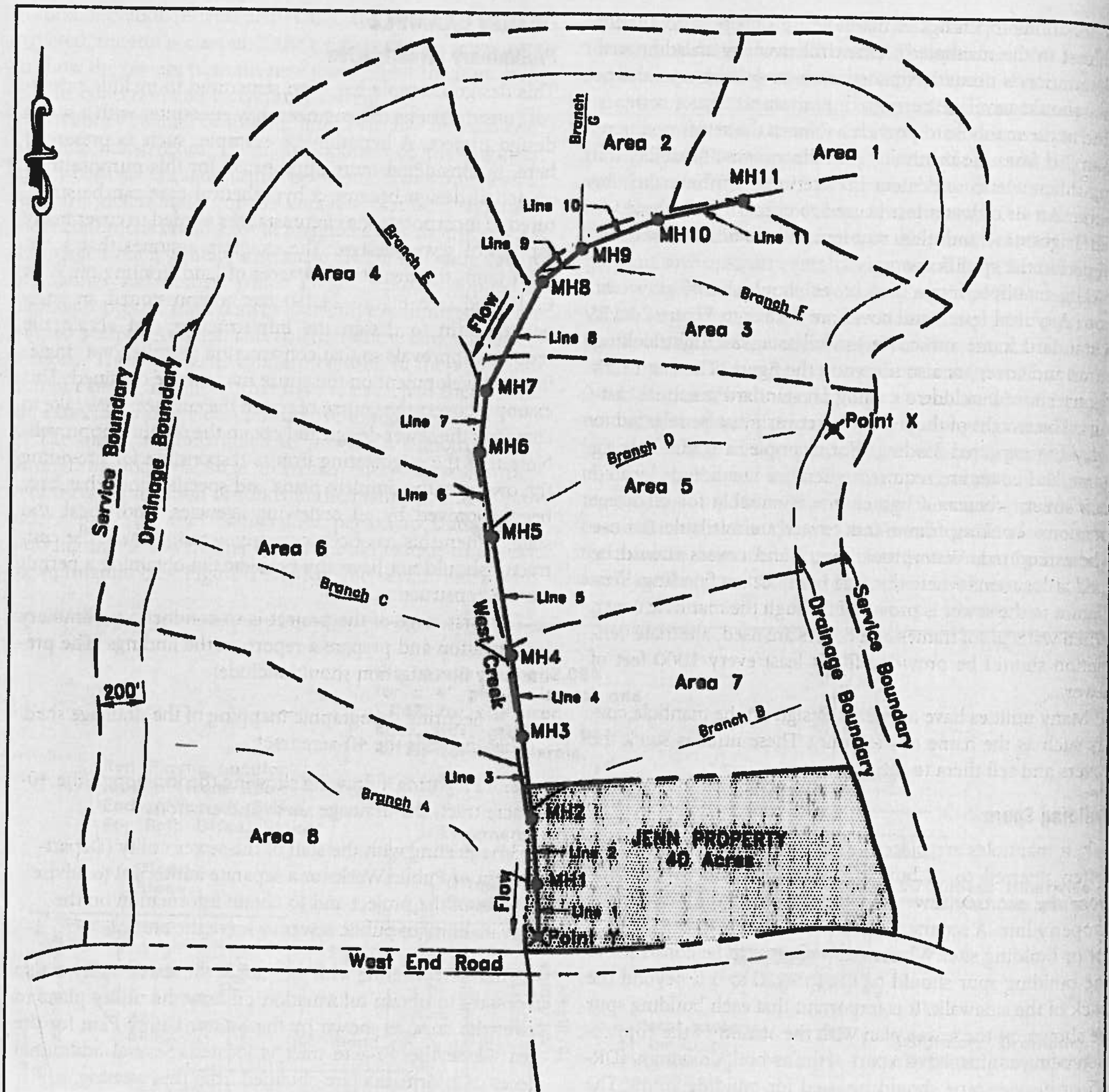


FIGURE 24.19 Sewer Shed West End Road service area.

tion of a pumping station on the adjacent sewer shed to the east that will pump the flow across the drainage area containing the 40-acre tract.

- This additional 28-acre parcel is zoned single family at four units per acre.
- The engineer was advised that the property south of West End Road is not planned for public sewer.
- A pumping station is to be located at Point Y (Figure 24.19) and a forcemain routed along the north side of West End Road to the existing trunk sewer.

The utility has a reimbursement policy whereby if Jenn, the owner of the 40-acre tract, constructs facilities that will serve off-site property, the utility will collect prorated funds from other landowners as their property is approved for sewer and pass these funds along to Jenn as they are collected. Jenn will only be entitled to reimbursement for any oversizing of utilities as required to provide service for off-site property. Note that this is only one procedure for managing a reimbursement policy, since the local policy may

TABLE 24.11 Acreage for Each Subarea as Shown in Figure 25.18

AREA	ACRES*	ADDITIONAL ACRES†	TOTAL ACRES
1	15.50	8.61	24.11
2	16.07	5.68	21.75
3	21.12	4.24	25.36
4	36.39	9.41	45.80
5	37.37	3.85	41.22
6	37.54	5.17	42.71
7	49.65	8.15	57.80
8	41.62	6.95	48.57
Totals	255.26	52.06	307.32

*Acres within the drainage boundary.
†Acres within the additional 200-ft-wide strip.

provide for reimbursement on the basis of acreage sewered or by some other procedure.

Early in the project, a meeting between the engineer and the utility staff is very beneficial to both parties and should always be scheduled.

The cost of off-site construction in this example may be more than would be economically feasible in the development of a 40-acre parcel. The principal off-site costs are those of a pumping station and 16,000 feet of forcemain. The engineer is able to apprise the owner of these costs as a part of the preliminary report. This report is completed and presented to the owner before starting design. Thus, the owner is able to make financing decisions before incurring substantial engineering costs. Project financing is not covered in this chapter, but note that the property owner has several options that should be explored. The owner himself or through his legal counsel should meet with other property owners in the sewer shed to determine whether other property owners are interested in participating in the project. The engineer may also assist the owner by sending a letter to each affected property owner advising them of a meeting to be held at the engineer's office for the purpose of determining their interest in participating in the project. The engineer would generally not be authorized to proceed with design until Jenn has made suitable financing arrangements for the construction of the project.

Establishing Sewer Service Shed

A drawing of the sewer shed is shown in Figure 24.19. The drainage boundary above West End Road is 255.26 acres. It should be recognized that, for a rural area, a 255.26-acre area may require many years for complete development, in which case something less than the entire drainage shed would be included in the planning and design associated with sewering the 40-acre tract. This is a decision that should be agreed on by the local government planners and the utility. In this example, it is assumed that the parcel is located in a major urban area where development will occur within a 10-year period once sewer service becomes available.

Contours are not shown on Figure 24.19 for clarity purposes. The figure is shown at a 1-in = 500-ft scale in order to include the entire area on a normal sheet. The engineer would normally use a 24-in by 36-in layout sheet. This would permit using a scale of 1 in = 200 ft. At this scale a 5-foot contour interval would be used, unless the terrain is extremely flat or steep.

The 255.26-acre sewer shed above West End Road has eight subsheds, each having an unnamed intermittent stream draining to West Creek. These intermittent streams are shown as Branch A through G. It is recommended that a maximum depth of 20 ft for the gravity sewer system be used for design purposes.

A 200-ft-wide strip has been used in the example to establish a sewer service boundary. This strip extends around the three sides of the drainage boundary. Lines are drawn to show the drainage limits of the eight subsheds. Also shown is the point where the off-site sewer is proposed to discharge to the West Creek shed. This is shown as Point X.

The engineer must determine how the area will be sewered. In this example, it is determined that the area will be sewered by constructing a main sewer along the east side of West Creek from Point Y to Area 1. Submains will connect at the confluence of each of the branches with West Creek. Each of the submains will sewer the respective area, with no area connecting directly to the main except through the submains. This deci-

TABLE 24.12 Planned Density

AREA NUMBER	PLANNED DENSITY
1	100% single-family at 4 units/acre
2	40% townhouses at 10 units/acre
	60% new elementary school—300 students
3	100% single-family at 4 units/acre
4	100% single-family at 4 units/acre
5	100% single-family at 4 units/acre
6	100% townhouses at 10 units/acre
7	100% single-family at 4 units/acre
8	40% commercial at 2000 gpd/acre
	60% apartments at 25 units/acre

sion is made by the engineer and based on the topography. In this example, the flow for the main sewer is computed. The design of the submains and the sewer laterals would be designed as a part of the development of each parcel.

Determining Average and Peak Flow

A topographic map that shows the boundaries for each planned land use should be prepared for the sewer shed. This information is taken from the comprehensive plan for the area. The area of each use for each subshed is then determined by planimetering. The area for each subshed, as determined by planimeter, is shown in Table 24.11. The planned density for each subshed is shown in Table 24.12.

A drawing similar to Figure 24.19 would be prepared at a scale not smaller than 1 in = 200 ft with all topographic features shown including contours and land use. A map of this type is not provided in this example because of size limitations.

Table 24.13 is formatted for tabulating average and peak flow for each main sewer segment. The length of each segment is shown in column 3 of the table. The peak design flow is provided in both gpm and cfs for general information.

Table 24.14 provides information on elevations for the proposed sewer route and the associated West Creek streambed. This table would not be required in a normal design, as the information would be taken directly from the topographic map data as the design proceeds.

Note the following general guidelines:

1. Keep the sewer depth to at least 6 ft. This minimizes the impact of surface loadings.

2. Design to allow for stream crossings. The top of the sewer for all stream crossings is to be at least 1 ft below the streambed. The sewer, in the area of the stream crossing, will be concrete encased.

Line Design

The sewer design is provided in Table 24.15. Notes are provided as a part of the table to show how the data was determined. In the design example, the design is started

TABLE 24.13 Computation of Sewage Flow

(1) From Point*	(2) To Point*	(3) Length Feet	(4) Area Added†	(5) Acres Total‡	(6) Density Added‖	(7) Added[a] Ave. (gpd) Flow§	(8) Total Flow[b] (gpd)	(9) Peak Factor[c]	(10) Design Flow[d] (gpm)	(11) Design Flow (cfs)
8	7	700	1(24.11)	24.11	96 S.F.[e]	35,520	35,520	6.4	158	0.35
7	6	220	2(21.75)	45.86	87 T.H[f]	26,100	61,620	6.0	257	0.57
6	5	470	3(25.36)	71.22	101 S.F.	37,370	98,990	5.9	406	0.90
5	4	680	4(45.80)	117.02	458 T.H.	137,400	236,390	5.4	886	1.97
4	3	470[g]	5(41.22)	158.24	164 S.F.	60,680				
			X(28.0)	186.24	112 S.F.	41,440	338,510	5.1	1199	2.67
3	2	670	6(42.71)	228.95	427 T.H.	128,100	466,610	4.9	1588	3.54
2	1	290	7(57.80)	286.75	231 S.F.	85,470	552,080	4.7	1802	4.01
1	Y	250	8(48.57)	335.32	Commercial 728 Apts.	38,856 218,550	809,486	4.5	2530	5.63

*Points refer to confluence of subshed drainage with West Creek and correspond with area added as shown in column 4.
†Acres taken from mapping by planimeter (see Figure 25.18 and Table 25.11).
‡Areas are totaled to confluence of indicated streams.
‖Allowable density obtained from comprehensive plan of local government (see Table 25.12).
§Average flow per unit obtained from Table 25.2.
[a]Flow added is unit flow times the number of units.
[b]Total flow is total to indicated downstream point.
[c]Peak Factor obtained from Figure 25.2.
[d]Design flow is obtained by multiplying average flow times the peak factor to obtain gpd and then dividing by 1440 minutes/day to give flow in gpm. Cfs is obtained by dividing gpm by 449 gpm/cfs. MGD is obtained by dividing gpm by 695 gpm/mgd or cfs divided by 1.55 cfs/mgd gives the flow in mgd. Cfs or gpm can then be used with the nomograph shown on Figure 25.7 to establish head loss and pipe size.
[e]S.F. = Single-family detached housing.
[f]T.H. = Town house.
[g]Note that when the distance between points in greater than 400 ft, and intermediate number is needed. See the design tabulations shown in Table 15.5.

TABLE 24.14 Elevations Determined from Field Surveys

Station§	Ground Elev. (ft)*	Stream Inv. Elev. (ft)†,‡	Station§	Ground Elev. (ft)	Stream Elev. (ft)
0+00	212.0	206.8	19+00	221.6	217.4
1+00	212.0	207.8	20+00	222.0	218.0
2+00	213.2	208.2	21+00	222.5	218.7
3+00	213.7	208.9	22+00	223.1	219.3
4+00	214.4	209.3	23+00	223.6	219.9
5+00	215.0	209.7	24+00	224.0	220.5
6+00	215.9	210.1	25+00	224.7	221.0
7+00	216.4	210.4	26+00	225.2	221.4
8+00	216.6	211.0	27+00	225.8	222.1
9+00	216.8	211.6	28+00	226.5	222.9
10+00	217.2	212.3	29+00	227.0	223.5
11+00	217.4	213.0	30+00	227.6	224.2
12+00	217.9	213.3	31+00	228.2	224.8
13+00	218.5	213.7	32+00	228.9	225.4
14+00	218.9	214.3	33+00	229.5	226.2
15+00	219.6	214.9	34+00	230.1	227.0
16+00	220.1	215.5	35+00	230.7	227.5
17+00	220.7	216.2	36+00	231.2	228.2
18+00	221.0	216.8	37+00	231.7	228.0

*Ground elevations were taken from field topography for the selected sewer route.
†Stream elevations are for the lowest point in the stream cross section at the respective station along the sewer route.
‡The sewer route is within 25 ft of the stream at all points.
§Station 0+00 is adjacent to proposed location of pumping station.

adjacent to the proposed pumping station and continued upstream. When designing sewers to serve relative flat terrain in which the ground slope along the sewer route is less than the slope required to provide minimum velocity in the sewer, the design should be started at the upper end of the system and continued downstream. This is because the sewer will continue to get deeper as the design proceeds. If the design is started at the lower end, the designer does not know what initial depth to use so that adequate cover will be provided upstream, making several trial-and-error attempts necessary.

Preparation of Construction Plans

The engineer now has all the information required to complete a set of construction drawings. It is important that the plans be clear and complete. These plans would normally be prepared on 24-in by 36-in sheets (local utility may have other requirements) and consist of:

- *A cover sheet* showing the project title, any identifying project number, a location map, and the name of the engineering firm. The cover sheet also often includes a sheet index for the drawings and the name of the owner. If the owner is a public body, the names of the elected officials such as mayor and council members may be shown as local practice requires.
- *A general notes and legend sheet.* This sheet should include specific notes for the construction of the improvements in the designated municipality or authority service area, especially if the improvements will be

TABLE 24.15 Sewer Design Tabulation*

(1) Line	(2) From	(3) To	(4) Length (ft)	(5) Peak Flow	(6) Ground Lower	(7) Elev. Upper	(8) Stream Elev. (m)	(9) Sewer Dia.	(10) Vel.	(11) Slope	(12) Head Loss	(13) Q Full	(14) Sewer Lower	(15) Invert Upper	(16) Control
1	Y	1	250	5.63	212.0	213.4	208.6	15	4.1	0.0056	1.40	5.0	204.75	206.15	Cover
2	1	2	290	4.01	213.4	215.5	209.9	15	3.9	0.0052	1.50	4.8	206.15	207.65	Stream
3	2	3	340	3.54	215.5	216.7	211.5	15	3.7	0.0047	1.60	4.6	207.65	209.25	Stream
4	3	4	330	3.54	216.7	217.9	213.4	15	3.6	0.0042	1.40	4.5	209.25	210.65	Cover
5	4	5	470	2.67	217.9	220.6	216.0	12	3.6	0.0059	2.75	2.9	210.85	213.60	Cover
6	5	6	340	1.97	220.6	222.1	218.2	12	3.1	0.0044	1.50	2.5	213.60	215.10	Cover
7	6	7	340	1.97	221.1	223.9	220.3	12	3.4	0.0053	1.80	2.8	215.10	216.90	Cover
8	7	8	470	0.90	223.9	226.7	223.1	8	2.6	0.0054	2.54	0	217.16	219.70	Cover
9	8	9	220	0.57	226.7	227.9	224.5	8	3.0	0.0070	1.53	1.1	219.70	221.23	Cover
10	9	10	350	0.35	227.9	230.1	227.0	8	2.8	0.0063	2.20	0.9	221.23	223.43	Cover
11	10	11	350	0.35	230.1	232.0	228.2	8	2.7	0.0054	1.90	0.93	223.43	225.33	Cover

Column 1 = Number of sewer segment—generally numbered consecutively upstream.
Column 2 = Number of manhole at downstream end of sewer segment.
Column 3 = Number of manhole at upstream end of sewer segment.
Sewer manholes are numbered consecutively upstream. Submains connecting to the main numbered from that point. For example, a submain or lateral connecting to the main sewer at manhole 5 would have manhole numbers of 5-1, 5-2, etc.
Column 4 = Length of sewer segment measured from center to center of manholes.
Column 5 = Peak flow calculated for sewer segment (see Table 25.13).
Column 6 = Ground elevation at downstream end of sewer segment.
Column 7 = Ground elevation at upstream end of sewer segment.
Note that the ground elevation is carried to the nearest 0.1 ft, whereas the sewer invert is carried to the 100th of a ft.

Column 8 = Stream elevation at upstream end of sewer segment. Must be able to cross stream with one ft of cover over top of pipe. Column 16 is added to show if stream elevation is controlling.
Column 9 = Diameter of sewer segment in inches.
Column 10 = Velocity of flow in sewer segment.
Column 11 = Slope of invert for sewer segment.
Column 12 = Feet of head loss in sewer segment.
Column 13 = Full capacity of sewer.
Column 14 = Sewer invert elevation at downstream end of segment.
Column 15 = Sewer invert elevation at upstream end of segment.
Column 16 = Condition controlling depth. This column is not shown on sewer design tabulations, but included here for instructional purposes. The design criteria employed in this example is that the sewer cover is to be at least 6 ft and that at least 1 ft below the stream invert. The invert of a sewer is the lowest elevation on the inside of the pipe. The invert elevation is maintained in sewer construction and design.

*Design procedure: For line 1 running from manhole Y to manhole 1, check ground slope for segment. Difference in elevation is found to be 1.4 ft, and slope is head loss divided by segment length = 1.4/250 = 0.0056 as shown in column 11. Manning's n is 0.013 (generally always used for design). Knowing n and S, from Figure 24.7 using a straight edge, the two points will establish a point on the pivot line. Maintaining the pivot point, rotate the straight edge to the required Q as shown in column 5. Note that the straight edge falls between 12 in and 15 in pipe diameter. The straight edge is rotated about the pivot point to the next larger pipe diameter of 15 in. At this alignment of the straight edge D, V, and Q full is established. The most economical condition is to lay the sewer on the same slope as the ground surface. In this example the grund slope is adequate to provide the minimum velocity of 2.0 fps. A 6-ft cover requires that the sewer invert be 6.0 ft + the 1.25-ft pipe diameter, 7.25 ft from the ground elevation at each end of the sewer segment. In some instances it is necessary to check cover at low point between manholes and the designer should always make this check. Columns 9, 10, and 13 data is obtained from the nomograph shown as Figure 25.7. Columns 14 and 15 are based on cover to be provided and head loss as shown by column 12. Check upper end invert for cover at stream crossing when submain is installed.

The rules for connecting sewer segments at manholes are: The crown of the entering and exiting pipe are at the same elevation. The 0.8th depth of the entering and exiting pipe are at the same elevation. This option has been used in this example. Note Manhole 4 where the entering sewer is 12 in and the downstream sewer is 15 in in diameter. In this case the drop in invert elevation across the manhole is 0.20 ft. If the crowns had been placed at the same elevation the drop would have been the difference in the diameter of the two sewers or 0.25 ft. Past practice has been to provide a 0.10-ft drop across the manhole where there is no change in pipe diameter. This practice is no longer considered to be necessary. If a smooth chsnnrl having the same diameter as the sewer is constructed across the manhole when the bench is made, there is no need for this drop. In fact, the change in velocity resulting from a 0.10-ft drop is likely to cause problems. Note in the data in Table 25.15 that no loss is shown across the manhole where there is no change in pipe diameter.

In cases where the sewer is at a depth greater than what is needed to provide minimum cover, the sewer slope to be used is generally the slope required to provide the minimum velocity, which is usually 2.0 or 2.25 fps. This permits minimizing the depth of construction. Never use a greater pipe size than what is required to carry the flow. Designers sometimes do this when designing sewers for flat terrain to reduce the slope required to provide the minimum velocity (Note in Eq. (25.7) that the larger the pipe diameter, the smaller the slope required to maintain a velocity). This practice will lead to maintenance problems because the actual velocity of the partially filled pipe will result in the deposition of solids.

dedicated to the municipality or authority after construction. The legend provides definitions for symbols and details used on the plan and profile sheets.

- *A general layout sheet* showing streets, lots, and sewer location. The layout sheet should be to a scale where the entire project can be shown on a single sheet. The design sheets should be overlaid on the layout sheet as an index.

- *Plan and profile sheets.* These sheets normally show the design in plan view on the top half and in profile on the lower half. All topographic data related to the design construction area should be shown. The manholes are shown and numbered. The lines are shown with bearings and distances with any angles being shown at changes in horizontal alignment. Sewer size, manhole numbering, and stationing are shown on both the plan and profile views. The plan view should show existing and proposed structures, houses, underground utilities, curbs, property lines, all storm drainage, and any other infrastructure. The profile shows the elevation of all critical points such as all manhole inverts and top of covers to 0.01 ft. Care must be taken to show the location and elevation of all subsurface utilities, foundations, and other elements important for safe, orderly, and economical construction. Field-test pits may be needed for the proper location of existing subsurface elements.

- *The plan view* is generally shown at a scale of 1 in equals 50 ft. The horizontal scale for the profile is shown at the same scale as the plan. The vertical scale for the profile is generally shown at a scale of 1 in equals 5 ft.

- *Standard details sheets.* Standard details for manhole, manhole frame and covers, special construction such as waterproof frame and covers, and stream crossings are shown on sheets as needed following the plan and profile sheets.

- *Submain and building spur connections.* In subdivisions it is common to locate the sewer near the middle of the street where possible and construct all building spurs to the property line and cap the end of the line. It is important to show the location and elevation of each spur on the drawings for later location. The spur should enter the sewer through a manufactured wye. House spurs generally do not connect to a manhole. Submains should enter the main at a manhole where the connection has been cast as a part of the manhole.

Note that the sewer design results in establishing the sewer invert profile. The sewer conduit is uniform and at steady flow conditions. The invert (lowest point inside the sewer conduit) is parallel to the hydraulic grade line (HGL). While the HGL will rise and fall with changes in flow conditions, it will always be within the pipe under gravity flow conditions. Since the HGL location varies, the invert slope is established in design and utilized in constructing the sewer.

PUMPING STATIONS

General Information

Pumping stations are common to most sewage collection systems. The collection system that does not include lift or pumping stations at one or more locations is rare. The engineer has available a wide choice of pump types giving flexibility in selecting the type of facility most suitable for a particular application.

The term *lift station* refers to a pumping facility located within a collection system to lift the flow to a higher elevation. Sewer systems located in relatively level terrain may become excessively deep, making it necessary to lift the flow up to minimum cover for the sewer. In Table 24.5, the slope of an 8-in sewer providing a velocity of 2.25 fps is 0.0042. This sewer has a head loss and, hence, a drop in the sewer invert of 4.2 ft per 1000 ft of length. If the sewer is long or if the sewer is not running with the ground slope, the depth may become so deep that a lift station is needed. The depth where a lift station becomes economical depends on subsurface conditions. The presence of a high ground water table, poor soil, or rock must be considered in making this determination. Also, the length of sewer that will be excessively deep must be considered. Utility maintenance personnel generally prefer that sewers not be over 20 ft deep if greater depths can be avoided.

A pumping station is a facility designed to lift the flow and convey it some distance, such as from one drainage area to another or from the collection system to a treatment works. A lift station is commonly referred to as a pumping station, but pumping stations are not called lift stations.

Location

A pumping station should be located so as to serve the entire sewer shed. In the sewer design example in the previous section, it was found as a part of the preliminary investigation that the limit of the sewer shed was to be West End Road. This may have been a political boundary or a decision made by the local government because of land planning. The station must be located so as to receive the sewage flow from the designated sewer shed. Some general considerations to be used in locating a pumping station are as follows:

- A lift station or pumping station should never be located in a public street.

- The station must be accessible by an all-weather service road, where the utility either owns the right-of-way or has a permanent access easement.

- The station must be located to provide protection from flood inundation. The station should as a minimum remain fully operational at the 25-year flood elevation and receive no damage at the 100-year flood level. The criteria used in determining flood protection are dictated by local policy and conditions related to the site, such as the potential for unacceptable environmental pollution when overflows or sewer backups occur. In no case should sewer backup due to pump station failure result in flooding of buildings with sewage.

- Consideration should be given to problems with noise and odor. Odor is most likely to be a consideration with flow from large long main or trunk sewers where the sewage becomes septic in the sewer before reaching the pumping station. The discharge from a forcemain is also usually septic. Equipment is available for removing odors from exhaust air. Screenings are another source of odors, and they should not be stored at locations where odors will impact the surrounding area. Accumulation of solids in a poorly designed wet well may also cause odors.

Noise will not normally be a problem at a properly designed facility. The noise level from the operation of pumps and motors is low and should not be noticeable outside the station. There will be some minor noise from the maintenance truck and crew that checks the station on a daily basis. If the station design includes on-site emergency power, the generator will need to be exercised weekly, but residential-type mufflers are available and should be used. The exercise period is usually one-half hour and should be scheduled during the normal workday.

- A buffer area of at least 100 ft around the station should be owned by the utility. A cyclone-type fence should be installed around the station to minimize problems with vandalism and, more important, reduce the possibility of accidents involving children.

- The exterior architecture of the station should not detract from the appearance of the neighborhood. A pumping station can be located in any residential neighborhood if it is designed and sited in an attractive manner. Windows should be avoided to reduce vandalism. False windows can be used where needed for architectural reasons. The owner should always be consulted about the exterior appearance of the station. The grounds should be properly landscaped for appearance and drainage. Use slow-growing shrubbery, gravel areas, and pavement to reduce maintenance requirements.

Design Types

The engineer should be familiar with the requirements of the utility that will own and operate the station. Utility personnel prefer to have equipment that they are familiar with for ease in maintenance. If the utility staff does not like the type of equipment provided, it probably will not be a successful project. It is also important that the equipment be procured through a manufacturer's representative who will be available and responsive if problems occur with the equipment.

A small lift station may serve such uses as a few houses, a business, a church, or a school. This type of station can be as simple as placing one or two small pumps in a manhole. It is widely used with submersible pumps, with the manhole housing the pumps serving as the wet well. Small grinder pumps are often used, in which case the forcemain may be a 1¼-inch plastic pipe. Pneumatic ejectors may also be used for small lift stations, but ejectors are generally located inside buildings to lift the flow from floors lower than the public sewer up into the sewer. Ejectors located inside buildings are part of the building plumbing. In most instances these small lift stations should include two pumps with automatic alternation for station reliability. Figure 24.20 shows a typical design of this type.

Moving up in capacity, the second type of pumping or lift station may be of the same general type as described earlier, except for the size or capacity of the equipment and the reliability of the station. A station of this size would always pump into a forcemain 4 inches in diameter or greater. The station should always have two pumps and generally have on-site emergency power. The engineer has many options available for selecting the type of pumping facility to be used. However, the submersible station design normally offers the most economical and functional choice for use in systems serving small communities.

Figure 24.20 shows a design employing two submersible pumps placed in a properly sized concrete manhole. The bottom of the manhole is filleted and sloped to direct the flow to the pump intake and to prevent solids from accumulating in the manhole. The pump mounting design provides for the pump to be removed and replaced by a lifting chain without the need to enter the manhole. A lifting hoist is provided as part of the station design. All valves and controls are located in a separate vault for ease in maintenance, thus eliminating the need to enter the wet well. This type of pumping station can be used for designs of any size. Where the average daily flow is greater than 0.5 mgd, the two pumps may be placed in separate chambers that are interconnected with isolation valves so that either chamber may be dewatered for maintenance.

There are several manufacturers of reliable submersible pumps. No alternative offers the simplicity, reliability, and economy that are achieved with a submersible pump station. Four-ft-diameter manholes are normally used as the wet well in collection systems; however, manholes are generally available in diameters up to 10 ft. When a large-diameter manhole is needed for locations where they are not available, reinforced concrete pipe can be placed vertically on a concrete pad. When concrete pipe is used, the design must connect the pad and pipe in a manner that prevents leakage at the joint.

While the submersible pumping station is recommended, there are hosts of package pump station designs available. Generally, the entire pumping station is shipped to the site already assembled and is placed on a concrete pad, connected to a wet well discharge, forcemain piping, and an electrical supply. Typically, this type of station is referred to as a suction-lift pumping station. The name defines how it operates: a suction pipe is placed within a wet well of simi-

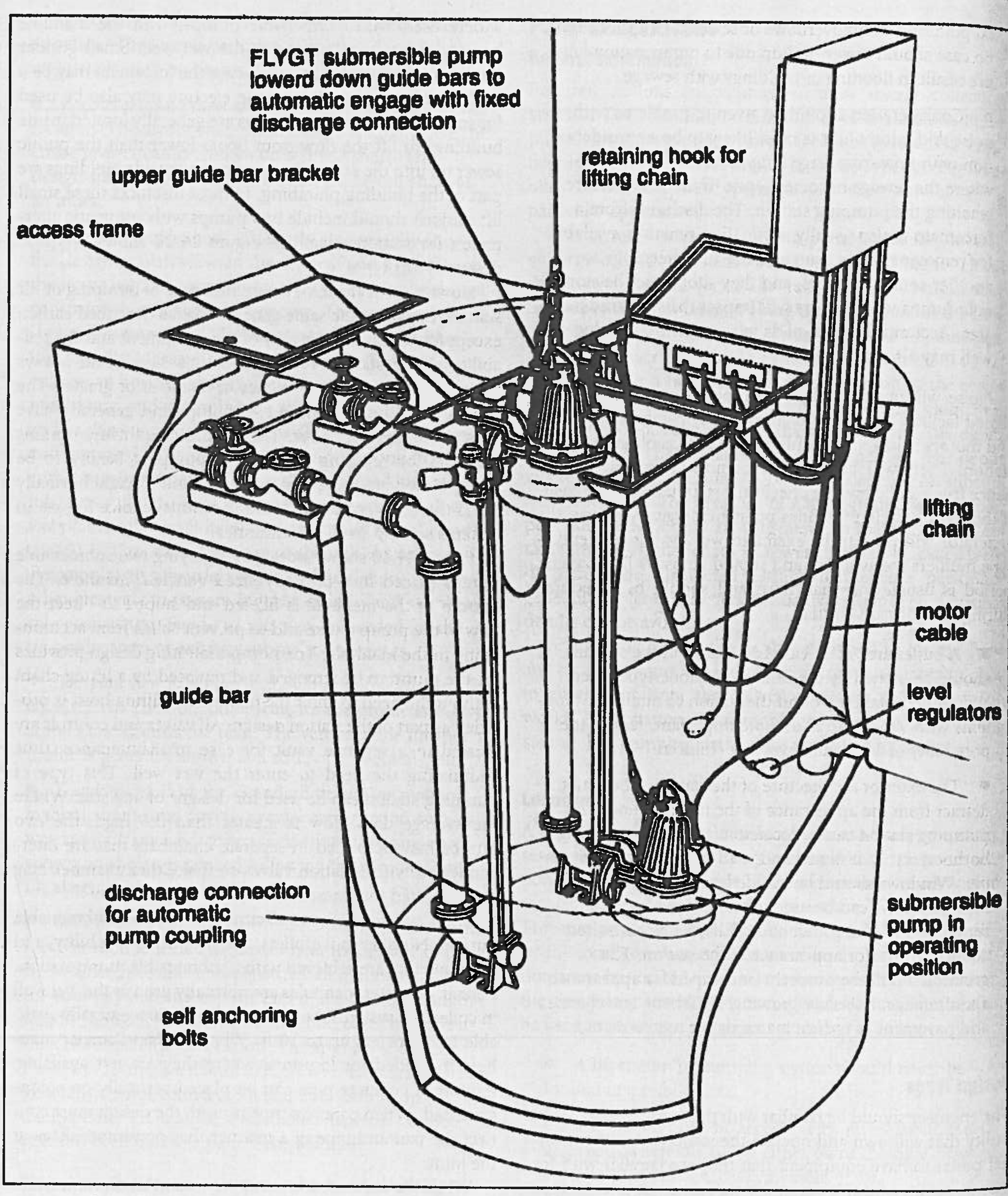


FIGURE 24.20 Typical submersible pump station. (Reprinted with permission from ITT FLYGT Corporation, 1993, Wastewater Pumps Catalog, Trumbull, CN)

lar shape and size as a submersible station, and the lines are connected to pumps placed aboveground. These pumps discharge into a forcemain that conveys the sewage to the receiving manhole in another collection system or the treatment works. The pumps and associated controls are typically housed in a prefabricated fiberglass or concrete station that is easily accessed by maintenance personnel. In addition to limitations that impact the operation of submersible pumping stations, the suction-lift station has limitations with the depth of the wet well. Based on the location of the pumping station in relation to sea level, the pumps are typically unable to pull the sewage vertically through a suction line greater than 25 ft. Suction-lift stations are generally more expensive to construct, but are typically easier to maintain than a submersible pumping station.

The engineer always has the option of designing a constructed-in-place wet well–dry well design where reinforced concrete basins are constructed and the pumping equipment, piping, and associated appurtenances are assembled in place. The wet well is the receiving chamber for the incoming flow. The wet well serves as a flow surge basin and as short-term storage during periods when the pumps are not operating. The dry well is isolated from the wet well except for the pump intake pipes. The dry well houses the mechanical equipment. The electrical equipment, including the pump motors, is generally placed in the ground floor housing located over the dry well. Both the wet well and the dry well must be suitably lighted and ventilated and have proper access for maintenance personnel. This type of pumping station can be designed to incorporate all specific requirements of the operating utility, but it is the most expensive to construct and to operate and is normally not an option for a station serving a land development project.

With designs where the maintenance personnel must enter the wet or dry wells, a minimum two-person crew must be present, whereas only one person can carry out the routine maintenance at a submersible station since no belowground tasks are required. It is important to note that all pumping station designs be checked for flotation. While more engineers and utilities are using submersible pump stations as described earlier, the older accepted practice of constructing a built-in-place wet well–dry well pumping station remains the standard design for the larger installations. Submersible pumps can be used in dry well installations.

Components of a Pumping Station

The flow schematic shown in Figure 24.21 is common to all pumping stations, except the gate valve is not needed between the wet well and the pump in a submersible design because the pump is installed in the wet well.

Screening. Screenings are the larger particles of floating and suspended matter that may clog pumps and other equipment. The quantity of screening normally does not exceed 0.5 ft^3 per million gallons of flow for residential sewage. Rags are the most troublesome items in pumping stations. The quantity of screenings to be removed depends on the type of housing and other sources of incoming sewage. The screening device should be located such that all incoming flow passes through the chamber before entering the wet well. The device should only remove material that interferes with the station operation because of the difficulty of handling removed material. Disposal methods include grinding and returning the material to the flow, burial, and incineration. Burial and incineration should be off-site from the pumping station. Maintenance crews generally carry the screenings from small pumping stations back to the treatment plant for treatment and disposal. Macerators are available for locating on the incoming sewer and should be considered for use on sewers serving industrial areas producing significant fibrous wastes.

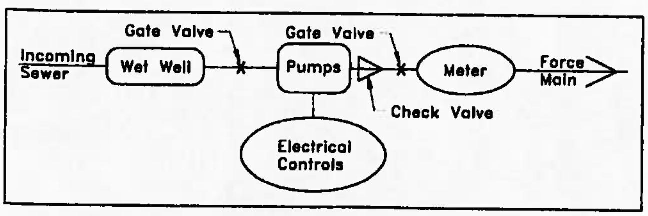


FIGURE 24.21 Schematic of a pumping station.

The screening device for pumping stations of small to moderate size, which primarily serve residential areas, may be a basket located such that the flow passes through the basket as it enters the wet well. Daily cleaning should be scheduled until experience shows that a less frequent schedule is adequate. The design permits overflow in case of stoppages without causing a sewer backup. The basket should be designed for easy removal of the screening. With submersible stations, the basket is lowered into place and removed by a cable hoist. The basket is held in place by guides or by hanging hooks, as shown in Figure 24.22. Hanging hooks are generally more satisfactory because guides are a maintenance problem.

Fixed bar screens are used in larger pumping stations. The bar screen is placed in a straight channel, usually located at the entrance to the wet well. The screen is inclined from 30 to 60 degrees from the horizontal. The corners of the channel should be filleted to minimize problems with accumulation of solids, particularly grit. The bars generally have a face dimension of ¼ to ⅜ inches and a depth from 2 to 2½ inches. The bar size for larger screens is determined by the structural requirements, with the face dimension limited to ⅜ inches. The clear space between the bars is normally 2 to 2½ inches. The bars are held together with a welded strip located on the downstream side at the bottom and top of the screen. A rake, having teeth size and spacing compatible with the screen, should be provided for cleaning the screen. The screenings are raked to the top of the screen, where they fall into a can having a perforated bottom for drainage or onto a drain rack. The top of the bars may be curved over to facilitate cleaning. Fiberglass racks and slide gates are available and should be considered for a design. Mechanically cleaned screens are used at large stations.

The approach velocity to the screen should not be greater than 2.5 fps at peak flow. The design must allow for the energy loss in the channel and through the screen so that the incoming sewer is not surcharged. Also, the maximum operating level in the downstream wet well must not surcharge the screen. When the screen is located in a subsurface chamber, suitable hoisting equipment should be provided for lifting the screening to the surface. A typical manually cleaned bar screen is shown in Figure 24.23. Calculations are shown for sizing a screen to handle the flow from the West Creek sewer.

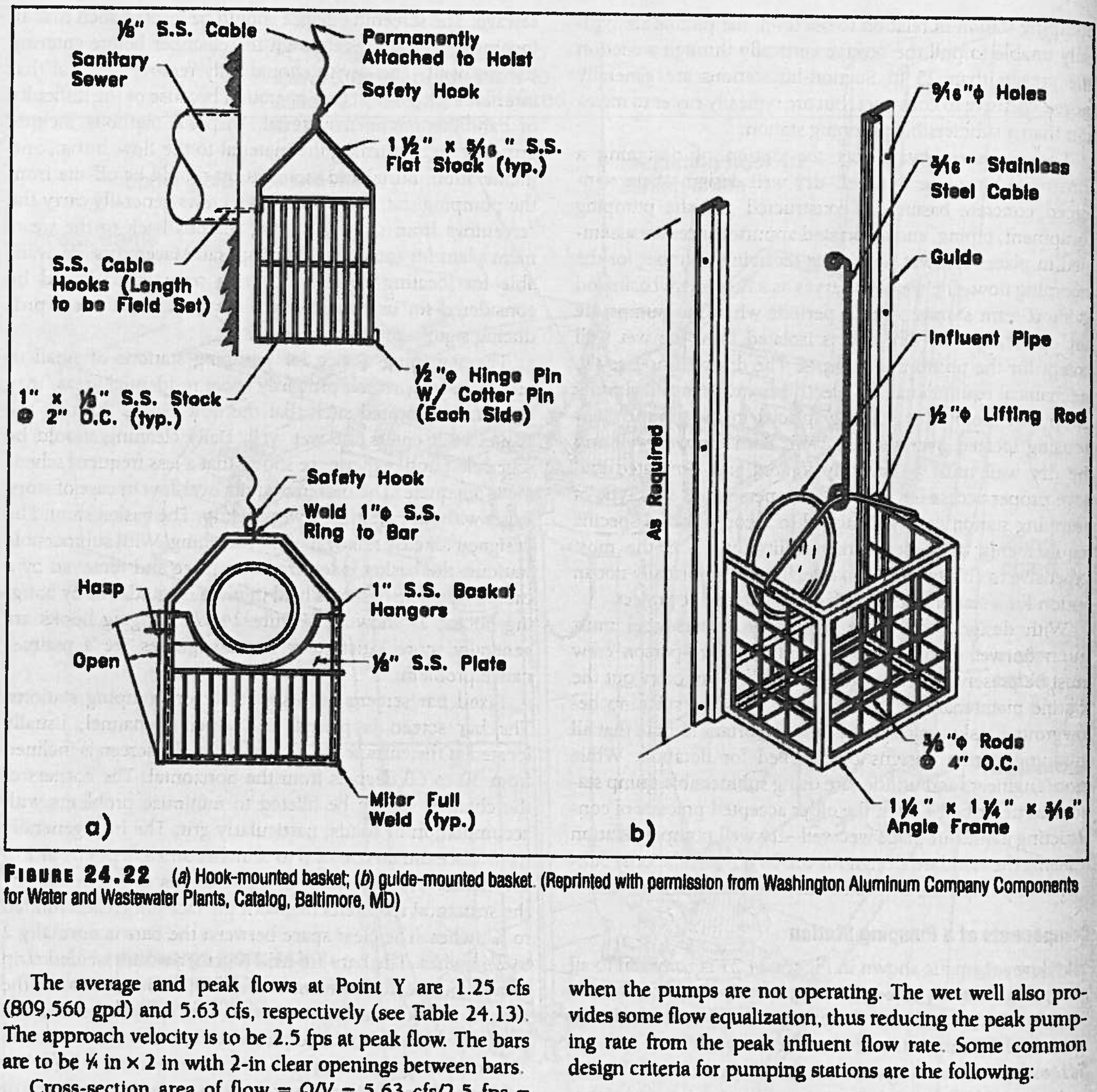


FIGURE 24.22 (*a*) Hook-mounted basket; (*b*) guide-mounted basket. (Reprinted with permission from Washington Aluminum Company Components for Water and Wastewater Plants, Catalog, Baltimore, MD)

The average and peak flows at Point Y are 1.25 cfs (809,560 gpd) and 5.63 cfs, respectively (see Table 24.13). The approach velocity is to be 2.5 fps at peak flow. The bars are to be ¼ in × 2 in with 2-in clear openings between bars.

Cross-section area of flow = Q/V = 5.63 cfs/2.5 fps = 2.25 ft^2.

Select Channel Depth. The incoming 15-in sewer at 0.8 depth of flow = 1 ft; therefore, a 1-ft depth is selected for the approach channel. Width of channel = 2.25 ft/1.0 ft = 2.25 ft = width of screen. Note in Figure 24.23 that free fall of about 3 in is provided at the influent pipe and 6 in of fall is provided through the screen chamber. The energy loss through a clean screen is small, generally less than 1 in. But most of the time the screen is partly clogged, which can materially increase the loss. Note the curved bars that aid in handling the screening. The screenings are raked onto a drain pad. After draining, the screenings are removed for disposal.

Sizing of the Wet Well. The wet well is a chamber that has been sized to receive and accumulate the incoming flow when the pumps are not operating. The wet well also provides some flow equalization, thus reducing the peak pumping rate from the peak influent flow rate. Some common design criteria for pumping stations are the following:

1. Pumps are selected to pump 2.5 times the average daily flow rate with the largest pump out-of-service. For example, if only two pumps are provided, each must have a pumping capacity of 2.5 times the average daily flow rate. If three pumps are provided that are not of equal size, the two smaller-capacity pumps must be capable of pumping 2.5 times the average daily rate. All pumping stations should have at least two pumps so that any single pump can be removed for maintenance.

2. There is a heat buildup in pump motors if the frequency of starting is too great. This can shorten the life of the motor. Most heavy-duty motors of the type used with pumps can be started up to about 10 times per hour without overheating. Starting frequency is con-

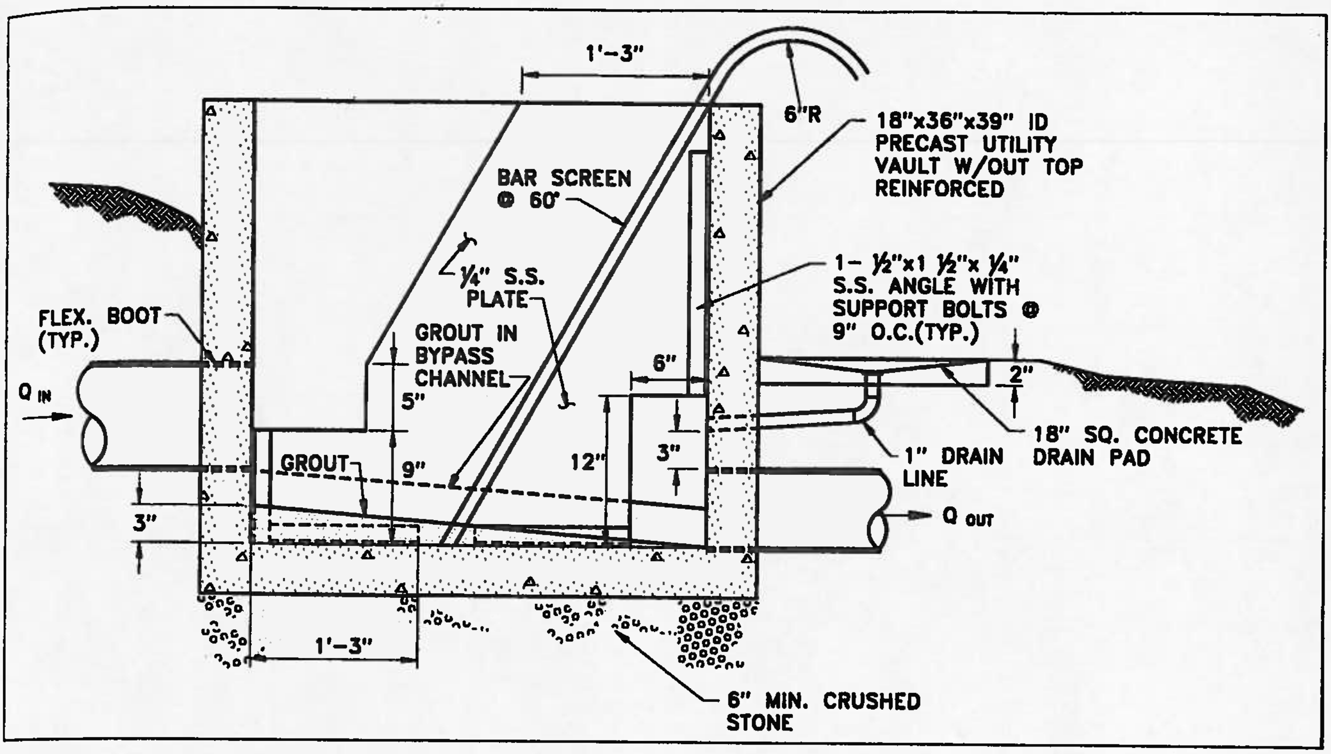


Figure 24.23 Manually cleaned bar screen.

trolled by the size of the wet well, and this should be coordinated with the electrical designer to ensure that the proper class of motor is specified.

3. The wet well should be filleted to eliminate corners where grit can accumulate.

4. The wet well should be divided into sections at larger stations so that the section supplying each pump can be isolated and dewatered for maintenance. This can be accomplished with submersible pumps by placing each pump in a separate manhole that is suitably interconnected and valved.

5. If maintenance crews are required to enter a chamber for routine maintenance, proper ventilation must be provided. The ventilation fan should be sized to provide 30 air changes per hour. Air is usually exhausted from near the floor so that fresh air is brought into the top of the chamber as the crew descends. The exhaust fan switch should be located at the top of the stairs or ladder so the fan can be started before entering the subsurface chamber.

6. A potable water hose bib should be available at each structure for use in cleaning. The potable source should be protected by installing a backflow preventer on the service line serving all outlets except the drinking fountain and bathroom.

7. A flow-measuring device should be included in the design. Ultrasonic meters are reliable and widely accepted.

8. The short intake piping and intrastation effluent piping may be designed for velocities of 6 to 8 fps, but force mains are not normally designed for velocities greater than about 5.0 fps at the peak pumping rate. Higher velocities scour and erode the pipe, and the energy loss increases exponentially with an increase in velocity. A minimum velocity of 2.0 fps should be provided at the minimum pumping rate.

9. Pump stations, except for the very small ones, should have an alarm system for the following malfunctions: power supply failure, pump fail to start, high level in wet well, failure of sump pump to operate, and failure of on-site generator to start. Other alarms may be needed at specific locations. The alarm should be relayed by telephone line or other means to a central staffed location.

10. A check valve should be placed in each pump discharge pipe to prevent backflow through the pump. A gate valve should follow the check valve so that the pump and check valve can be isolated for maintenance.

11. The top of the pump bowl on horizontal impeller shaft pumps should contain an air release valve that is vented back to the wet well.

12. No air should enter the dry well from the wet well.

13. Pumping stations located in an area where sewage backup or overflow would result in a health hazard should include an on-site power generator. These gener-

ators are normally driven by diesel engine and designed to start automatically when the off-site power supply fails and return to standby conditions when the off-site power supply returns. The units are designed to keep the facility completely operational. An on-site fuel supply for 36 to 48 hours of operation is provided as a part of the design.

Pump Selection

Sewage pumps are typically centrifugal volute nonclog impeller design. The impeller should be designed to pass a 2½-inch-diameter sphere. Other pumps, such as grinder pumps, utilize a cutting-type impeller that reduces the size of the solids that are being pumped and allows for a reduction in the discharge forcemain diameter. The pump manufacturer provides performance curves for each impeller available for the specific pump. A typical performance chart for a submersible pump is shown in Figure 24.24.

The performance curve shown in Figure 24.24 shows only the characteristics of the particular pump for a single impeller. The manufacturer casts the largest impeller that can be used with the pump. Impellers are then trimmed to the diameter required to provide the head and flow needed.

Figure 24.25 provides performance data for eight impeller diameters. The diameters, in millimeters, are shown along the ordinate of the graph. Efficiency curves are shown for the operational range of the pump, with the peak efficiency being 87 percent for the 670-mm impeller. Additional data available from catalogs includes intake and discharge connection diameter, brake horsepower, and rotational speed. Rotational speed is also an important characteristic in pump selection. Higher rotational speed increases flow and head, but bearing life is generally materially reduced. A larger pump with lower speed may cost more, but reliability is significantly improved. The pump selected should operate at less than 1700 rpm and specify any diameter between 7.0 in and 10 in. Net positive suction head (NPSH) is the difference between the absolute pressure at the suction point and the vapor pressure of the liquid. Failure to maintain the NPSH can result in vaporization of the liquid being pumped, with the resulting cavitation causing damage to the pump. The low water pump shutoff level is usually set no lower than the midpoint of the pump bowl so that pump suction is not a problem. Pump efficiency is shown to vary between 58 percent and 72 percent. Brake horsepower is shown for each of the four impeller sizes.

The total head imparted to the flow is shown as the ordinate for the indicated flow rate. The head imparted is not the same as the discharge pressure. The imparted head is the discharge energy minus the intake energy of the flow. Note that, for centrifugal pumps, the discharge head decreases as the motor horsepower requirement increases. Never oversize a centrifugal pump, as this may lead to an overloaded motor.

It is sometimes necessary to install two centrifugal pumps in series to provide the required energy input. When centrifugal pumps are installed in series, the flow through each pump is the same and the energy input is the sum imparted by the two pumps. The two pumps are designed as separate pumps and piped to operate in series. With water pumps, series impellers can be installed in a single multistage pump housing (multistage pump has two or more impellers on the same shaft within a single housing). However, this cannot be done with sewage pumps because the pump must be designed to pass solids. Therefore, sewage pumps are designed as separate pumps where series operation is required to induce the required head.

When centrifugal pumps are installed in parallel, the flows cannot be summed to obtain the flow from the two (or more) pumps. Figure 24.26 shows a pump curve that has been constructed for four pumps installed in parallel. The pump curve is constructed by selecting a head and drawing a line parallel to the abscissa. Each pump discharge for that head is added along the line. In Figure 24.26, all four pumps are identical, but the same principle applies if the pumps are different. When pumps having different performance characteristics are installed in an installation, the smaller pump will not function if the discharge head of the larger pump is greater than the shutoff head of the smaller pump. The shutoff head is the head produced by a pump at zero flow and is shown by the ordinate-intercept of the pump curve.

A system curve is then constructed by starting with the static head at the ordinate-intercept and adding the system energy loss at two or more flow rates to permit drawing the curve. The interception of the system curve with the performance for the indicated number of pumps shows the capacity for the respective number of pumps. Note that in Figure 24.26, since four pumps are identical, the distance between the curves at the total head value are the same because each pump has the same pumping capacity at any given head. However, as more pumps come online, the energy loss increases as defined by the Hazen-Williams equation and shown by the system curve. Thus, the flow capacity of two pumps is less than twice that of the single pump operating. The capacity of each pump is the same and is one-half, one-third, or one-fourth of the total, depending on the number of pumps operating.

Pumping stations serving a long forcemain should have a surge relief valve to relieve the surge from water hammer. This valve is located on the station discharge piping and is vented back to the wet well.

The incoming 15-inch sewer at Point Y has an invert elevation of 204.75 ft (see Table 24.15). The fall in the piping and bar screen is 1.00 ft, giving an invert elevation entering the wet well of 203.75 ft. Each of the two pumps are to be installed in an 8-ft diameter manhole (volume per vertical foot = 375.8 gal per manhole = 100 ft^3 for the two manholes). The minimum inflow rate is taken as about 10 percent of the average rate or 50 gpm. The peak inflow rate is 2530 gpm (see Table 24.13) and the pumping rate will be 2.5 times the average daily inflow rate (2.5 × 562.2 gpm = 1405 gpm).

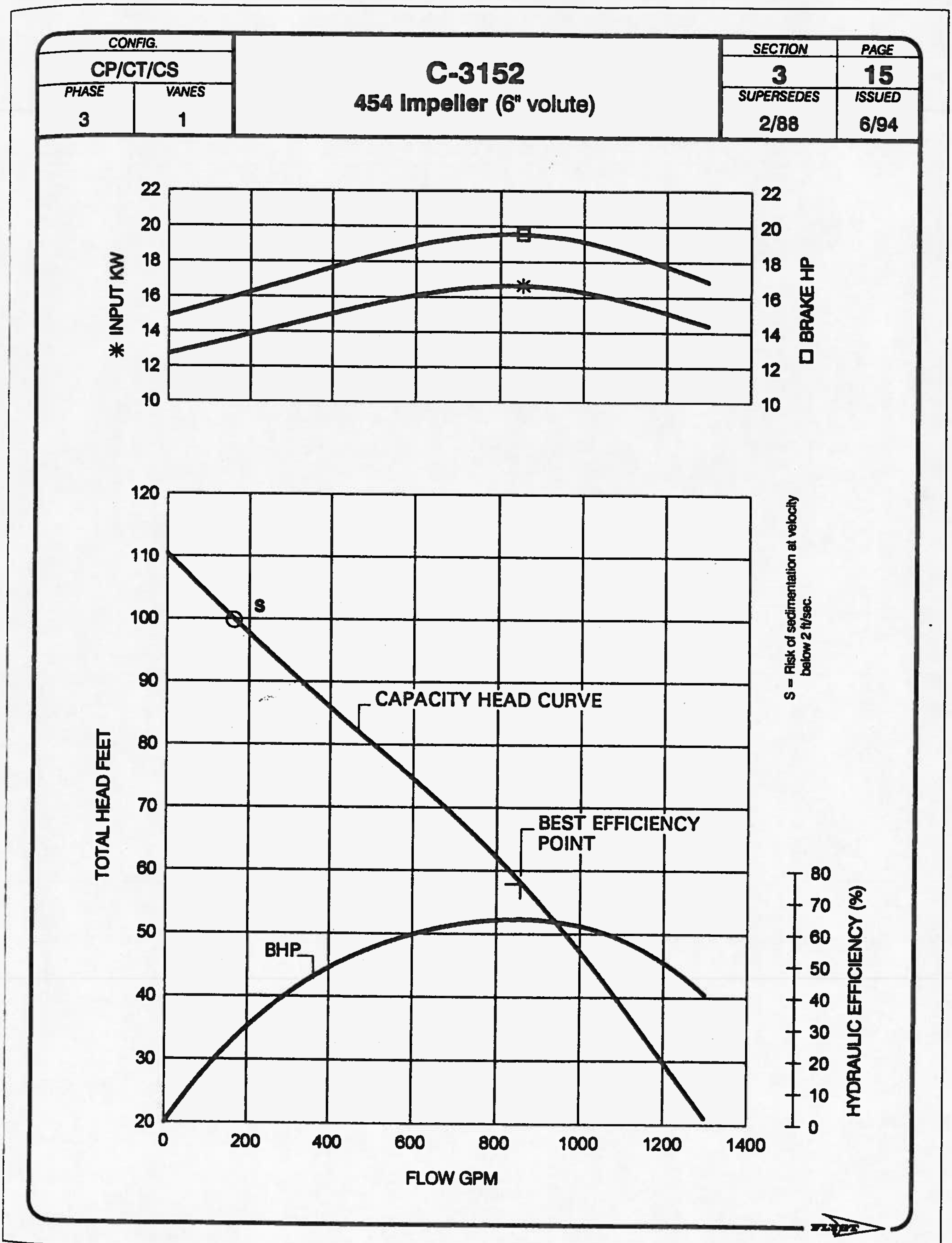


FIGURE 24.24 Pump performance curve. (Reprinted with permission from ITT FLYGT Corporation, 1993, Wastewater Pumps Catalog, Trumbull, CN)

CONFIG.		C-3602 1040 Impeller	SECTION	PAGE
CP/CT			3	3
PHASE	VANES		SUPERSEDES	ISSUED
3	4		12/91	6/94

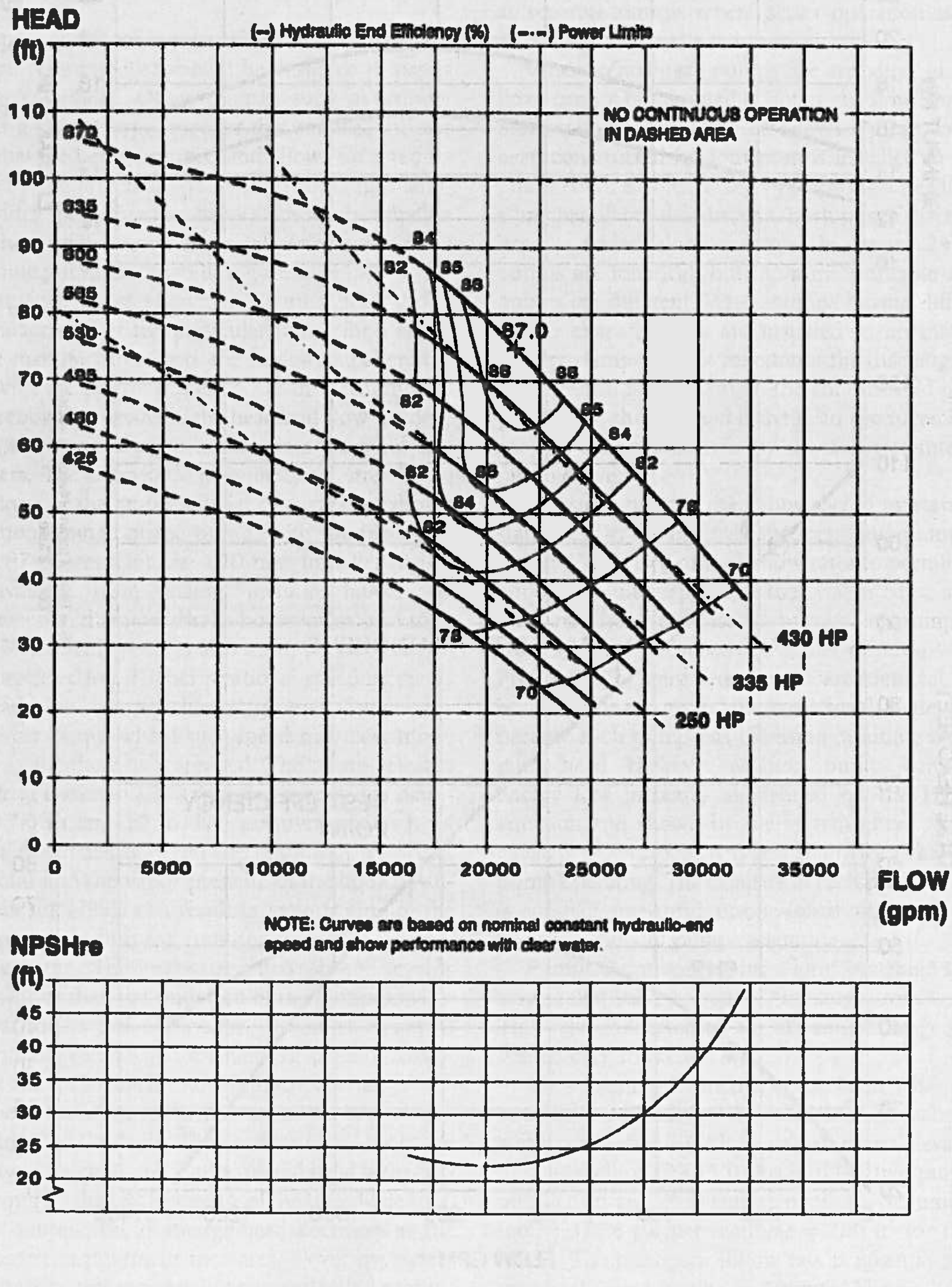


FIGURE 24.25 Centrifugal pump performance curve. (Reprinted with permission from ITT FLYGT Corporation, 1993, Wastewater Pumps Catalog, Trumbull, CN)

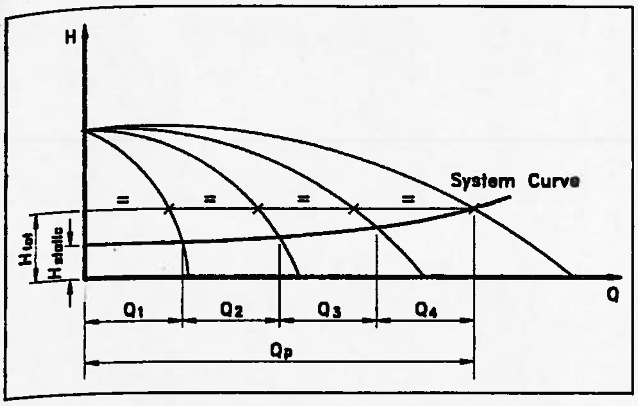


FIGURE 24.26 Performance curve for up to four identical pumps operating in parallel.

The wet well should be sized to provide at least a 4-minute run time at minimum flow. Therefore, the volume of the wet well is the pumping rate minus the inflow rate = (4 × 1405) − (4 × 50) = 5420 gal = 724.6 ft^3. The operating depth of storage in the wet well = 724.6 ft^3/100 ft^3 of volume per foot = 7.25 ft. Therefore the pump cutoff depth = 203.75 − 7.25 = 196.50 ft. These depths are shown in Figure 24.27. Note that the bottom of the manhole is the distance from the pump connection flange to the bottom of the pump mounting plate, which is lower than the pump stop level. This dimension is obtained from pump catalog information.

A suction lift exists when the hydraulic grade line drops below the center of the pump bowl. Suction lifts should be avoided with sewage pumps unless the installation is specifically designed to be self-priming. Total dynamic suction lift is the vertical distance in feet from the center line of the pump to the free liquid level in the wet well plus all energy losses in suction pipe and fittings.

The total dynamic head imparted by a pump is the difference in energy across the pump and is given by the E term in Equation 24.16:

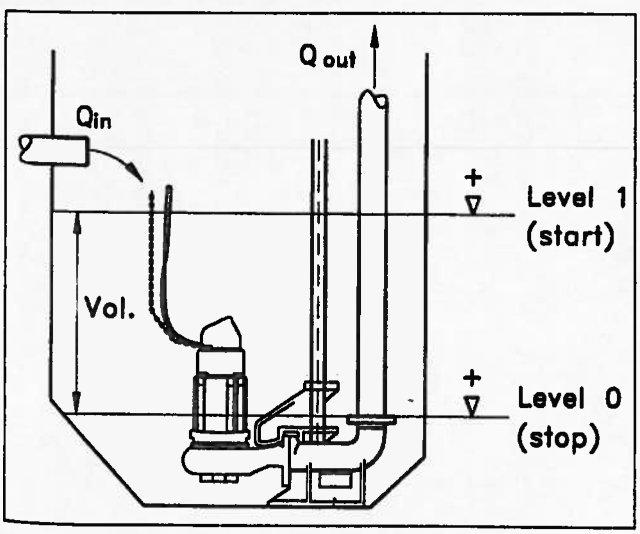


FIGURE 24.27 Schematic showing pump wet well elevations.

$$\frac{V^2_1}{2g} + P_1 + Z_1 + E_P = \frac{V^2_1}{2g} + P_2 + Z_2 + H_{1\text{-}2} \qquad (24.16)$$

Points 1 and 2 are located on the suction and discharge side of the pump, respectively. Point 1 may be the water surface level in the wet well and point 2 the free discharge point at the end of the forcemain. Points 1 and 2 must be the same for all parameters in Equation 24.16. V is the velocity, Z is the vertical distance from the same datum to the respective points, E is the energy imparted by the pump, and H is the energy loss between the two points. Each term in the equation has the dimension of feet.

Brake horsepower is the shaft input to the pump and is given by Equation 24.17:

$$\text{BHP} = \frac{\text{(gpm)(Head in Feet)}}{\text{(3.960)(Pump Efficiency)}} \qquad (24.17)$$

Brake horsepower is the work done by the pump expressed in foot-pounds per minute. Raising a gallon of water 10 ft represents 8.34 lb/gal × 10 ft = 83.4 ft-lbs of work. One horsepower is doing work at a rate of 33,000 ft-lb per minute. In Equation 24.17, 3960 is obtained by dividing 33,000 by 8.34. Pump efficiency is expressed as a decimal and not in percent (obtained from pump performance curves). Velocity head is usually ignored in pump selection because it is insignificant. For example, at a velocity of 5 fps, the velocity head (see Equation 24.5) = 25/64.4 = 0.39 ft.

The profile for the forcemain, planned for construction between the West Creek pumping station located at Point Y in Figure 24.19 and the existing gravity trunk sewer, is shown in Figure 24.28. The low water level in the station wet well is 196.50 ft. The invert of the forcemain at the discharge point is 308.15 ft (obtained from field survey or taken from as-built drawings available at utility), giving a static head of 111.65 ft.

The energy loss due to pipe roughness for pressure flow is generally determined by the Hazen-Williams equation. The terms in Equation 24.18 are: V = velocity in fps, C is the Hazen-Williams coefficient, R is the hydraulic radius = $D/4$ in feet for circular pipe, and S is the hydraulic gradient =

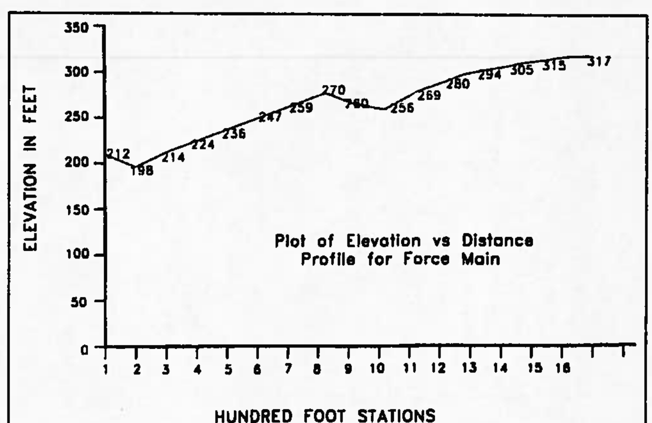


FIGURE 24.28 Profile of ground surface along forcemain.

head loss/length. Equation 24.19 is the continuity equation for noncompressible flow. Equations 24.20 and 24.21 are rearrangements of Equation 24.18. Note that the flow varies directly as C for a constant hydraulic gradient and pipe diameter. The technology of pipe manufacture is such that all pipe is near smooth wall. Most information supplied by pipe manufacturers indicates that a C value of 140 to 150 should be used in design; however, engineers generally use a value between 100 and 120. This allows for normal minor losses in the pipe and also accounts for a somewhat deteriorated pipe wall condition after the pipe has been in use for several years.

$$V = 1.318CR^{0.63}S^{0.54} \tag{24.18}$$

$$Q = AV \tag{24.19}$$

$$\frac{Q_1}{Q_2} = \frac{C_1}{C_2} \tag{24.20}$$

$$H_1 = \left[\frac{V}{1.318CR^{0.63}}\right]^{1.85} L \tag{24.21}$$

Various types of nomographs and slide rules are available for solving the Hazen-Williams equation. One type of nomograph is shown as Figure 24.29.

As determined previously, the static head for the pumping station is 111.65. The friction loss is based on 1600 ft of pipe plus the minor losses in the pumping station. The pumping rate is 1405 gpm or 3.13 cfs. Forcemains are generally designed for a velocity at peak flow of around 5 fps. From the continuity equation and Figure 24.29, it is found that a 12-in pipe at a velocity of 5.0 fps will have a capacity

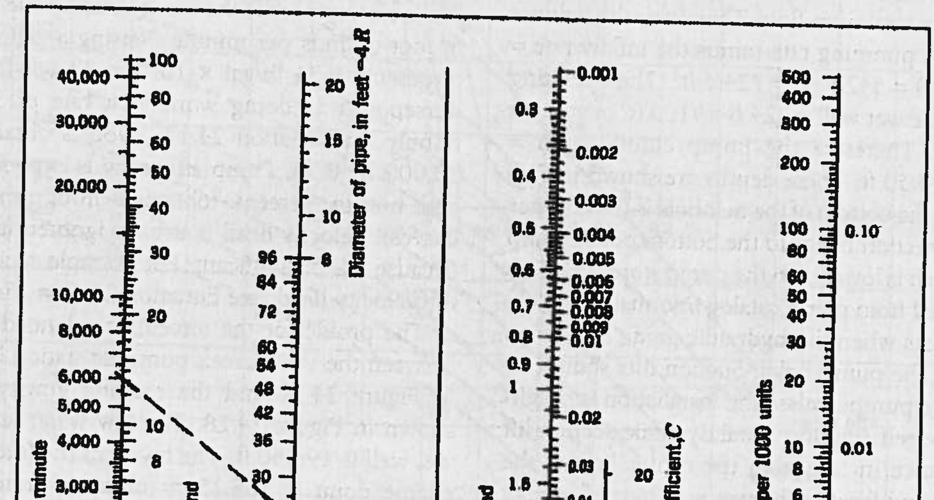


FIGURE 24.29 Alignment chart for Hazen-Williams formula for pipe flow (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York)

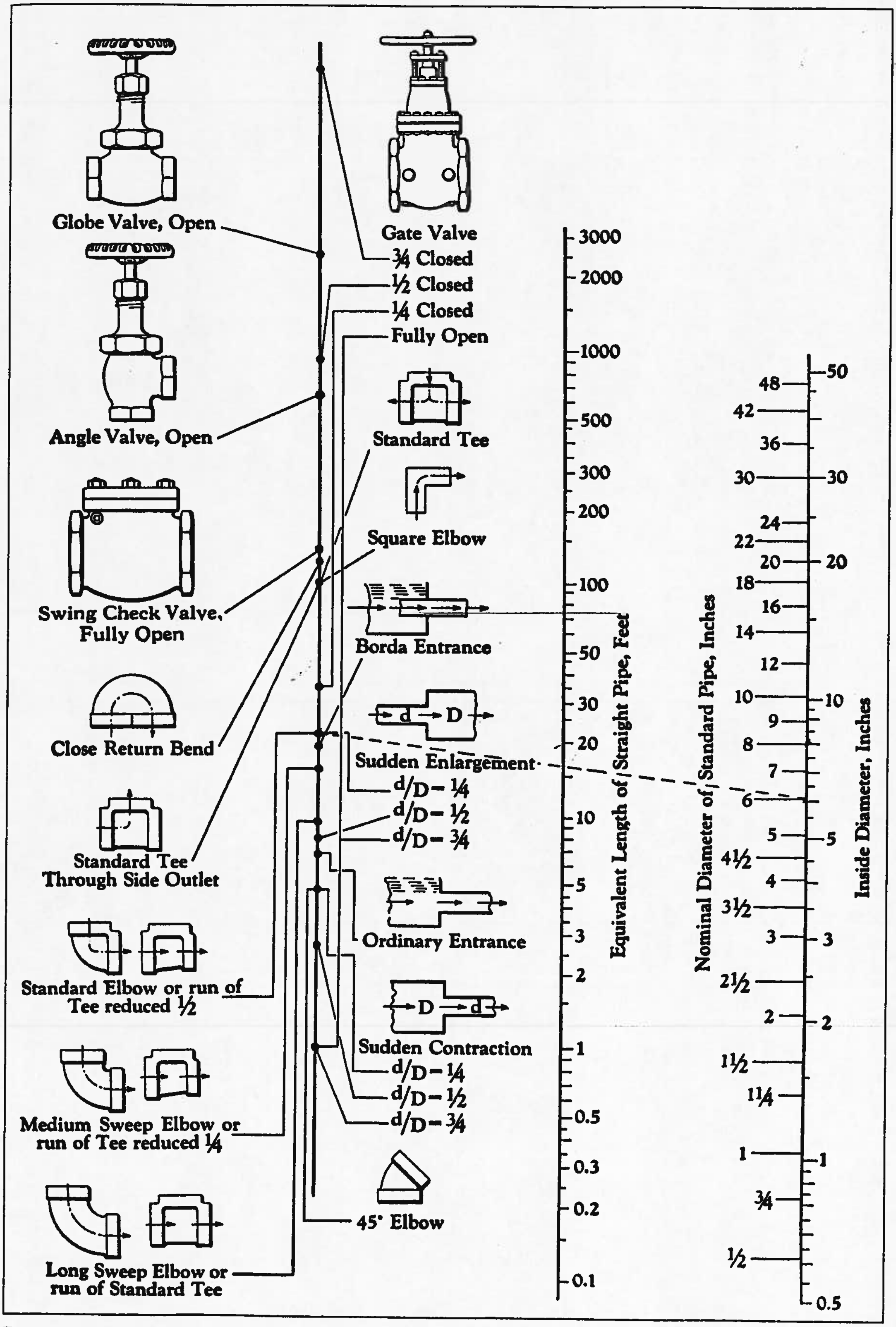


FIGURE 24.30 Equivalent lengths for minor losses. (Reprinted with permission from Crane Co., Stamford, CN)

of 1737 gpm and that a 10-in pipe requires a velocity of 5.7 fps to carry 1405 gpm. The 12-in pipe is selected for the forcemain because the friction loss will be less and because there is another area at station 10+00 that may use capacity in the forcemain at a later date.

Since the flow from the pumping station will be small for several years, it is decided to install less than the full pumping capability at this time. A forcemain should have a minimum velocity of 2 fps when the pumps are operating. Thus, the initial pumping capacity will be at least 710 gpm. Figure 24.29 shows a hydraulic gradient of 0.0016 when using a design C of 120 and a flow of 710 gpm in the 12-inch pipe. The hydraulic gradient at 1405 gpm is 0.0056.

Minor losses are losses in valves, bends, and segments of a conduit that produce turbulence over that for straight pipe. These losses are usually neglected in a pipeline, but they can become significant in a pumping station. One method of accounting for minor losses is by use of Figure 24.30. Knowing the source of the minor loss, the information from the figure can be used to determine an equivalent length of straight pipe. For example, a check valve is equivalent to 80 ft of 12-in pipe. The following minor losses apply to the West Creek pumping station: one check valve, one gate valve, two elbows, and one tee, giving equivalent lengths of 80 ft + 6 ft + 33 ft (2) + 70 ft and a total equivalent line length of 1600 + 222 = 1822 ft. At the hydraulic gradient of 0.0056 the initial friction head loss will be 10.2 ft, which is added to the static head and gives a pumping head of 111.65 + 10.2 = 122 ft. The pump selected, as shown by the performance curve in Figure 24.24, will deliver about 750 gpm at the required head. This pump will operate at about 53 percent efficiency and require a 40 Hp motor.

The pump selected will accept larger impellers and motors as shown by the catalog data. These can be changed when the need arises without having to replace the pump. The daily cost of pumping can be determined as follows:

BHP = (750 gpm)(122 ft)/(3960)(0.53)
= 43.6 (See Equation 24.15)
Electric Power = BHp/Efficiency of motor
= 43.6/0.92 = 47.4 Hp
Cost = (Hp) (0.746 kW/Hp) (Hours)[1] ($/kWH)
= (47.4) (0.746) (24) (0.40)[2] ($0.08)[3]
= $27.16/day

Other appurtenances to be included in a forcemain design include air relief valves and blowoff valves. Air relief valves must be placed at high points in the forcemain, as an accumulation of air will restrict the area of flow and result in a decreased carrying capacity of the line. It is seen from the line profile in Figure 24.28 that an air valve is needed at about station 8+00. An air relief valve for sewage must be used. The valve is placed in an enclosure such as a manhole. Blowoff valves are located at low points in the line to permit draining the line for maintenance. A blowoff valve is usually a gate valve or a plug valve. The profile of the main shows that a blowoff valve is needed at about station 10+00. Plug valves are generally preferred over gate valves because of the ease of operation. A plug valve only needs to be turned 90 degrees to open or close, whereas a gate valve requires much more effort and time to open or close.

Solid-state frequency controllers are a very reliable means of varying the speed of the pump motor. Variable-speed pumps should be considered for pumping stations, as their use minimizes surges in the forcemain from the constant-speed pump cycle. The variable-speed feature results in the pumping rate matching the incoming flow rate. Variable-speed pumping stations at sewage treatment plants greatly reduce surge waves through the plant.

DESIGN OF SANITARY SEWERS WITHIN SUBDIVISIONS

General Information

The previous section presents procedures for the design of a main or trunk sewer such as might be needed for sewering an entire sewer shed. This section presents the principal elements that are generally applicable to the layout and design of a sewage collection system to serve a subdivision or development.

The design of a sewage collection system for a subdivision generally requires fewer hydraulic calculations than is needed in the design of a main sewer, such as presented in Tables 24.13 and 24.15. There are many important aspects in the layout and design of any sewage collection system, and an experienced engineer should be involved in the process. A poorly conceived layout of a collection system will increase the construction costs and generally result in additional maintenance requirements. Care must also be taken to ensure that all lots can be properly sewered.

The hydraulics of sewers as presented, along with the data in Table 24.5, apply to all sewer designs. However, the calculations required in the design of collector sewers are generally much less complicated than what is required for larger sewers. As shown in Table 24.5, an 8-in sewer flowing full at a slope of 0.0042 ft/ft, and a roughness coefficient n of 0.013, has a capacity of 354 gpm (0.51 MGD). At this flow the peaking factor is 5.0 (see Figure 24.2), giving an average daily flow for design of 0.1 MGD (hydraulic design flow divided by peaking factor = average daily flow from source). Thus, an 8-in sewer will sewer 270 single-family houses (270 SF units times 370 gpd/unit = 99,900 gpd) or 333 town houses (see Table 24.2 for average daily flow per unit). Likewise, a 10-in sewer will serve 476 single-family units or 587 town houses

[1] Although 24 hours is used in the example, pump may operate fewer hours per day and this would be used in the equation period.
[2] Since the pumping rate is 2.5 times the average daily flow, the pump will operate only 40 percent of the time when the flow reaches the design rate. The flow and pump run time will be less until the sewer shed is developed to design density.
[3] Power costs are assumed as $0.08 per kilowatt-hour.

(peak factor = 4.5). A 12-in sewer will serve 700 single-family units (peak factor = 4.4) or 863 townhouses.

These sewer capacities are based on a minimum velocity of 2.25 fps. When a sewer is placed on a slope greater than required to provide the minimum velocity, the sewer capacity is increased in accordance with the Manning equation (see Figure 24.7). Generally, the sewer design for the normal subdivision does not require the calculations of flow and tabulation of data as shown for the larger-capacity main sewers by Tables 24.13 and 24.15. For subdivisions where the number of units is less than 500, the engineer is concerned with selecting the most appropriate sewer layout and ensuring that the sewer depth is sufficient to provide gravity drainage from the units served and that the sewer slope is equal to or greater than required to provide a velocity of 2.25 fps (slope greater than 0.0042 ft/ft). When designing the sewer collection system for single-family subdivisions greater in size than 500 units or town house developments greater than about 800, the design engineer should perform the design calculations as shown for large sewer mains. The upstream terminal segment of a sewer line should be placed on a 1 percent slope because this segment will serve only a few houses and the greater slope is needed to ensure that adequate scour is provided at the low flow. The minimum diameter for a public sewer is 8 in. Also, as noted previously, infiltration should essentially be zero in a properly constructed sewer. The unit flow data given in Table 24.2 includes an allowance for infiltration.

Feasibility Investigation

Often an engineering firm will become involved in a feasibility investigation. This investigation, generally conducted prior to the developer purchasing the property, is important in that it confirms the zoning of the parcel, the availability of an adequate water supply, and sewer service. The investigation should also discover any limitations to development, such as environmental or physical limitations.

Zoning for the parcel dictates several features of the development that impact the sewer layout. These features include street width, lot size, sidewalk and curb and gutter requirements, and building setbacks from property lines. This chapter is concerned only with items related to the sewer design. A more complete description of items associated with a feasibility investigation is included in Chapter 5.

The Preliminary Plan

An experienced engineer should direct the preliminary plan design. Many important aspects leading to the success of the development are related to the preliminary plan. Items such as the boundary shape of the parcel, topography, and the location of existing connecting roads and utility lines influence the location of streets and consequently the planimetric design of the sanitary sewer system. While it is preferable that utilities such as sewer, water, and gas be located within the public street for ease of maintenance, this may not be possible for all sewer lines because of topographic considerations. When the sewer is located within the street, it is generally located along the center line except for curvilinear street segments where the manholes are located so as to keep the sewer within the paved street area. Locating the sewer as near as feasible to the center of the street results in equal lengths of building spurs to the units on each side of the street. The sewer should never be located under the curb because access for maintenance is thereby limited. Most organized utility departments have regulations on the location of utility lines because uniformity in location and construction methods throughout the system aids in maintenance.

During preparation of the preliminary plan, the engineer's task is to select a sewer layout that will most effectively serve an area at the least cost while providing long-term reliable service with minimum maintenance. The principal items affecting construction costs are the length of sewer, sewer depth, and subsurface obstacles to construction—all of which must be investigated and confirmed as part of the preliminary design. Deep sewers are particularly expensive when construction is in rock. The principal items affecting maintenance costs include infiltration and stoppages. Both items are related to the quality of the material specified by the engineer and to the quality of the construction. A properly designed and constructed sewer should require little, if any, maintenance. Good engineering includes specifying the proper materials and construction requirements and providing competent inspection of the construction, the fine details of which are determined during development of the final plan.

Final Plan

The engineer should not proceed with the preparation of the final plans for a development until the preliminary plan review has been completed and all departments of the local government have approved the plan. In most instances, one department will act as the control department and distribute the plans to the necessary departments and all review comments will come back through the control department. While the sewage utility, whether it is the Department of Public Works or another authority, will review and comment on the sewer system proposed, the preliminary plan will be reviewed by a number of departments and agencies.

A public or common sewer is a sewer that serves two or more building units. A private ownership segment of sewer should never serve more than one customer. All segments of common or public sewers that are not constructed within public right-of-way should be within dedicated easements. A fifteen-foot-wide easement is desirable, with the sewer line located at its center. Easements should not straddle lot lines because homeowners often plant shrubbery or construct fences along property lines, making access to the sewer difficult. Easements should extend to the development boundary at locations where the sewer will need to be extended in the future to serve off-site property.

Proper separation must be maintained between water and sewer lines. Regulatory agencies generally require a minimum horizontal separation of 10 ft between the two conduits. Where field conditions preclude a 10-ft separation, some agencies will permit construction with a 5-ft horizontal plus an 18-in vertical separation with the water line higher. Special design requirements may be used where suitable separation cannot be maintained. Special conditions may include the use of ductile iron pipe for the sewers or other safeguards to ensure that the water lines do not become contaminated in case of line leakage or breakage. Sewer lines placed within 20 ft of buildings may require special construction, such as concrete encasement, to ensure that any leakage will not seep around the building foundation. Also, if a building is located near or over a sewer, it may not be possible to access the sewer for any required maintenance.

Sewer depth is established by the elevation of the buildings to be served. The sewer should be sufficiently deep to permit gravity drainage from all buildings if possible. For residential subdivisions, the sewer depth is based on the building construction. If the homes have basements, the sewer should be at sufficient depth to gravity sewer the basement. Basement floors will generally be about 6 ft below finished grade at the highest point next to the house. An additional 2 ft of depth is needed to allow for the basement foundation, as the house sewer should be located below the wall foundation and not through the foundation. The building spur (sometimes referred to as the house lateral or house sewer) should be placed on a minimum slope of ¼ in per foot or 2 ft per 100 ft (slope = 0.02 ft/ft). Plumbing codes allow a slope of 1 ft per 100 ft (slope = 0.01 ft/ft) for sewers under conditions where the greater slope cannot be conveniently provided. Thus, the invert of the house spur at the connecting point to the public sewer, where the public sewer is located in the center of a 50-ft street and the setback for the house is 50 ft, is 9.5 ft lower than the highest point of the finished grade at the house wall (from above 6 ft + 2 ft = 8 ft at outside of building foundation plus 1 ft drop in elevation in the 50-ft distance between the house and front property line plus ½-ft drop in elevation in the 25-ft distance from the property line to center of street). The invert of the public sewer is the sewer diameter plus 1 ft to allow for the wye connection, lower than that of the incoming house spur (see Figure 24.18 for typical spur connection to public sewer). Thus, a total vertical drop of about 12 ft between the highest proposed grade at the house wall and the invert of the public sewer is needed to ensure gravity drainage when the house is constructed.

In relatively level terrain, there will be a drop across the lawn of about 2 ft between the front building wall and the street thats results in a sewer about 8 ft to 10 ft lower than the street. Sewers should not be constructed deeper than 20 ft except where alternatives are not available because of the difficulty in maintenance of deep sewers. Topographic conditions may make it necessary to locate sewer lines along the back of some lots in order to provide gravity drainage at a reasonable sewer depth.

Since, as noted previously, design calculations are seldom required for sewers within a subdivision, sewer slopes are shown on the profile for each segment of sewer. The slope is constant between manholes.

The design engineer should follow certain rules in selecting sewer slope. The slope for each segment of sewer running between adjacent manholes is selected on the basis of design conditions. The overall design objective is to design the collection system that can be constructed most economically while maintaining good engineering practice. This generally means keeping the sewer as shallow as possible while complying with the following criteria:

1. Minimum sewer slope—the slope required to provide a velocity of 2.25 fps, or the slope required to carry the required flow, whichever is greatest.
2. Minimum cover
   a. Sewers located in streets should have at least 6 ft of cover. If less than 6 ft of cover is provided, special bedding may be needed because of the superimposed load from traffic.
   b. Sewers located within off-street easements should be placed below the freeze depth or at least to a depth of 3 ft.
   c. Sewer segments serving a building lateral should be at sufficient depth to provide gravity drainage from the building served.

Figure 24.31 shows a portion of a final plan. The planimetric sewer layout is highlighted on this drawing. The final grading contours are shown along with the storm drainage. The manholes have been numbered and the locations of the house laterals are shown.

Plan and Profile

A plan and profile is prepared showing all utilities including the sewers. If the sewer is located within a street, the lines are a part of the street plan and profile. All existing and proposed underground utilities should be shown on these drawings. If underground utilities are not shown, the construction contractor is likely to damage them, leading to additional costs.

A plan and profile sheet is used for presenting this design. The plan view is shown on the top half of the sheet and the profile is shown directly below on the bottom half of the sheet. The plan and profile view for the final plan of Figure 24.31 is shown in Figure 24.32. The scale of the original drawing is 1 in = 50 ft. In the profile view the horizontal scale of the profile is the same as that of the plan view. The vertical scale is 1 in = 5 ft. The larger scale is used to show the detail needed for construction. The drawing is

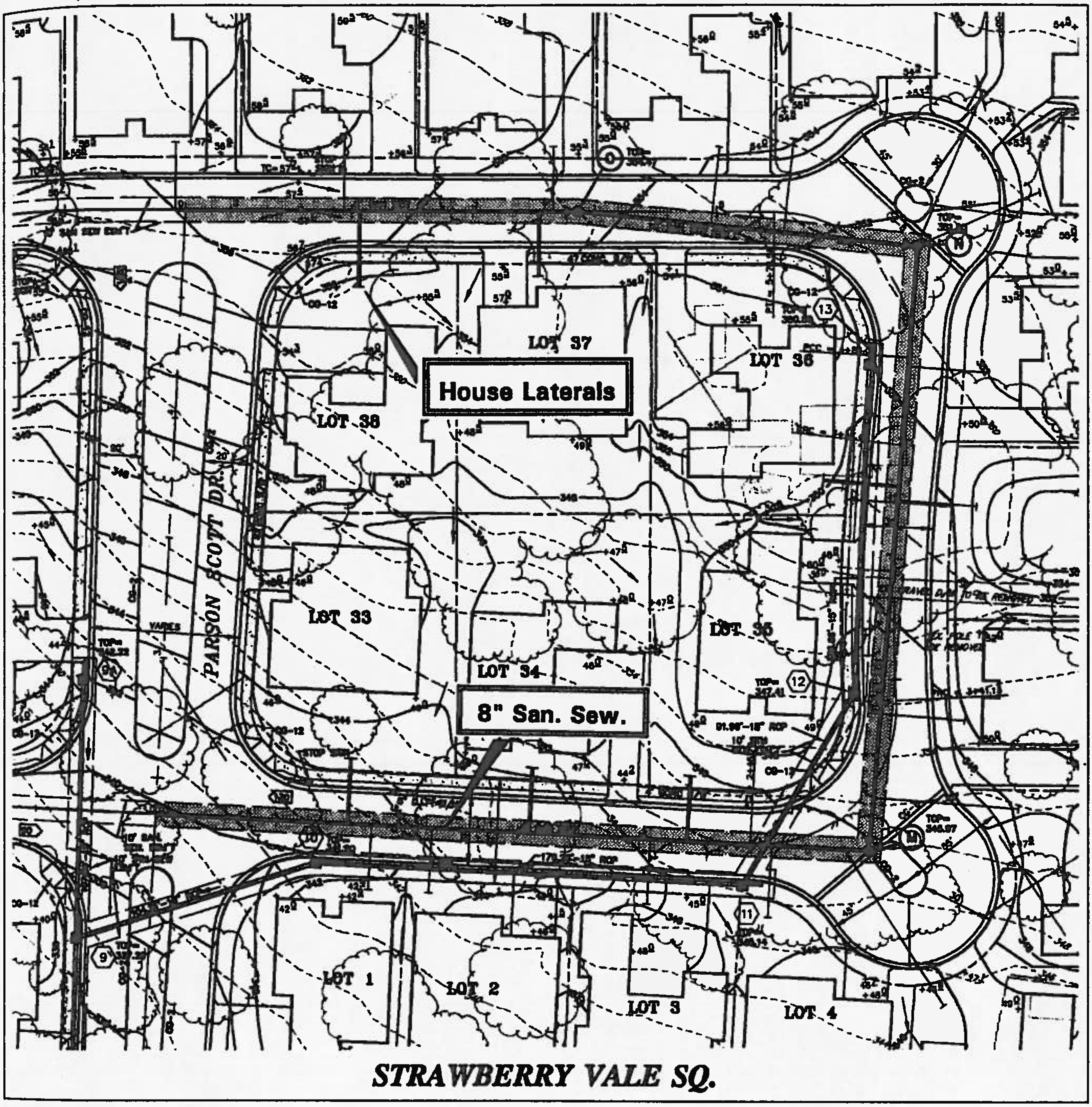


FIGURE 24.31 Final plan.

carefully constructed, as the contractor may need to scale some dimensions from the drawing during the project construction.

Corresponding manhole numbers are shown on both the plan and profile views. The building lateral locations are shown on the plan view, but note that the stationing and elevation of each building lateral at the lot line are also shown on the profile view. The contractor must construct the sewer as shown. Sewers located in a street are constructed to just inside the property line and marked at the surface with a stake so that the house plumber can access the sewer without disturbing the street. He keeps a set of plans in the field that are kept current with field conditions being noted as the construction progresses. This field set of plans is used to construct as-built drawings that become record drawings. The utility crews can then use these drawings for maintaining the sewers in future years.

The manhole stationing along with the sewer slopes and diameter is shown for each segment of sewer. Note that the sewer grades are considerably greater than the 0.0042 ft/ft required to provide the minimum velocity of 2.25 fps. The water line is placed to provide 4 ft of cover, whereas the sew-

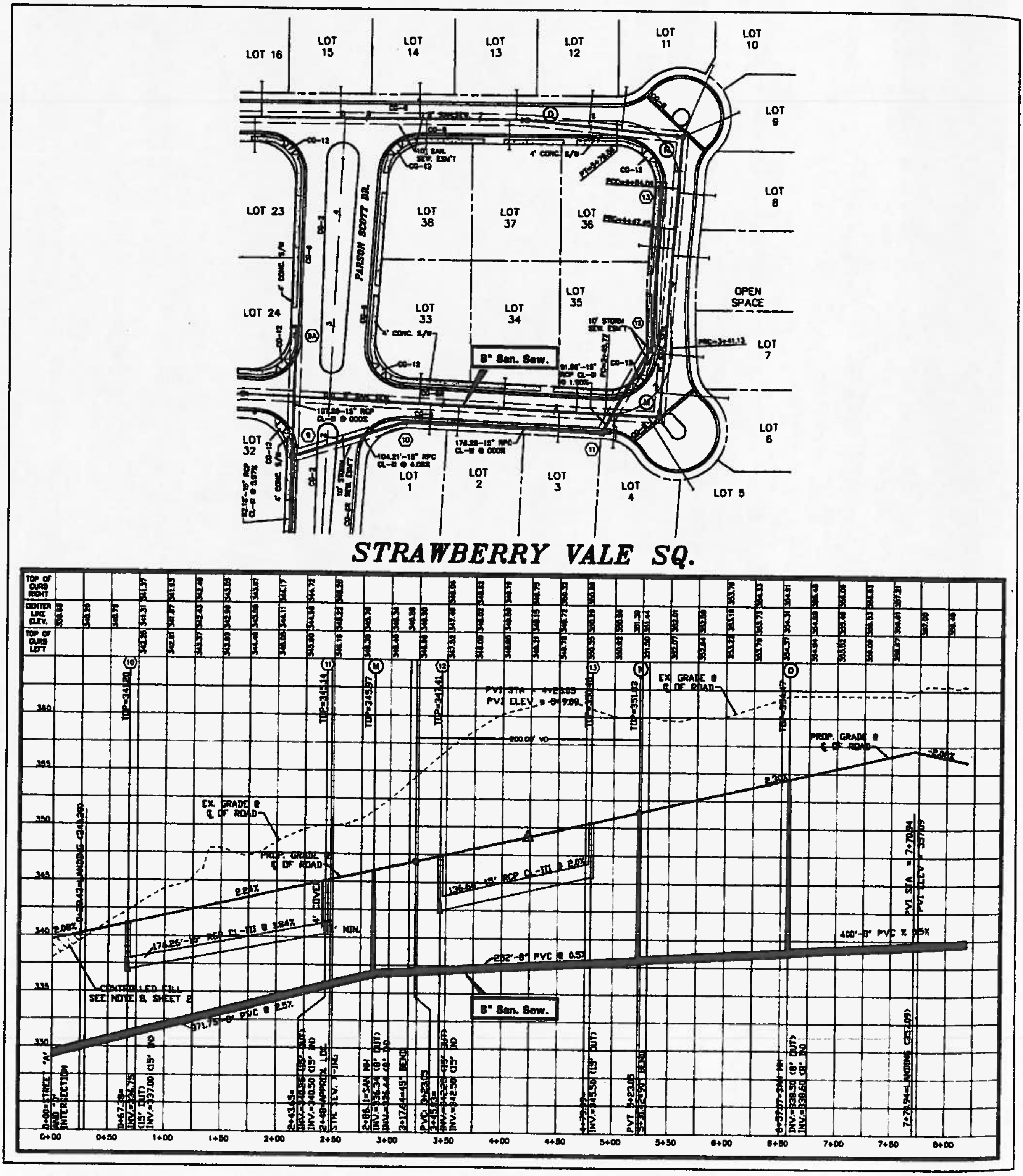


FIGURE 24.32 Example of a plan and profile view for a sanitary sewer line.

ers are required to be much deeper in order to provide gravity drainage from the buildings. All information needed by the contractor to construct the sewer, roads, and other utilities is shown on the plan and profile. The engineer must be thorough in ensuring that all underground conditions are shown on the final plan and profile. Existing underground utility location can be obtained from as-built drawings and supplemented through field surveys and test pits. Specifications are prepared to establish the allowed type of pipe material, the type of bedding required, and the acceptable infiltration. This information can also be included on the plans as notes.

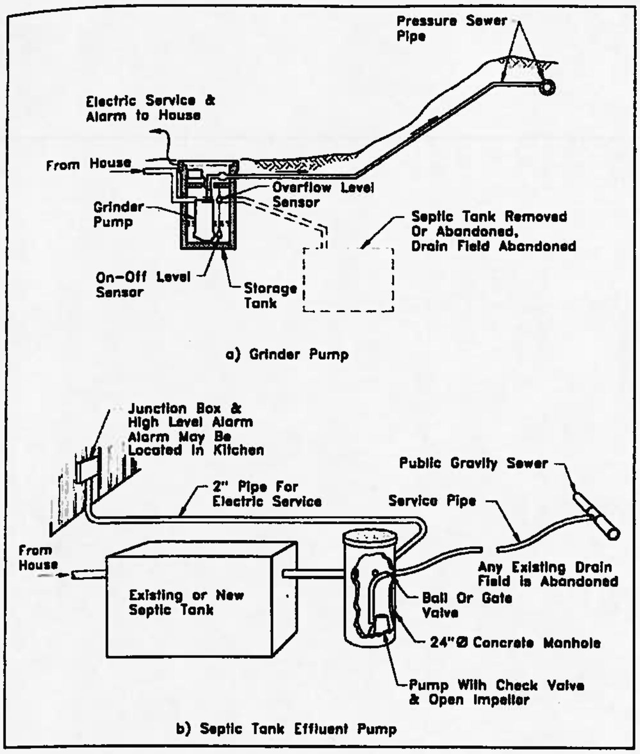


FIGURE 24.33 Flow diagram for pressure pumping systems.

PRESSURE SEWERS

A pressure sewer collection system is used to reduce costs relative to the cost of a conventional gravity system. The technology may be the only feasible means of sewering some areas. Pressure sewers are particularly applicable for sewering less populated areas, and developments or communities located in hilly or rocky terrain. Conventional gravity sewers may need to be deep and costly to construct in areas where the topography is undulating. Also, a high water table may make the construction of gravity sewers economically unfeasible. Pressure sewers also have merit other than lower construction cost. Reliable pumping equipment is available. The technology requires no modification to the house plumbing and, therefore, the use of pressure systems causes little inconvenience to the homeowner.

The two major types of pressure sewer systems are the grinder pump (GP) system and the septic tank effluent pressure (STEP) system. Figure 24.33 shows the basic elements of both systems. A typical grinder pump is shown in Figure 24.34. The GP system consists of a grinder pump that receives the flow from a dwelling or other activity and pumps the flow into a pressure forcemain. The GP system may also be installed to replace an existing septic tank system, as shown in Figure 24.33. The STEP system follows a conventional septic tank where the flow is pumped into a pressure forcemain.

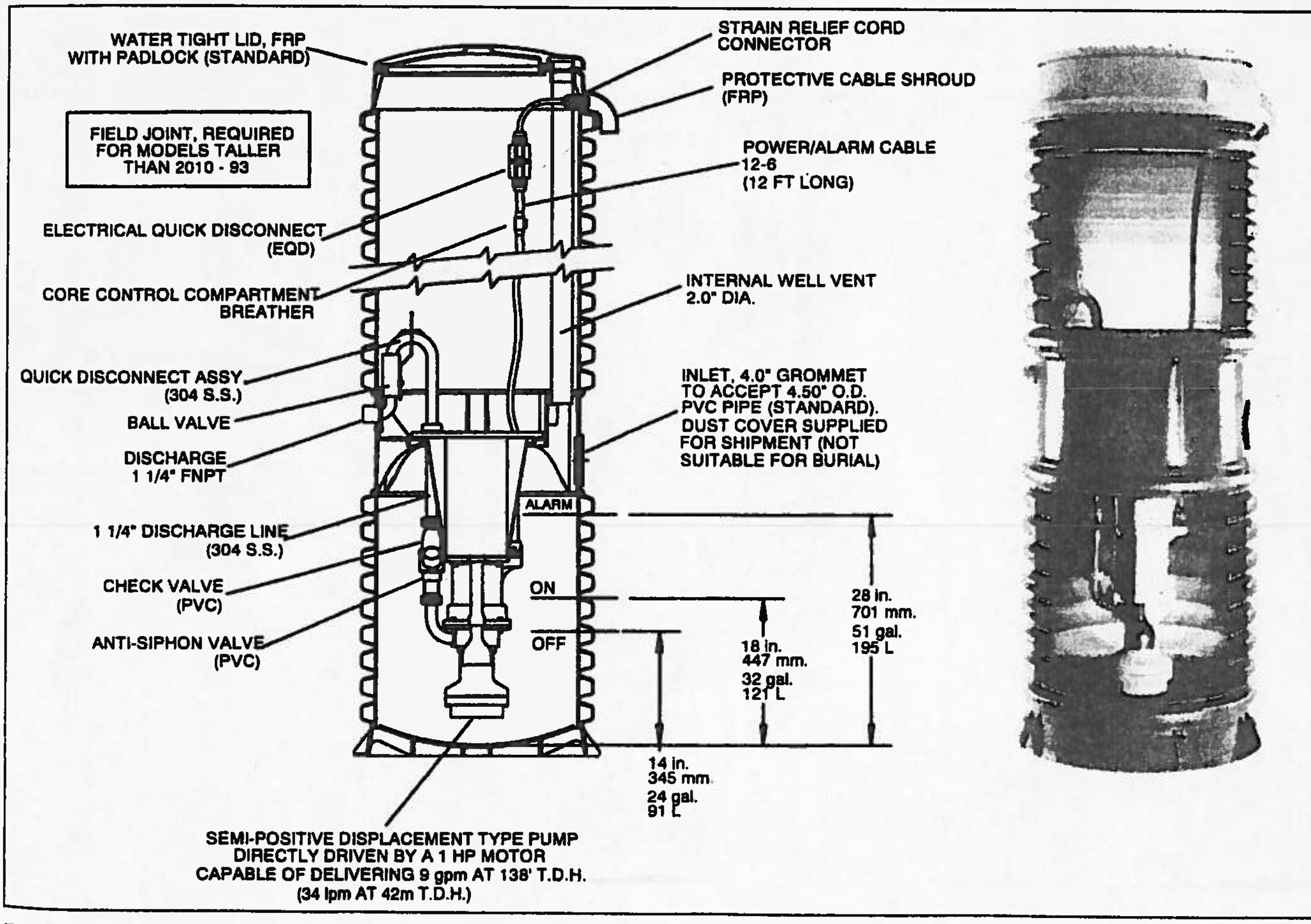


FIGURE 24.34 Typical grinder pump complete with housing. (Reprinted with permission from Environmental One Corp. 1994. Grinder Environment One product catalog, Schenectady, NY)

Pumping equipment for the GP technology is well developed. The pump shown in Figure 24.34 is available in several sizes and capacities to serve uses from a single-family dwelling to a small commercial or industrial flow. The unit is supplied in a prepiped and wired fiberglass enclosure that can be installed in a basement, crawl space, or below ground level in a lawn. The units are available with one or two pumps for reliability. The unit is installed and the 4-inch building sewer connected. Pumps are available for pumping against discharge heads of more than 100 ft. Pressure systems are designed similarly to forcemains, as discussed in the section on pumping stations, except for a determination of the number of pumps operating on the main at any one time and the resulting flow. GP suppliers have developed this type of statistical data, and it can be found in the respective catalog.

The normal wet well size for a single-family dwelling is 60 gallons (pump enclosure provides wet well volume). This constitutes some wastewater storage availability during periods of power outages. The units are wired to permit placing a high-water alarm at an appropriate location in the home, usually in the kitchen, so that the homeowner is made aware that the pump is not operating and that the wet well is full. The pumping units are designed for easy removal of the entire unit in case of pump failure. The pumps should give 10 or more years of service under normal conditions. It is desirable for the community with a GP system to have service provided by a utility or by a private plumbing company that will keep spare pumps in stock for rapid installation. The nonfunctioning pump is then taken to the shop for repair or replacement and kept in stock for the next replacement need. If rapid service is not available, the homeowner should install the duplex unit so that a backup pump is always available, allowing time to have the nonfunctioning unit repaired.

The septic tank effluent pressure system (STEP) is a means of eliminating the need for on-site treatment and disposal, such as the soil absorption field. In past years, septic tanks' absorption fields have been installed at locations that are no longer environmentally acceptable. Criteria for siting absorption fields have improved as more knowledge on soil percolation and potential for ground water pollution has become available. The STEP systems are being installed to eliminate failing absorption fields and as a means of providing central sewerage service to both existing and new communities. The septic tank located ahead of the pump eliminates most of the grease and solids in the flow to be pumped. The forcemain design is similar to that of the GP system except for the type of pump required. The homeowner must continue to maintain a septic tank as a part of the STEP design.

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