CHAPTER 6
SITE UTILITY SYSTEMS

This chapter will describe various pressurized and gravity flow utility and service systems that are found outside of buildings on the site. These also include services that extend from buildings to points of connection or disposal. A common connection point for site work is considered to be 5 ft from the building wall.

Site utilities discussed here include subsurface drainage, storm water drainage and retention methods, sanitary drainage, water supply, and industrial and laboratory site drainage systems. Also included are sections that provide information and fundamentals of hydrology and design of buried piping.

BASIC GEOLOGY AND HYDROLOGY

Because of the close relationship of geology and hydrology to site utility work, it is useful to have an elementary knowledge of the hydrological cycle and underlying geological formations.

HYDROLOGIC CYCLE

The continuous circulation of water through surface water, the atmosphere, and the land is called the *hydrologic cycle.* Inflow to the hydrologic system arrives as precipitation (rain or snowmelt) on the surface. Three things may then happen to this water. It may be pulled into the soil surface by capillary action and evaporated back into the atmosphere, be absorbed by plant roots and evaporated back into the atmosphere by transpiration through leaves and roots, or infiltrate down through the soil until it reaches the groundwater table. The hydrologic cycle is schematically illustrated in Fig. 6.1.

From surface sources, including rainfall, streams, rivers, and lakes, water infiltrates down into the soil. Basic geological formations are illustrated in Fig. 6.2. By infiltration, capillary action, and percolation, some portion of that water eventually reaches the groundwater table. Water in the soil between the surface of the ground and the water table is called subsurface water. The slow movement of subsurface water through the ground places this water in direct contact with soluble minerals that make up Earth's crust. The water, therefore, may have a wide variation in its chemical character even within small geographic regions.

In some areas, water may accumulate in a local zone of saturation above an impervious stratum and be prevented from reaching the level of the water table. This is called *perched water,* and its free surface is called the *perched water table.*

The term *groundwater* is classically defined by geologists as water found below the water table or in a geological formation that is fully saturated. *Subsurface water* is defined as near surface water that infiltrates the soil but is not absorbed into the

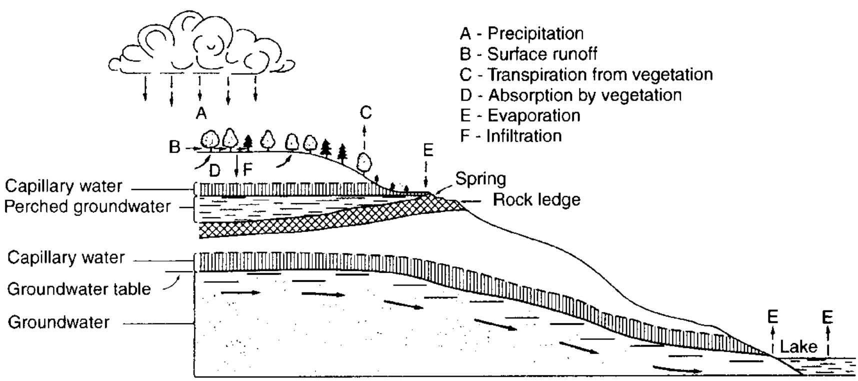


FIGURE 6.1 The basic hydrologic cycle. (*Courtesy of U.S. Army Corps of Engineers.*)

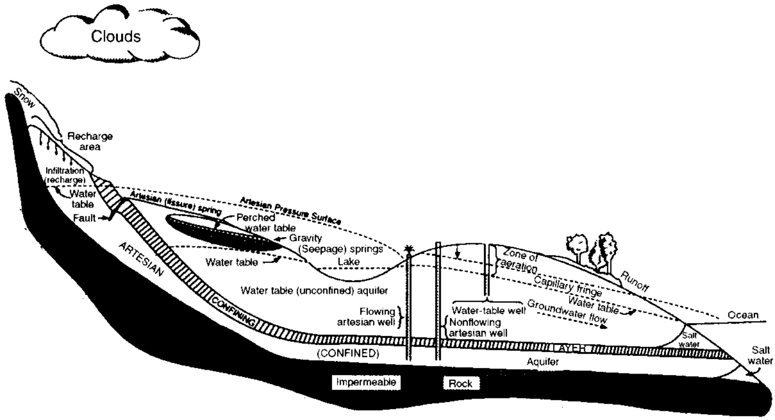


FIGURE 6.2 Basic geological formations.

ground at a lower level. It is primarily subsurface water that requires draining from excavations.

Groundwater and subsurface water are part of the hydrologic cycle and have two important applications. The first is using groundwater to obtain a supply of domestic water. The second is the necessity of collecting and removing subsurface water when required.

The *lithosphere* is the geological term for Earth's crust. All earth materials are known collectively as rock. *Consolidated* rock is the common hard rock such as granite or limestone, which will be called simply rock in this book. It is classified by origin into three main groups: igneous, sedimentary, and metamorphic. Unconsolidated rock is made up of small particles created from rock and composed of soils such as gravel, clay, and sand. It is classified into groups described in Table 6.20.

Water falling on the ground either collects on or near the surface or infiltrates downward into the subsurface. Subsurface water exists in pores and occurs in two distinct zones. The upper zone is called the *unsaturated zone,* with pores that contain both air and water. It is also called the zone of aeration. In the lower one, called the *saturated zone,* all the pore spaces are completely filled with water.

The unsaturated zone is divided into different layers. The porous, upper part of the unsaturated zone is called the *soil water belt.* This belt, composed mostly of soil, is a layer that extends down from the ground surface at least to the bottom of major plant roots, and often, much lower. This layer is where water is held in suspension by capillary action against the force of gravity. Below the soil water belt is an area called the vadose zone, which is the lower part of the unsaturated zone. This is the area where water under hydrostatic pressure that cannot be held in the belt by gravity moves downward into the saturated zone.

The saturated zone is the region where all the pores are completely filled with water. The top of the saturated zone is called the *water table.* Water found in the saturated zone is called groundwater and is the source of drinking water obtained from wells. Groundwater is replenished, or recharged, from precipitation that infiltrates down into the saturated zone. Groundwater is discharged through wells.

AQUIFERS

An *aquifer* is a water-bearing formation or stratum capable of storing or transmitting water in sufficient quantities to permit development for a specific purpose. To qualify as an aquifer, the formation must have pores or open spaces large enough for water to move in reasonable quantities. Where pores are so small that water cannot easily move through them, for example, in clay, the formation is not considered an aquifer even though it is saturated.

Aquifer Classifications

Aquifers can be classified by the nature of the rock that the water is stored in. *Unconsolidated* aquifers consist of a high percentage of permeable granular material. This material is often referred to as alluvial since it is usually sediment deposited by running water. This type of aquifer produces the greatest amount of water and is recharged directly from precipitation or streams.

The *semiconsolidated* aquifer is similar to the unconsolidated type except for weak bonding materials, usually calcium carbonate and iron oxide, that fill a portion of the pore spaces. These materials tend to be geologically older than unconsolidated materials and are usually found at a greater depth.

Consolidated aquifers consist of different types of solid rock that have some porosity, with the available water depending on the number of fractures present in the layer.

Another possible classification of aquifers is based on the intended use of the water. These classifications, or similar ones, are determined by local and state agencies. The general classifications are

1. Class 1: groundwater of special ecological significance
2. Class 2: groundwater for potable water supply
3. Class 3: groundwater for uses other than potable water

Aquifer Categories

Aquifers are often categorized by the conditions in which water is stored and whether a free water surface (or water table) exists under atmospheric conditions. *Unconfined* aquifers occur in the unconsolidated layer and have water and air filling the pores, and the top of the layer is at atmospheric conditions. The top of the layer is the water table, which fluctuates with the amount of water present.

When an aquifer is found between impermeable layers, the water is confined in the same way as it would be inside a pipe. Because of the confining strata, the aquifer is under pressure greater than atmospheric. This aquifer is called an *aquiclude,* but is better known by the more popular terms *confined* aquifer or *artesian* aquifer. Hydrostatic pressure within an artesian aquifer is sometimes high enough to cause water in a well to rise above the surface level of the ground above it.

When a confining bed is located beneath an aquifer, it prevents water from continuing its downward movement. The limited amount of water that accumulates above this confining bed is known as a *perched aquifer.*

SUBSURFACE DRAINAGE

This section discusses the drainage aspects of subsurface water entering excavations and the drainage requirements resulting from intrusion of subsurface water into footing drains.

When water in any form interferes with the construction of a project or could potentially cause damage to any structure or installation placed in the ground, it becomes a problem rather than a resource and must be removed. Problems occur when the presence of water could cause a structure to float or when groundwater could cause soil-bearing resistance to be lost.

Subsurface water is removed by placing a drainage system below the level required to be kept dry. In many cases, this drainage system will remove excess water by gravity. Two of the most common systems make use of trenches filled with pervious backfill and drain pipes. The drain pipes may have holes in them or may be installed with open joints to allow water to enter, and are pitched to provide a flow path. A special layer of pervious backfill is placed above the trench bottom or drain pipes to allow subsurface water to flow easily through them. This pervious backfill is often called *filter material.*

There are three general methods of draining subsurface water, categorized according to their purpose. The first is *subbase drainage,* which is generally used for draining road subbases. This system consists of shallow drain pipes laid near and parallel to the edge or end of the pavement to be drained. The second is called *subgrade drainage,* which is generally used when it is necessary to lower a high water table from around and inside the project area. It consists of either trenches filled with filter material or pipes placed to collect and route the water away. The third is called an *interceptor drainage* system, which consists of pipes or trenches placed in a manner that will intercept the groundwater before it gets to the project site.

When none of the gravity drainage methods are possible, a pumped system is required. The most common method for draining excavations of a project under construction uses well points, which serve the same purpose as intakes for a well. A *well point* is a reinforced, pointed metal cylinder with holes in it for water to enter. The points are driven into the ground below the level of the excavation and connected to the surface with a discharge pipe. The top of the pipes are connected by a header to the suction side of a pump, which collects the groundwater and discharges it to an approved location. The groundwater level is brought below the bottom of the excavation, keeping it dry. A typical well-point system is illustrated in Fig. 6.3.

INFLOW INTO EXCAVATIONS

The development of predictive methods for the inflow of water into excavations or from foundation drains has not kept up with the solutions to other problems in groundwater hydrology. It is safe to say that soil exploration must be done before a structure is built on any site. Analysis of the borings will establish if the site has groundwater that must be removed. In addition, after the excavation has been started, the amount of water drained from well points (if the site is a wet one) is the most accurate indication of the quantity of water that must be drained after the

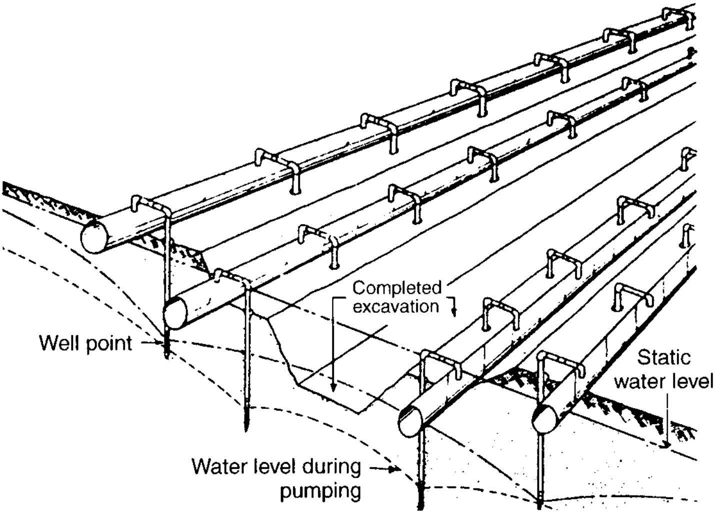


FIGURE 6.3 Typical well-point system.

foundation is in place. In addition to this information, the engineer must research the prior weather conditions to determine if there has been a dry spell or drought that might have resulted in a smaller amount of groundwater present. Allowance must also be made for additional water that could result from storms creating a higher groundwater level.

In the absence of such information, the following is a method that is useful in estimating the probable water quantity discharged from a subsurface drainage pipe. This method was established by the U.S. Army Corps of Engineers. The formula is:

$$Q = \frac{KHC}{60} \tag{6.1}$$

where K = horizontal permeability of soil, ft/min (This is a measure of how fast the water will flow through any type of soil. Refer to Table 6.1 and Fig. 6.4 depending on whether the material is backfill or undisturbed soil.)

H = difference in elevation between the center of the pipe and the ground surface, L distance from the drain (An example is illustrated in Fig. 6.5.)

C = shape factor dependent upon L and H. L is the distance to the edge of the excavation around a building. (Use $L = 50$ when K is greater than 10^3 ft/min. Refer to Fig. 6.6 to find the shape factor.)

Q = discharge quantity in cubic feet of water per second (cfs) for each foot of subsurface drainage pipe

TABLE 6.1 Permeability of Backfill

The following tabulation may be used for preliminary estimates of average coefficients of permeability for remolded samples of sand and gravel bases:

Coefficient of permeability for remolded samples,* fpm	
Percent by weight passing 200-mesh sieve	Coefficient of permeability for remolded samples,* fpm
3	10^1
5	10^2
10	10^3
15	10^4
25	10^5

*The coefficient of permeability of crushed rock and slag, each without many fines, is generally greater than one foot per minute.

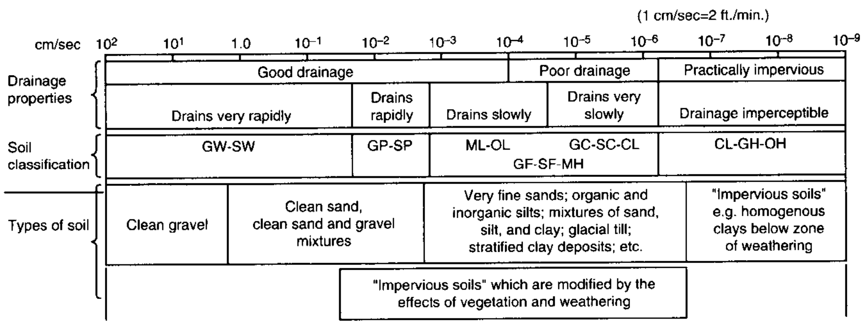


FIGURE 6.4 Permeability of soils.

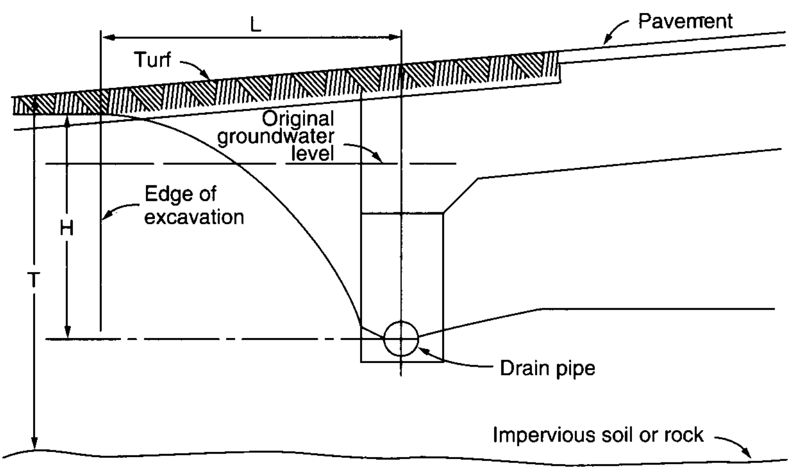


FIGURE 6.5 Example diagram.

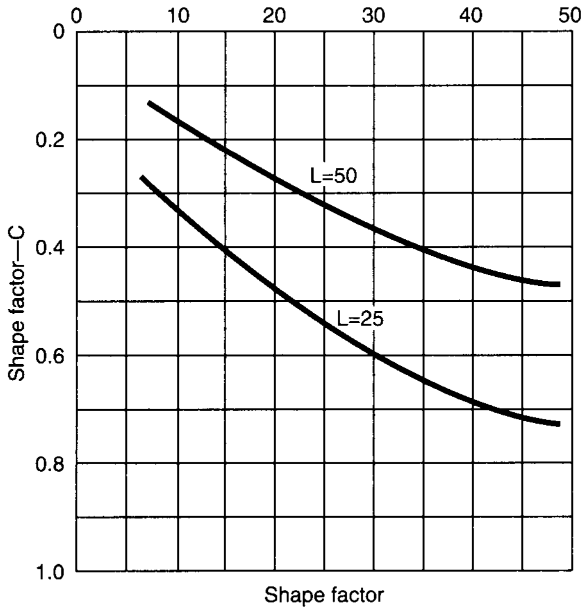


FIGURE 6.6 Shape factor.

The following example will illustrate the use of Eq. (6.1):

$K = 10^{-4}$ fpm
$H = 6$ ft
$C = 0.53$ (from Fig. 6.5)
$L = 25$ ft (This is the dimension by examining the borings from grade to the impervious layer under the water table.)

Therefore:

$$Q = \frac{1}{60} \times 0.0001 \times 6 \times 0.53 = 0.0000053 \text{ cfs/ft of pipe}$$

To this figure, add 25 percent additional flow if the height of a wet period occurred in the immediate past and 50 percent additional flow if the height of the dry season is occurring at present. These figures are the author's estimate for a reasonable allowance. It is important to check with local building officials for confirmation based on their experience.

There is always the possibility of groundwater being polluted. Before groundwater is discharged into the public sewer, samples of the water should be taken. Acceptable sampling and testing procedures must be used. The key terms used are sampling for *priority pollutants* and *contaminants of concern.* After the tests are run, a full assessment by the local, state, and federal authorities must be made to determine both the extent of any contamination and the ultimate method of disposal.

Because of sand present in discharge water coming from subsurface drainage piping, a sand interceptor is usually required by plumbing codes. The sand interceptor should be placed in the drainage line before it discharges into a building sump or ejector pit for disposal. This will avoid the necessity of installing the sand interceptor at the public sewer, where discharge of sand is prohibited. There are several commercially available sand interceptors, or one can be built on site.

The drainage lines should be sized using standard flowcharts, such as nomographs using the Manning, Kutter, or Hazen-Williams formula, with a roughness coefficient corresponding to the type of pipe used in the system and the pitch of the drainage pipe.

SURFACE DRAINAGE

This section is intended as a guide for development of a storm water drainage system for smaller uncomplicated sites of less than 2 square miles. Procedures follow established engineering principles and allow a simplified, conservative method of design and piping layout for small watersheds.

Included is rain water removal from parking lots, roadways, and undeveloped areas of the site. The selection and location of inlets, catch basins, manholes, and piping along with storm sewer design must be accomplished. The design of storm water removal, for example, from building roofs, is discussed in Chap. 9, in the section entitled Interior Storm Water Drainage.

When systems are located on larger sites or complications are encountered, it may be necessary to request the help of a consultant specializing in this type of work to ensure that the design is adequate.

PRELIMINARY INVESTIGATIONS

The following investigations must be concluded before designing the proposed system:

1. Determination of the site storm water discharge location or method of disposal, such as public sewers, waterways, surface absorption, recharge basins, or drywells.

2. If streams or rivers are to be used as a final outfall, local, state, or federal agencies must be contacted regarding permitted flow, outfall structures, and required permits. Such agencies may be the U.S. Army Corp of Engineers, Environmental Protection Agency (EPA), or local sewer department. EPA permits are required if such a waterway is considered navigable.

3. If a public sewer is to be the final outfall, public agencies must be contacted regarding permitted flow, connection details, plan submittal, and required approvals.

4. High and low water levels of all streams and rivers must be known, including past history of floods.

5. If site storm water will be discharged directly into smaller streams, it must be determined whether a possible flood condition downstream could be created.

6. Provision should be made for any future development of surrounding site areas of building additions.

Layout of System

At this point, it is assumed a survey plan and a preliminary site plan with contours are available for a general layout of the system. Runouts from buildings should now be located, outfall of the storm sewer selected, and work started on a trial location of storm water inlets. A trial layout of the site piping should be developed at this time and coordinated with water mains, electrical or telephone duct banks, sanitary sewers, gas mains, and other piping to ensure proper clearance and allow for proper and reasonable slope for the storm sewer lines. Obstacles to be avoided, such as trees, underground structures, and rock, should be located. Recommendations for final grading could be made to the landscape architect to possibly reduce the cost of storm water sewers by advantageous placement of inlets to minimize piping runs.

In general, the following should be considered when locating and designing inlets:

1. Where streets or roads intersect, inlets must be located upstream of traffic flow.

2. Where a series of inlets are located in a road with continuous slope, each intermediate inlet should be designed for approximately 75 to 90 percent of design flow, passing 10 to 25 percent on to the next downstream inlet. If the slope is shallow and flow is not great, 100 percent capacity should be considered. More than one inlet should be provided at the bottom to accept the additional flow.

3. Inlet capacity should be limited to approximately 5 cfs (0.15 m^3/s) of water.

4. All site low points must be provided with an inlet if off-site gravity flow is not possible. Combination inlets are used for streets or roadways. Parking lot inlets should be flat grates only, even when located at a curb.

5. Distance between drainage inlets should be a maximum of about 300 ft.

6. In a roadway with a sag vertical curve, more than one inlet should be considered if a large quantity of runoff is anticipated. Additional inlets should be located on either side of the low point inlet to minimize flooding and sediment buildup resulting from large flows.

7. Manholes should be used at all changes of direction and slope of pipe and at multiple pipe connections. Drain structures also may serve the same purpose. Distances between manholes should be 250 to 600 ft (75 to 188 m) depending on sewer size.

8. Steep slopes should have inlets located closer together than normally required to limit inflow.

9. A hooded catch outlet should be used as a final structure before connection to a combined sewer to prevent the passage of unwanted gases.

10. The efficiency of inlets in roadways depends on gutter flow, water velocity, roadway slope, and inlet depression, if any. In many cases, freedom from clogging or interference with traffic may take precedence over hydraulic considerations.

In general, the following should be considered:

- The road crown should be as steep as possible.
- Where traffic will not travel close to curb and clogging is not a problem, use a depressed gutter inlet. Use a depressed curb or combination-type inlet where clogging may occur.
- Where traffic moves close to the curb, use an undepressed inlet and gutter grate with longitudinal bars only.

STORM WATER INLET SELECTION

Drainage Structures

A drainage structure is an assembly of elements outside of piping built or installed below ground. Inlet structures are intended to collect surface storm water and route the effluent into a piping system.

While the piping system is being designed, after the storm water drainage inlet structures have been located, the type and size of inlets and grates must be selected. In most cases, the authority having jurisdiction has design standards available for such purposes. If no standards are available, the following can be used as a guide. Storm water drainage inlet structures are classified according to their functions as follows:

1. *Drainage inlets* (DIs) are structures that admit storm water into the piping system. They are generally located in areas that are reasonably free from sediment or debris such as paved, lightly vegetated, or unimproved areas. The bottom of a DI is level with the invert of the outlet pipe. A shallow DI is illustrated in Fig. 6.7 and a deep DI is illustrated in Fig. 6.8.

2. *Catch basins* (CBs) are similar to drainage inlets except that there is a space below the inlet and outlet pipes for retention of debris or sediment. A CB is usually located in paved areas where debris can be easily washed in. Experience has indicated that inadequate maintenance negates their benefits, and catch basins are not generally used unless good maintenance can be assured. A typical CB is illustrated in Fig. 6.9.

3. *Manholes* (MHs) do not allow surface rainfall to enter but are provided for ease of pipe connections for cleaning purposes. Drop manholes provide for pipe connections where a difference of more than two feet in elevation exists between inlet and outlet pipe. A precast MH is illustrated in Fig. 6.10 and a drop connection into a MH is illustrated in Fig. 6.11.

Gratings

Gratings allow water to enter the drainage structure, and some gratings will prevent the entrance of debris into the piping system. Gratings are classified as plain, curb, or combination.

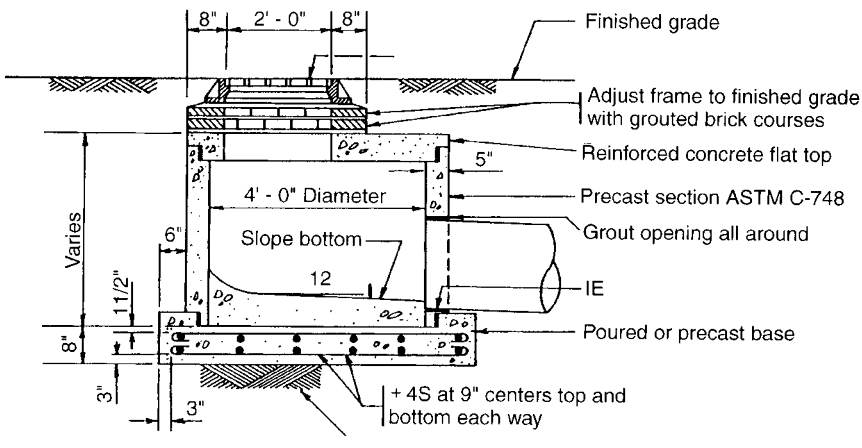


FIGURE 6.7 Typical shallow drainage inlet.

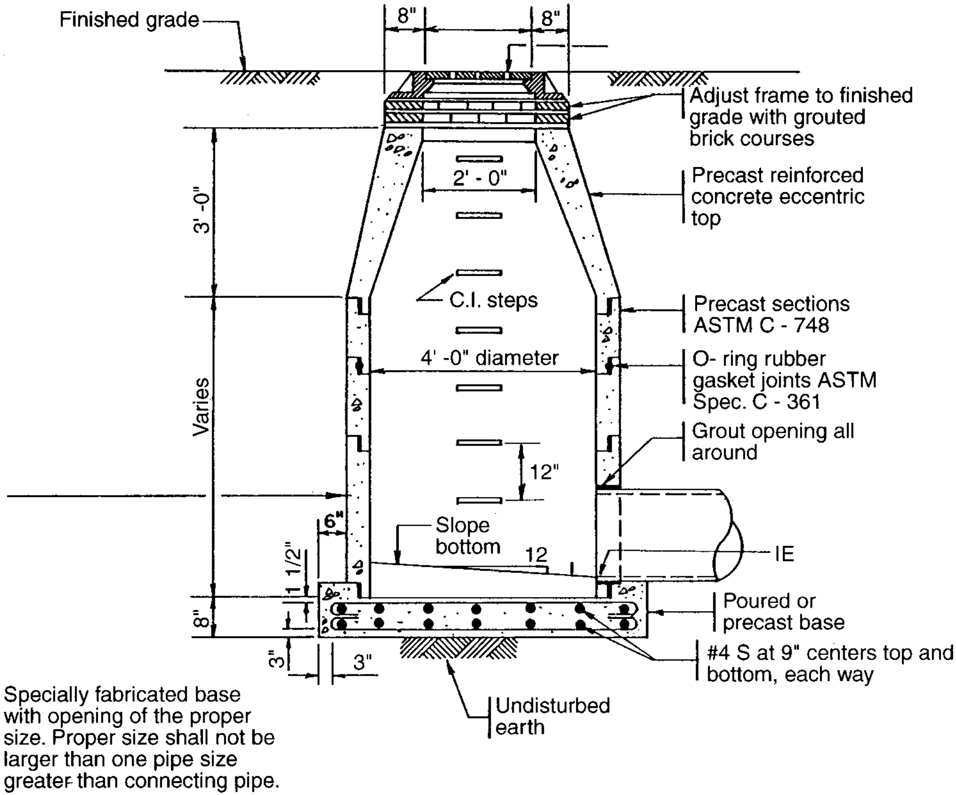


FIGURE 6.8 Typical deep drainage inlet.

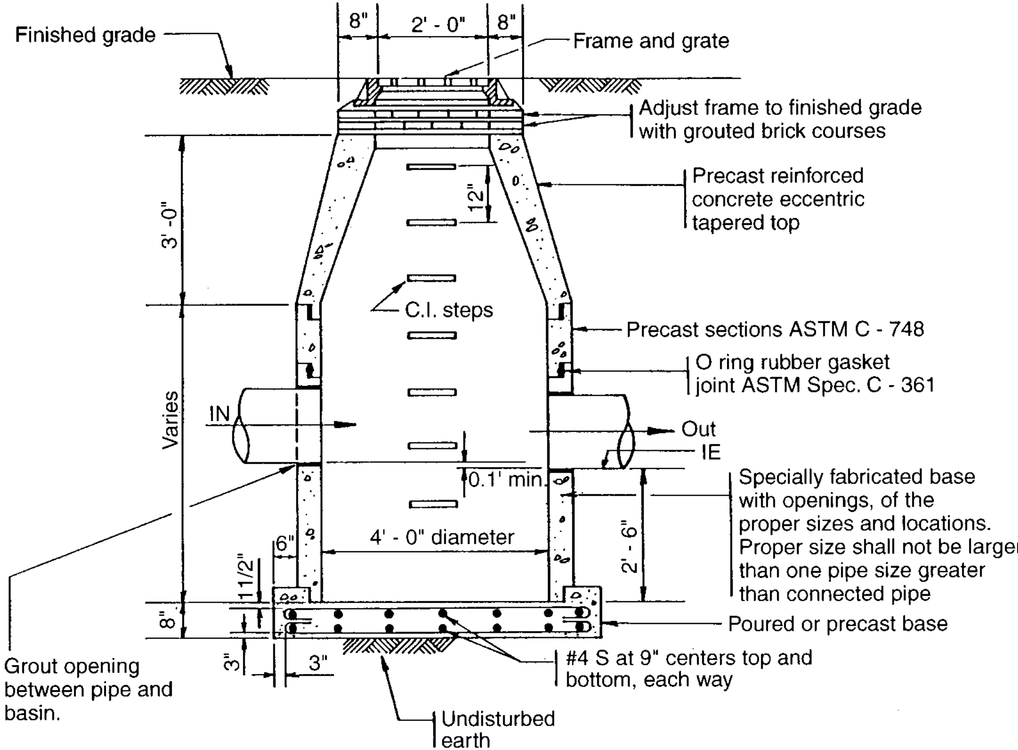
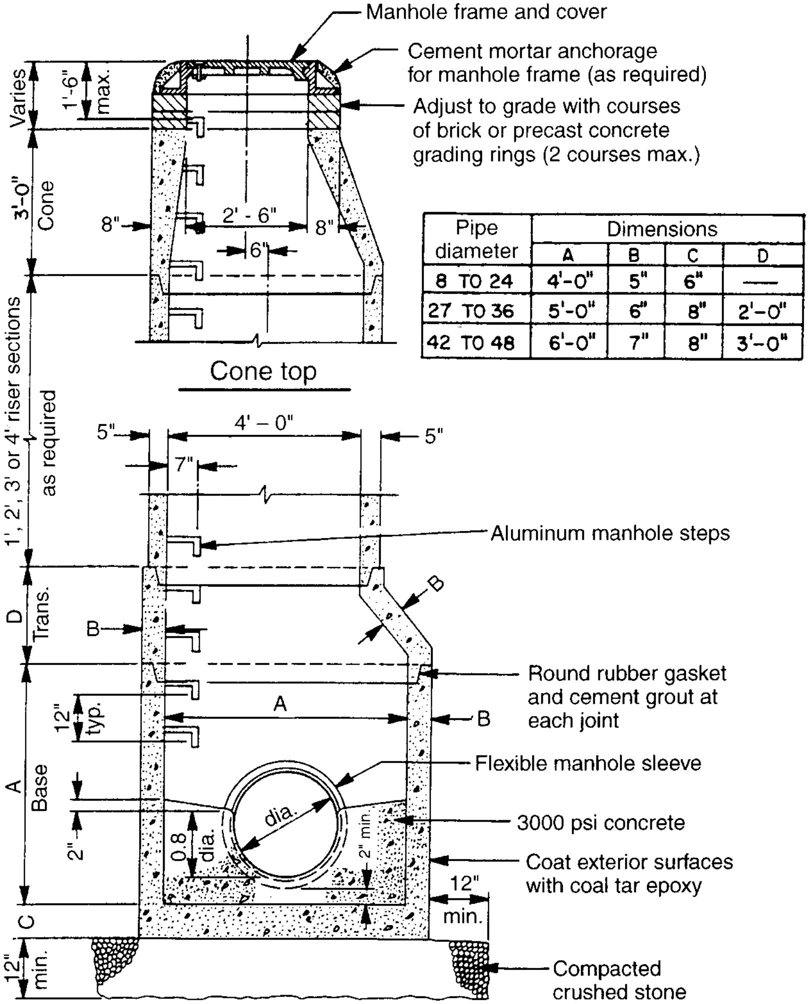


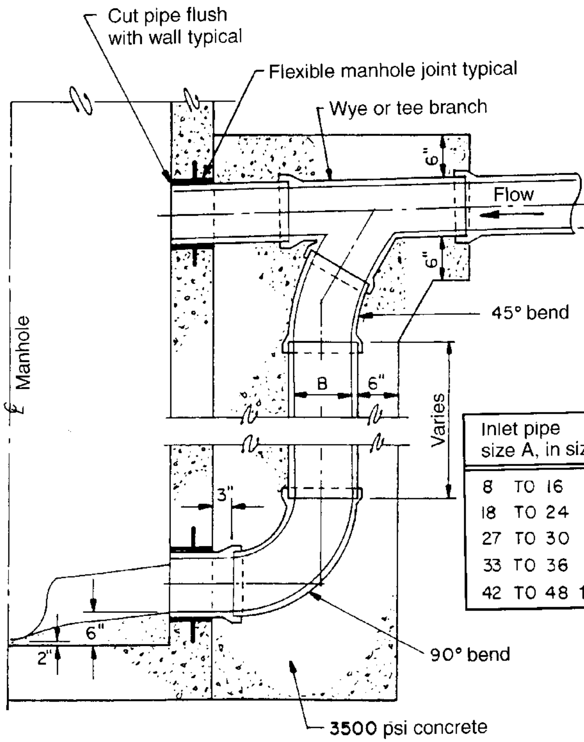
FIGURE 6.9 Typical catch basin.



Pipe diameter	Dimensions			
	A	B	C	D
8 TO 24	4'-0"	5"	6"	—
27 TO 36	5'-0"	6"	8"	2'-0"
42 TO 48	6'-0"	7"	8"	3'-0"

FIGURE 6.10 Typical precast manhole.

- A plain grate (or gutter grate) is flat and allows water to enter from all unobstructed sides. Bars prevent the entrance of debris.
- A curb grate has a single vertical opening in the side of a curb. Water inters only from the front. Debris can enter freely.
- A combination grate is a plain grate and curb grate acting as a single unit with one placed in front of the other.
- Multiple inlets are two closely placed inlets acting together.
- A round grating cannot fall into a drainage structure.
- Flat grates are assumed to be flush with grade and flooded.



Inlet pipe size A, in size A DN		Drop connection size B, in DN	
8 TO 16	150 to 400	8	150
18 TO 24	450 to 600	12	300
27 TO 30	700 to 750	18	450
33 TO 36	800 to 900	24	600
42 TO 48	1050 to 1200	30	750

Note:
Drop pipe to be used in all cases where difference between inlet invert and lowest outlet invert is 2 feet or greater.

FIGURE 6.11 Typical drop manhole connection.

Grates Installed in Flat Areas (1 Percent or Less). The following empirical formulas have been developed for the selection of gratings:

Plain Grate

$$SF \times Q = C\,L\,H^{2/3} \tag{6.2}$$

where Q = quantity of water entering grate, cfs
C = constant of 3.0
L = total perimeter of grate, ft
H = depth of water, above level of road, ft
SF = safety factor

Discussion:

1. H is usually 2 to 3 in, but never more than 6 in.
2. A safety factor is required to allow for blockage by debris. In paved areas, use 1.25. Other areas use 1.5 or 2.0, according to conditions of possible debris accumulation. 2.0 is not unusual.

3. Where one side of a grate is against a curb, only three sides are used to compute the perimeter.
4. Figure 6.12 depicts a graphical solution of inlet capacity for a flooded plain grate, based on Eq. (6.2).
5. Use a manufacturer's catalog to select a standard size grate with the calculated perimeter as a minimum starting point.

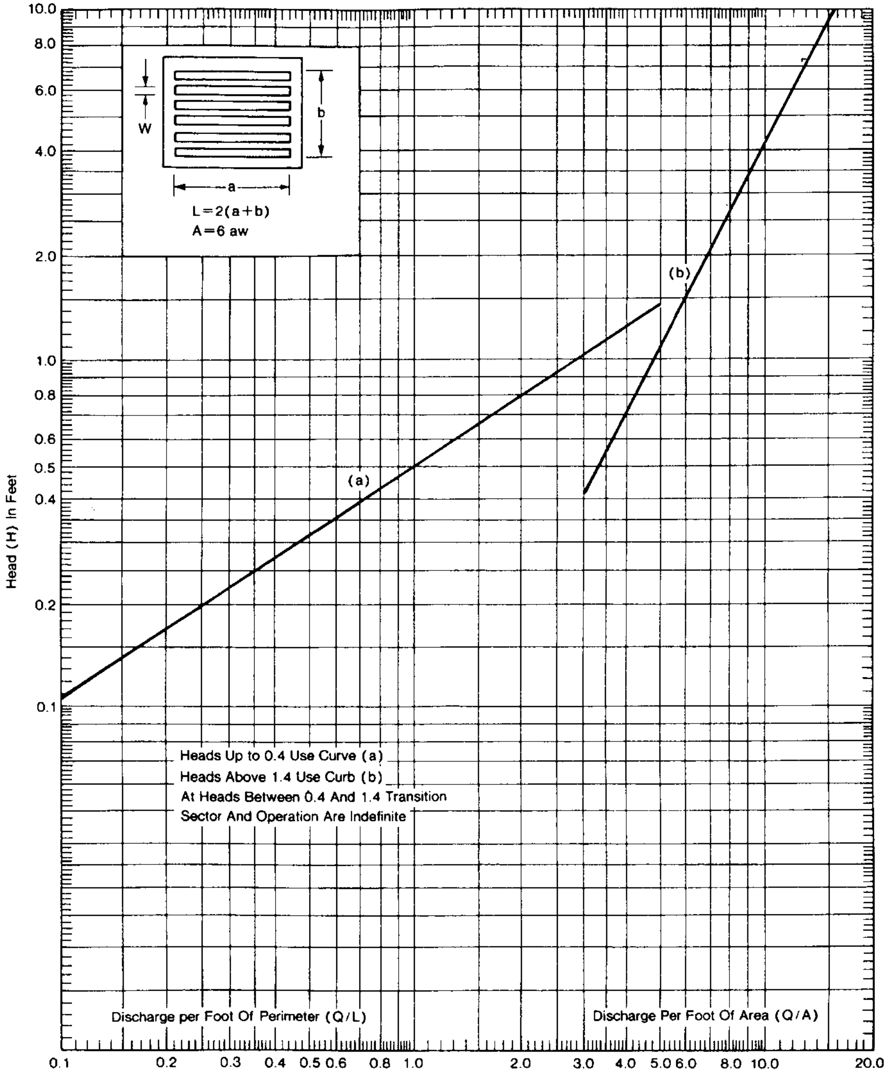


FIGURE 6.12 Graphical solution for a plain flooded grate. (*Courtesy Bureau of Public Roads, Baltimore County.*)

Curb Grate. This type of inlet is rarely used alone unless conditions do not permit any other type and, in addition, favorable inlet depression and contour is possible. Figure 6.13 can be used to determine flooded curb grating size.

Combination Grate. Experiments have shown that combination grates do not admit substantially more water than a flat grate; therefore, Eq. (6.2), for the plain grate, should be used to determine the size. An additional 10 percent for paved areas and up to 75 percent for other areas should be added as safety factors plus an additional 25 to 75 percent as a debris factor. This type of grate is preferred where provision to collect all storm water is required. The curb grate acts as a safety valve when a flat grate is clogged with debris.

Grates Installed in Sloped Areas (Greater Than 1%)

Plain Grate. The size of a flat grate located in a sloping roadway can be determined from the trial length derived from the following formula.

$$L = \frac{V}{2}\sqrt{Y + D} \tag{6.3}$$

where L = minimum length of opening, ft
V = velocity of water in gutter, fps
Y = depth of water in curb, ft
D = thickness of grate, ft (from a manufacturer's catalog)

Discussion:

1. A minimum length of 3 ft (1 m) and a minimum width of 2 ft (1.3 m) are recommended.
2. Net opening of the grate should be 50 percent of width or greater.
3. The nomograph in Fig. 6.14 should be used to determine water velocity in a gutter or channel.

Curb Grate. This type of inlet is very inefficient, and a long length is usually required to collect the entire design flow. The capacity for complete interception is determined from the following formula.

$$Q = 0.7L(A + Y)^{1.5} \tag{6.4}$$

where Q = capacity of grate, cfs
L = length of clear opening, ft
A = depth of depression at inlet, ft (below roadway level)
Y = depth of water in gutter, ft (above roadway level)

Combination Grate. The capacity of combination inlets has been determined by a series of complex equations. Since actual experiments have proven that the capacity of combination inlets is not much greater than that of flat inlets, it is much easier to design the grate as if it were flat. Since some allowance can be made for the combination grate, the size of the flat grate portion can be reduced if the calculated size is larger than that of the standard size produced by a manufacturer.

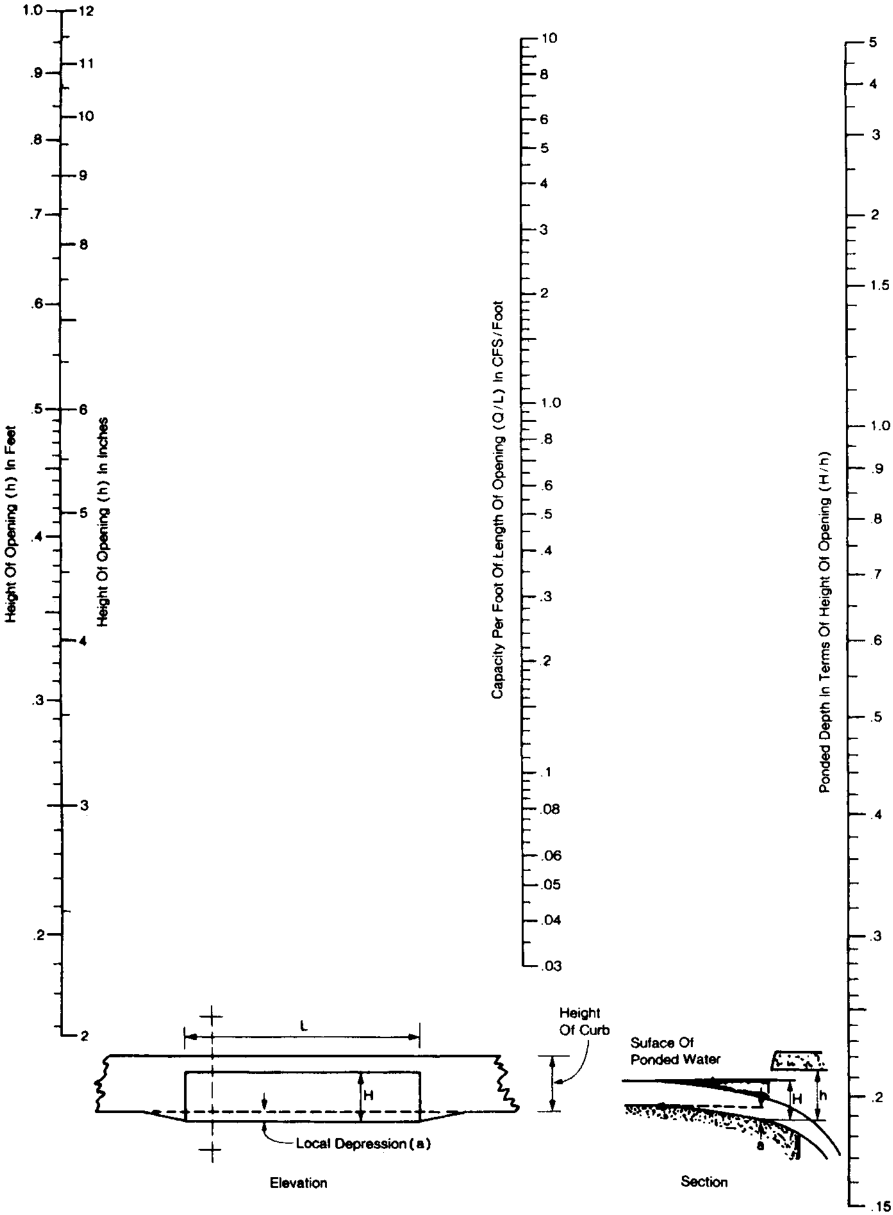


FIGURE 6.13 Graphical solution for a curb grate.

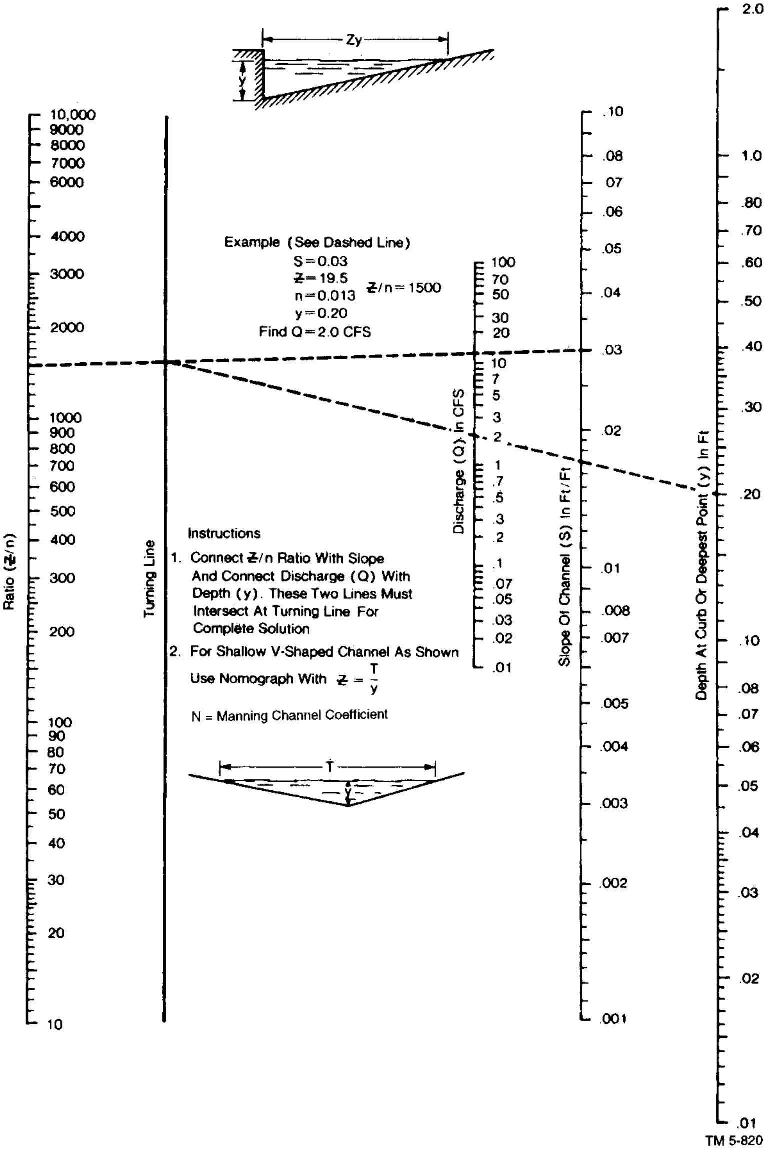


FIGURE 6.14 Water velocity in a gutter.

SYSTEM DESIGN CRITERIA

General

The *Rational Method* will be used to determine theoretical water inflow into a storm sewer system. It has been widely used since its introduction slightly before the turn of the century and has a proven record of acceptability. It is an empirical formula well suited for the design of small watersheds. The formula is

$$Q = A \times I \times R \tag{6.5}$$

where Q = quantity of storm water runoff, cfs
A = area to be drained, acres
I = imperviousness factor of surface comprising drainage area, dimensionless
R = rate of rainfall, in/h

The following discussion will provide an understanding of the basic values that will be used in the Rational Method formula.

Design Storm

The Rational Method reduces an inexact set of conditions into an exact formula. Such variables as rainfall rate, overland water flow, and the amount of storm water that actually reaches the DI can never be exactly determined. Because of this unpredictability, it is important to realize that some judgment is required in applying the calculated values and given information in the design of a storm water system.

The storm water drainage system is designed to remove the maximum amount of expected runoff as quickly as it falls to avoid ponding or flooding. The ability to calculate the flow rate is complicated by the inability to accurately predict many of the factors affecting the actual amount of runoff resulting from any given storm. In order to calculate the maximum estimated runoff, an artificial "design storm" must be created. This simulation can be used to predict runoff volume accurately enough to provide a basis for the piping network design.

The design storm is based on actual rainfall records, and has been presented in a convenient form by the National Oceanic and Atmospheric Administration/National Weather Service (NOAA). Design storms are available as either intensity-duration-frequency curves or as charts and formulas appearing in several technical memoranda covering different areas of the United States. The intensity-duration-frequency charts are in a very convenient, easy-to-use format and are considered accurate for the small watersheds that are the subject of this handbook. Charts for various cities throughout the United States are presented in Fig. 6.15. However, because of the longer period of data collection, the charts have been superseded by technical memorandum *NWS Hydro 35,* which must be considered the latest information available. This handbook presents the intensity-duration-frequency charts only because the values obtained from them fall well within the acceptable accuracy for calculations for small watersheds and because the charts are easy to use.

Imperviousness Factor

The imperviousness factor allows for the loss of rainwater as it flows over the ground from a remote point of the DI tributary area until it enters the DI. The causes for this loss include infiltration of water into the soil, ponding, and water remaining on vegetation. The average figures for imperviousness of various surfaces are found in Table 6.2.

Inspection, if possible, is the best way to determine the nature of an existing surface. In places where the exact nature of future development is uncertain, the figure selected should represent the minimum expected loss, to allow the largest quantity of storm water to reach the DI. Where any area consists of different types of surfaces and/or soil combinations, a weighted overall value may be assigned for ease of calculation.

Rate of Rainfall (Rainfall Intensity)

The rate, or intensity of rainfall, is obtained from the rainfall intensity-duration-frequency curves, shown in Fig. 6.15. The intensity is measured in in/h. In order

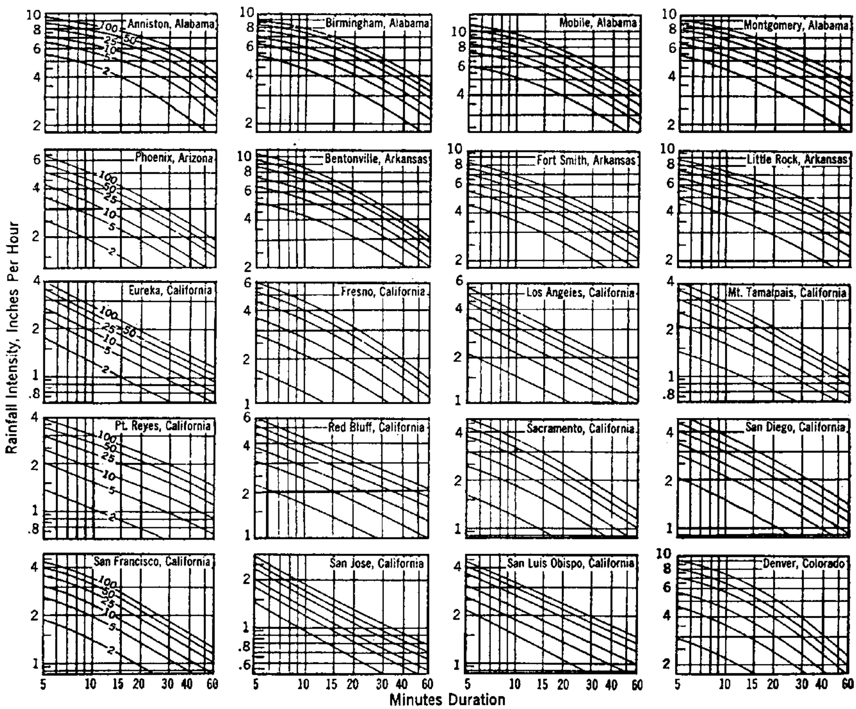


FIGURE 6.15 Rainfall intensity-duration-frequency charts.
The rainfall intensity-duration-frequency curves are abstracted from Technical Paper No. 25 of the Weather Bureau. Included are data for 200 selected stations in the United States and Puerto Rico. The data are substantially in the form found in the Technical Paper, except that the curves are cut off at the 60-minute duration line, enabling data to be presented in six pages.

Rainfall data in this form are intended for use in designing the modest storm drainage system associated with buildings and with industrial plants and their surrounding parking and lawn areas.

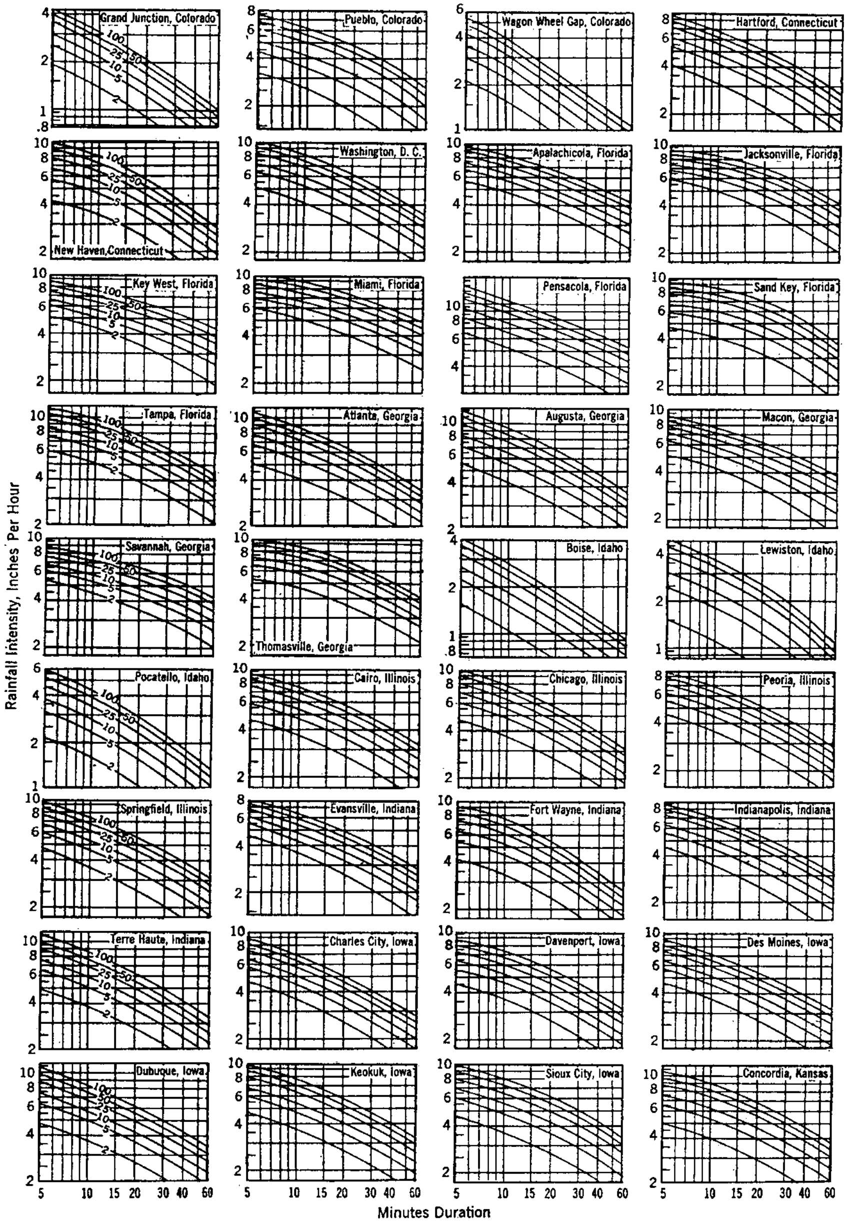


FIGURE 6.15 (*Continued*)

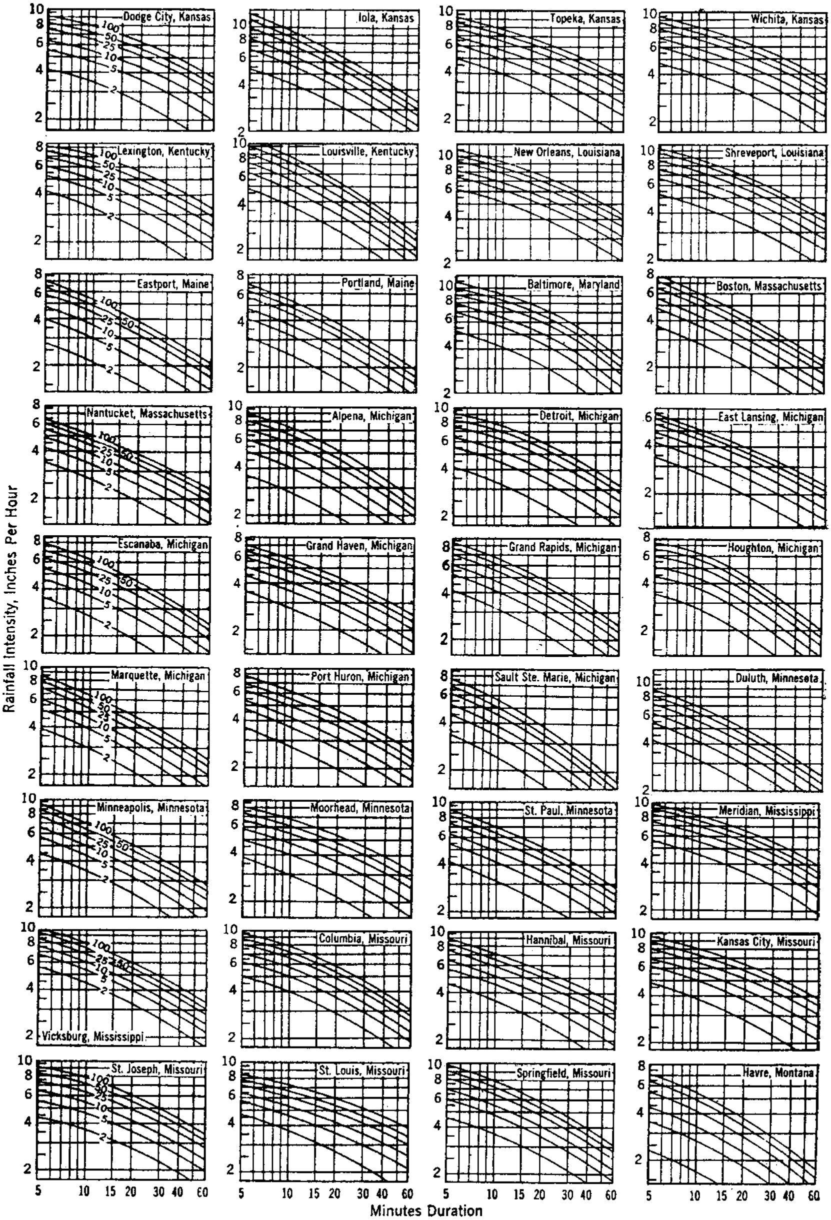


FIGURE 6.15 (*Continued*)

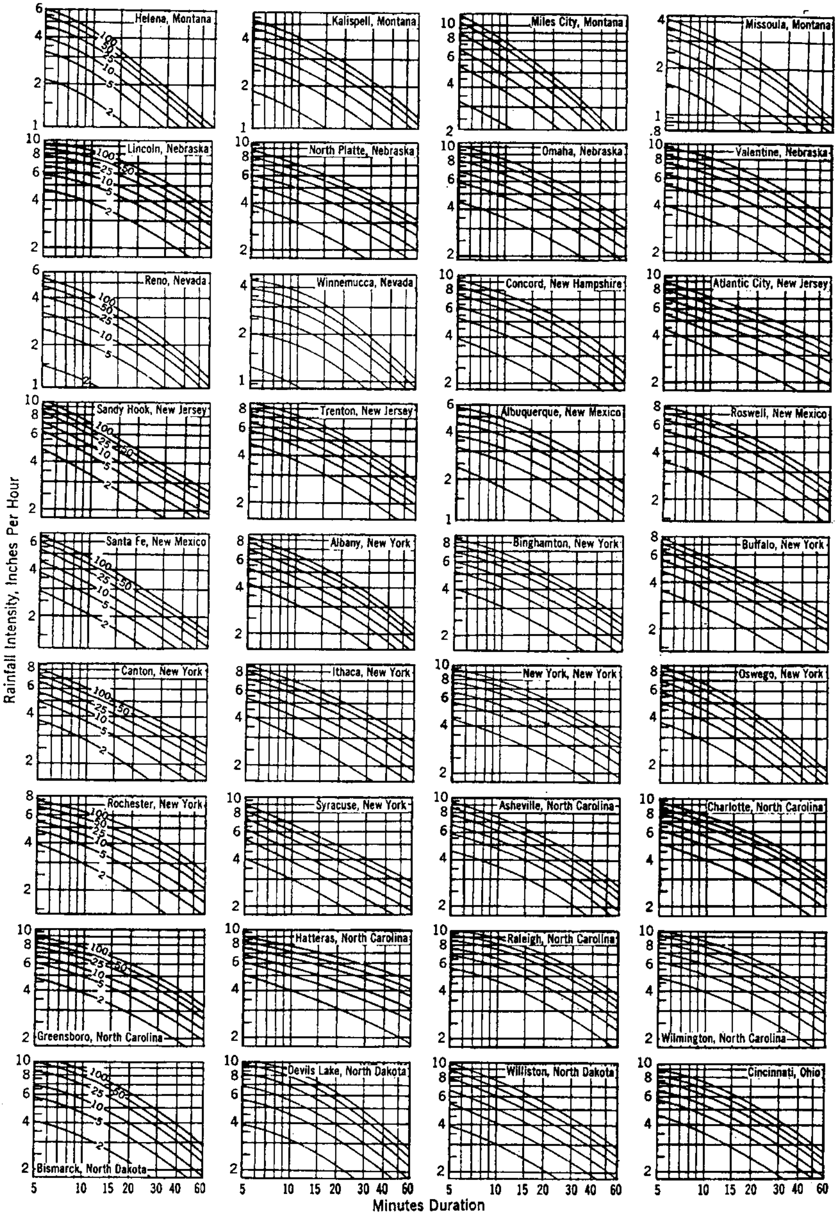


FIGURE 6.15 (*Continued*)

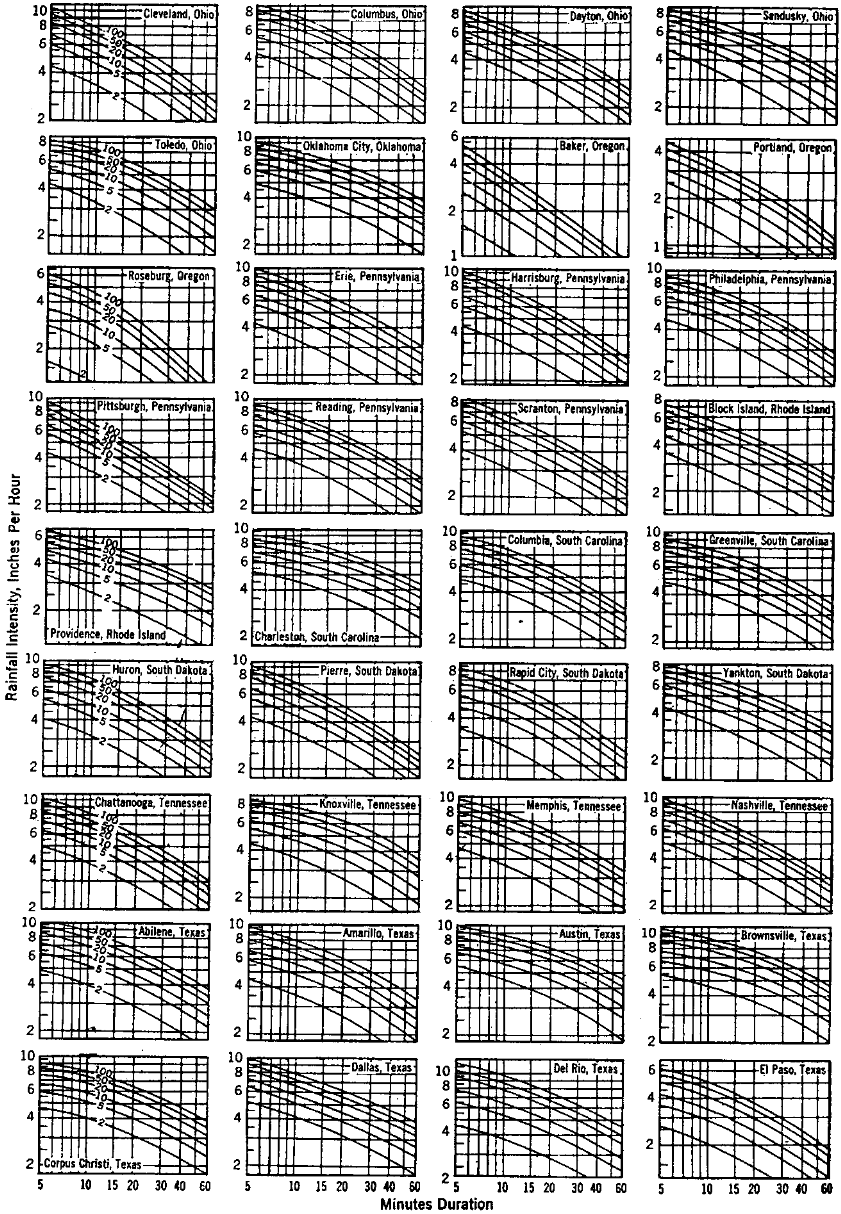


FIGURE 6.15 (*Continued*)

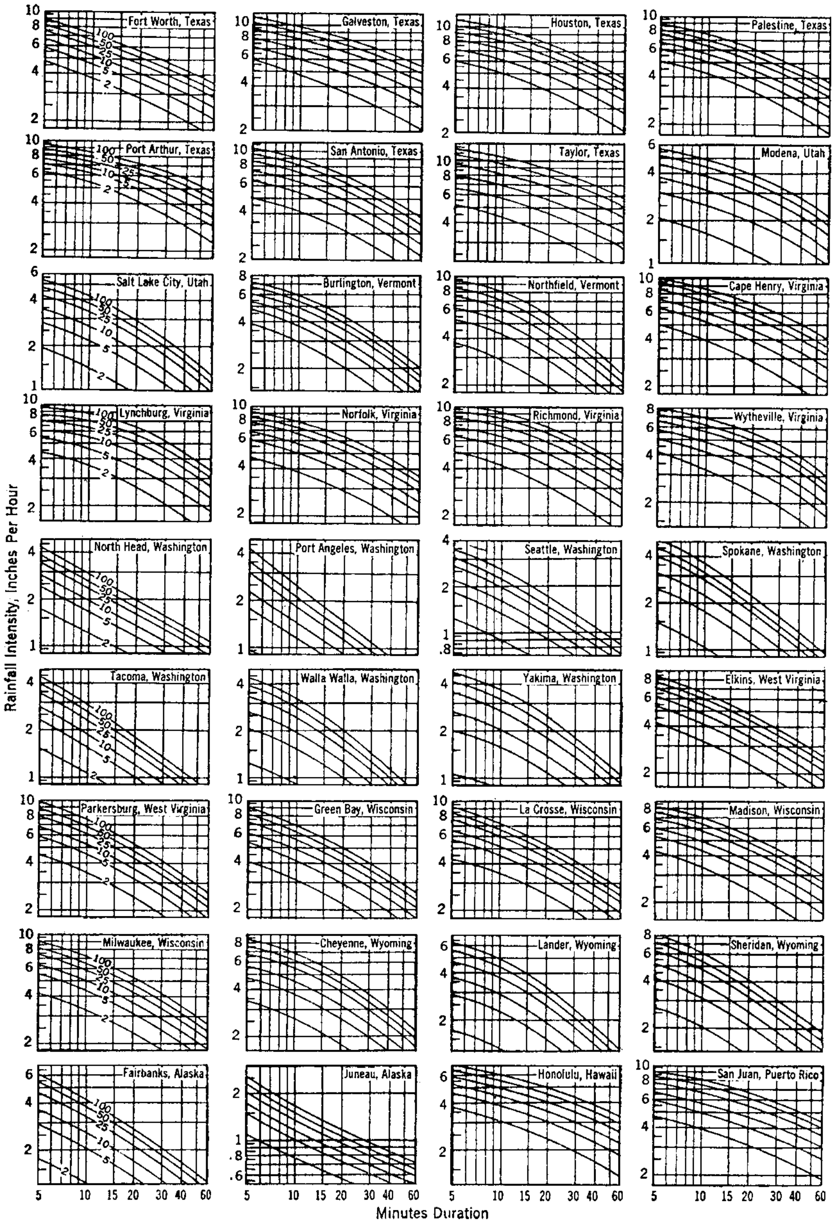


FIGURE 6.15 (*Continued*)

TABLE 6.2 Imperviousness Factor for Surfaces

Surface	Flat slope, less than 2%	Average slope, 2.0 to 7.0%	Steep slope, 7.1% or more
Roofs & pavement (all types)*	0.95	0.95	0.95
Clay-sparse vegetation	0.40	0.55	0.70
Clay-lawn	0.15	0.20	0.30
Clay-dense vegetation	0.10	0.15	0.20
Clay-dense woods	0.07	0.12	0.17
Sand-sparse vegetation	0.20	0.30	0.40
Sand-lawn	0.07	0.12	0.17
Sand-dense vegetation	0.05	0.10	0.15
Sand-dense woods	0.03	0.08	0.13

*For ease of calculation, the factor for roofs and pavement could become 1.0 if excessive runoff will not result in a significant overdesign.

Source: Baltimore County Design Standards.

to enter Fig. 6.15, the return period (or frequency) and the duration (or time of concentration) of the design storm must be found. Determining the intensity requires consideration of many factors.

Return Period (Frequency). The design storm return period (or frequency) is the statistical period of years that must elapse to produce the most severe design storm once in that period of time. The theory is that the worst storm that would ever be expected to occur once in 100 years would be much more severe than the worst storm that would occur once in two years.

The curve for a given frequency is actually a plot of different storms of varying lengths, each with a different duration, rather than a single storm plotted against time. Therefore, a storm of 10-min duration is one that lasts for exactly 10 min and then stops. It is not an extension of any other plot. Any storm with a longer duration will have a lower instantaneous peak flow.

Flooding will result if the design storm is exceeded. Special consideration should be given to the degree of protection provided for the building and its contents by rapid removal of rainfall by the storm water drainage system. This depends on the importance of the facility, how flooding may affect access, the importance of uninterrupted service, and the value of equipment or material installed or stored. Since a severe thunderstorm or hurricane may produce rainfall rates greater than anticipated, the property value may require the selection of a frequency longer than the minimum determined.

The return period can be based on client preference, code requirements, or the degree of safety desired. The following criteria may be helpful as a guide. For average sites, a design storm frequency of 10 years is generally used. Certain clients design their projects for 40 years of useful life, which may require a 50-year design storm if flooding might cause a problem. Many municipal authorities use a 2- or 5-year design storm for city sewers. Some use a 10-year storm. The U.S. Corps of Engineers normally uses a 10-year storm, but for military installations a 25-year storm is used.

Duration (Time of Concentration). The duration (or time of concentration) is measured in minutes. It is found by calculating the overland flow time and the time in pipe of a theoretical drop of water from the most remote point on the site to any design point on a branch or main drainage line.

It is important because the shorter the time, the more intense the rate of rain. This heavy rainfall, multiplied out to 1 h, would be much greater than the rainfall expected for a 1-h period. As an example, for New York City, the heaviest total amount of rain on record for a 5-min period is 0.75 in. Multiplied out to determine the 1-h rate using the 5-min reading gives an hourly figure of 9 in of rain per hour. Yet the largest amount of rainfall every recorded over a 1-h period is 3.11 in. It can thus be seen that the rate of rainfall measured during a short period is much more intense than the rate measured over a longer period of time.

The underlying principle is that the rainfall will stop at the exact moment the entire design area is contributing to the flow in this particular section of the sewer. Therefore, calculations must establish the shortest amount of time a drop of water takes to run from the farthest point of the site to any design point of the sewer. This is necessary so that the shortest time of concentration is obtained and the largest hourly rainfall rate is used for design purposes.

Overland Flow Time (*Inlet Time*). The overland flow time, often called the inlet time, is the number of minutes it takes a drop of water to travel from the farthest point of the area contributing flow to a DI until it spills into the inlet. The impedance to overland flow of water by a grass surface is greater than that of an asphalt surface. For equal distances and slope, it takes longer for a drop of water to enter a DI from the farthest point of a lawn than from a paved parking lot. Slope of the land is also a factor, since for any given surface, the steeper the slope, the faster water will flow.

Figure 6.16 offers a solution to determine overland flow time when slope of grade and type of surface are known. During a rainstorm, the flow of water from surfaces covered with pavement is in the form of a very shallow sheet of water covering the surface of the ground. This phenomenon is known as *sheet flow.* Finding the velocity of sheet flow in feet per second and multiplying that figure by the actual distance in feet gives the time in seconds. This should be converted to minutes to calculate the overland flow time.

Time in Pipe. The time in pipe is the number of minutes the theoretical drop of water takes to flow from the DI to the design point. This requires that the pipe be sized and the velocity of the storm water known.

Finding the velocity in feet per second and multiplying that figure by the actual pipe length in feet gives the time in pipe, in seconds. This should be converted to minutes to calculate the time in pipe.

Final Calculation. Adding the overland flow time and the time in pipe will give the time of concentration. Experience has shown that for most projects, use of a short duration will result in excessive runoff volume and excessive pipe sizes. It is recommended that a 10-min duration be used as the minimum for typical design conditions.

SYSTEM DESIGN PROCEDURE

1. The drainage structures should be located, and final coordination completed, with site planner. Select the type of drainage structure, either DI or CB.

2. The return period for the design storm should be chosen.

3. A site plan showing the location of the drainage structures and contour lines should be used to determine the exact surface area and type of surface contributing to flow into each DI. A chart or tabulation in any convenient form should be prepared indicating the type of surface and the area in square feet of each type of

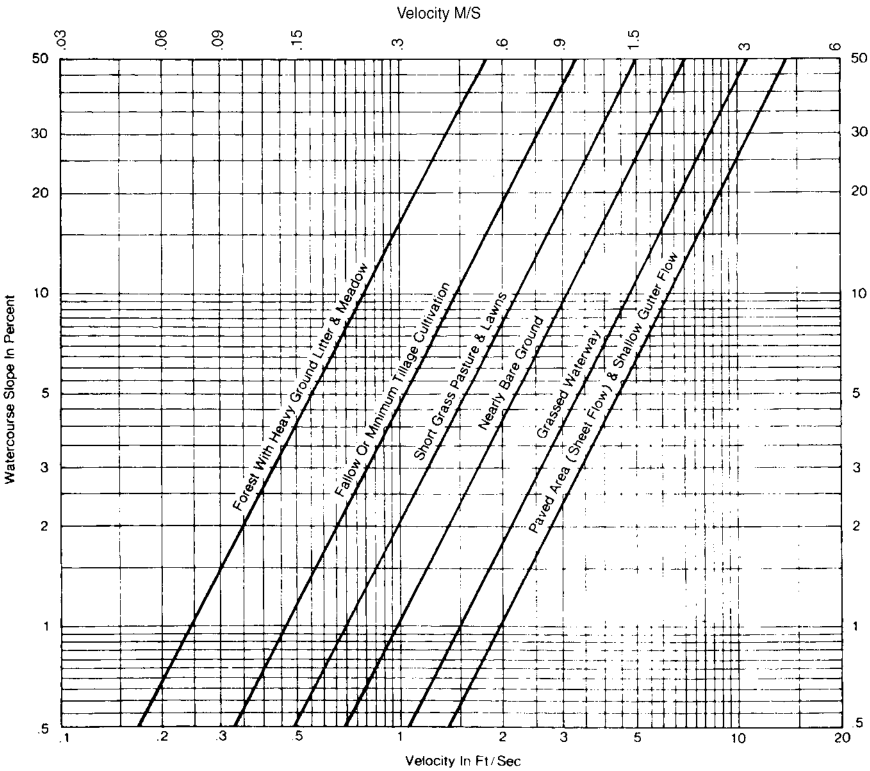


FIGURE 6.16 Overland flow time.

surface draining into each DI. Room should be left for additional information that will be added later. For each DI, the distance from the farthest point on the watershed that could contribute water to the DI should be measured. Also, the slope of the land into each DI should be calculated.

4. The imperviousness factor of the various surfaces in the watershed should be determined, keeping in mind the average slope of the area and the type of surface or surface combinations. A weighted factor for areas having more than one surface type should be used and recorded on the chart for reference.

5. Using Fig. 6.16, the overland flow time into the furthest DI should be determined using the slope of the ground and the type of surface. The minimum flow time should be 10 min. Rarely will a smaller time be economically justifiable.

6. Utilizing the rainfall intensity-duration-frequency curves (Fig. 6.15) for the city closest to the project location, the rainfall intensity should be selected using the return period and the time of concentration previously chosen. For each station, six curves are given. Although specifically so designated in the first column only, each set corresponds to storm frequencies (or return periods) of 2, 5, 10, 25, 50, and 100 years. For the normal industrial site, the 10-year curve should be used. The more conservative the design, the larger the design period that will be chosen.

7. From data in the form just prepared, and using the rational formula, the total inflow to each individual DI should be calculated. A weighted factor should be used for areas having more than one surface type, and this information placed in the form for reference. The individual pipe from each DI will be sized from this information.

8. With the total inflow in cfs to individual DIs now known, the grate type and size can be selected. The grate type should be entered on the form for reference. DI locations should be adjusted for flow requirement if necessary. Most manufacturers have developed inlet flow charts for specific grates and their catalogs will give the grate size that can accept the calculated flow.

9. The piping system from the DI to the point of disposal can now be laid out. MHs can be located and the pipe material selected.

10. Building roof drain runouts shall be connected to the storm water drainage lines and their drainage areas noted.

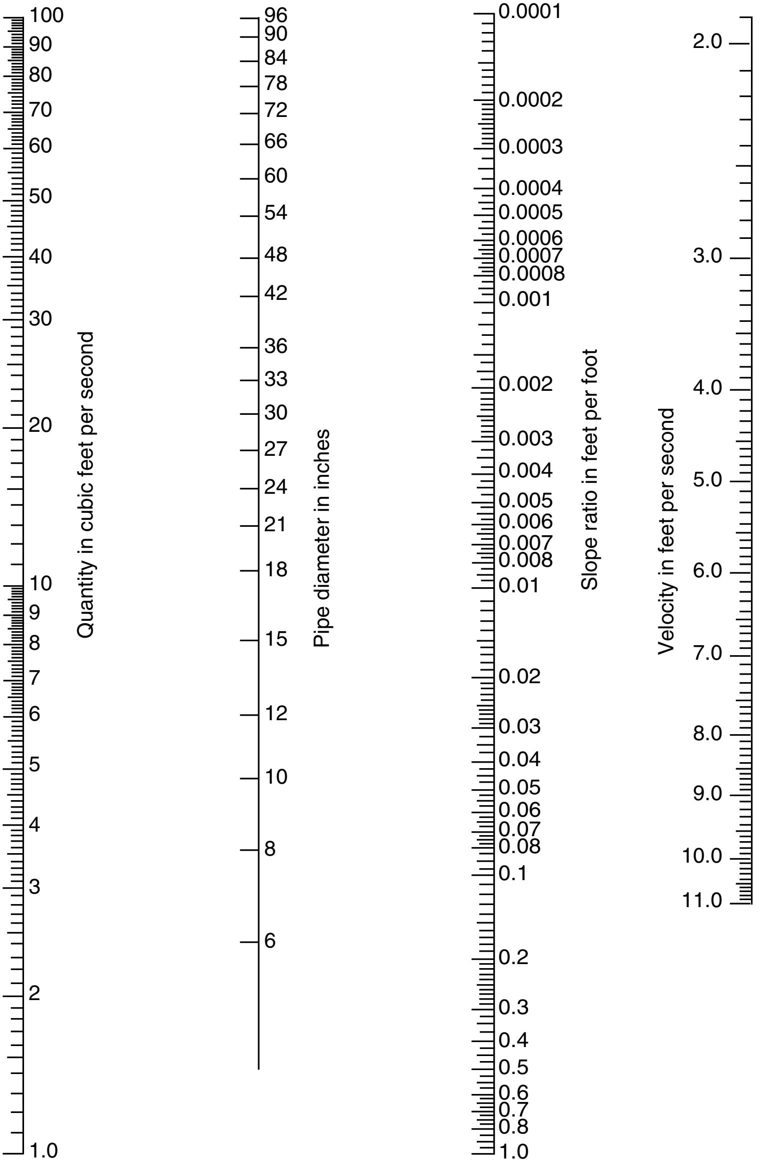
11. The critical inlet is the one that produces the maximum combination of inlet time plus flow time in sewer to the very first connection with any other branch. The critical inlet should be chosen now. The inlet at the furthest end of the drainage system should be selected as the starting point. This point will be the longest in time, not necessarily in distance. This may require some trial calculations of several DIs at various far ends in order to determine which drainage inlet actually is critical. Distance will not be the only criterion. An area composed of asphalt, which has a fast inlet time, might be a much greater distance from an inlet than an area consisting of grass, which has a longer inlet time. The slope and length of sewer pipe must also be considered.

12. With the selection of the critical inlet, the individual sewer pipe line can now be sized from the critical inlet up to the first point of intersection with any other contributing source of storm water. The layout of the sewer system will establish the slope of the pipe, and the form will provide the flow rate of storm water in cfs. The pipe material will have been selected and value of n for the pipe for use in the Manning formula will be known. Refer to Table 6.3 to select the n value. Entering Fig. 6.17 with the pipe material, connecting a straight line from the flow rate to the slope of piping will now establish the pipe size and water velocity. The necessary adjustment must be made, as indicated by the conversion factor if a different n is used from that of the chart. The desired figure is now multiplied by

TABLE 6.3 *n* Value of Pipe Used in the Manning Formula

Pipe material	Range of *n* values	Generally accepted value*
Asbestos-cement	0.011–0.015	0.013
Corrugated metal pipe	0.022–0.026	0.024
Cast iron	0.011–0.015	0.013
Concrete pipe	0.011–0.015	0.013
Ductile iron (cement lined)	0.011–0.015	0.013
Plastic pipe, all kinds	0.010–0.015	0.011
Steel pipe	0.012–0.020	0.015
Vitrified clay	0.011–0.015	0.013

*Values will vary based on condition of pipe.



Value of n	0.008	0.010	0.011	0.012	0.013	0.015	0.019	0.021	0.024
Conversion factor for discharge and velocity	1.62	1.30	1.18	1.08	1.00	0.87	0.68	0.62	0.54

FIGURE 6.17 Pipe flow chart. Diagram for solution of Manning formula for circular pipes flowing full (n = 0.013).

the conversion factor found under the actual *n*. Generally accepted practice assigns a value of 0.013 to most pipe materials.

13. When the first junction with another contributing source has been reached, the entire area of all contributing sources up to this first junction should be totalled. The new time of concentration should be calculated from the most remote inlet and the time in pipe to the design point of the storm sewer. Entering the intensity-duration-frequency chart to find the rainfall rate will now give all necessary information required for insertion into the Rational Method formula [Eq. (6.5)] to find total cfs. The resultant flow into the pipe is used to size that particular segment downstream from the design point. This is continued in succession to the end of the system, requiring a longer time of concentration each time a new design point is reached.

14. As progress is made along the route of the sewer, all branches to the main sewer line should be sized individually as their connection is reached. The branch is sized using the time of concentration required for that branch only. To size the main, all the areas contributing to the flow at any particular junction should be added up, using the longer time of concentration determined from the initial overland flow time from the critical inlet added to the flow time of the water in the pipe.

SYSTEM DESIGN CONSIDERATIONS

1. The velocity in the piping system should not be less than 2 fps (0.6 m/s), to allow for the movement of sediment and any other solids in the pipe.

2. The maximum velocity should be limited to approximately 30 fps to prevent erosion of the pipe interior. If large quantities of sand are present in the water, the scouring action will increase and so the velocity should be reduced for this condition.

3. To limit velocity, the slope of the piping system should be decreased rather than using oversized pipe. There are other methods available, also. If the slope of the ground is steep, a drop manhole should be used where the difference in elevation between inlet and outlet inverts is more than 2 ft (0.6 m).

4. A manhole should be located at every change in pipe size, slope, or direction. In many cases, a drainage structure can be used for this purpose if such placement is practical.

5. The minimum size pipe should be 12 in (DN 300) to reduce the possibility of stoppage by debris. A smaller size for runout from a building roof storm water system is generally acceptable.

6. When a change in pipe size occurs, the pipe should be installed crown to crown to eliminate a surcharge of the upstream portion of the pipe.

7. Factors such as snowmelt may add additional unexpected quantities of water. Snowmelt does not usually add a significant amount of inflow, but it may if the depth of snow is large. However, increasing one pipe size may lead to a problem by reducing velocity too much. In that case, it is advisable to increase the slope of the sewer line. As a general rule, one should be a little generous with pipe sizes. The slight increase in cost will be more than made up by the safety factor of additional capacity provided.

8. Exfiltration occurs when water leaks out of a pipe through bad joints or cracks into the surrounding ground. When a pipe passes under a road or railroad, a joint should be chosen that will permit very little or no leakage to avoid washing away the subgrade.

9. Piping should be installed roughly parallel to the slope of the ground. Standard percentages of slopes if possible should be used to permit contractors to easily install the pipe.

STORM WATER RETENTION METHODS

There are circumstances where it is not possible or desirable to remove storm water from a site as quickly as it collects. Some built up urban areas generally have sewer systems originally designed for low projected land development and population density, which may result in the design outflow from the site exceeding the permitted flow into the sewer or the environment. Therefore, many cities have adapted a storm water management program to limit the outflow of storm water from developed sites to a flow rate equal to that before the development was started. Similarly, if a small stream is used as an outfall, it may not be possible to drain a large paved area that was formerly woodland into this stream without flooding the downstream portion.

In these circumstances, some method of temporarily storing the storm water must be provided. The methods used most often are to retain the storm water on the roof, on the site, or both.

ROOF RETENTION

Refer to Chap. 9, Plumbing Systems, section entitled, Limited Discharge Roof Drainage Systems, for the method used to retain water on the roof.

SITE RETENTION METHODS

Storage of water on roofs, if allowed, provides only a small portion of the retention generally required for an entire site drainage system. Therefore, additional provisions must be made. For smaller sites where space is limited, this is best accomplished by temporarily retaining the excess storm water in a retention basin either at or below ground. Another popular method is to greatly oversize the discharge piping, in effect, using the pipe to store the required volume. On-ground retention is called *ponding.*

The least costly method is the oversized pipe. Substituting a 72-in (DN 1800) or larger pipe for the designed size is a possibility if the ground is deep enough to accommodate it. If not, a retention basin should be considered.

Several factors should be considered in the selection of a retention basin. First is cost. An owner may want a pond in an out-of-the-way corner of the site rather than the more expensive retention basin. Second is available depth. The depth of an underground basin may be limited by the invert of the sewer to which it would be connected, or the depth of the stream or river which would provide the final outfall. In many cases, an underground basin may be the only solution in the development of an urban site, possibly with a pump to reach a higher sewer line.

The design of a retention basin begins with the concept that the total volume of storage required will be the difference between storm water inflow and allowable outflow for a given period of time. This requires the calculation of three variables:

1. Outflow from basin, which varies as a function of time

2. Inflow to basin, which varies as a function of time

3. Storage, which is the difference between 1 and 2 above for a specific time period

The information for retention basin design has been extracted from Technical Release Number 55, U.S. Department of Agriculture, Soil Conservation Service. The methodology differs from that of the Rational Method for storm water design. For example, total volume of rainwater produced by a given storm during a particular time is used rather than rate of rainfall in inches per hour.

When original agricultural and wooded areas are replaced with new, less impervious surfaces such as buildings and roads, both the volume and the peak rate of discharge will increase. The additional peak rate of discharge for the storm period is the amount of water to be stored on site while being released at a lower predetermined flow. Following are brief explanations of the factors to be considered in obtaining a reliable estimate of the peak rate of discharge and total runoff volume:

1. *Storm distributions.* There are two storm distributions that are used for different climatic regions: Type I and Type II. Type I storms are located in maritime climates and are found only in limited areas west of the Rocky Mountains. Type II storms are found in regions where high rates of runoff are usually generated from summer thunderstorms or tropical storms.

2. *Hydrologic soil group.* There are groups of soils that have the same runoff potential under similar storm conditions. Nationwide, over 8000 specific soils have been classified into four soil groups as follows:

Group A: Low runoff potential soils having a high infiltration rate even when thoroughly wet and consisting chiefly of deep, well- to excessively drained sand or gravel

Group B: Soils having a moderate infiltration rate when wet and consisting chiefly of moderately deep to deep, moderately well- to well-drained soil with moderately fine to moderately coarse texture

Group C: Soils having a slow infiltration rate when wet and consisting of soil with a layer that impedes downward movement of water, or soil with moderately fine to fine texture

Group D: High runoff potential soils having a very slow infiltration rate when wet and consisting chiefly of clay soil with a high swelling potential, soil with a high permanent water table, soil with a clay pan or clay layer at or near the surface, and shallow soil over nearly impervious material

Caution is advised when determining the soil group for the final developed conditions, as machine compaction or removal of upper soil layers can change the group drastically. Advice of the Soil Conservation Service is recommended when determining the soil group.

3. *Average watershed slope.* The steeper the average slope of the watershed, the greater the peak discharge will be.

4. *Land use.* The most common effect of urbanization is reduced infiltration, resulting in increased runoff volume, decreased overland travel time, and higher peak rates of flow. The volume of runoff is governed primarily by infiltration characteristics, and is related to soil type, type of vegetative cover, impervious surfaces, and surface retention. Travel time is governed by slope, flow length, and surface roughness. The peak discharge is based on the relationship of these factors.

5. *Runoff curve number* (*CN*). This is a dimensionless number selected for use in calculations that takes into account the previous land use information. The CN allows information to be conveyed in easy tabular or graphical form.

6. *Amount of rainfall.* The criteria for the selection of rainfall quantity used to determine the design parameters for the storage requirements are different than those used to design the piping system. Total rainfall is used rather than the instantaneous rate. In fact, the total rainfall for an entire 24-h period is used because of a phenomenon called *abstraction,* which is the sum of interception, depression storage, and infiltration. *Interception* is rain caught by foliage, leaves, and twigs that evaporates before it reaches the ground. *Depression storage* is rain caught in low points on grade and thus not available as runoff. *Infiltration* is water absorbed into the ground. Investigations have shown that for areas having a CN of 60 to 65, 2 in (50 mm) of rain must fall before runoff even starts. For this reason, a 24-h period is chosen. A heavy concentration of rain falling later in a given storm event will produce a greater peak discharge than at the beginning of the same storm.

The 24-h time period is also used to determine total volume of runoff for the same reasons. As will be seen, modifications and assumptions must be made to properly estimate runoff from the rainfall amount chosen.

The first step in retention basin design is to calculate the peak rate of discharge from the existing site and then from the site as it will be developed. To do this, the CNs of both the original and developed site must be found. When there is only one type of land use and one soil group, this is easy. Otherwise, a weighted CN must be found. This is done by determining the CN for each type of soil condition found and the number of acres for each CN, and multiplying these two figures. The sum of all the products is divided by the total area to find the weighted CN.

EXAMPLE A site consists of 100 acres with two types of land use. One type is 70 acres of pasture land in good condition with soil group C, and the other type is 30 acres of grain field with soil group B. Find the weighted CN for the site.

solution Refer to Table 6.4 to find the runoff CN. The pasture has a CN of 88, the grain field, a CN of 71.

$$\begin{array}{rlr} 70 & \text{acres} \times 88 = & 6160 \\ \underline{+30} & \text{acres} \times 71 = & \underline{+2130} \\ 100 & & 8290 \end{array}$$

$$\frac{8290}{100} = 82.9 \text{ (rounded to 83)}$$

Therefore, 83 is weighted CN for the entire site.

A similar calculation is made to find the CN for the developed site.

1. After the CN has been determined for both the undeveloped site and the developed site, the rate of rainfall and return period should be selected. Local rainfall standards must be used. However, if no standards exist, the maps in Figs. 6.18 (10-year, 24-h rainfall), 6.19 (25-year, 24-h rainfall), 6.20 (50-year, 24-h rainfall), and 6.21 (100-year, 24-h rainfall) should be used. Interpolation between isobars is necessary for intermediate values. A recommended average storm would be a 50-year, 24-h rainfall, Fig. 6.20.

2. Next, an adjustment to the 24-h rainfall figure must be made to allow for abstraction and the other considerations that were made when the CN was determined. Table 6.5 reduces the amount of runoff depth according to the various factors previously discussed.

TABLE 6.4 Runoff Curve Number

Land use description	Average % impervious§	A	B	C	D
		Hydrologic soil group			
Cultivated land*:					
without conservation treatment		72	81	88	91
with conservation treatment		62	71	78	81
Pasture or range land:					
poor condition		68	79	86	89
good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or forest land:					
thin stand, poor cover, no mulch		45	66	77	83
good cover†		25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition: grass cover on 75% or more of the area		39	61	74	80
fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (75% impervious)		81	88	91	93
Residential‡:					
Average lot size	Average % impervious§				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.¶		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers¶		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

*For a more detailed description of agricultural land use curve numbers refer to *National Engineering Handbook,* Section 4, Hydrology, Chapter 9, Aug. 1972.

†Good cover is protected from grazing and litter and brush covered soil.

‡Curve numbers are computed assuming the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur.

§The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

¶In some warmer climates of the country a curve number of 95 may be used.

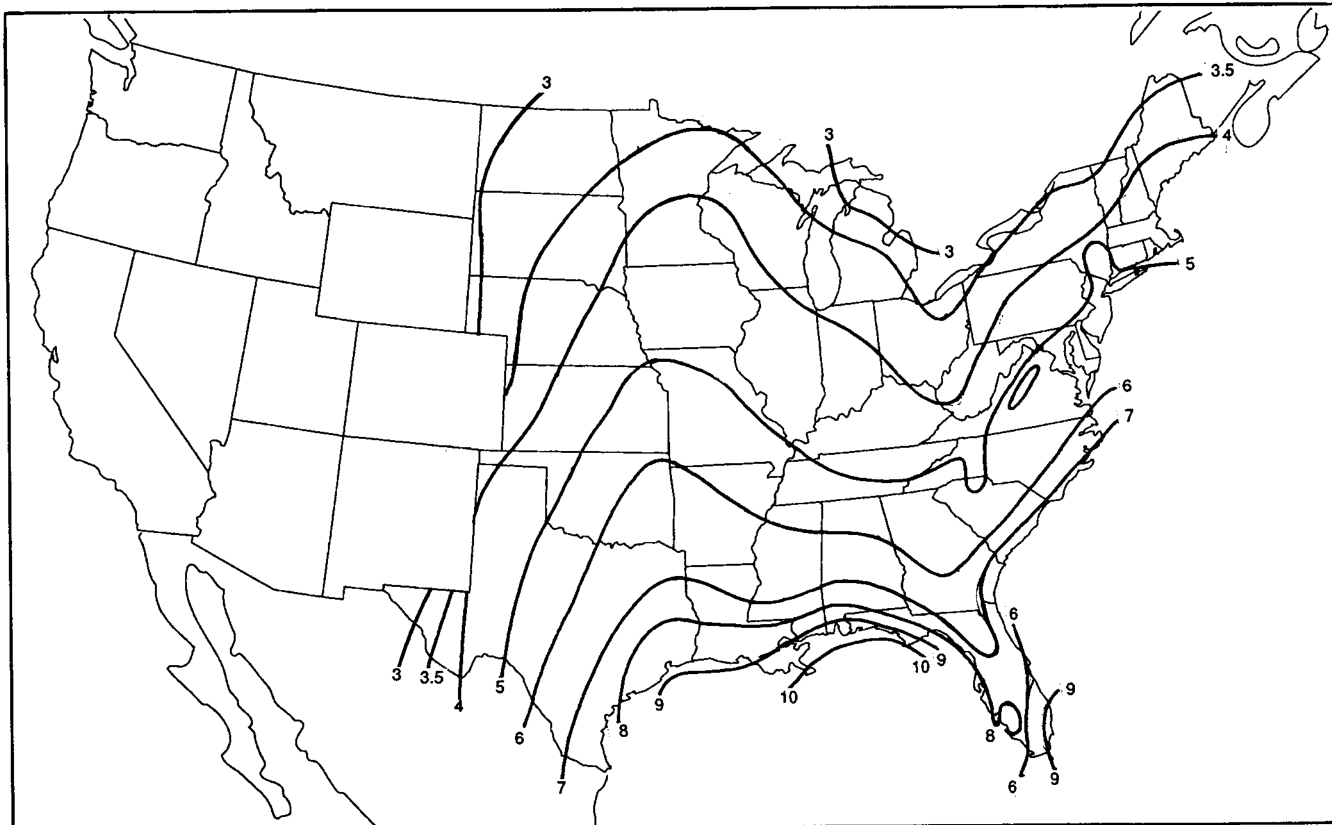


FIGURE 6.18 10-year, 24-hour rainfall. (*Adapted from U.S. Weather Bureau map.*)

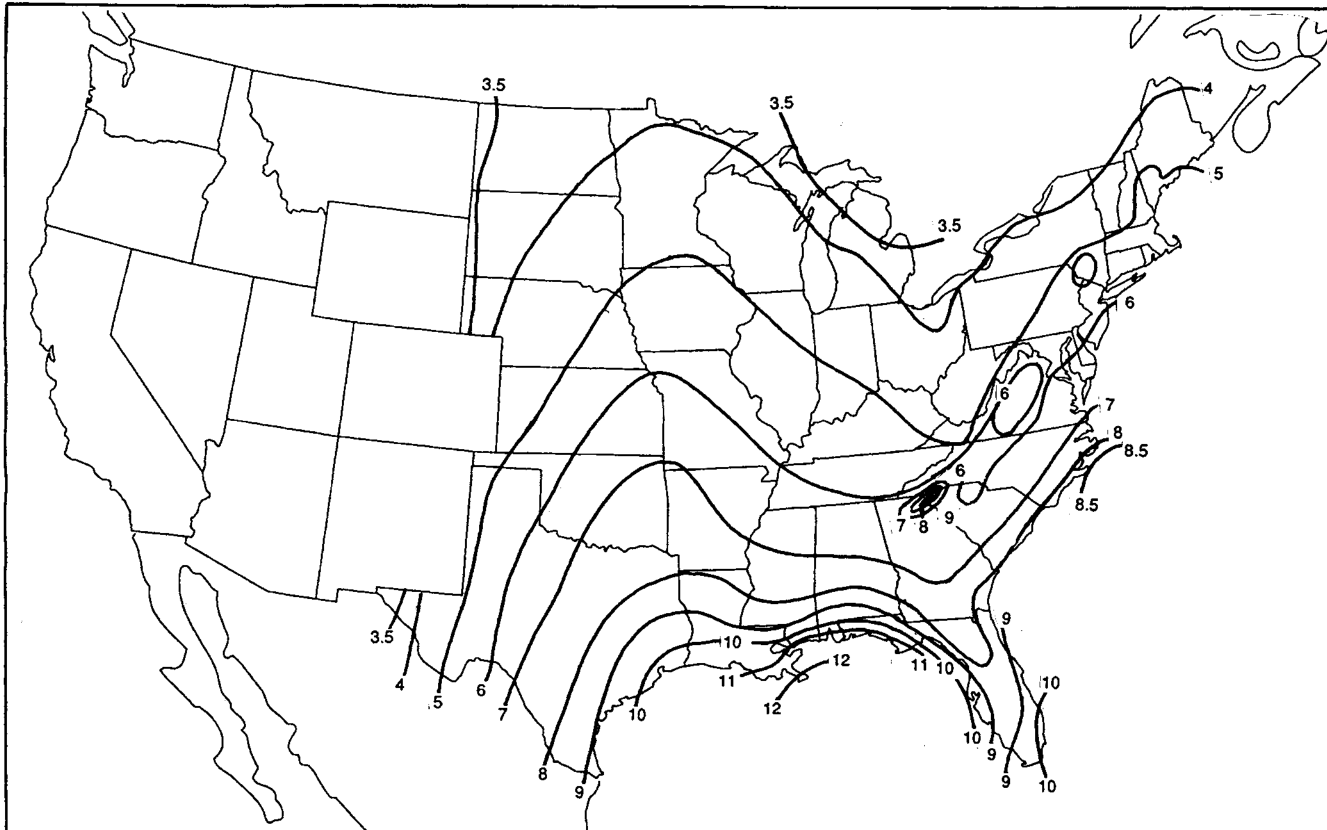


FIGURE 6.19 25-year, 24-hour rainfall. (*Adapted from U.S. Weather Bureau map.*)

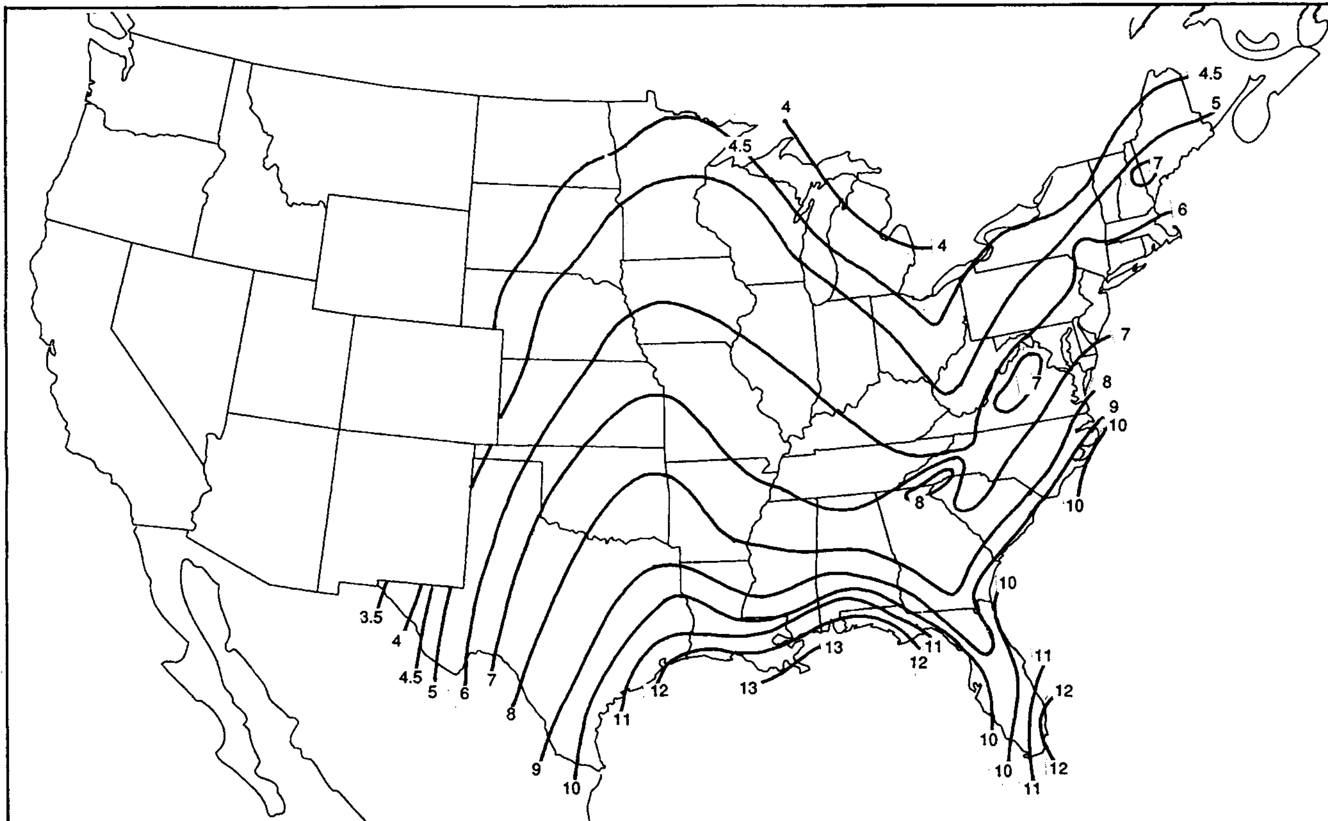


FIGURE 6.20 50-year, 24-hour rainfall. (*Adapted from U.S. Weather Bureau map.*)

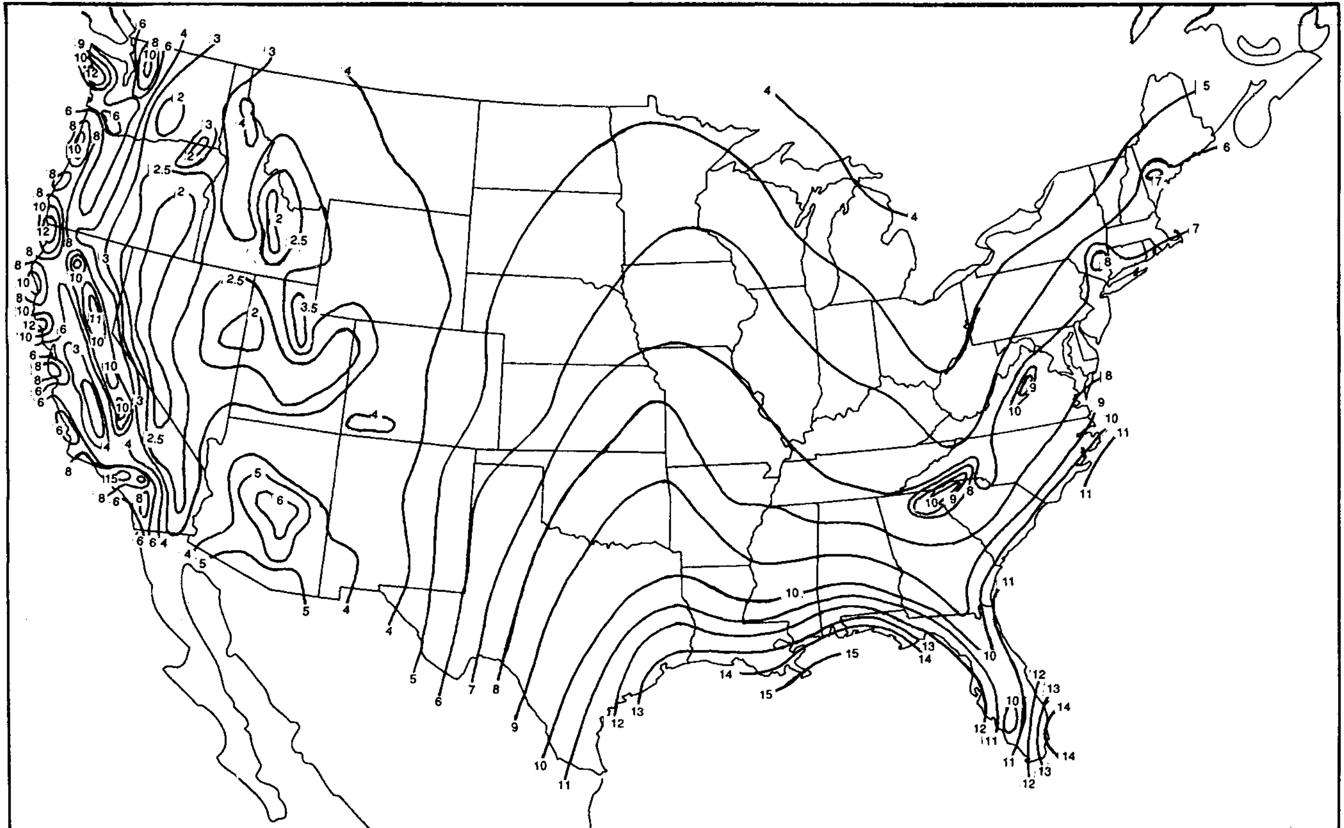


FIGURE 6.21 100-year, 24-hour rainfall. (*Adapted from U.S. Weather Bureau map.*)

TABLE 6.5 Reduction of Runoff Depth

Rainfall, in	Curve number (CN)*								
	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	0.56	0.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	0.74	0.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	0.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

*To obtain runoff depths for CNs and other rainfall amounts not shown in this table, use an arithmetic interpolation.

†1 in = 24.5 mm.

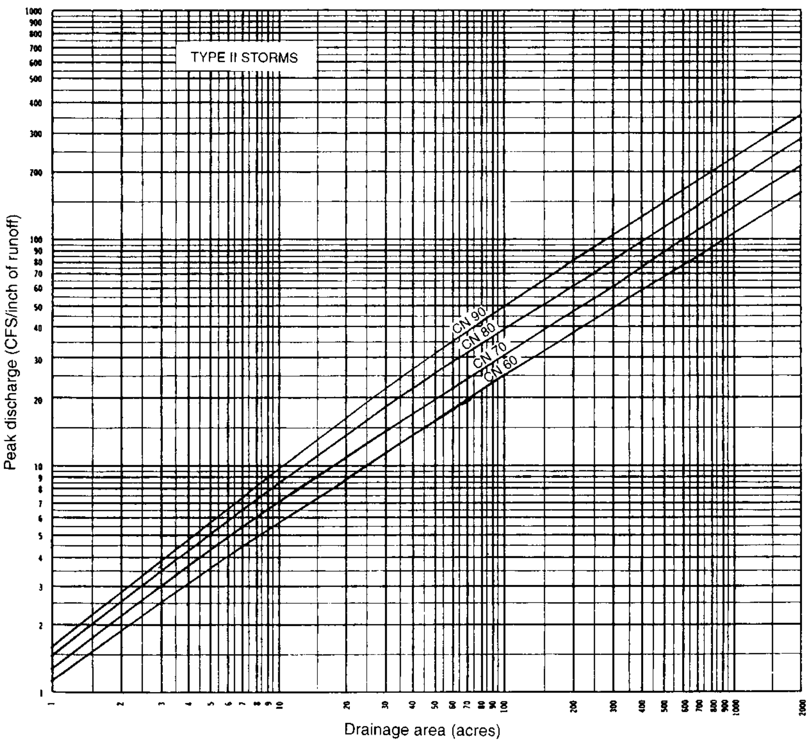


FIGURE 6.22 Adjustment to peak rate of discharge, flat slope, up to 2 percent.

3. Another item required for the determination of peak rate of discharge for the site is the adjustment to rainfall based on the average slope of the watershed. This is the average slope of grade to the storm water inlets, not of the piping system. Figures 6.22 (up to 2 percent slope), 6.23 (up to 7 percent slope), and 6.24 (up to 50 percent slope) give the discharge in cfs per inch of adjusted rainfall on the site, according to the average slope for Type II storms. Table 6.6 gives adjustment factors for intermediate slopes other than those found in Figs. 6.22 to 6.24. Type I storm tables are not available.

With the CN calculated, it is now possible to calculate the peak rate of discharge for the site. For example, a 300-acre site is to be developed in the northern part of New Jersey. The present weighted CN has been determined to be 75. The average slope of the watershed is found to be 4 percent. A 50-year, 24-h storm has been selected.

1. The actual rainfall must be found. From Fig. 6.20, for a 50-year, 24-h storm, 6 in of rain is read.

2. The actual rainfall must be adjusted. From Table 6.5, for present CN of 75 and 6 in of rain, an adjusted figure of 3.28 in of rain is used. This number is known as VR.

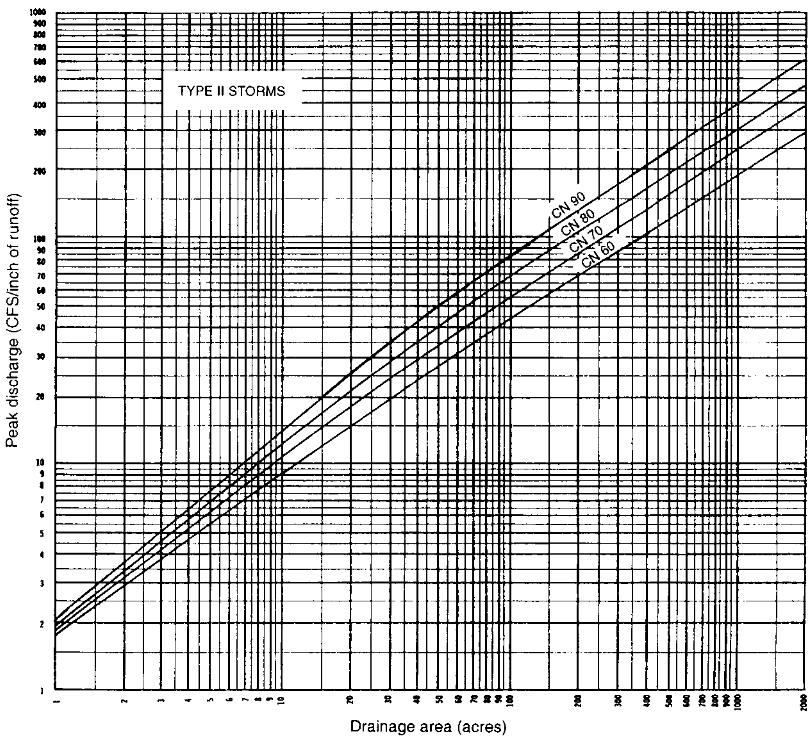


FIGURE 6.23 Adjustment to peak rate of discharge, moderate slope, up to 7 percent.

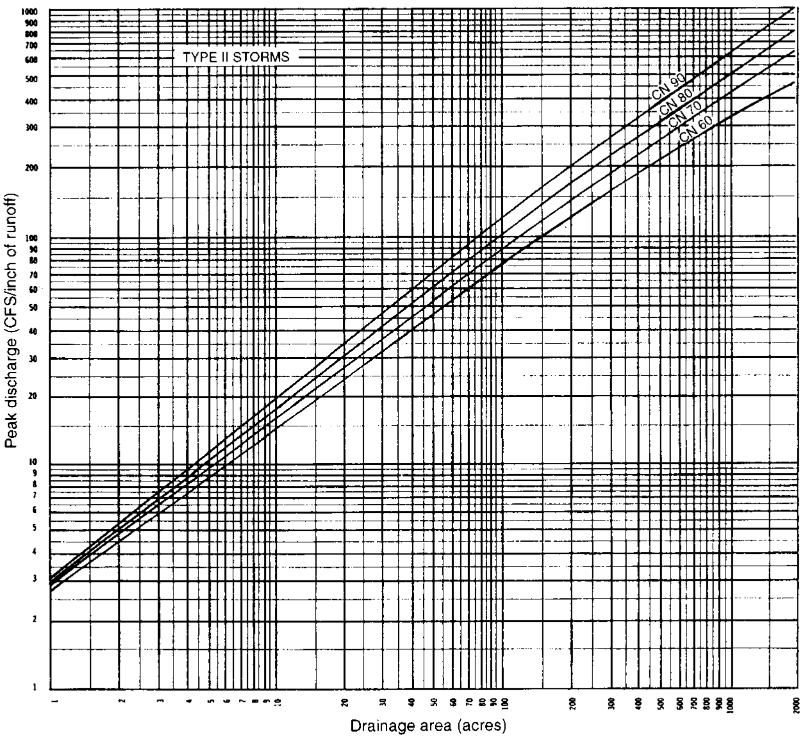


FIGURE 6.24 Adjustment to peak rate of discharge, steep slope, up to 50 percent.

3. The actual peak discharge must be determined based on original slope of the site. From Fig. 6.23 (for a 4 percent slope) enter with a CN of 75 and a watershed area of 300 acres. Read 125 cfs per in of rainfall. Therefore, $125 \times 3.28 = 399$ cfs peak discharge for the undeveloped 300-acre site. (Table 6.6 should be used for intermediate slope values, if required.)

To establish the peak discharges for the proposed development, two additional adjustments must be made to the rainfall amount. The first adjustment will increase the peak runoff due to the amount of impervious area that will replace the undeveloped land. The second will increase the peak runoff due to the faster overland flow rate that will result from a change in surface characteristics. This results in a shorter time of concentration. Continuing with the site as before, the next part of the procedure follows.

1. The future condition weighted CN must be calculated. It is found to be 80.

2. Assuming the same slope, the actual peak discharge must be found for future condition. Using Fig. 6.23, this time entering with a CN of 80 and 300 acres, 140 cfs per inch of rainfall is read. The future base discharge is $140 \times 3.28 = 459$ cfs peak discharge.

TABLE 6.6 Adjustment to Peak Rate of Discharge, Intermediate Slopes

Slope (percent)	10 acres	20 acres	50 acres	100 acres	200 acres	500 acres	1000s acres	2000 acres
				Flat slopes				
0.1	0.49	0.47	0.44	0.43	0.42	0.41	0.41	0.40
0.2	0.61	0.59	0.56	0.55	0.54	0.53	0.53	0.52
0.3	0.69	0.67	0.65	0.64	0.63	0.62	0.62	0.61
0.4	0.76	0.74	0.72	0.71	0.70	0.69	0.69	0.69
0.5	0.82	0.80	0.78	0.77	0.77	0.76	0.76	0.76
0.7	0.90	0.89	0.88	0.87	0.87	0.87	0.87	0.87
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.13	1.14	1.14	1.15	1.16	1.17	1.17	1.17
2.0	1.21	1.24	1.26	1.28	1.29	1.30	1.31	1.31
				Moderate slopes				
3	0.93	0.92	0.91	0.90	0.90	0.90	0.89	0.89
4	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	1.04	1.05	1.07	1.08	1.08	1.08	1.09	1.09
6	1.07	1.10	1.12	1.14	1.15	1.16	1.17	1.17
7	1.09	1.13	1.18	1.21	1.22	1.23	1.23	1.24
				Steep slopes				
8	0.92	0.88	0.84	0.81	0.80	0.78	0.78	0.77
9	0.94	0.90	0.86	0.84	0.83	0.82	0.81	0.81
10	0.96	0.92	0.88	0.87	0.86	0.85	0.84	0.84
11	0.96	0.94	0.91	0.90	0.89	0.88	0.87	0.87
12	0.97	0.95	0.93	0.92	0.91	0.90	0.90	0.90
13	0.97	0.97	0.95	0.94	0.94	0.93	0.93	0.92
14	0.98	0.98	0.97	0.96	0.96	0.96	0.95	0.95
15	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.98
16	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10
25	1.06	1.08	1.12	1.14	1.15	1.16	1.17	1.19
30	1.09	1.11	1.14	1.17	1.20	1.22	1.23	1.24
40	1.12	1.16	1.20	1.24	1.29	1.31	1.33	1.35
50	1.17	1.21	1.25	1.29	1.34	1.37	1.40	1.43

3. The number of additional acres that will be covered with an impervious cover must now be determined and expressed as a percent compared to the existing condition. This includes areas that will be developed by adding buildings, roads, parking lots, and the like. Assuming that 25 percent will be covered, Fig. 6.25 gives the adjustment factor based on percent of imperviousness. Entering with the CN of 80 and percentage of new impervious area of 25, 1.16 is read.

4. The length of run along the slope of the new development that is modified, and which will affect the overland flow time, must also be determined and expressed as a percent of the total length. Assuming that 50 percent of the original hydraulic length is modified, Fig. 6.26 gives the adjustment factor based on hydraulic length modified. Entering with a CN of 80 and 50 percent length, 1.31 is read.

5. The future condition peak discharge can now be found by multiplying the base peak discharge by both the impervious area factor and the hydraulic length modified factors. Thus,

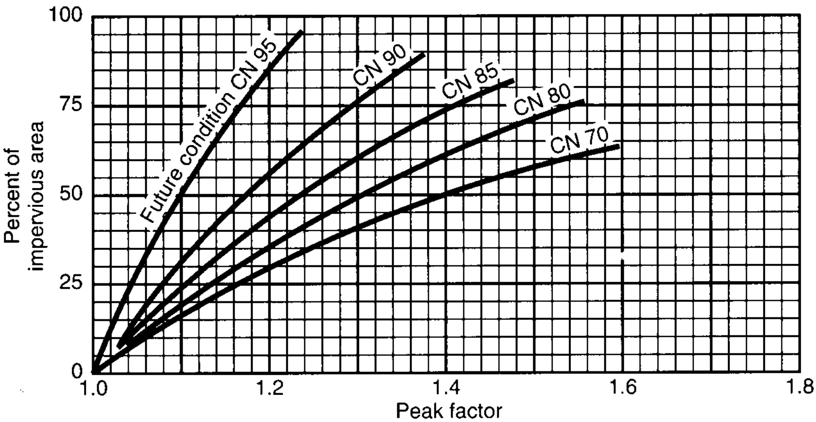


FIGURE 6.25 Factor for adjusting future condition runoff CN based on percent of imperviousness.

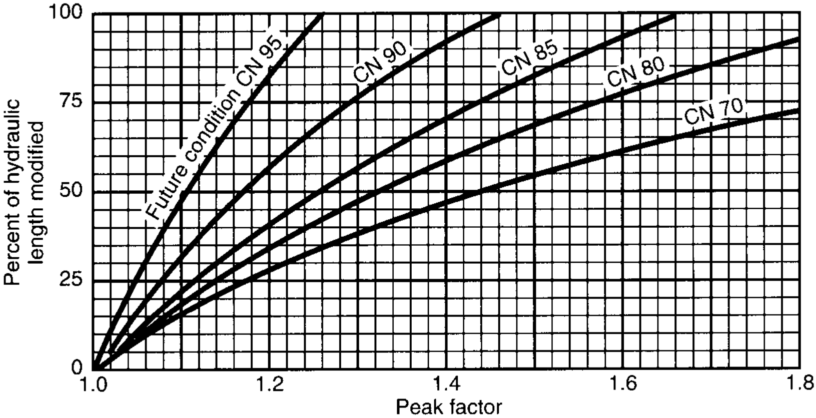


FIGURE 6.26 Factor for adjusting future condition runoff CN based on hydraulic length modified.

$$459 \times 1.16 \times 1.31 = 697.5 \text{ (rounded to 698 cfs)}$$

The effect of this proposed development is to increase the peak discharge from 399 cfs to 698 cfs.

Now that the basic watershed parameters have been established, the final design of the storage basin can proceed. This method is not as accurate as the computer program developed by the Soil Conservation Service for this purpose, but Eq. (6.6) will provide a conservative approach based upon average storage effects on peak discharges.

$$V_B = \frac{V_S \times A}{12} \tag{6.6}$$

where V_B = volume of storage basin, acre ft
V_S = volume of storage, watershed in
A = area of watershed, acres (note: 640 acres = 1 square mile)
12 = inches in 1 ft (for conversion of VS into acre ft)

Discussion:

1. Figures 6.27 and 6.28 are graphs giving the solution for the single-stage structure routing. Figure 6.27 is used for an allowable peak discharge of up to 300 cubic feet per second per mile (csm) when a pipe is used as the outlet, or 150 csm for a weir outlet. Figure 6.28 is for peak discharges above those amounts. To convert cfs to csm, use the following formulas:

$$\frac{\text{cfs} \times 640}{\text{watershed area in acres}} = \text{csm} \tag{6.7}$$

$$\frac{\text{cfs}}{\text{area in miles}^2} = \text{csm} \tag{6.8}$$

2. To use Fig. 6.27, two parameters must be known before entering the graph. One is the allowable peak discharge from the basin in csm, and the second is the

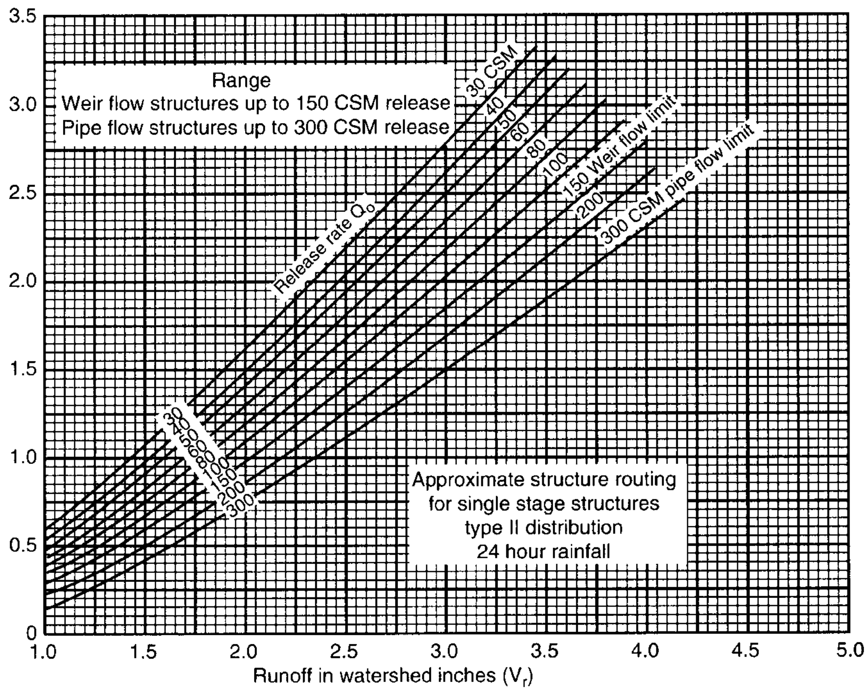


FIGURE 6.27 Single-stage structure routing, up to 300 csm.

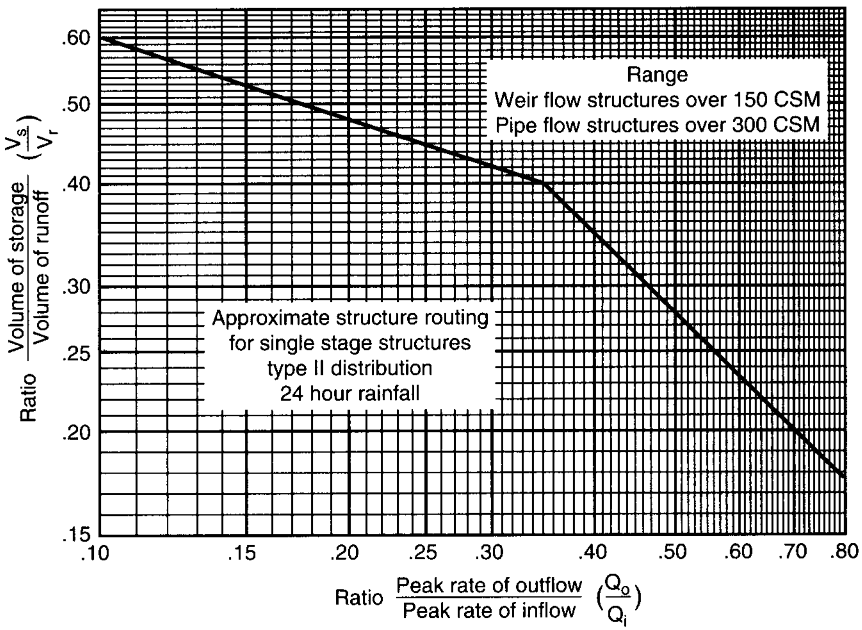


FIGURE 6.28 Single-stage structure routing, over 300 csm.

adjusted rainfall figure (VR) from Table 6.5. Entering the figure with each of these numbers, V_S is found for use in Eq. (6.6).

3. To use Fig. 6.28, additional steps are required due to different parameters used to construct the graph. The peak rate of outflow (Q_O) and peak rate of inflow (Q_I) are known from previous calculations. The allowable peak discharge from the basin in csm must also be calculated, but only to use the appropriate figure. Also, the adjusted rainfall figure (VR) from Table 6.5 must be found. Entering Fig. 6.28 with the ratio of Q_O over Q_I, reading up to where it intersects the heavy line, and then left, the actual ratio of storage volume over runoff volume is found. With the ratio now known, the ratio just found is multiplied by VR (adjusted rainfall figure from Table 6.5) to find V_S used in Eq. (6.6). The following examples show how Figs. 6.27 and 6.28 are used.

EXAMPLE 1 Continuing with the 300-acre site, assume the maximum permissible discharge from the site equal to that established for the undeveloped site. This is 399 cfs. A pipe will be used for discharge.

1. Convert cfm to csm (using Eq. 6.7):

$$\frac{399 \times 640}{300} = \frac{255{,}360}{300} = 851.2 \text{ (rounded to 852 csm)}$$

Figure 6.28 must be used since the result is greater than 300 csm.

2. Find ratio:

$$\frac{Q_I}{Q_O} = \frac{399}{698} = 0.507 \text{ (rounded to 0.51)}$$

3. Determine VR: From above, 3.28 in of rain is used.

4. Find V_S: First enter Fig. 6.28 with $Q_O/Q_I = 0.51$. Read 0.27 ratio on the left side of chart. Therefore:

$$V_S = 3.28 \times 0.27 = 0.88$$

5. Find basin volume [Eq. (6.6)]:

$$V_B = \frac{0.88 \times 100}{12} = \frac{88}{12} = 7.3 \text{ acre ft}$$

EXAMPLE 2 A 0.4 square mile watershed with an adjusted rainfall of 3.28 in is required to limit discharge to 103 cfm through a piped outlet. What basin volume is required?

1. Find csm [use Eq.(6.8)]

$$\frac{103}{0.4} = 258 \text{ csm}$$

Therefore use Fig. 6.27 since this is less than 300 csm.

2. Find V_S. Entering Fig. 6.27 with V_R of 3.28, find V_S of 1.55.

3. Find V_B. Use Eq. (6.6).

$$V_B = \frac{V_S \times A}{12} = \frac{1.55 \times 640 \times 0.4}{12} = 33.1 \text{ acre ft}$$

To convert from acre feet to cubic feet, multiply acre feet by 43,560 and divide the product by 9.

The following design criteria must be established to select the size of the retention basin and the size of the outflow pipe:

1. Allowable cfs to outflow pipe
2. Height from the top of the basin to centerline of outlet pipe
3. Location of the retention basin
4. Completed site drainage system
5. Trial size of outlet pipe
6. Trial slope of outlet pipe

First, the retention basin must be located on the site plan. The size restrictions must be determined, if any, and then a trial depth found. Next, assuming a trial size pipe, one-half of the pipe diameter is subtracted from the basin depth. Also, at this time, the rate of discharge from the site to the outfall must have been definitely established. With the above information, the size discharge pipe can be calculated from the following orifice formula. Note that the allowable discharge will only occur when the design head is reached.

$$A = \frac{Q}{C \times 2GH} \tag{6.9}$$

where A = area of outlet pipe, ft^2 (refer to Table 6.7)
Q = cfs discharge allowable
C = orifice coefficient (use 0.60, an average value)
G = 32.2, acceleration due to gravity, 32.2 ft/sec^2
H = design head from surface of water to center of outlet pipe

To convert pipe size from square feet to diameter in feet the following formula is used:

$$D = \frac{4A}{\sqrt{\pi}} \tag{6.10}$$

where D = interior diameter of pipe, ft (Table 6.7)
A = interior area of pipe, ft^2 (Table 6.7)
π = 3.14

Refer to Table 6.7 to select design properties of pipe. If an exact cfm discharge to the outfall is required, using a standard size pipe, the depth of the basin can be adjusted to achieve the desired result. If the height of the basin is fixed, an orifice plate of the exact calculated size can be cut and installed in the outlet pipe. If a weir outlet is to be used, the outflow is determined by the shape of the weir and height of water over the weir bottom. Standard references should be consulted for discharge through such weir openings.

TABLE 6.7 Design Properties for Pipe

DN	Nominal size, in	Inside area, ft^2	Nominal I.D., ft	Outside diameter, ft						
				Cast iron	Ductile iron	Plain concrete	Reinforced concrete	Clay	Corrugated steel (⅛-in corr.)	Composite
250	10	0.545	0.84	0.89	0.93	1.04		1.04	0.92	0.94
300	12	0.785	1.00	1.06	1.10	1.29	1.33	1.25	1.08	1.13
350	14	1.18	1.18		1.28					1.33
375	15	1.227	1.25	1.32		1.56	1.62	1.54	1.33	1.44
400	16	1.53	1.33		1.46					1.52
450	18	1.767	1.50		1.62	1.87	1.92	1.86	1.58	1.68
500	20	2.26	1.70		1.82					1.87
525	21	2.405	1.75			2.20	2.20	2.14	1.83	1.98
600	24	3.142	2.00		2.12	2.62	2.62	2.39	2.08	2.27
700	27	3.976	2.25			2.92	2.92	2.77	2.33	2.55
750	30	4.909	2.50		2.66	3.20	3.20	2.08	2.58	2.84
800	33	5.940	2.75			3.50	3.50	3.31	2.83	3.14
900	36	7.069	3.00		3.19	3.79	3.79	3.66	3.08	3.39

DESIGNING BURIED PIPING

This section describes the criteria and methods used in the design and installation of underground piping on a site. The loads placed on the pipe, the method of calculating these loads, and determination of the required supporting strength of the pipe will be discussed.

GENERAL

All buried pipe is subject to stresses imposed upon it by the nature of burial. Such factors as bedding methods, type of backfill material, type and shape of trench, loading from live loads, and storage of material over the trench will each contribute to the total load transmitted to the pipe and the ability of the pipe to support such loads. The pipe selected, combined with the method of installation, must have the strength to withstand the entire load placed upon it without crushing, cracking, or deforming.

CODES AND STANDARDS

1. ASTM and ANSI standards for pipe and installation
2. AWWA standards for pipe, installation, and disinfection
3. NFPA-20 standard for installation of private water mains

PIPE AND INSTALLATION CLASSIFICATIONS

Pipe Installation

Pipe installation is divided into three general classifications:

1. *Pipe in a tunnel.* This is a classification for pipe forced through the earth by a ram or other means that leaves the surrounding earth undisturbed.

2. *Pipe in an embankment condition.* This is a classification for pipe installed on the surface of the ground (positive projection) or in a trench (negative projection) with a layer of earth placed above the original ground surface.

3. *Pipe in a trench or ditch.* This is a classification for pipe installed in an excavation below grade, with the trench backfilled approximately to the original ground line. Trench loading will be the only condition considered here, since it is the most common means of installation.

Pipe Classifications

Piping is divided into two classifications, rigid and flexible. Rigid pipe material is concrete, composite, metal, or clay. Rigid pipe fails when it breaks under a three-edge-bearing test. Concrete pipe fails when the three-edge-bearing test produces a crack 0.01 in wide and 2 in long. Flexible pipe is plastic, composite, or corrugated steel. Flexible piping is considered to fail when loads produce a vertical deflection of 5 percent or more of pipe diameter.

LOADS ACTING ON A PIPE

Definitions

The *earth load* is the weight of all earth backfill over the pipe. The *superimposed load* is produced by either a moving object (live load) passing over the pipe or a uniformly distributed (dead) load placed at ground level over the pipe (in addition to the earth load). A uniformly distributed load and a live load acting concurrently on a pipe will not be considered.

Earth Loads

General. The earth load is the vertical force earth transmits to a pipe buried in a trench. The pipe is subject to a very complex relationship among many factors. The type of fill and the depth and width of a trench must all be considered. Original research at Iowa State College under the direction of Professor Anson Marston produced a formula for determination of earth loads. This, and additional research, resulted in Eqs. 6.11 and 6.12.

Earth Load Calculation. The earth load is obtained by selecting the lower result from either Eq. (6.11) or (6.12). Actual tests have proven that the loads for a positive projection condition should be used when it is the lower of the two even if the pipe is placed in a trench.

$$L_e = C_p\, W\, D \text{ (positive projection condition)} \tag{6.11}$$

$$L_e = C_d\, W\, B \text{ (ditch condition)} \tag{6.12}$$

where L_e = earth load on the pipe (lb/ft of pipe length)
C_d = coefficient for load calculation of ditch condition
C_p = coefficient for load calculation of positive projection condition
W = weight of backfill, lb/ft^3
B = width of trench at top of pipe, ft
D = outside diameter of pipe, ft

Discussion:

1. To determine C_d or C_p, refer to Figs. 6.29 and 6.30 or 6.31,

where H = height of backfill over top of pipe, ft
D = outside diameter of pipe, ft
B = width of trench at top of pipe, ft

Curves A, B, C, D, and E represent the following soils in Fig. 6.30.

A = sand and sandy loam

B = sand and gravel

C = saturated topsoil

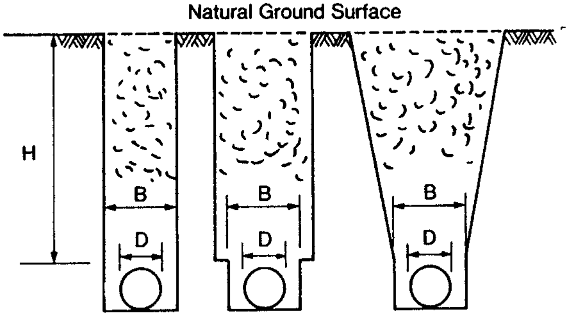


FIGURE 6.29 Installation conditions for earth load calculations.

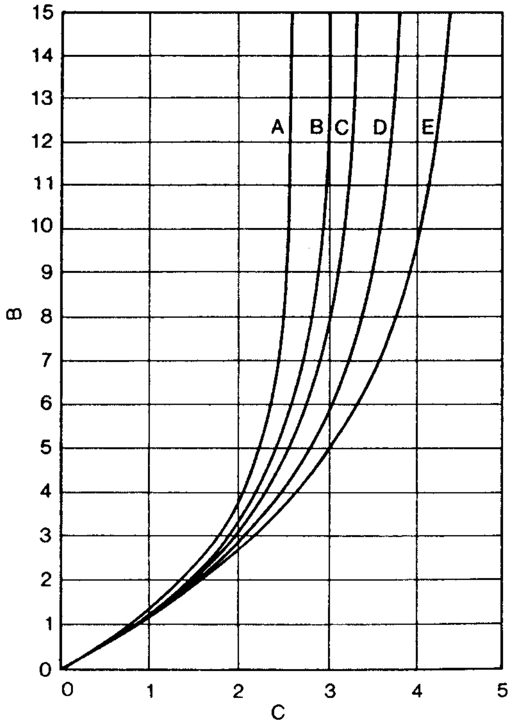


FIGURE 6.30 Calculation coefficients for ditch condition. (*Courtesy: ANSI 21.1.*)

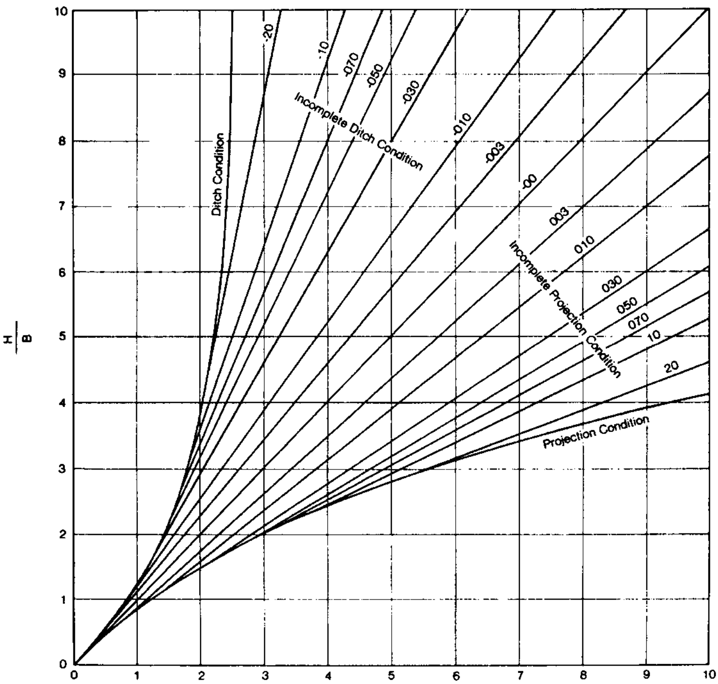


FIGURE 6.31 Coefficient for positive projection condition. (*Courtesy: ANSI 21.1.*)

D = average clay

E = saturated clay

Curve 0.70 is an average value used for standard calculations in Fig. 6.31 (incomplete projection condition).

2. To determine *W*, refer to Table 6.8.

3. Trench width has an important influence on the soil load transmitted to the pipe. Any trench must provide sufficient working space for the installation with tolerance allowed for accepted construction practices and field conditions. Experi-

TABLE 6.8 Weight of Soil

Type of soil	Kg/m^3	Weight, lb/ft^3	Correction factor	Average moisture content, %
Sand and sandy loam	1600	100	0.83	15
Sand and gravel mix	1760	110	0.90	20
Saturated topsoil	1840	115	0.95	25
Average clay	1920	120	1.00	30
Wet clay, peat	2080	130	1.10	100

ments have proven that only the trench width at the top of the pipe need be considered in calculating the earth load. If the trench is widened above the top of pipe, it does not contribute any additional load to the buried piping. If sheeting is used, the width of the trench is computed to the inside of the sheeting if it is to be removed. It is recommended that the sheeting remain, if possible.

At any given depth, for a particular size pipe and type of soil, a trench width is reached beyond which there is no longer any additional load added to the buried pipe. This is called *transition width,* and is the widest dimension that need be considered to compute the earth load. Table 6.9 gives the transition width for various combinations of pipe size and depth of burial.

It is common practice to allow 1 ft (0.3 m) on either side of a pipe in a trench for working space. Figure 6.32 provides direct determination of earth load for this condition. If soil weight is other than 120 lb/ft^3 (1920 kg/m^3), proportionally decrease or increase the resulting weight according to the correction factor in Table 6.8.

SUPERIMPOSED LOADS

Live Loads

Whenever pipe is buried in the ground there is always the possibility of heavy equipment or trucks riding over the buried pipe during construction. When pipe is placed under a road or railroad, this is a normal occurrence. This load must be added to the earth load to determine the total force exerted on the buried pipe. The general contractor should be consulted to determine if piping will be buried to its full depth during construction or if additional cover will be added later.

A live load consists of two separate types of loads: the actual weight of any vehicle passing over the pipe and the impact load that places additional stress on the pipe due to the fact the load is moving.

Live Load from Trucks. The type of road surface over the pipe may have a reducing effect on the amount of live load reaching the pipe. A flexible surface such as light-duty asphalt will have little or no reducing effect. A concrete road or heavy-duty asphalt will greatly reduce the load intensity. Railroad tracks are constructed over a standard bed consisting of rock ballast and ties, which gives a uniform distribution of load to the piping.

For calculating the live load transmitted to a pipe under a road, the following formulas are used.

Unpaved or light-duty pavement:

$$L_t = CRPF \tag{6.13}$$

Rigid or heavy-duty pavement:

$$L_t = CBPF \tag{6.14}$$

where L_t = truck superload, lb/ft of pipe
C = surface load factor
R = reduction factor
P = wheel load

TABLE 6.9 Transition Width for Trenches

Cover over top of pipe*	Nominal pipe size, in												
	10	12	14	15	16	18	20	21	24	27	30	33	36
2′0″	1′9″	1′11″	2′1″	2′2″	2′3″	2′5″	2′8″	2′9″	3′1″	3′5″	3′7″	3′10″	4′3″
2′6″	1′10″	2′0″	2′3″	2′4″	2′5″	2′8″	2′10″	2′11″	3′3″	3′6″	3′10″	4′1″	4′4″
3′0″	1′11″	2′3″	2′5″	2′6″	2′7″	2′10″	3′0″	3′2″	3′5″	3′8″	3′11″	4′3″	4′7″
4′0″	2′0″	2′5″	2′8″	2′10″	2′11″	3′1″	3′4″	3′6″	3′9″	4′1″	4′5″	4′6¨	4′11″
5′0″	2′2″	2′7″	2′11″	3′1″	3′2″	3′5″	3′8″	3′10″	4′1″	4′6″	4′8″	5′1″	5′3″
6′0″	2′5″	2′9″	3′1″	3′2″	3′4″	3′8″	3′11″	4′1″	4′5″	4′8″	5′0″	5′5″	5′8″
7′0″	2′5″	2′10″	3′2″	3′4″	3′6″	3′10″	4′1″	4′4″	4′8″	5′1″	5′5″	5′8″	6′0″
8′0″	2′6″	2′11″	3′3″	3′6″	3′7″	4′0″	4′3″	4′6″	4′10″	5′3″	5′7″	6′1″	6′2″
9′0″	2′8″	3′0″	3′5″	3′7″	3′9″	4′1″	4′5″	4′7″	5′0″	5′6″	5′10″	6′3″	6′8″
10′0″	2′9″	3′1″	3′6″	3′8″	3′11″	4′3″	4′6″	4′9″	5′2″	5′7″	6′1″	6′5″	6′10″
12′0″	3′0″	3′4″	3′7″	3′10″	4′0″	4′5″	4′10″	5′0″	5′5″	6′0″	6′5″	6′10″	7′4″
14′0″	3′1″	3′5″	3′11″	4′0″	4′2″	4′7″	5′1″	5′3″	5′8″	6′3″	6′9″	7′2″	7′8″
16′0″	3′2″	3′8″	4′0″	4′3″	4′5″	4′9″	5′2″	5′5″	5′11″	6′6″	7′0″	7′5″	7′11″
18′0″	3′3″	3′10″	4′3″	4′5″	4′7″	5′0″	5′4″	5′6″	6′1″	6′8″	7′3″	7′9″	8′2″
20′0″	3′5″	4′1″	4′4″	4′6″	4′9″	5′2″	5′7″	5′8″	6′2″	6′10″	7′5″	7′11″	8′6″

*1 ft = 0.3 m.

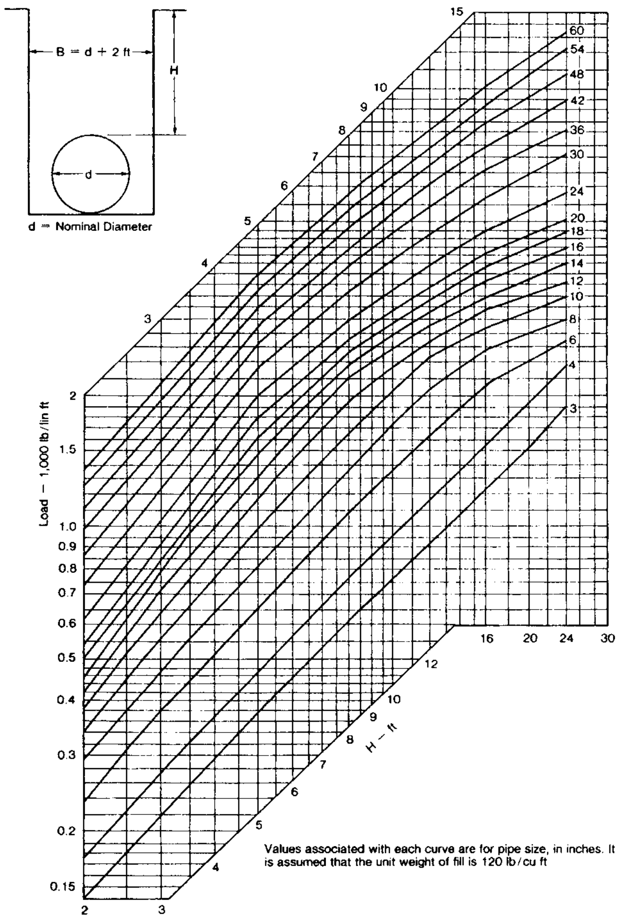


FIGURE 6.32 Earth load on pipe in trench. (*Courtesy: ANSI 21.1.*)

F = impact factor
B = outside diameter of pipe, ft

Discussion:

1. The surface load factor is the weight transmitted to the pipe from a truck on a road passing over the pipe. Tables have been prepared for both one truck and two trucks passing over the pipe simultaneously. The proper chart should be consulted based on road construction and number of trucks. Refer to Table 6.10 (one truck on flexible pavement, Table 6.11 (two trucks on flexible pavement), or Table 6.12 (one or two trucks on rigid pavement).

2. The reduction factor allows for the fact that adjacent portions of pipe that are not directly under the load assist in carrying some portion of the live load. Refer to Table 6.13.

TABLE 6.10 Surface Load Factor—One Truck on Flexible Pavement*

DN	Pipe size, in	Depth of cover, ft†												
		2	2½	3	3½	4	5	6	8	10	12	16	20	24
80	3	0.028	0.020	0.014	0.011	0.009	0.006	0.004	0.002	0.0015	0.001	0.0006	0.0004	0.0002
100	4	0.034	0.024	0.017	0.013	0.011	0.007	0.005	0.003	0.002	0.0015	0.0008	0.0005	0.0003
150	6	0.048	0.034	0.025	0.020	0.015	0.010	0.007	0.004	0.003	0.002	0.001	0.0007	0.0004
200	8	0.062	0.044	0.033	0.026	0.020	0.013	0.009	0.006	0.0035	0.0025	0.0013	0.0008	0.0005
250	10	0.074	0.054	0.040	0.031	0.025	0.016	0.012	0.007	0.004	0.003	0.0016	0.001	0.0006
300	12	0.087	0.063	0.048	0.036	0.030	0.019	0.014	0.008	0.005	0.0035	0.002	0.0012	0.0007
350	14	0.099	0.072	0.055	0.042	0.034	0.022	0.016	0.010	0.006	0.004	0.0025	0.0015	0.0008
400	16	0.110	0.082	0.061	0.047	0.038	0.025	0.018	0.011	0.007	0.005	0.003	0.0017	0.001
450	18	0.122	0.090	0.068	0.052	0.042	0.028	0.020	0.012	0.008	0.0055	0.0035	0.002	0.0012
500	20	0.132	0.098	0.075	0.058	0.046	0.031	0.022	0.013	0.009	0.006	0.004	0.0025	0.0015
600	24	0.150	0.113	0.087	0.068	0.054	0.037	0.026	0.015	0.010	0.007	0.0045	0.003	0.0017
750	30	0.171	0.132	0.102	0.081	0.065	0.045	0.031	0.019	0.012	0.009	0.005	0.0035	0.002
900	36	0.188	0.148	0.117	0.093	0.076	0.052	0.037	0.022	0.015	0.010	0.006	0.004	0.0025

*These factors are for a single concentrated wheel load centered over an effective pipe length of 3 ft.
†1 ft = 0.3 m.
Source: ANSI 21.1.

TABLE 6.11 Surface Load Factor—Two Truck on Flexible Pavement*

DN	Pipe size, in	Depth of cover, ft†												
		2	2½	3	3½	4	5	6	8	10	12	16	20	24
80	3	0.019	0.012	0.008	0.006	0.005	0.004	0.0035	0.003	0.0025	0.002	0.001	0.007	0.0005
100	4	0.032	0.022	0.016	0.012	0.009	0.006	0.005	0.004	0.0035	0.003	0.002	0.0013	0,0010
150	6	0.058	0.042	0.032	0.024	0.020	0.014	0.010	0.007	0.006	0.005	0.003	0.0019	0.0015
200	8	0.076	0.058	0.044	0.036	0.030	0.022	0.017	0.011	0.009	0.007	0.004	0.0027	0.0021
250	10	0.092	0.072	0.056	0.046	0.039	0.028	0.021	0.014	0.011	0.008	0.005	0.0033	0.0026
300	12	0.108	0.086	0.070	0.056	0.047	0.034	0.027	0.018	0.012	0.009	0.006	0.0039	0.0030
350	14	0.122	0.098	0.078	0.065	0.055	0.040	0.031	0.020	0.014	0.010	0.007	0.0046	0.0036
400	16	0.136	0.110	0.090	0.074	0.062	0.046	0.036	0.024	0.016	0.012	0.009	0.0060	0.0046
450	18	0.149	0.122	0.101	0.084	0.070	0.052	0.041	0.027	0.019	0.014	0.010	0.0066	0.0051
500	20	0.162	0.136	0.115	0.096	0.080	0.060	0.048	0.032	0.022	0.016	0.012	0.0079	0.0061
600	24	0.185	0.152	0.126	0.106	0.091	0.067	0.053	0.036	0.026	0.119	0.013	0.0086	0.0067
750	30	0.212	0.176	0.146	0.124	0.107	0.080	0.064	0.044	0.032	0.024	0.016	0.0106	0.0081
900	36	0.235	0.202	0.169	0.146	0.127	0.095	0.075	0.052	0.038	0.028	0.019	0.0125	0.0097

*These factors are for two trucks with 6-ft rear wheel spacing passing with inside rear wheels 3-ft apart. Effective pipe length is 3 ft coinciding with the distance between the adjacent inside wheels.

†1 ft = 0.3 m.

TABLE 6.12 Surface Load Factor—One or Two Trucks on Rigid Pavement*

Depth of cover, ft‡	Pavement thickness, in†							
	One truck				Two passing trucks			
	4	6	8	10	4	6	8	10
2	0.0244	0.0149	0.0101	0.0076	0.0410	0.0263	0.0186	0.0142
2½	0.0213	0.0139	0.0097	0.0072	0.0364	0.0246	0.0177	0.0136
3	0.0186	0.0126	0.0090	0.0070	0.0333	0.0228	0.1067	0.0129
3½	0.0164	0.0114	0.0085	0.0066	0.0290	0.0206	0.0156	0.0122
4	0.0144	0.0102	0.0079	0.0061	0.0262	0.0187	0.0146	0.0117
5	0.0114	0.0084	0.0066	0.0054	0.0210	0.0156	0.0123	0.0102
6	0.0093	0.0071	0.0057	0.0047	0.0170	0.0133	0.0107	0.0088
8	0.0065	0.0052	0.0043	0.0036	0.0114	0.0097	0.0081	0.0069
10	0.0046	0.0039	0.0033	0.0029	0.0080	0.0070	0.0062	0.0055
12	0.0034	0.0030	0.0026	0.0023	0.0059	0.0054	0.0049	0.0045
16	0.0022	0.0019	0.0017	0.0016	0.0034	0.0032	0.0030	0.0028
20	0.0013	0.0011	0.0010	0.0009	0.0024	0.0023	0.0022	0.0021
24	0.0008	0.0007	0.0006	0.0005	0.0015	0.0014	0.0013	0.0012

*These factors were computed by the methods explained in "Vertical Pressure on Culverts under Wheel Loads on Concrete Pavement Slabs," Portland Cement Assn., Chicago, IL in Bulletin ST65.
†1 in = 25 mm.
‡1 ft = 0.3 m.

Source: ANSI 21.1.

TABLE 6.13 Reduction Factor

Pipe size, in	Depth of cover, ft*			
	2½–3½	4–7	8–10	>10
3–12	1.00	1.00	1.00	1.00
14	0.92	1.00	1.00	1.00
16	0.88	0.95	1.00	1.00
18	0.85	0.90	1.00	1.00
20	0.83	0.90	0.95	1.00
24–30	0.81	0.85	0.95	1.00
36–60	0.80	0.85	0.90	1.00

*1 ft = 0.3 m.

Source: ANSI 21.1.

3. Wheel load is the actual weight of the truck and its cargo on a wheel passing over the pipe. Refer to Table 6.14. The H-20 loading is most commonly used.

4. Impact factor is a dynamic load caused by a moving object. Refer to Table 6.15 to determine the factor based on depth of burial.

5. Table 6.16 gives the outside diameter of pipe, based on pipe size and material.

TABLE 6.14 Truck Wheel Load

AASHTO truck	Gross weight, tons	Wheel load (P), lb*
H-10	10	8,000
H-15	15	12,000
H-20	20	16,000

*1 lb = 2.2 kg.

TABLE 6.15 Truck Impact Factor

Depth of cover*	Impact factor (F)
Up to 1 ft-0 in	1.30
1 ft-1 in to 2 ft-0 in	1.20
2 ft-0 in to 2 ft-11 in	1.10
3 ft-0 in or more	1.00

*1 ft = 0.3m.

TABLE 6.16 Outside Diameter of Pipe

Nominal size, in	Inside area, ft^2	Nominal I.D., ft	Outside diameter, ft						
			Cast iron	Ductile iron	Plain concrete	Reinforced concrete	Clay	Corrugated steel, ⅛ in	Composite
10	0.545	0.84	0.89	0.93	1.04	—	1.04	0.92	0.94
12	0.785	1.00	1.06	1.10	1.29	1.33	1.25	1.08	1.13
14	1.18	1.18		1.28	—	—	—	—	1.33
15	1.227	1.25	1.32		1.56	1.62	1.54	1.33	1.44
16	1.53	1.33	—	1.46	—	—	—	—	1.52
18	1.767	1.50	—	1.62	1.87	1.92	1.86	1.58	1.68
20	2.26	1.70	—	1.82	—	—	—	—	1.87
21	2.405	1.75	—		2.20	2.20	2.14	1.83	1.98
24	3.142	2.00	—	2.12	2.62	2.62	2.39	2.08	2.27
27	3.976	2.25	—		2.92	2.92	2.77	2.33	2.55
30	4.909	2.50	—	2.66	3.20	3.20	3.08	2.58	2.84
33	5.940	2.75	—		3.50	3.50	3.31	2.83	3.14
36	7.069	3.00	—	3.19	3.79	3.79	3.66	3.08	3.39

Live Load from Trains. For calculating the live load transmitted to a pipe under a railroad, refer to Table 6.17. Recommendations of the American Railway Engineering Association are used. This figure should be added to the weight of the track structure. The following criteria are used:

1. Copper E72 railroad design load is assumed.
2. The weight of track structure and ballast is assumed to be 200 lb/ft
3. An impact factor of 1.4 is used, decreasing to 1.0 with 10 ft of cover. The impact factor has been included in the calculations used to prepare Table 6.17.

TABLE 6.17 Railroad Loads on Circular Pipe, Pounds per Linear Foot*

Pipe size, in	Height of fill H above top of pipe, 10 ft																
	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20	25	30
12	3270	3059	2750	2431	2109	1808	1550	1330	1138	982	788	646	528	442	380	259	185
15	3986	3729	3352	2964	2571	2204	1890	1622	1388	1198	960	788	644	540	463	315	226
18	4702	4400	3955	3497	3033	2599	2229	1913	1637	1413	1133	930	759	636	546	372	266
21	5416	5067	4555	4027	3493	2904	2568	2204	1886	1627	1305	1071	874	733	629	428	307
24	6132	5738	5158	4560	3955	3390	2908	2495	2135	1842	1478	1212	990	830	712	485	348
27	6849	6408	5760	5093	4417	3786	3247	2786	2384	2058	1650	1354	1106	927	796	542	388
30	7562	7075	6360	5623	4877	4180	3586	3077	2633	2272	1822	1495	1221	1024	879	598	428
33	8279	7746	6963	6156	5339	4576	3925	3368	2882	2487	1995	1637	1336	1120	962	655	469
36	8995	8416	7565	6689	5801	4972	4265	3660	3132	2702	2167	1778	1452	1217	1045	711	510
42	10420	9754	8768	7752	6724	5763	4943	4242	3630	3132	2512	2061	1683	1411	1211	824	591

*Cooper E72 design loading consisting of four 72,000-lb axles spaced 5 ft c/c. Locomotive load assumed uniformly distributed over an area 8 ft × 20 ft. Weight of track structure assumed to be 200 pounds per linear foot. Impact included. Height of fill measured from top of pipe to bottom of ties. Interpolate for intermediate pipe sizes and/or fill heights.

Source: *Concrete Pipe Design Manual.*

Loads from Other Equipment. For calculating the live load transmitted to a pipe from any other heavy equipment (crane, bulldozer, or others) the total weight of the vehicle must be found. The weight on each of its tracks or wheels is calculated by dividing the number of wheels or tracks into the vehicle weight. With the track or wheel weight and the pipe depth of bury known, refer to Table 6.18 for the percent of load transmitted to the pipe. The track or wheel weight is then multiplied by the percent load transmitted to calculate the actual load on the pipe. Regardless of how slowly the vehicle is capable of moving, an impact factor must be used. The calculated actual load on the pipe is then multiplied by the impact factor obtained from Table 6.15. The result of this calculation is the total actual live load on the buried pipe.

Uniformly Distributed Load

When an additional load, such as fill or lumber, is stored at grade on top of the buried pipe, this is called a uniformly distributed load. This load is added to the earth load over the pipe. The formula for calculating the uniform load is:

$$L_s = CBW \tag{6.15}$$

where L_s = superimposed load on pipe, lb/ft of pipe
C = coefficient, dimensionless
B = width of trench at top of pipe, ft
W = weight of superimposed load, lb/ft^2

Discussion:

1. The additional load L_s is limited in dimension.

2. To find C, refer to Table 6.19 to determine the coefficient for vertical load on pipe installed in trench.
Table 6.19 is entered with the following values:

TABLE 6.18 Percentage of Wheel Loads Transmitted to Pipes*

Tabulated figures show percentage of wheel load applied to one linear foot of pipe.

Depth of backfill over top of pipe, ft	Pipe size, in													
	6	8	10	12	15	18	21	24	27	30	33	36	39	42
	Outside diameter of pipe, ft (approximate)													
	0.64	0.81	1.0	1.2	1.5	1.8	2.1	2.4	2.7	3.0	3.3	3.5	3.9	4.2
1	12.8	15.0	17.3	20.0	22.6	24.8	26.4	27.2	28.0	28.6	29.0	29.4	29.8	29.9
2	5.7	7.0	8.3	9.6	11.5	13.2	15.0	15.6	16.8	17.8	18.7	19.5	20.0	20.5
3	2.9	3.6	4.3	5.2	6.4	7.5	8.6	9.3	10.2	11.1	11.8	12.5	12.9	13.5
4	1.7	2.1	2.5	3.1	3.9	4.6	5.3	5.8	6.5	7.2	7.9	8.5	8.8	9.2
5	1.2	1.4	1.7	2.1	2.6	3.1	3.6	3.9	4.4	4.9	5.3	5.8	6.1	6.4
6	0.8	1.0	1.1	1.4	1.8	2.1	2.5	2.8	3.1	3.5	3.8	4.2	4.3	4.4
7	0.5	0.7	0.8	1.0	1.3	1.6	1.9	2.1	2.3	2.6	2.9	3.2	3.3	3.5
8	0.4	0.5	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.3	2.5	2.6

*These figures make no allowance for impact.

Source: *Clay Pipe Engineering Manual.*

TABLE 6.19 Vertical Load Coefficients

	$L/2H$													
D/H	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.124	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.244	0.248
0.3	0.053	0.103	0.149	0.190	0.224	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.284	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.481	0.505	0.525	0.540	0.548
0.6	0.089	0.174	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.546	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.584	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.894	0.916
2.0	0.124	0.244	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.849	0.894	0.930	0.956

Source: *Clay Pipe Engineering Manual.*

a. D is either length or width of material at grade, in ft.
b. L is either length or width of material at grade, in ft.
c. H is the height of backfill over top of pipe in ft, prior to placement of new load.

BEDDING

General

Bedding is the contact between pipe and earth and is used when pipe is installed in a trench, prior to the major backfilling operation. The type of bedding used has an important influence on the total load any pipe can support.

Bedding Methods

Rigid Pipe. The four most popular bedding methods are shown in Fig. 6.33. The load factors are indicated in each detail. The most common are the Class B methods. The least desirable is Class D, which is not recommended. In all cases, the bell holes are dug out prior to placement of pipe.

Flexible Pipe. The recommended bedding method for flexible pipe is shown in Fig. 6.34.

Pipe Bedding Material Classification

ASTM D 2321 presents a method of classifying soils and aggregates that are used as bedding and backfill around pipes. Refer to Table 6.20 for the classification of embedment and backfill material.

Selection of Bedding and Backfill Material

General recommendations for the selection and installation of soils for bedding and backfill are given in Table 6.21.

Rigid Pipe

The load factor for a bedding condition is used to determine the actual supporting strength of a pipe. As will be seen, the load factor increases the total load a pipe can support. The laboratory-calculated pipe strength multiplied by a load factor will give the *field supporting strength* of a pipe. These load factors have been determined experimentally for the various bedding methods described and are indicated in Fig. 6.33 in each bedding diagram. However, they do not contain a safety factor. The result of the calculations will determine whether the field supporting strength of the pipe is great enough to resist the imposed load.

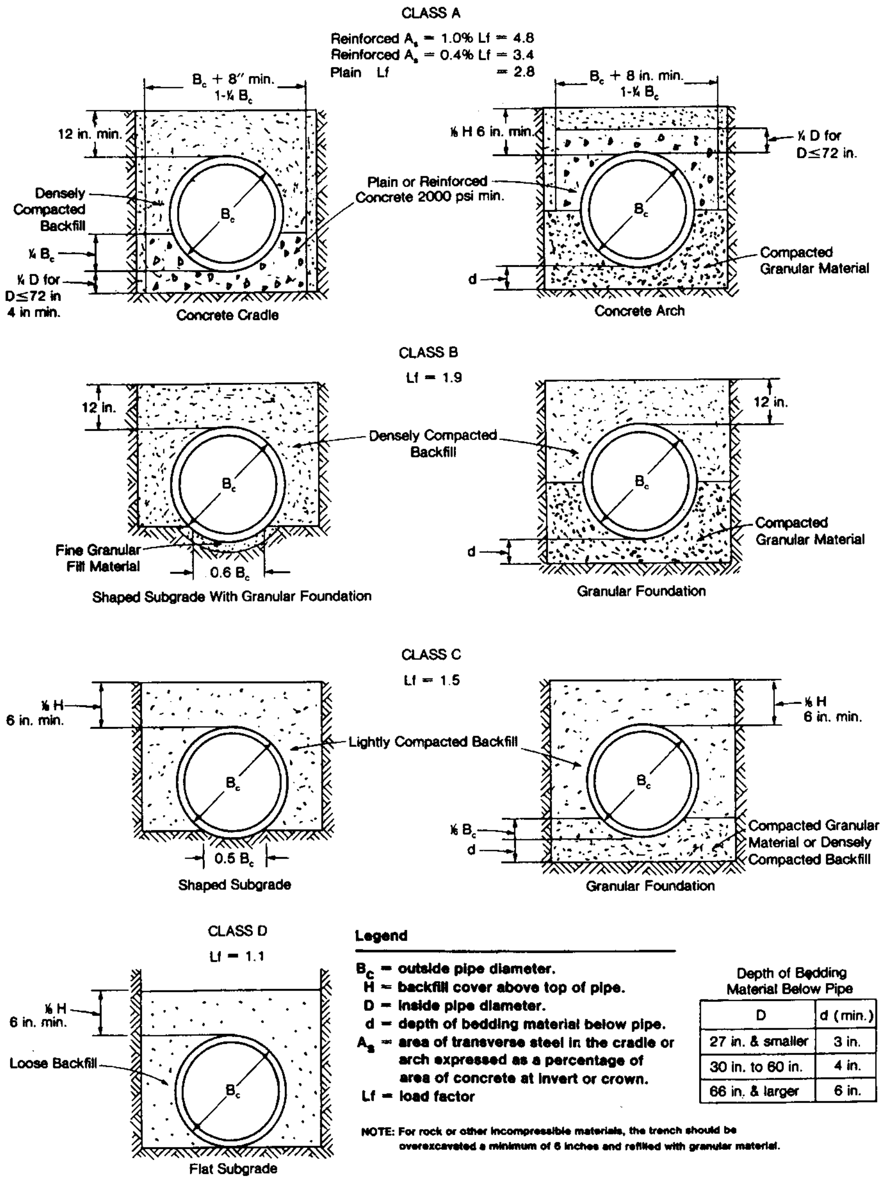


FIGURE 6.33 Rigid pipe bedding methods. (*Courtesy: ASPE Data Book.*) Note: 1 in = 25 mm.

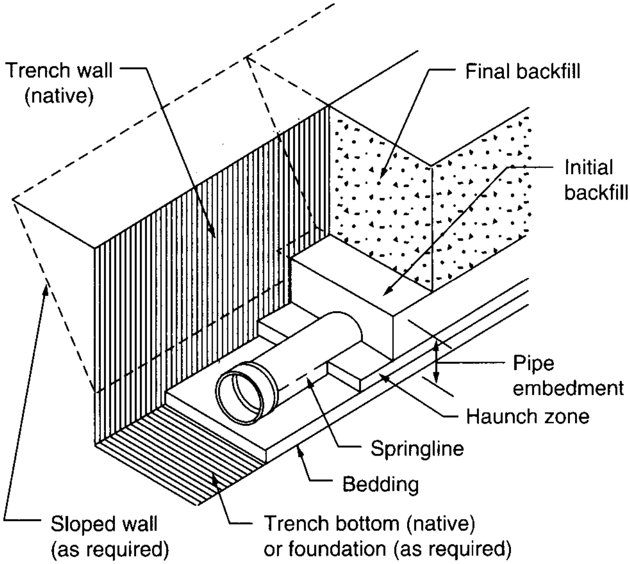


FIGURE 6.34 Flexible pipe bedding.

Flexible Pipe

Pipe Deformation (Deflection)

1. *Radial deflection.* Commonly called *ovalling,* radial deflection is a change in pipe diameter caused by an exterior load on a portion of the pipe. This will occur in buried pipe from a load on the top portion of the pipe if the support of the surrounding soil is not enough to resist the force.

2. *Longitudinal deformation.* Often referred to as *sagging,* longitudinal deformation is a change along the length of a pipe that causes the pipe to bend.

3. *Deformation at structures.* Not limited to flexible pipe, this type of deformation is caused by uneven settlement at wall penetrations and manholes.

The structural behavior of flexible pipe is not the same as for rigid pipe. Flexible pipe does not have the structural strength and stiffness to resist traffic and backfill loads without support from surrounding embedment. Being flexible, the pipe deforms (or deflects) under load. This generally takes the form of radial elongation. This elongation structurally stresses the surrounding embedment material. The resistance to this stress is required for long-term support for the pipe. In order to achieve long-term resistance, proper placement of acceptable material around the pipe is required.

STRENGTH OF PIPE MATERIALS

Rigid Pipe

All rigid pipe has been tested for strength in the laboratory by means of a three-edge-bearing test. In order to simplify selection of some piping, a method was

TABLE 6.20 Classification of Soils for Embedment and Backfill

Class	Type	Soil group symbol D 2487	Description	Percentage passing sieve sizes			Atterberg limits		Coefficients	
				1½ in (40 mm)	No. 4 (4.75 mm)	No. 200 (0.075 mm)	LL	PI	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, clean	None	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 (see X1.8).	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 to 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 to 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 to 12	Nonplastic		Same as for GW, GP, SW and SP	

TABLE 6.20 Classification of Soils for Embedment and Backfill (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	PI	Coefficients: Uniformity C_u	Curvature C_c
IIII	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*	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.				>50	<A line		
	Highly organic	PT	Peat and other high organic soils.							

*Includes Test Method D 2487 borderline classifications and dual symbols depending on plasticity index and liquid limits.

NOTE—"Coarse fraction" as used in this table is defined as material retained on a No. 200 sieve.

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TABLE 6.21 Recommendations for Installation and Use of Soils and Aggregates for Foundation, Backfill, and Embedment

	Soil class (see Table 6.20)*				
Use	Class IA	Class IB	Class II	Class III	Class IV-A
General recommendations and restrictions	Do not use where conditions may cause migration of fines from adjacent soil and loss of pipe support. Suitable for use as a drainage blanket and underdrain in rock cuts where adjacent material is suitably graded.	Process materials as required to obtain gradation which will minimize migration of adjacent materials. Suitable for use as drainage blanket and underdrain.	Where hydraulic gradient exists check gradation to minimize migration. "Clean" groups suitable for use as drainage blanket and underdrain.	Do not use where water conditions in trench may cause instability.	Obtain geotechnical evaluation of proposed material. May not be suitable under high earth fills, surface-applied wheel loads, and under heavy vibratory compactors and tampers. Do not use where water conditions in trench may cause instability.
Foundation	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 6-in maximum layers.	Suitable as foundation and for replacing over-excavated and unstable trench bottom. Install and compact in 6-in maximum layers.	Suitable as a foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 6-in maximum layers.	Suitable as foundation and for replacing over-excavated trench bottom as restricted above. Do not use in thicknesses greater than 12 in total. Install and compact in 6-in maximum layers.	Suitable only in undisturbed condition and where trench is dry. Remove all loose material and provide firm, uniform trench bottom before bedding is placed.
Bedding	Suitable as restricted above. Install in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in for rock cuts).	Install and compact in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in for rock cuts).	Suitable as restricted above. Install and compact in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in for rock cuts).	Suitable only in dry trench conditions. Install and compact in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in for rock cuts).	Suitable only in dry trench conditions and when optimum placement and compaction control is maintained. Install and compact in 6-in maximum layers. Level final grade by hand. Minimum depth 4 in (6 in for rock cuts).

TABLE 6.21 Recommendations for Installation and Use of Soils and Aggregates for Foundation, Backfill, and Embedment (Continued)

	Soil class (see Table 6.20)*				
Use	Class IA	Class IB	Class II	Class III	Class IV-A
Haunching	Suitable as restricted above. Install in 6-in maximum layers. Work in around pipe by hand to provide uniform support.	Install and compact in 6-in maximum layers. Work in around pipe by hand to provide uniform support.	Suitable as restricted above. Install and compact in 6-in maximum layers. Work in around pipe by hand to provide uniform support.	Suitable as restricted above. Install and compact in 6-in maximum layers. Work in around pipe by hand to provide uniform support.	Suitable only in dry trench conditions and when optimum placement and compaction control is maintained. Install and compact in 6-in maximum layers. Work in around pipe by hand to provide uniform support.
Initial backfill	Suitable as restricted above. Install to a minimum of 6 in above pipe crown.	Install and compact to a minimum of 6 in above pipe crown.	Suitable as restricted above. Install and compact to a minimum of 6 in above pipe crown.	Suitable as restricted above. Install and compact to a minimum of 6 in above pipe crown.	Suitable as restricted above. Install and compact to a minimum of 6 in above pipe crown.
Embedment compaction†	Place and work by hand to ensure all excavated voids and haunch areas are filled. For high densities use vibratory compactors.	Minimum density 85% Std. Proctor.‡ Use hand tampers or vibratory compactors.	Minimum density 85% Std. Proctor.‡ Use hand tampers or vibratory compactors.	Minimum density 90% Std. Proctor.‡ Use hand tampers or vibratory compactors. Maintain moisture content near optimum to minimize compactive effort.	Minimum density 95% Std. Proctor.‡ Use hand tampers or impact tampers. Maintain moisture content near optimum to minimize compactive effort.
Final backfill	Compact as required by the engineer.	Compact as required by the engineer.	Compact as required by the engineer.	Compact as required by the engineer.	Suitable as restricted above. Compact as required by the engineer.

*Class IV-B (MH-CH) and Class V (OL, OH, PT) materials are unsuitable as embedment. They may be used as final backfill as permitted by the engineer.

†When using mechanical compactors avoid contact with pipe. When compacting over pipe crown maintain a minimum of 6-in cover when using small mechanical compactors. When using larger compactors maintain minimum clearances as required by the engineer.

‡The minimum densities given in the table are intended as the compaction requirements for obtaining satisfactory embedment stiffness in most installation conditions.

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developed to eliminate any difference due to pipe size. Therefore, a D load has been developed by dividing the actual laboratory strength by the pipe size in feet. In cases where different sizes of pipe have different strengths, this is not possible. Table 6.22 gives the laboratory design loads for various pipe material.

Flexible Pipe

Flexible pipe is plastic or corrugated steel pipe. The failure of this type of piping is considered to occur when the total calculated load on the pipe produces a radial deflection of 5 percent or more of the pipe diameter. Refer to Table 6.23 for the maximum allowable deflection of various size pipes.

TOTAL LOAD ON BURIED PIPE

To calculate the actual load used to select any rigid pipe, the following formulas can be used:

For pipe using D load:

$$L_D \times F = \frac{L_T}{D} \times \text{SF} \tag{6.16}$$

For pipe using ordinary load:

$$L_A \times F = L_T \times \text{SF} \tag{6.17}$$

where L_D = maximum allowable D load on pipe, lb/ft (Table 6.22)
L_A = maximum allowable load on pipe, lb/ft (Table 6.22)
L_T = total calculated combined load (earth, superimposed, railroad), lb/ft
D = outside diameter of pipe, ft (Table 6.16)
F = load factor (Fig. 6.33)
SF = safety factor

Discussion:

1. L_D is the allowable D load for pipe.

2. L_A is the allowable load for all other types of rigid pipe.

3. L_T is the actual total combined load on pipe after determining the earth load, superimposed load, and/or railroad load.

4. F is the load factor determined from the pipe bedding method (Fig. 6.33).

5. SF is the safety factor. A practice normally followed is to allow for the possibility of all extreme conditions occurring simultaneously, thereby producing maximum loading on the buried pipe. Therefore, a safety factor is added to the calculated loads on the buried pipe. This factor ranges between 1.25 and 1.50. How closely and competently the excavation and backfill specification is both written and supervised will determine the number selected. When using reinforced concrete pipe, with the 0.01 crack as failure criterion, no safety factor is required. This is the only exception. An SF of 1.5 is to be used unless close supervision of the job will be maintained and a very descriptive and tight specification is written. If such

TABLE 6.22 Pipe Design Loads, lb

DN	Pipe size, in	Cast iron	Clay pipe, regular strength	Clay pipe, extra strength	Concrete pipe, standard strength	Concrete pipe, extra strength	Composite pipe, strength class	D loads: Reinforced concrete pipe		
								Strength class	0.01 crack	Ultimate
250	10	2265	1600	2400	1400	2000	1500	I	800	1200
300	12	2231	1800	2600	1500	2250	2400	II	1000	1500
375	15	2302	2000	2900	1750	2750	3300	III	1350	2000
450	18		2200	3300	2000	3300	4000	IV	2500	3000
525	21		2400	3850	2200	3850	5000	V	3000	3750
600	25		2800	4700	2400	4000				
700	27		2800	4700						
750	30		3300	5000						
900	36		4000	6000						

TABLE 6.23 Maximum Allowable Deflection for Pipe

DN	Size, in	Maximum deflection, in
300	12	0.60
375	15	0.75
450	18	0.90
525	21	1.05
600	24	1.20
700	27	1.35
750	30	1.50
800	33	1.65
900	36	1.80

is the case, a 1.25 SF can be used. In addition, a 1.25 SF should be used for cast iron pipe in all conditions.

6. The allowable (or field supporting strength) must be greater than the actual load so that the pipe will not fail under design conditions.

A formula has been developed to calculate the deflection under the total calculated load transmitted to the buried pipe. It is called the Iowa formula.

$$\text{DEF} = \frac{\text{LF} \times K \times L_T \times R^3}{E \times I + 0.061 \times S \times R^3} \tag{6.18}$$

where DEF = deflection of pipe, in
LF = deflection lag factor
K = bedding constant
L_T = total load transmitted to pipe, lb/in
R = mean radius of pipe, in
E = modules of elasticity
I = moment of inertia
S = modulus soil reaction, psi

Discussion:

1. LF is the lag factor, which allows for continued deflection of pipe after the total load has been developed. It is an empirical number, with a recommended value of 1.25 for good backfill (85 percent density), or 1.50 for excellent backfill (95 percent density).

2. K is a factor depending on the width of bedding. An average value of 0.1 is recommended.

3. L_T is the total load (earth, superimposed, railroad) in lb/in of pipe length.

4. R is the radius of pipe measured from the center of the corrugation. For this formula, one-half of the nominal size should be used.

5. To determine I, refer to Table 6.24 for the correct value of corrugated pipe, based on the gauge of steel selected. For plastic pipe, individual manufacturers should be consulted because of the great diversity of materials available.

6. E is 30 3 10^6 for all pipe sizes.

TABLE 6.24 Moment of Inertia for Corrugated Steel Pipe

Corrugation pitch & depth	16 gauge	14 gauge	12 gauge
$2 \times \frac{1}{2}$	0.00194	0.00246	0.00354
$2\frac{2}{3} \times \frac{1}{2}$	0.00189	0.00239	0.00343

7. *S* is soil reaction modulus in psi. *S* is dependent upon the degree of compaction of backfill around the pipe. This number has not been fully correlated for backfilled soil. Estimates by the Bureau of Public Roads are 700 for good backfill at 85 percent compaction and 1400 for excellent backfill at 95 percent compaction. The value established for good or excellent backfill should be used in Eq. (6.18).

In certain cases, other structural failures for corrugated steel pipe may have to be considered during design. Seam, buckling, handling, and installation strengths and conduit wall compression strength may have to be determined. Refer to the *Handbook of Steel Drainage and Highway Construction Products,* published by the American Iron and Steel Institute, for a description of the method of calculating all the above design parameters, if they are required.

SEWER CLEANING AND REPAIR

CLEANING METHODS

There are many options available to clean sewer piping, manholes, and drainage inlets. The following paragraphs will describe the methods and applications associated with each.

Cleaning Ball

The cleaning ball method is limited to light cleaning of sewer pipe, for example, new piping recently installed or pipe in which almost no accumulation of debris is present. This method uses a ball with serrated ridges around its circumference, which is slightly smaller than the pipe to be cleaned. The ball is placed in the pipe and water is added behind the ball. The water pushes forward and forces its way through the serrations. The water pressure on the ball causes the ball to rotate and dislodge any light dirt. The water forcing its way around the ball flushes the loosened debris away.

Hydraulic Jet Rodder

Application of the hydraulic jet rodder (HJR) is limited to pipes filled to no greater than 30 percent of their depth with dirt or debris. This method uses a bullet-shaped head, with water nozzles both in front and in the rear, attached to a flexible hose. The head is inserted into a sewer through a manhole and uses the water jet to force its way into the pipe. This action drives some debris in front of it, but also leaves much debris at its rear, behind the head. The head is then withdrawn back to the manhole where it was inserted, using the rear-facing water jets to force the loosened dirt and debris to a point where they can be easily removed from the manhole. Depending upon the particular machine used, water pressure up to 4000 psi can be developed. Care must be exercised to avoid damage to the pipe, which is a limiting factor to the amount of debris which can be loosened. In addition, a circular rotating blade, operated hydraulically, can be attached to the head for cleaning grease, with the rear-facing jets of water removing it back to the manhole. It is anticipated that the jet rodder will become the most popular method of pipe cleaning.

Bucket Machine

Application of the bucket machine is limited to piping filled to between 30 and 99 percent with debris. An open bucket capable of remote closing and attached to a cable is inserted into the sewer pipe from one manhole with its open end into the pipe. The cable must be pulled from two ends, usually from one manhole to another. The cable, with bucket attached, is pulled forward into the sewer. When filled with debris, the bucket end closes and the other end of the cable pulls the bucket backwards to the manhole, where it is emptied. This process is repeated until the line is clear. If a sewer line is completely blocked, preventing a cable from being stretched from one point to another, this cleaning method cannot be used.

Excavation and Disassembly

When a pipe is completely blocked, and standing water is observed in the manhole, there may be no other alternative than to excavate around the pipe at a joint, disassemble the joint, and remove the accumulated debris from a trench by hand. This method also requires a portable pump.

Manhole or Drainage Inlet Cleaning

This is accomplished with a ''clam digger,'' a small-diameter, articulated finger-type machine that can reach down into a manhole or inlet for debris removal.

Vapor Rooting

Vapor rooting is used only to chemically remove roots inside a sewer and retard their future growth outside the sewer for several years. Its application is limited to sewers that rarely run full. This method uses a foam dispenser that is run through the sewer pipe, producing a chemical foam having the consistency of thick soapsuds. Since roots do not occur in a pipe full of water, the foam adheres to the roots and pipe above the water line, killing the roots and retarding future growth for approximately three years. This chemical does not remove the roots immediately, but is very effective in about three to six months. If the roots must be removed immediately, the use of an HJR is necessary. In addition to being used alone, this chemical can be added to the grout used to repair cracks.

REPAIR OF CRACKS AND JOINT SEPARATIONS IN PIPING

In many cases, cracks have appeared in pipes and joints have separated due to uneven settlement of the earth surrounding the sewer pipe. When this occurs, the pipe will leak, washing away the supporting earth fill and also allowing roots to enter the pipe. Repair of these deficiencies is of the highest priority.

The most common method used for the repair of piping up to 24 in diameter is called *chemical grouting* (CG). This method uses a polymer adhesive with the viscosity of water that hardens into a solid in a period of time determined by the mixture of ingredients. It is applied from inside a pipe by a device called a *packer,* which is, in essence, a 2½-ft length of pipe, slightly smaller than the pipe being repaired, with an inflatable donut at either end. The packer must be positioned correctly by the use of a video camera. Once in position, the donuts inflate creating a seal, and the adhesive is injected under pressure between them. The adhesive is forced out of the pipe into the surrounding soil, where it hardens, forming a tough elastic and waterproof barrier to inhibit further leakage. If the rooting chemical is added to this adhesive, any roots growing toward the hardened soil will die. This is considered a permanent repair. If any adhesive hardens on the inside of the pipe, it must be cleaned, generally with an HJR with a rotating blade.

REPAIR OF MANHOLES

Manholes are generally repaired using a waterproof epoxy cement or other hydraulic coating applied to all areas of the manhole until a smooth finish is achieved. This is very labor intensive.

PREVENTIVE MAINTENANCE

After the piping network is initially cleaned and repaired, the following scheduled preventive maintenance program is recommended:

1. After the initial vapor rooting procedures, a second application is recommended after one year and again every three years. This should be applied only where there are trees. The price would be the same as for VR cleaning.

2. A regularly scheduled cleaning using the HJR once every two years is recommended.

3. A video inspection of the piping system once every five years is recommended to see if any problems have developed in the piping network.

SANITARY GRAVITY SEWERS

This section will discuss gravity sewers collecting domestic sanitary waste and acceptable industrial discharge. These sewers carry this combined waste from small areas or buildings from the property line or building wall to a connection with a public sanitary sewer. Sanitary sewers are called house sewers for private residences and service laterals for larger facilities. On large sites, long runs of private sewers receiving discharge from various buildings route the sewage into the public sewer for disposal.

If a public sewer is available, it is the least expensive method of disposing sanitary waste. In urban areas, a sewer could be considered available if it is within 500 ft of a property line. This distance varies based on local requirements. It may be necessary for a house sewer to travel a considerable distance, often in a public street.

If a public sewer is not available, the sanitary effluent must be treated by a septic tank or a sewage treatment system to the extent required by local authorities. The treated effluent can then be discharged to the environment.

Plumbing codes govern the size and installation of house sewers and building sewers, or building laterals. Any discharge into the public sewer other than sanitary effluent must be treated so that the quality of the effluent is within guidelines established by the local authorities. These guidelines vary based on the municipal sewage measurements. It could combine with the sanitary house sewer prior to discharge into the public sewer to save installation costs.

The following information should be obtained before designing a sewer:

1. The size, location, and depth of the existing public sewer, and whether the public sewer is a sanitary or combined sewer
2. The location and size of spurs and manholes in the public sewer
3. The allowable method of connection to the public sewer
   - Do the local authorities require a manhole? Will the local authority build the manhole? If so, who will bear the cost?
   - If a pipe is installed in a public street, are there special installation or other requirements?
4. Topographical map along the proposed route of the sewer
5. Minimum depth of bury if mandated by the local authorities

SEWER COMPONENTS

The components of the sewer system consist of appurtenances, or structures, such as manholes, cleanouts to grade for smaller lines, and the piping network.

MANHOLES

A manhole is an underground structure that facilitates pipe connections, access into the sewer for observation and maintenance, and keeps hydraulic interference from connections and changes in direction to a minimum. A typical manhole is illustrated in Fig. 6.10.

There are several different types of manholes, each used for a different purpose. A shallow manhole (Fig. 6.35) is used when the depth from the invert of the sewer to grade is less then approximately 3 ft (1 m). A drop manhole (Fig. 6.11) should be used when the invert of the incoming pipe is more than 2 ft (60 cm) higher than the discharge pipe. This is to avoid the inflow spilling onto the channel of the manhole because it would quickly wear the channel away. Therefore, a drop pipe is provided to bring the effluent into the manhole to lessen the hydraulic impact. Two pipe entries into the manhole are provided in case the drop pipe has a stoppage. When the peak flow fills the pipe to more than three-quarters full, it is common practice to make the drop pipe one size larger than the incoming sewer line to minimize stoppages.

Manholes consist of a frame and cover, structure body, steps, pipe connections, sewage channel, and bottom slab.

Frame and Cover

Frames and covers are available in round and square configurations and in a wide variety of sizes. The advantage of a square cover is the large space for entry into the manhole. The advantage of the round cover is that it cannot fall into the structure. The frame and cover materials is normally cast iron, but ductile iron and aluminum are also commonly available. They are manufactured in light- and heavy-duty, based on the load that could be safely placed on them.

It is common practice to install the top of the cover 1 in above the road surface or grade to keep infiltration of storm water between the frame and cover to a minimum. If distance is greater than 3 in, use concrete rings or brick firmly bedded in mortar to raise the frame. The casting is set to final grade elevation by means of a mortar bed. Care should be taken to select frames and covers that fit well in order to prevent rattling in traffic, provide reasonably tight closure, and resist unauthorized entry.

The size of the cover is determined by the intended use. For workers to enter, the minimum size is 2 ft (60 cm), which is considered small. The common size is 2 ft, 6 in (75 cm). It is common practice to cast into the cover the word "sanitary" or "storm" (or other) as required for each utility to easily identify the system.

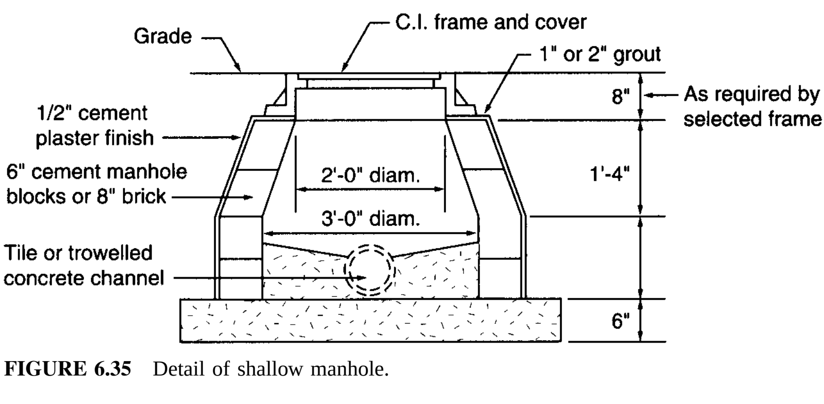


FIGURE 6.35 Detail of shallow manhole.

Manhole Construction

Most manholes are circular, with the inside dimensions large enough for a worker to perform necessary functions without difficulty. The minimum recommended diameter of the major section is generally 3 ft (1 m), with a 4-ft (1.25 m) inside diameter used most often. It is common practice to use eccentric conical sections at the top to give a straight vertical side for the steps. If the cover is 2 ft, 6 in (75 cm), the clear inside dimension of the top section should be a minimum of 3 ft (1 m).

Manholes can be constructed of brick, precast concrete sections, poured (cast-in-place) concrete, and concrete block. The most common method of construction is with precast sections called risers. The typical precast manhole consists of a base, riser sections, and a conical top section ready for installation. The sections fit together with tongue-and-groove joints with gaskets used in the joints to make them watertight. Gaskets should be installed in accordance with manufacturers' recommendations regarding lubricants, cements, and other special installation requirements. Risers are available in a large variety of sizes and shapes.

Poured-in-place or precast reinforced concrete structures must use concrete with a compressive strength of 4000 psi after 28 days in accordance with ASTM C-478. Manhole steps should be installed into forms. A concrete base should be provided under the structure. Rebars must conform to ASTM A-615, grade 40, and wire fabric, to ASTM A-185.

Brick and block manhole walls should be constructed 8 in (20 mm) thick for up to 8 ft of depth, and 12 in (30 mm) for greater depths. The outside of the manholes should be parged (coated) with at least one coat of cement mortar at least one-half in thick for protection against deterioration. Two coats are often used. For additional protection in wet soils, a coating of coal tar epoxy is often applied over the parging after it has dried for 30 days.

Steps

Manhole steps are generally constructed of cast iron or aluminum. Common step diameters are three-quarters in (19 mm) or 1 in (25 mm). Width is 16 in (40 cm) if it is desired to have two feet on one rung; 12 in (30 cm) rungs are the most common size. Steps are spaced 12 in (30 cm) to 16 in (50 mm) apart, with 12 in the most common. Good practice is to have a distance of approximately 6½ in (16 mm) from the wall to the inside of the rung.

The Base and Channel

Structures are built on concrete bases or footings. For precast manholes a recessed center is provided to receive the male tongue and groove joints of a riser section. The invert channel is formed either by shaping poured concrete after the drainage lines have been installed or by a cast iron pipe that has the top cut away after installation.

A channel is the lowest internal part of the manhole and should be a smooth continuation of the pipe. It provides a U-shaped open flow path for the liquid. Its height should be at least one-half the pipe size for 12 in (30 cm) pipe and three-quarters the height for 15 in (38 cm) and larger.

SIZING THE SANITARY SEWER

The sanitary sewer pipe size is selected after calculating the peak flow rate, determining the slope of the pipe, and selecting the sewer pipe material.

Peak Flow Rate Determination

Four factors must be considered when calculating the peak flow:

1. Peak sanitary discharge and flow from fixtures
2. Peak nonsanitary discharge, i.e., drainage and estimated leakage flow from all process, utility, and manufacturing sources, as well as blowdown and similar sources
3. Allowance for future
4. Infiltration for long runs of piping

Peak Sanitary Discharge. Peak sanitary discharge is calculated using fixture units in accordance with local plumbing codes. A method of converting sanitary fixture units into gpm is presented in Fig. 6.36. Private sewers serving multiple buildings are also sized in the same manner until the fixture unit count becomes too large. After the code fixture unit value is exceeded, the drainage flow rate figure is then calculated by using the facility water fixture unit flow rate demand.

Peak Nonsanitary Discharge. The process, utility, and manufacturing discharge can only be calculated by a study of the entire facility and a determination of the total gpm expected to be discharged from all sources. It is doubtful that all discharges will occur at the same time, therefore some diversity might be used to reduce the peak flow. If there is any doubt, use the highest figure.

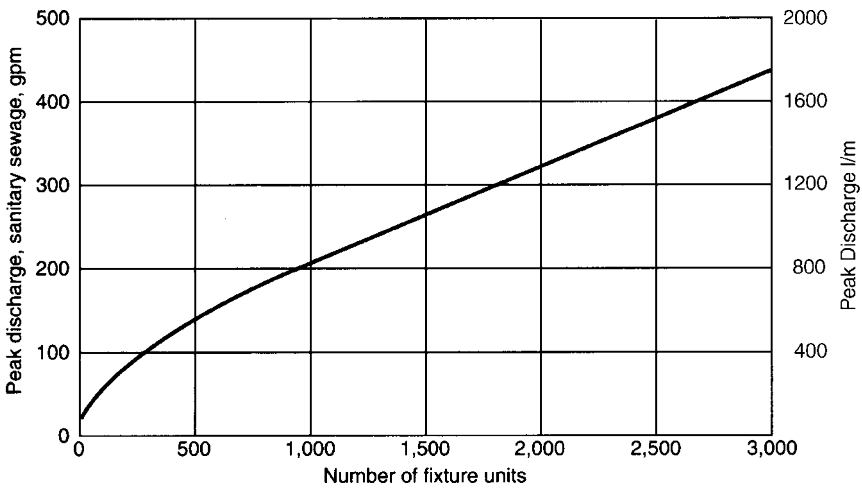


FIGURE 6.36 Fixture unit conversion to gpm.

It is often necessary to prepare a preliminary estimate for pipe size before the exact information is available. Table 6.25 provides the gallons per day for various types of facilities based on population or other easily found criteria.

To estimate a size, find the total gallons per day for the facility. Divide that figure by the number of hours the facility is in operation to give the average hourly flow. Double the hourly flow rate and divide by 60 to find the average peak flow rate in gpm. This is only an average figure. To determine pipe size, allow an additional 10 percent safety factor for peak flow rate.

Allowance for the Future. Allowance for the future should be obtained and added to the peak discharge.

Infiltration. Infiltration is the amount of groundwater or storm water entering the sewer piping network from faulty joints in underground piping, leakage around manholes and manhole covers, or cracked pipe. Another major contributor to infiltration is poorly installed laterals. Infiltration should be considered only for a very long run from the building to the sewer or in a large network serving multiple facilities on a single site. Existing sewers are frequently found to be quite leaky.

The allowance for infiltration is based on the sewer system when it is reaching the end of its useful life, not when it is new. The types of joints and the pipe material used have an effect on the figure selected. Common figures used for private sewer systems are 500 gallons per day per mile for smaller sewers and 1000 gal/day/mi for larger sewer lines. The figure selected for infiltration does not relate to the infiltration allowance used to test for sewer acceptance.

For a single facility with a short run to a public sewer, no allowance should be made. If the run is more than about 1500 ft (450 m), some allowance should be made. For lengths of run shorter than 1500 ft (450 m), some consideration should be made depending on the local authorities, stability of ground conditions, and the pipe-jointing methods selected.

Slope of the Piping. The pipe slope is generally determined either by the topography where the sewer will be installed or the elevation of the outfall where the sewer line will discharge. Often, it is a combination of both. Keeping a fairly uniform depth of bury will establish the general slope, with adjustment made to keep the slope uniform in different sections of the run. If grade is flat, the slope should be based on the optimum velocity, if possible. If the outfall elevation is the controlling factor, there is little choice in the slope. If an adequate slope to provide the necessary velocity can't be maintained, it will be necessary to use a lift station and pump the effluent. The slope of the line shall be steep enough to provide a minimum scouring velocity of 2 fps.

Selecting the Pipe Material. The pipe material selected is based on many factors. Since no single material will meet all conditions, the material selected should be based on the most important characteristics. Some factors to consider are:

1. Flow characteristics (friction coefficient)

2. Life expectancy

3. Resistance to scour

4. Resistance to effluent and surrounding soil

5. Ease of handling and installation

6. Physical strength to resist loading

TABLE 6.25 Quantities of Sewage Flow

Type of establishment	Gallons per person per day (unless otherwise noted)	
Miscellaneous facilities	Typical	Range
Apartments—multiple family (per resident)	60	
Apartments—multiple family (per apartment)	110	79–132
Bathhouses and swimming pools	10	
Camps		
Campground with central comfort stations	35	
With flush toilets, no showers	25	
Construction camps (semipermanent)	50	
Day camps (no meals served)	15	11–18
Resort camps (night and day) with limited plumbing	50	
Luxury camps	100	
Labor camp	45	37–53
Cottages and small dwellings with seasonal occupancy	50	
Country clubs (per resident member)	100	80–105
Country clubs (nonresident)	20	16–26
Dwellings:		
Boarding houses	50	
Additional for nonresident boarders	10	
Luxury residences and estates	150	
Multiple family dwellings (apartments)	60	
Rooming houses	40	
Single family dwellings	75	
Factories (gallons per person, per shift, sanitary only)	25	11–30
Institutions other than hospitals (per bed space)	125	106–160
Laundries, self-service (gallons per wash, i.e., per customer)	50	
Mobile home parks (per space)	250	
Motels (per bed space)	50	
Picnic parks (toilet wastes only) (per picnicker)	5	
Picnic parks with bathhouses, showers, and flush toilets	10	
Restaurants (toilet and kitchen wastes per patron)	10	
Restaurants (kitchen wastes per meal served)	3	
Restaurants additional for bars and cocktail lounges	2	
Schools:		
Boarding	100	
Day, without gym, cafeteria, or showers	15	
Day, with gym, cafeteria, and showers	25	
Day, with cafeteria, but without gym or showers	20	
Service stations (per vehicle served)	10	
Swimming pools and bathhouses with toilet and shower	14	11–16
Theaters:		
Movies (per auditorium seat)	5	
Drive-in (per car space)	5	
Per seat	4	3–5
Travel trailer parks without water and sewer hook-ups (per space)	50	
Travel trailer parks with individual water and sewer hook-ups (per space)	140	130–160
Workers:		
Construction	50	
Day, at schools and offices (per shift)	15	

(*Continued*)

TABLE 6.25 Quantities of Sewage Flow (*Continued*)

Source	Unit	Wastewater flow, gallons per day per unit: Range	Wastewater flow, gallons per day per unit: Typical
Commercial sources			
Airport (per passenger)	4	3–5	
Automobile service station	Vehicle services	7.9–13.2	10.6
	Employee	9.2–15.8	13.2
Bar & cocktail lounge	Customer	1.3–5.3	2.1
	Employee	10.6–15.8	13.2
Bowling Alley (per alley)	20	16–26	
Bowling Alley (per sq. ft.)	0.256 GPD		
Hotel	Guest	39.6–58.0	50.1
	Employee	7.9–13.2	10.6
Industrial building (excluding industry and cafeteria)	Employee	7.9–17.2	14.5
Laundry (self-service)	Machine	475.0–686	580.0
	Wash	47.5–52.8	50.1
Motel	Person	23.8–39.6	31.7
Motel with kitchen	Person	50.2–58.1	52.8
Office	Employee	7.9–17.2	14.5
Restaurant	Meal	2.1–4.0	2.6
Restaurant (per seat)	40	32–48	
Rooming house	Resident	23.8–50.1	39.6
Service area, roadway			
Counter seat	350	265–420	
Table seat	175	150–210	
Store, department	Toilet room	423.0–634	528.0
	Employee	7.9–13.2	10.6
	Sq. ft	0.22 GPD	
Shopping center	Parking space	0.5–2.1	1.1
	Employee	7.9–13.2	10.6
	Sq. ft	0.160 GPD	
Institutional and recreational sources			
Hospital, medical	Bed	132.0–351.0	250.0
	Employee	5.3–15.9	10.6
Hospital, mental	Bed	79.3–172.0	106.0
	Employee	5.3–15.9	10.6
Prison	Inmate	79.3–159.0	119.0
	Employee	5.3–15.9	10.6
Rest home	Resident	52.8–119.0	92.5
	Employee	5.3–15.9	10.6
School, day:			
With cafeteria, gym, showers	Student	15.9–30.4	21.1
With cafeteria only	Student	10.6–21.1	15.9
Without cafeteria, gym, showers	Student	5.3–17.2	10.6
School, boarding	Student	52.8–106.0	74.0
Restort	Person	52.8–74.0	58.1
Barber shop	Chair		55
Beauty parlor	Chair		250

TABLE 6.25 Quantities of Sewage Flow (*Continued*)

Source	Unit	Wastewater flow, gallons per day per unit: Range	Typical
Institutional and recreational sources			
Cabin, resort	Person	34.3–50.2	42.3
Cafeteria	Customer	1.1–2.6	1.6
	Employee	7.9–13.2	10.6
Campground (developed)	Person	21.1–39.6	31.7
Cocktail lounge	Seat	13.2–26.4	19.8
Coffee shop	Customer	4.0–7.9	5.3
	Employee	7.9–13.2	10.6
Country club	Member present	66.0–132.0	106.0
	Employee	10.6–15.9	13.2
Dining hall	Meals served	4.0–13.2	7.9
Dormitory, bunkhouse	Person	19.8–46.2	39.6
Hotel, resort	Person	39.6–63.4	52.8
Public park with toilets	Visitor	5–11	9
Store resort	Customer	1.3–5.3	2.6
	Employee	7.9–13.2	10.6
Swimming pool	Customer	5.3–13.2	10.6
	Employee	7.9–13.2	10.6
Theater	Seat	2.6–4.0	2.6
Visitor center	Visitor	4.0–7.9	5.3

7. Type and flexibility of joints

8. Requirements of the local authorities

9. Cost

Laterals and service connection runs for sanitary discharge are often made of extra-heavy cast iron with compression gasket joints. Another popular material is plastic, usually PVC with butt-fused joints.

Sizing the Sewer Pipe

The pipe size is selected after determining the pipe material, laying out the run of the sewer to determine the slope, and calculating the peak flow rate.

Sanitary effluent has the same characteristics as storm water. Sewers are sized using the Kutter or Manning formula. Both will yield the same results. The easiest method, rather than using the actual equation, is to find readily available prepared charts. Refer to Fig. 6.17, which solves for the Manning equation. Another set of charts (also based on the Manning formula) providing a direct reading of the size based on gpm flow rate, pitch, and depth is given in Table 6.26.

TABLE 6.26 Drainage Pipe Sizing Tables

Discharge of circular sewer N = .013

	Grade		½ Full			⅔ Full*			Full		
			Discharge		Velocity	Discharge		Velocity	Discharge		Velocity
Pipe size, in	Inch per foot	%	gpm	cfs	ft/s	gpm	cfs	ft/s	gpm	cfs	ft/s
2	⅛	1.0	7	0.02	1.3	10	0.02	1.5	13	0.03	1.3
	¼	2.1	9	0.02	1.8	14	0.03	2.0	18	0.04	1.8
	½	4.2	14	0.03	2.9	22	0.05	3.2	28	0.06	2.9
	1	8.3	20	0.05	4.0	32	0.07	4.5	40	0.09	4.0
3	⅛	1.0	18	0.04	1.7	29	0.06	1.9	36	0.08	1.7
	¼	2.1	26	0.06	2.4	41	0.09	2.7	51	0.11	2.4
	½	4.2	40	0.09	3.7	64	0.14	4.1	80	0.18	3.7
	1	8.3	57	0.13	5.3	90	0.20	5.9	114	0.25	5.3
4	⅛	1.0	39	0.09	2.0	61	0.13	2.2	77	0.17	2.0
	¼	2.1	55	0.13	2.8	87	0.20	3.1	110	0.25	2.8
	½	4.2	87	0.20	4.5	138	0.31	5.0	174	0.39	4.5
	1	8.3	123	0.28	6.3	194	0.44	7.1	245	0.55	6.3
6		0.5	79	0.18	1.8	124	0.28	2.0	157	0.35	1.8
	⅛	1.0	110	0.25	2.5	174	0.39	2.8	220	0.49	2.5
		1.5	135	0.30	3.1	213	0.47	3.5	269	0.60	3.1
	¼	2.0	157	0.35	3.6	248	0.55	4.0	314	0.70	3.6
		2.5	175	0.39	4.0	277	0.62	4.5	350	0.78	4.0
	⅜	3.0	193	0.43	4.4	305	0.68	4.9	386	0.86	4.4
		3.5	207	0.46	4.7	327	0.73	5.3	413	0.92	4.7
	½	4.0	225	0.50	5.0	355	0.79	5.6	449	1.00	5.0
	⅝	5.0	247	0.55	5.6	391	0.87	6.3	494	1.10	5.6
	¾	6.0	270	0.60	6.1	426	0.95	6.8	539	1.20	6.1
	⅞	7.0	292	0.65	6.6	461	1.03	7.4	583	1.30	6.6

See last page of table for footnotes.

(*Continued*)

TABLE 6.26 Drainage Pipe Sizing Tables (*Continued*)

Discharge of circular sewer N = .013

	Grade		½ Full			⅔ Full*			Full		
			Discharge		Velocity	Discharge		Velocity	Discharge		Velocity
Pipe size, in	Inch per foot	%	gpm	cfs	ft/s	gpm	cfs	ft/s	gpm	cfs	ft/s
8		0.2	108	0.24	1.6	170	0.38	1.8	215	0.48	1.6
		0.4	153	0.34	2.0	241	0.54	2.2	305	0.68	2.0
		0.6	191	0.43	2.4	302	0.67	2.7	382	0.85	2.4
		0.8	236	0.53	2.9	372	0.83	3.2	471	1.05	2.9
	⅛	1.0	247	0.55	3.2	391	0.87	3.6	494	1.10	3.2
		1.5	303	0.68	3.8	479	1.07	4.3	606	1.35	3.8
	¼	2.0	348	0.78	4.5	550	1.22	5.0	696	1.55	4.5
		2.5	392	0.88	4.9	621	1.38	5.5	785	1.75	4.9
	⅜	3.0	427	0.95	5.4	674	1.50	6.0	853	1.90	5.4
		3.5	449	1.00	5.8	710	1.58	6.5	893	2.00	5.8
	½	4.0	494	1.10	6.2	780	1.74	6.9	987	2.20	6.2
		4.5	516	1.15	6.6	816	1.82	7.4	1032	2.30	6.2
10		0.2	211	0.47	1.7	334	0.74	1.9	422	0.94	1.7
		0.4	303	0.68	2.4	479	1.1	2.7	606	1.35	2.4
		0.6	359	0.80	2.9	568	1.3	3.2	718	1.60	2.9
		0.8	438	0.98	3.5	692	1.5	3.9	875	1.95	3.5
	⅛	1.0	472	1.05	3.8	745	1.7	4.3	943	2.10	3.8
		1.5	561	1.25	4.5	887	2.0	5.0	1122	2.50	4.5
	¼	2.0	651	1.45	5.3	1029	2.3	5.9	1302	2.90	5.3
		2.5	741	1.65	5.7	1170	2.6	6.4	1481	3.30	5.7
	⅜	3.0	808	1.80	6.4	1277	2.8	7.2	1616	3.60	6.4
		3.5	853	1.90	6.8	1348	3.0	7.6	1706	3.80	6.8

TABLE 6.26 Drainage Pipe Sizing Tables (*Continued*)

Discharge of circular sewer N = .013

Pipe size, in	Grade: Inch per foot	Grade: %	½ Full: Discharge gpm	½ Full: Discharge cfs	½ Full: Velocity ft/s	⅔ Full*: Discharge gpm	⅔ Full*: Discharge cfs	⅔ Full*: Velocity ft/s	Full: Discharge gpm	Full: Discharge cfs	Full: Velocity ft/s
12		0.2	337	0.8	1.9	533	1.2	2.1	674	1.5	1.9
		0.4	472	1.1	2.7	745	1.7	3.0	943	2.1	2.7
		0.6	584	1.3	3.3	922	2.1	3.7	1167	2.6	3.3
		0.8	718	1.6	4.1	1135	2.5	4.6	1436	3.2	4.1
	⅛	1.0	763	1.7	4.3	1206	2.7	4.8	1526	3.4	4.3
		1.2	831	1.8	4.7	1313	2.9	5.3	1661	3.7	4.7
		1.4	898	2.0	5.0	1409	3.2	5.6	1795	4.0	5.0
		1.6	965	2.2	5.3	1525	3.4	5.9	1930	4.3	5.3
		1.8	1010	2.3	5.7	1596	3.6	6.4	2020	4.5	5.7
	¼	2.0	1077	2.4	6.0	1702	3.8	6.7	2154	4.8	6.0
		2.2	1122	2.5	6.2	1773	4.0	6.9	2244	5.0	6.2
		2.4	1167	2.6	6.6	1844	4.1	7.4	2334	5.2	6.6
14		0.1	382	0.8	1.6	603	1.3	1.7	763	1.7	1.6
		0.2	516	1.2	2.2	816	1.8	2.5	1032	2.3	2.2
		0.3	651	1.5	2.7	1029	2.3	3.0	1302	2.9	2.7
		0.4	763	1.7	3.1	1206	2.7	3.5	1526	3.4	3.1
	1⁄16	0.5	853	1.9	3.5	1348	3.0	3.9	1706	3.8	3.5
		0.6	920	2.1	3.8	1454	3.2	4.3	1840	4.1	3.8
		0.7	1010	2.3	4.1	1596	3.6	4.6	2020	4.5	4.1
		0.8	1077	2.4	4.4	1702	3.8	4.9	2154	4.8	4.4
		0.9	1145	2.6	4.6	1816	4.0	5.2	2299	5.1	4.6
	⅛	1.0	1212	2.7	4.8	1915	4.3	5.4	2424	5.4	4.8
		1.1	1257	2.8	5.1	1986	4.4	5.7	2513	5.6	5.1
		1.2	1324	3.0	5.3	2092	4.7	5.9	2648	5.9	5.3
		1.3	1369	3.1	5.5	2163	4.8	6.2	2738	6.1	5.5
		1.4	1436	3.2	5.8	2269	5.1	6.5	2872	6.4	5.8
		1.5	1481	3.3	5.9	2340	5.2	6.6	2962	6.6	5.9
		1.6	1526	3.4	6.0	2412	5.4	6.7	3052	6.8	6.0
		1.7	1571	3.5	6.3	2483	5.5	7.1	3142	7.0	6.3

(*Continued*)

TABLE 6.26 Drainage Pipe Sizing Tables (*Continued*)

Discharge of circular sewer N = .013

Pipe size, in	Grade: Inch per foot	Grade: %	½ Full: Discharge gpm	½ Full: Discharge cfs	½ Full: Velocity ft/s	⅔ Full*: Discharge gpm	⅔ Full*: Discharge cfs	⅔ Full*: Velocity ft/s	Full: Discharge gpm	Full: Discharge cfs	Full: Velocity ft/s
15		0.1	439	1.0	1.6	694	1.6	1.8	878	2.0	1.6
		0.2	628	1.4	2.3	993	2.2	2.6	1257	2.8	2.3
		0.3	763	1.7	2.7	1206	2.7	3.0	1526	3.4	2.7
		0.4	898	2.0	3.2	1419	3.2	3.6	1795	4.0	3.2
		0.5	1010	2.3	3.6	1596	3.6	4.0	2020	4.5	3.6
		0.6	1100	4.5	3.9	1738	3.9	4.4	2199	4.9	3.9
		0.7	1190	2.7	4.3	1880	4.2	4.8	2379	5.3	4.3
		0.8	1279	2.9	4.6	2021	4.5	5.2	2558	5.7	4.6
		0.9	1347	3.0	4.8	2128	4.7	5.4	2693	6.0	4.8
	⅛	1.0	1437	3.2	5.2	2270	5.1	5.8	2873	6.4	5.2
		1.1	1481	3.3	5.3	2340	5.2	5.9	2962	6.6	5.3
		1.2	1549	3.5	5.5	2447	5.5	6.2	3097	6.9	5.5
		1.3	1616	3.6	5.8	2553	5.7	6.5	3231	7.2	5.8
		1.4	1683	3.8	5.9	2660	5.9	6.6	3366	7.5	5.9
		1.5	1751	3.9	6.2	2766	6.2	6.9	3501	7.8	6.2
		1.6	1796	4.0	6.4	2837	6.3	7.2	3591	8.0	6.4
		1.7	1863	4.2	6.6	2943	6.6	7.4	3725	8.3	6.6

TABLE 6.26 Drainage Pipe Sizing Tables (*Continued*)

Discharge of circular sewer N = .013

Pipe size, in	Grade: Inch per foot	Grade: %	½ Full: Discharge gpm	½ Full: Discharge cfs	½ Full: Velocity ft/s	⅔ Full*: Discharge gpm	⅔ Full*: Discharge cfs	⅔ Full*: Velocity ft/s	Full: Discharge gpm	Full: Discharge cfs	Full: Velocity ft/s
16		0.1	539	1.2	1.7	851	1.9	1.9	1077	2.4	1.7
		0.2	786	1.8	2.4	1242	2.8	2.7	1571	3.5	2.4
		0.3	967	2.2	2.9	1528	3.4	3.2	1933	4.3	2.9
		0.4	1122	2.5	3.4	1773	4.0	3.8	2244	5.0	3.4
		0.5	1257	2.8	3.8	1986	4.4	4.3	2513	5.6	3.8
		0.6	1392	3.1	4.2	2199	4.9	4.7	2783	6.2	4.2
		0.7	1504	3.4	4.5	2376	5.3	5.0	3007	6.7	4.5
		0.8	1594	3.6	4.8	2518	5.6	5.4	3187	7.1	4.8
		0.9	1683	3.8	5.1	2660	5.9	5.7	3366	7.5	5.1
	⅛	1.0	1796	4.0	5.4	2837	6.3	6.0	3591	8.0	5.4
		1.1	1885	4.2	5.6	2980	6.6	6.3	3770	8.4	5.6
		1.2	1975	4.4	5.8	3121	7.0	6.5	3950	8.8	5.8
		1.3	2020	4.5	6.1	3192	7.1	6.8	4040	9.0	6.1
		1.4	2110	4.7	6.3	3333	7.4	7.1	4219	9.4	6.3
		1.5	2177	4.9	6.6	3440	7.7	7.4	4354	9.7	6.6
18		0.1	719	1.6	1.8	1136	2.5	2.0	1437	3.2	1.8
		0.2	1010	2.3	2.6	1596	3.6	2.9	2020	4.5	2.6
		0.3	1257	2.8	3.3	1986	4.4	3.7	2514	5.6	3.3
		0.4	1459	3.3	3.7	2306	5.1	4.1	2918	6.5	3.7
		0.5	1616	3.6	4.1	2554	5.7	4.6	3232	7.2	4.1
		0.6	1796	4.0	4.5	2837	6.3	5.0	3591	8.0	4.5
		0.7	1953	4.4	4.8	3085	6.9	5.4	3905	8.7	4.8
		0.8	2043	4.6	5.2	3228	7.2	5.8	4085	9.1	5.2
		0.9	2155	4.8	5.5	3405	7.6	6.2	4309	9.6	5.5
	⅛	1.0	2244	5.0	5.7	3546	7.9	6.4	4488	10.0	5.7
		1.1	2357	5.3	6.2	3724	8.3	6.9	4713	10.5	6.2
		1.2	2469	5.5	6.8	3901	8.7	7.6	4937	11.0	6.8
		1.3	2581	5.8	7.3	4078	9.1	8.2	5162	11.5	7.3
		1.4	2693	6.0	7.8	4255	9.5	8.7	5386	12.0	7.8

(*Continued*)

TABLE 6.26 Drainage Pipe Sizing Tables (*Continued*)

Discharge of circular sewer N = .013

Pipe size, in	Grade: Inch per foot	Grade: %	½ Full: Discharge gpm	½ Full: Discharge cfs	½ Full: Velocity ft/s	⅔ Full*: Discharge gpm	⅔ Full*: Discharge cfs	⅔ Full*: Velocity ft/s	Full: Discharge gpm	Full: Discharge cfs	Full: Velocity ft/s
20		0.1	965	2.2	2.0	1525	3.4	2.2	1930	4.3	2.0
		0.2	1392	3.1	2.8	2199	4.9	3.1	2783	6.2	2.8
		0.3	1706	3.8	3.5	2695	6.0	3.9	3411	7.6	3.5
		0.4	1975	4.4	4.0	3121	7.0	4.5	3950	8.8	4.0
		0.5	2200	4.9	4.5	3476	7.7	5.0	4400	9.8	4.5
		0.6	2401	5.4	4.8	3794	8.5	5.4	4802	10.7	4.8
		0.7	2648	5.9	5.3	4184	9.3	5.9	5296	11.8	5.3
		0.8	2805	6.3	5.6	4432	9.9	6.3	5610	12.5	5.6
		0.9	2918	6.5	6.0	4610	10.3	6.7	5835	13.0	6.0
	⅛	1.0	3150	7.0	6.3	4977	11.1	7.1	6300	14.0	6.3
		1.1	3254	7.3	6.5	5142	11.5	7.3	6508	14.5	6.5
		1.2	3366	7.5	6.8	5319	11.9	7.6	6732	15.0	6.8
21		0.1	1100	2.5	2.1	1738	3.9	2.4	2200	4.9	2.1
		0.2	1571	3.5	2.9	2483	5.5	3.2	3142	7.0	2.9
		0.3	1930	4.3	3.6	3050	6.8	4.0	3860	8.6	3.6
		0.4	2200	4.9	4.1	3476	7.7	4.6	4400	9.8	4.1
		0.5	2491	5.6	4.6	3936	8.8	5.2	4982	11.1	4.6
		0.6	2693	6.0	5.0	4257	9.5	5.6	5386	12.0	5.0
		0.7	2918	6.5	5.5	4610	10.3	6.2	5835	13.0	5.5
		0.8	3150	7.0	5.8	4977	11.1	6.5	6300	14.0	5.8
		0.9	3299	7.4	6.2	5212	11.6	6.9	6597	14.7	6.2
	⅛	1.0	3479	7.8	6.5	5497	12.2	7.3	6957	15.5	6.5
		1.1	3658	8.2	6.7	5780	12.9	7.5	7316	16.3	6.7
24		0.05	1122	2.5	1.6	1773	4.0	1.8	2244	5.0	1.6
		0.1	1616	3.6	2.3	2554	5.7	2.6	3232	7.2	2.3
		0.2	2244	5.0	3.2	3546	7.9	3.6	4488	10.0	3.2
		0.3	2805	6.3	3.9	4432	9.9	4.4	5610	12.5	3.9
		0.4	3187	7.1	4.5	5035	11.2	5.0	6373	14.2	4.5

TABLE 6.26 Drainage Pipe Sizing Tables (*Continued*)

Pipe size, in	Grade: Inch per foot	Grade: %	½ Full: Discharge gpm	½ Full: Discharge cfs	½ Full: Velocity ft/s	⅔ Full*: Discharge gpm	⅔ Full*: Discharge cfs	⅔ Full*: Velocity ft/s	Full: Discharge gpm	Full: Discharge cfs	Full: Velocity ft/s
		0.5	3591	8.0	5.1	5673	12.6	5.7	7181	16.0	5.1
		0.6	3927	8.8	5.5	6205	13.8	6.2	7854	17.5	5.5
		0.7	4264	9.5	6.0	6738	15.0	6.7	8528	19.0	6.0
		0.8	4488	10.0	6.4	7092	15.8	7.2	8976	20.0	6.4
27		0.05	1482	3.3	1.7	2341	5.2	1.9	2963	6.6	1.7
		0.1	2110	4.7	2.5	3333	7.4	2.8	4219	9.4	2.5
		0.2	3130	6.8	3.5	4787	10.7	3.9	6059	13.5	3.5
		0.3	3703	8.3	4.3	5851	13.0	4.8	7406	16.5	4.3
		0.4	4309	9.6	4.8	6808	15.2	5.4	8617	19.2	4.8
		0.5	4713	10.5	5.5	7446	16.6	6.2	9425	21.0	5.5
		0.6	5162	11.5	6.0	8156	18.2	6.7	10323	23.0	6.0
		0.7	5610	12.5	6.5	8864	19.8	7.3	11220	25.0	6.5
30		0.05	2020	4.5	1.9	3192	7.1	2.1	4040	9.0	1.9
		0.1	2805	6.3	2.7	4432	9.9	3.0	5610	12.5	2.7
		0.2	4040	9.0	3.8	6383	14.2	4.3	8079	18.0	3.8
		0.3	4938	11.0	4.6	7802	17.4	5.2	9876	22.0	4.6
		0.4	5610	12.5	5.3	8864	19.8	5.9	11220	25.0	5.3
		0.5	6508	14.5	6.0	10282	22.9	6.7	13016	29.0	6.0
		0.6	7181	16.0	6.5	11346	25.3	7.3	14362	32.0	6.5
33		0.05	2581	5.8	2.0	4078	9.1	2.2	5162	11.5	2.0
		0.1	3703	8.3	2.8	5851	13.0	3.1	7406	16.5	2.8
		0.2	5162	11.5	4.0	8156	18.2	4.5	10323	23.0	4.0
		0.3	6512	14.5	4.9	10289	22.9	5.5	13023	29.0	4.9
		0.4	7406	16.5	5.6	11701	26.1	6.3	14811	33.0	5.6
		0.5	8303	18.5	6.4	13119	29.2	7.2	16606	37.0	6.4
		0.6	9201	20.5	7.0	14537	32.4	7.8	18401	41.0	7.0
36		0.05	3254	7.3	2.1	5142	11.5	2.4	6508	14.5	2.1
		0.1	4713	10.5	3.0	7446	16.6	3.4	9425	21.0	3.0
		0.2	6733	15.0	4.3	10638	23.7	4.8	13465	30.0	4.3
		0.3	8303	18.5	5.2	13119	29.2	5.8	16606	37.0	5.2
		0.4	9425	21.0	6.0	14892	33.2	6.7	18850	42.0	6.0
		0.5	10772	24.0	6.8	17019	37.9	7.6	21543	48.0	6.8

*The depth of flow is equal to two-thirds the pipe diameter.

Source: After Chezy.

SANITARY SEWER DESIGN CONSIDERATIONS

Minimum and Maximum Effluent Velocity

The minimum velocity of the effluent should be sufficient to prevent solids from being deposited on the bottom of the pipe. This is called a *scouring velocity.* Tests have established this minimum velocity at 2 fps or 0.62 meter per second (mps) when the pipe is flowing full. Accepted practice has a recommended minimum of 2.5 fps (0.77 mps). Since pipes do not run full most of the time, it may be necessary to flush the sewer in order to remove accumulated sediment from time to time.

The maximum velocity for clear water in hard-surfaced pipe is quite high. Tests have shown that storm water velocities in excess of 40 fps (12 mps) have been found harmless to concrete channels. In practice, sanitary sewers that have continuously high velocities and where grit is expected to be a problem should limit the highest velocity to 10 fps (3 mps). For ordinary sewers without grit and only occasional periods flowing full, a maximum velocity of 20 fps (7 mps) would be considered acceptable.

For large facilities, if the house sewer is sized for the peak flow rate, the velocity during peak flow rate should be adequate to flush the pipe clean of any deposits left during periods of lower flow, for example, during the night.

Tests have shown that pipe size is not a factor in determining the velocity, provided that the pipe is large enough to prevent surcharging.

Connections to the Public Sewer

Connecting the house lateral or the private sewer to the public sewer is generally governed by the local sewer authority.

Most public sewers in public streets have built-in connections, called *spurs,* installed during construction. The location, size, and invert of spurs is often available on utility survey maps. If the size of the lateral is small enough to use a spur, it should be used.

Local authorities usually require that the connection to public sewers be made using manholes where spurs are not available. If the public sewer is very large compared to the house sewer, a special method of connection is required to pierce the main sewer and install the new pipe. One such method is illustrated in Fig. 6.37.

Depth of Bury

There are no hard and fast rules to determine the depth at which sanitary sewers should be buried. Often, local authorities have established this depth. When no guidance exists, a reasonable starting point would be 3 ft of cover, with a depth of 2 ft in areas where the pipe will not be disturbed. Consideration should be given to the depth necessary to resist possible pipe breakage by traffic or other vehicles passing over the pipe. When crossing other utilities, the sewer should be placed below them whenever possible. Since it is good practice to allow building laterals to have a ¼-in pitch when connecting to a sewer, the sewer should be deep enough to accommodate this pitch.

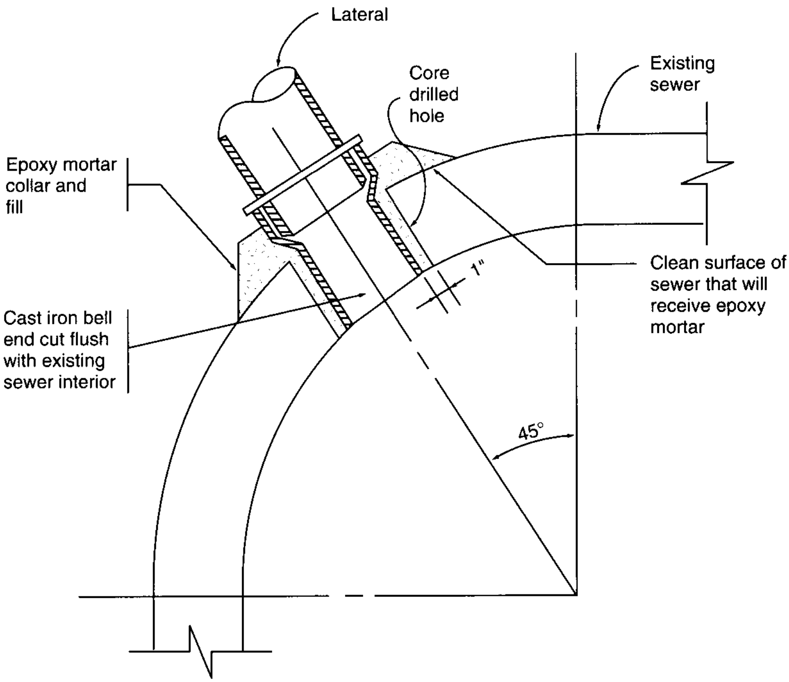


FIGURE 6.37 Method of connection to a large sewer.

Design Depth of Flow

Sewers serving facilities should be designed to carry the peak design flow rate, including allowance for the future, flowing between one-half to two-thirds full. This practice allows a safety factor for higher than expected peak flow rates. Another reason for this practice is that the air space above the flowing effluent allows for ventilation of the piping network.

Spacing of Manholes

When manholes are installed in a public street, local authorities usually mandate the distance between manholes based on sewer size. When sewers are installed on private property, the following guidelines are used for placement and spacing of manholes for sewer pipes up to 24 in (60 cm) in diameter. A maximum spacing of between 150 and 400 ft (80 and 125 m) along straight runs of pipe. Generally accepted practice has manholes spaced between 200 and 250 ft apart, with the longer distance for the larger pipe size. Sewers larger than 24 in usually have manholes 500 to 600 ft (155 to 190 m) apart. Smaller building laterals should have cleanouts brought up to grade 50 to 100 ft apart rather than manholes. Cleanouts to grade are illustrated in Fig. 6.38.

Manholes should be placed at:

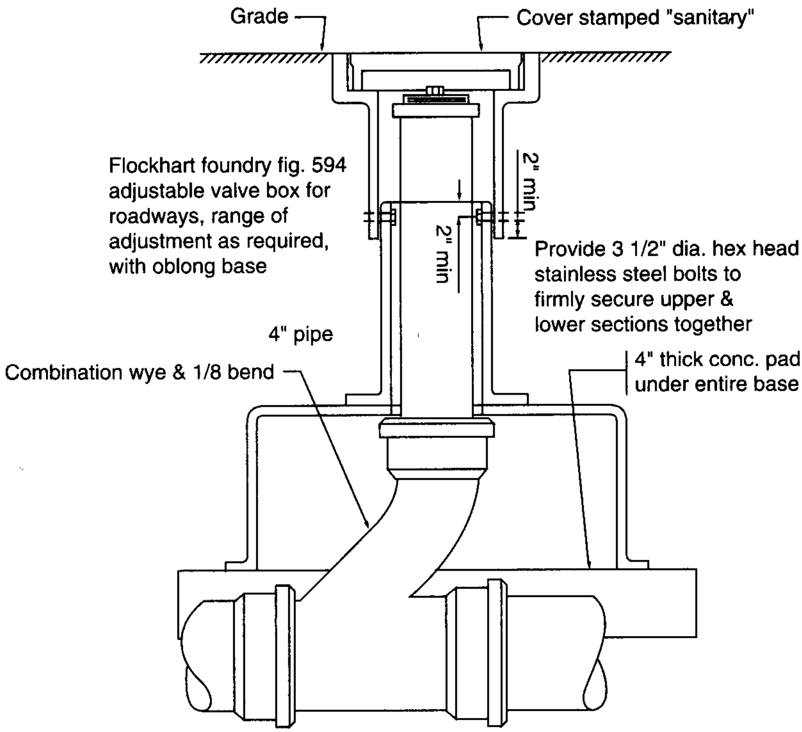


FIGURE 6.38 Cleanout to grade.

1. All changes in direction 45° or greater
2. Changes of sewer alignment
3. Changes in the size of the sewer
4. Changes of grade
5. Intersections with other sewers
6. The end of the sewer system

Clearances

Clearance is the distance between the sewer pipe and any other pipe, measured from exterior to exterior. Clearances are usually mandated either by code requirements or local authorities. There are different dimensions required horizontally and vertically.

The most stringent requirements are between potable water pipes and the sewer. Generally accepted practice is to have a minimum 10-ft (3-m) separation between water and sewer lines horizontally, with the sewer line below the water line. If that horizontal distance cannot be maintained, the sewer line should be considerably lower than the parallel water main, at the often recommended minimum distance of 4 to 6 ft (1.25 to 1.85 m) below the water line. When crossing, the sewer should be a minimum of 1 ft (0.30 m) below the water main. If this is not possible, the sanitary pipe shall be encased in concrete for a distance of 1 ft (0.30 m) past the water main. Consult local authorities for minimum distance required.

For other utilities, a 6-in clearance is adequate and encasement is not necessary.

SEWAGE LIFT STATIONS

When due to topography or low elevation the discharge of sanitary effluent from a facility is lower than the public sewer intended for disposal, pumping up to the sewer point is required. A sewage lift station, or sewage pumping station, is the system used to accomplish this.

There are three general categories of sewage pumping stations: municipal systems that are designed to serve a specific drainage area and are part of the public sanitary sewer system; industrial types that serve a single facility or site with multiple buildings; and residential types that serve a group of individual or multiple dwellings. This section will discuss methods used to pump sewage discharged from industrial/commercial types of facilities.

The difference in terminology between a sewage ejector and a lift station is one of scope. In general, the ejector system is used to pump discharge from a portion of a building up to the main house sewer for disposal, while the lift station is used to pump discharge from an entire building or site to a public sewer for disposal. The components of sewage ejector systems and sewage lift station systems are very similar to the ejector systems discussed in Chap. 9, Plumbing Systems.

CODES AND STANDARDS

There are requirements in regional model plumbing codes that apply to sewage pumping installations. The specific plumbing code used for the area where the facility is built must be followed.

SYSTEM COMPONENTS

The components of a sewage lift station are sewage pumps, basin, discharge pipe (routed to the public sewer or point of disposal), controls, and alarms. The sewage pump discharge line is often referred to as a force main.

Sewage Pumps

The types of pumps used in sewage lift stations are similar to the ejector systems discussed in Chap. 9, Plumbing Systems.

The pump selected most often is the submersible type, because of its low initial cost, wide range of capacities, and tolerance of many starts. Another advantage is that a smaller basin can be used because additional height below grade is not required to house the motor of a conventional vertical, submerged ejector pump. A typical submersible pump assembly is illustrated in Fig. 6.39.

Pump casings, impellers, and other components are available in a wide variety of materials to resist chemical corrosion, abrasion, and suspended solid size.

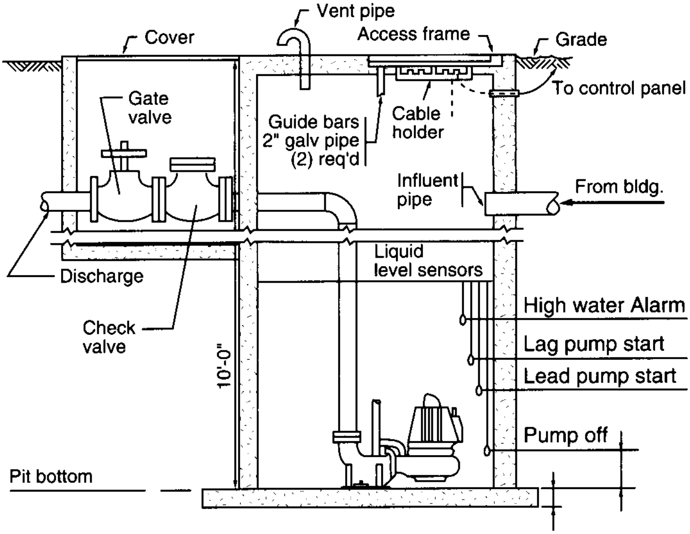


FIGURE 6.39 Detail of typical exterior submersible pump.

Storage Basin

Basins, often referred to as sumps or wet wells, are typically manufactured from cast iron, fiberglass, poured-in-place concrete, and steel.

Large basin capacities require that the design of the basin include principles important to proper pump functioning. These principles are:

1. The flow of effluent from the liquid entrance should be directed to the pump inlet in a manner that will reduce swirl and hydraulic loss.

2. Water depth shall be the minimum established by the manufacturer to avoid surface vortices.

3. Excessive turbulence should be avoided. A small amount of turbulence shall be limited to that required to prevent stagnation of stored water.

4. To prevent sedimentation and accumulation of solids, the floor of the basin should be sloped to the pumps and the floor joints with walls shall have a fillet.

5. Where the inflow into the sump is at a high elevation, the falling water entrains air as it spills onto the surface of water in the sump. This entrained air should be allowed to rise to the surface before reaching the pump. This is accomplished by providing a baffle wall in the sump and/or a dissipation arrangement at the inflow. If the basin is large, there should be a long distance from the point of liquid entry to the pumps to permit air to dissipate.

6. Pumps should be installed as close to the outside walls as practical to avoid stagnant areas that allow solids to accumulate.

7. A small sump should be installed in the basin to permit total drainage for maintenance.

A majority of systems consist of relatively small units. For this type of installation, a standard basin should be obtained from the pump manufacturer. The basin sizes correspond to the number and size of the pumps selected; the inlet and discharge pipe are preinstalled in the basin; and the depth is predetermined to contain the desired quantity of liquid. The complete basin is installed as a single, integrated unit.

Basins receiving sanitary effluent shall be provided with a gasketed cover that has an atmospheric vent to the outside. The location shall be such that an odor emanating from the basin would be easily dissipated before causing any discomfort to people. The cover shall be easy to remove so that the pumps can be raised to the surface for servicing. Guide rails are attached to the inside of the basin for this purpose. A hose bibb or lawn hydrant should be provided to wash off the pump.

Discharge Piping

The discharge pipe, or force main, is the pipe from the pump discharge to the point of disposal. The force main includes valves, clean-outs, air and vacuum release valves, and thrust restraints.

The piping material should be selected primarily based on soil and effluent corrosion resistance and pressure rating. Other considerations are strength of pipe material and what materials are allowed by the local authorities. When there is a choice among several different materials, the total cost of installation will be the deciding factor.

Commonly used materials include carbon steel, ductile iron, PVC, CPVC, FRP, and PE.

COMPONENT DESIGN AND SELECTION

General

The design of the complete sewage lift station as a whole is an iterative one, where selection of each component is somewhat dependent on other components for size and capacity.

Pump Selection

Pump Flow Rate. For a facility sewage pump, it is accepted practice to have a pump capacity equal to the highest instantaneous flow rate expected from that facility. The reasoning is that in the event the basin is incapable of being used, a single pump must be capable of discharging all possible effluent from all sources. It is also critical that there be at least a duplex set of pumps, each with the same capacity, in order to ensure that the facility will be kept in operation if one pump is out of service. If there is the possibility of a wide range of instantaneous flow rates, a three-pump system, with each pump sized at 75 percent of maximum flow, should be considered. One pump would be on standby.

The maximum discharge is calculated for two conditions. The first is for the maximum possible inflow. This is done by adding the plumbing fixture load in gpm obtained from Hunter's WFU curve to the discharges from other sources to arrive at a maximum instantaneous flow rate. If the facility is mostly discharging effluent from plumbing fixtures, add 10 percent to that figure as a safety factor. The second condition is based on a reasonable number of starts per hour.

Required Head. The required system head is calculated by adding the static height (in feet) from the bottom of the basin to the point of discharge (or the highest point of the force main run) to the friction loss (in feet of head) of the liquid flowing through the pipe based on the equivalent length of run. The friction loss is obtained from standard engineering charts for the material selected, using water as the liquid for sanitary effluent. For more viscous liquids, appropriate charts shall be used.

Calculation of Basin Capacity

The capacity of the basin is based on a single primary pump discharge flow rate and the desired number of starts per hour. Recommended practice is to have an average of six or seven starts per hour up to a maximum of 12 starts per hour, and a minimum running time of 1 min. Within these parameters, and with consultation of the pump manufacturer, the storage capacity can be selected. Cost will also be a factor if the basin is large. Since the exact inflow into the basin in any time period will never be known, a minimum running time shall be selected based on the number of gallons stored, and if additional flow into the basin occurs, the running time of the pump is extended, which is an added benefit.

The actual size may be limited by space conditions at the location where the basin is located. Additional depth may be used to obtain the desired volume if the length or width cannot be adjusted. To size the basin, refer to Table 9.8 for capacities in gallons per foot of various basin sizes.

Force Main Pipe Material Selection and Sizing

The discharge pipe, commonly called a force main, is sized to convey the effluent economically from the pump to its ultimate point of discharge. The pipe material selected is based on flow rate, friction loss of the fluid in the pipe, chemical resistance to the effluent, and fluid velocity. A minimum size of 4 in is highly recommended to allow solids to pass easily through the pipe with little chance of producing stoppages.

Selection of Pipe Material. The selection of pipe material depends on the type of effluent expected and the strength of the pipe, based on its installed condition. For sanitary effluent, pressure-rated PVC or PE is often selected. Soil contamination may not permit plastic to be used. When the force main is to be routed in public streets, the local authorities may have pipe material and specific installation requirements. Where strength of pipe material is a factor (for example, burial close to the surface of a public road), ductile iron should be considered. It is common practice to coat the exterior of the pipe or protect it with a film of PE in corrosive or contaminated soils. Several methods of installation are described in ANSI A-21.3/AWWA C-105. A typical installation of a PE protective wrap is shown in Fig. 6.40.

Flow Rate. The flow rate used to size the force main is based on two conditions. The first (which is the normal condition) has one pump of a multiple pump system running. The second (emergency condition) has all the pumps running at the same

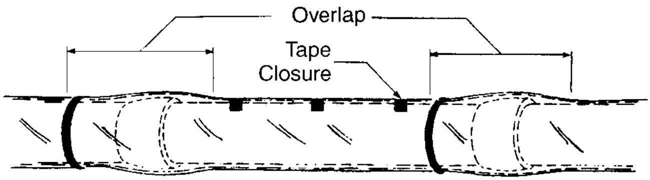


FIGURE 6.40 Typical polyethylene pipe wrap.

time. In the second case, the flow rate is the sum of the flow rates of all the pumps together.

Selecting the pipe size is an iterative procedure done when selecting the pump. A comparison of the pump system head curve is made to see ''what if'' when the pipe size is changed. The total friction loss is selected to give the optimum choice between the cost of the pipe and the cost of the additional horsepower required to pump the fluid at the higher head due to the smaller pipe size.

Friction Loss. The friction loss shall be found by using the maximum flow possible, that is, with the maximum number of pumps running. Use friction loss charts similar to Fig. 9.28 for water piping, based on the pipe material selected, flow rate, and its size.

Velocity. The velocity in the pipe should be within a range of between a minimum of 2 fps and the generally accepted maximum of about 10 fps. The low flow velocity is based on the flow rate when one pump is running, and the highest velocity occurs when all pumps are running together. Since the flow rate for all pumps running is considered a rare occasion, a slightly higher velocity should be considered acceptable if the possibility of the development of excessive water hammer does not occur. One recommendation is to use a slow-closing check valve to limit the generation of excessive pressures every time the pump(s) stop.

Pipe Sizing. The force main pipe size is based on flow rate, velocity, and friction loss. The criteria used to size the pump are capacity and total discharge head. After deciding on the pump capacity, the only criterion that can be adjusted is the pipe friction loss. A balance must be made between a large pipe size with low friction loss and a small pipe size with large friction loss. A small pipe size may also require a high velocity, which should be avoided. With different pipe sizes, at some point the friction loss will change the horsepower of the pump. The optimum economical point is reached when a larger pipe size will no longer reduce the horsepower of the pump(s). It is also possible that for a long pipe run, a smaller pipe size will justify a larger pump size. The force main size depends upon whichever condition is present.

SYSTEM DESIGN CONSIDERATIONS

Lift Station Location

The basin should not be located at a point on the site that is subject to flooding unless it is the only possible location. Borings of the area should be made to determine if the area has high groundwater or other underground obstacles that would interfere with the installation of the basin. The basin shall be installed with its top several inches above grade so that storm water will not be directed toward the basin.

Floatation

Any structure extending below the groundwater table, or extending into a soil that may become saturated, must be checked to ensure that the buoyant force will not cause the basin to float. To determine the buoyant force, calculate the upward and downward forces. If the sum of the upward force exceeds the downward force, the structure will float.

The downward forces are the sum of the equipment and basin weight, the weight of the concrete base slab under the basin, if present, and the dry weight of the soil on top of the basin, if any. The dry weight is calculated by determining the wet weight minus the weight of the water. For typical soils, refer to Table 6.8 to obtain the weight per cubic foot or meter, and subtract the weight of a cubic foot (or meter) of water, which is 62.4 lb/ft^3. The effective weight of concrete is 87 lb/ft^3 in water (150 lb/ft^3 of concrete minus 63 lb/ft^3 for water). For displaced saturated soil, the figure used is 50 lb/ft^3, with the assumed weight of soil of 100 lb/ft^3.

The buoyant force is the volume of displaced weight of groundwater measured to the highest level possible or the weight of the displaced saturated soil.

The above calculations do not consider additional restraining forces such as skin friction between the basin and the soil and the shear between overburden and adjoining soil, which may have to be considered for large basins. Common practice regards these figures as optional safety factors for smaller basins.

If it is determined that the basin will float, the most common method to offset floating is to use a concrete slab under the basin to provide the necessary added weight. The basin must be anchored to the slab. It is common practice to use a 25 percent safety factor when calculating the slab weight.

Control Considerations

The basic operating levels are decided in a manner similar to those of an ejector system, discussed in Chap. 9. If mercury switches are used, experience dictates that no less than 6 in (150 mm) between levels should be used.

Vacuum and Pressure Venting

It is good practice to provide an air pressure vent at the high point of the piping run. If the high point of the piping system occurs prior to the point of discharge, it is desirable to place a combination pressure/vacuum device in the pipe to allow accumulated air at the high point to be eliminated, and since the remainder of the run is downhill, a vacuum may be produced by the flowing liquid that will have to be broken to allow free flow of the liquid. Since there is a probability that air relief devices will discharge some water, the device is usually put in a large valve box with a gravel bottom to adsorb the liquid. Some authorities require that air release devices be installed in manholes. Ease of maintenance is the most important consideration. A detail of a typical device in a manhole is shown in Fig. 6.41, and a

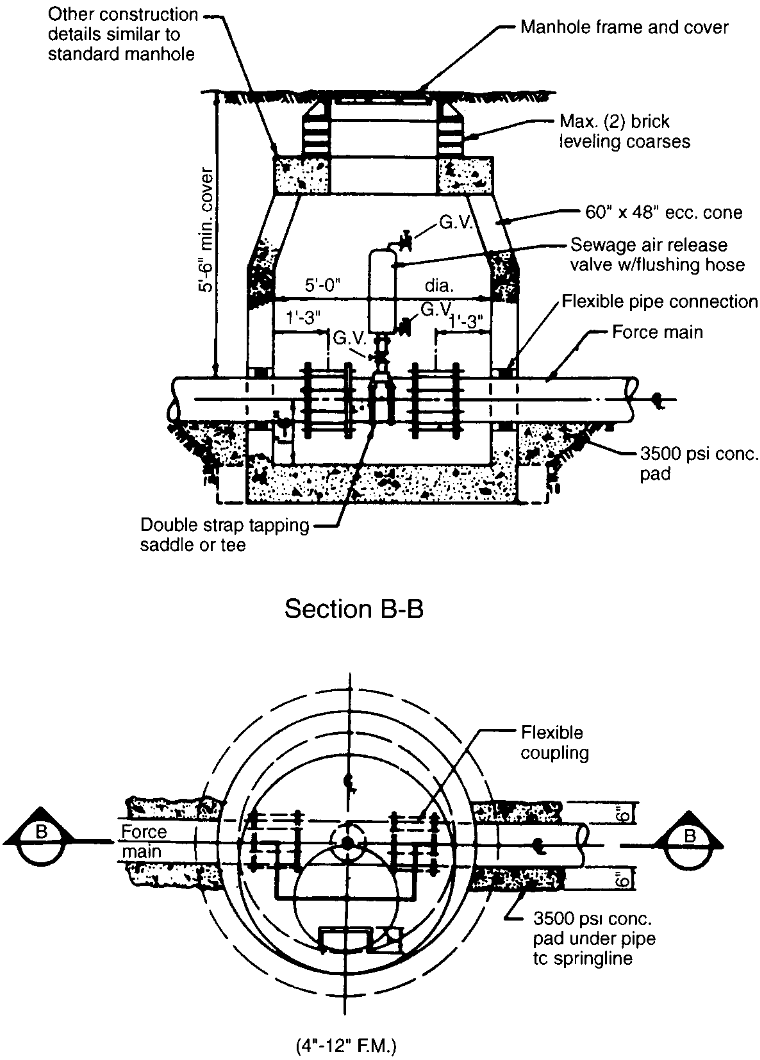


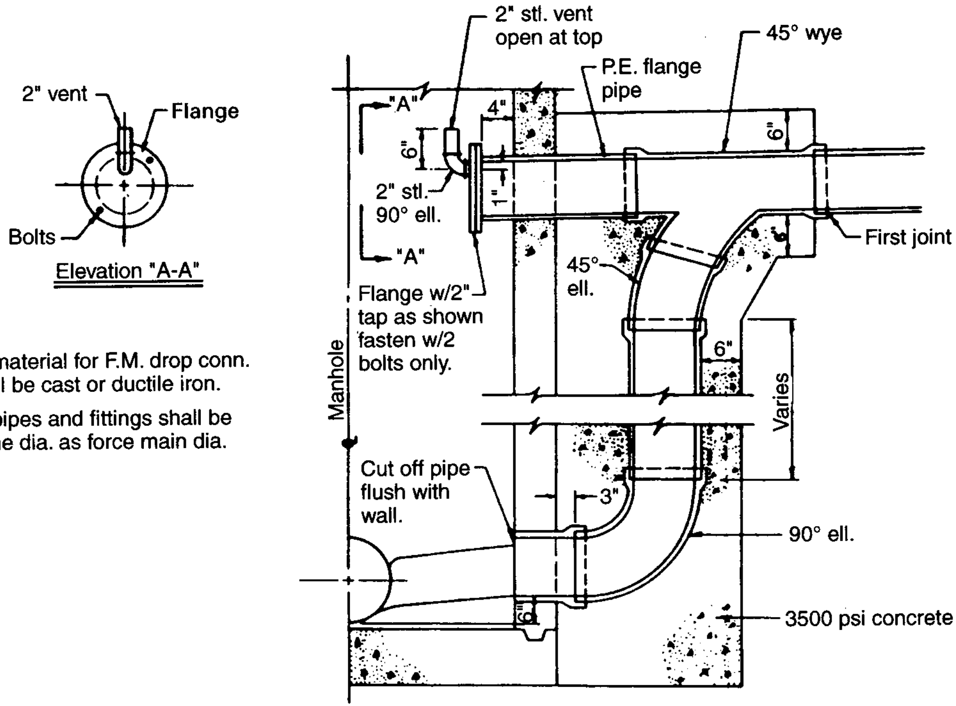
FIGURE 6.41 Air and vacuum release in manhole.

recommended air release at the final discharge into a public sewer manhole is illustrated in Fig. 6.42.

STORM WATER DISPOSAL

Storm Water Discharge Permit

Historically, the EPA National Pollutant Discharge Elimination System (NPDES) has focused primarily on the discharge of industrial process wastewater from mu-



<u>Notes</u>

1. All material for F.M. drop conn. shall be cast or ductile iron.
2. All pipes and fittings shall be same dia. as force main dia.

FIGURE 6.42 Air release valve at manhole.

nicipal wastewater treatment plants. Between 1978 and 1983, the EPA funded additional studies to measure the pollutants in storm water runoff.

The results of these studies concluded that a considerable amount of pollutants were carried to nearby lakes, streams, and the ocean. In 1990, the EPA issued storm water regulations that apply to both municipal and industrial storm water discharge. These regulations define who must apply for and obtain a NPDES permit for storm water discharge. Additional information can be obtained from the *Guidance Manual for the Preparation of NPDES Applications for Storm Water Discharge Associated with Industrial Activity,* published by the U.S. EPA Office of Water Enforcement and Permits.

Industrial facilities conforming to a specific ''industrial activity'' list and that discharge either directly into U.S. waters or into separate municipal storm water systems must apply for permits. Table 6.27 presents the list of industrial activities. There is a difference between industrial and municipal discharges. Only the industrial type discharges will be discussed. Storm water discharges into combined sewers do not require any permits. Also included in the list of industrial activities are construction sites and landfills associated with facilities.

Once it is established that a permit is required, the permit application must be prepared, usually by some person or organization specializing in this type of work. The field work (which consists of taking samples of storm water discharge) associated with the permit application must also be started. The following activities are necessary:

1. All survey and site plans of the facility, and also storm water sewer site plans, must be reviewed. A walk-through should be done to verify that the system has been installed the way it was designed.

2. A survey of the facility is required to note the amount, if any, of dry weather flow. In addition, any previously unknown or undocumented connections to the storm water sewer must be found, to the greatest extent possible. If any illegal connections to this system are found, they must be corrected. Another possibility is to permit each of the illegal connections separately, thereby making them ''legal.'' Experience with this approach has proven very time consuming and expensive. One of the requirements is to fill out an EPA form certifying that all outfalls have been tested for non-storm-water and nonapproved discharges.

3. It is an absolute necessity to find all of the storm water outfalls since experience has shown that the EPA has a very broad definition of outfalls. One way to identify outfalls is to find all the conveyances. A *conveyance* is any channel or passage that conducts or carries storm water, including any pipe, ditch, channel, tunnel, conduit, well, container, or discrete fissure. The EPA may determine that some outfalls are identical, thereby reducing the number that has to be monitored.

4. Any kind of flow from any conveyance, including that from grassy areas, are included. However, common sense must be applied. If a sample can't be collected from a particular area, then it probably would not be considered an outfall.

5. Samples taken must report actual quantitative data resulting from any storm water event. There are two types of samples required. The first type is called a *first flush grab sample* and must be taken during the first 30 min of a storm event. The second type is a *flow-weighted composite sample* for the entire event, and requires that a sample be taken during the event based on a predetermined volume of water flowing past a meter. A summary of pollutants to be analyzed are listed in Table 6.28.

TABLE 6.27 Industrial Activity for Storm Water Runoff

1. Facilities subject to storm water effluent limitations guidelines, new source performance standards, or toxic pollutant effluent standards under 40 CFR, subchapter N.
2. Facilities classified as:

SIC 24	Lumber and wood products (except 2434)	SIC 32	Stone, clay, and glass products (except 323)
SIC 26	Paper and allied products (except 265 and 267)	SIC 33	Primary metal industries
SIC 28	Chemicals and allied products (except 283)	SIC 3441	Fabricated structural metal
SIC 29	Petroleum and coal products	SIC 373	Ship and boat building and repairing
SIC 311	Leather tanning and finishing		

3. Facilities classified as SIC 10 through 14 including active or inactive mining operations and oil and gas exploration, production, processing, or treatment operations, or transmission facilities that discharge storm water contaminated by contact with or that have come into contact with, any overburden, raw material, intermediate products, finished products, by-products or waste products located on the site of such operations.

SIC 10	Metal mining	SIC 13	Oil and gas extraction
SIC 12	Coal mining	SIC 14	Nonmetallic minerals, except fuels

4. Hazardous waste treatment, storage, or disposal facilities, including those that are operating under interim status or a permit under subtitle C of RCRA.
5. Landfills, land application sites, and open dumps that receive or have received any industrial wastes including those that are subject to regulation under subtitle D or RCRA.
6. Facilities involved in the recycling of materials, including metal scrapyards, battery reclaimers, salvage yards, and automobile junkyards, including but not limited to those classified as:

SIC 5015	Motor vehicle parts, used
SIC 5093	Scrap and waste materials

7. Steam electric power generating facilities, including coal-handling sites.
8. Those portions of transportation facilities that are involved in vehicle maintenance, equipment cleaning operations, or airport deicing operations, or which are otherwise identified as industrial activities.

SIC 40	Railroad transportation	SIC 44	Water transportation
SIC 41	Local and interurban passenger transit	SIC 45	Transportation by air
SIC 42	Trucking and warehousing (except 4221-25)	SIC 5171	Petroleum bulk stations and terminals
SIC 43	U.S. Postal Service		

9. Wastewater treatment works with a design flow of 1 million gallons per day or more or required to have an approved pretreatment program under 40 CFR part 403.
10. Construction activity including clearing, grading, and excavation activities except operations that result in the disturbance of less than five acres of total land area which are not part of a larger common plan of development or sale.

(*Continued*)

TABLE 6.27 Industrial Activity for Storm Water Runoff (*Continued*)

11. Facilities where material handling equipment or activities, raw materials, intermediate products, final products, waste materials, by-products, or industrial machinery are exposed to storm water.

SIC 20	Food and kindred products	SIC 31	Leather and leather products (except 311)
SIC 21	Tobacco products	SIC 323	Products of purchased glass
SIC 22	Textile mill products	SIC 34	Fabricated metal products (except 3441)
SIC 23	Apparel and other textile products	SIC 35	Industrial machinery and equipment
SIC 2434	Wood kitchen cabinets	SIC 36	Electronic and other electric equipment
SIC 25	Furniture and fixtures	SIC 37	Transportation equipment (except 373)
SIC 265	Paperboard containers and boxes	SIC 38	Instruments and related products
SIC 267	Misc. converted paper products	SIC 39	Miscellaneous manufacturing industries
SIC 27	Printing and publishing	SIC 4221	Farm product warehousing and storage
SIC 283	Drugs	SIC 4222	Refrigerated warehousing and storage
SIC 285	Paints and allied products	SIC 4225	General warehousing and storage
SIC 30	Rubber and misc. plastics products		

TABLE 6.28 Summary of Pollutants to Be Analyzed

	Industrial	
Pollutant	First flush grab sample	Flow-weighted composite sample
Oil and grease	√	
pH	√	
Biological oxygen demand (BOD)	√	√
Chemical oxygen demand (COD)	√	√
Total suspended solids (TSS)	√	√
Total phosphorous	√	√
Nitrate and nitrite nitrogen	√	√
Total kjeldahl nitrogen	√	√
Any pollutant in the facility's effluent guideline	√	√
Any pollutant in the facility's NPDES permit	√	√
Any pollutant in EPA Form 2F Tables believed to be present	√	√

6. A system for monitoring storm water runoff shall measure rainfall and flow rates from outfalls and take a first flush grab sample and flow-weighted composite sample.

7. The flow meter measures open channel flow in a pipe or channel. There are a variety of methods used to measure flow rates in open channels, including ultrasonics, submerged pressure transducers, and bubblers. Weirs and flumes can also be used. When it is not practical to install any of the above, the Manning formula can be used. This is done by relating the channel shape, slope, roughness, and level to the flow rate. A meter can also be connected to a rain gauge. This combination would report on the date, duration, and amount of rainfall from each event. The flow rate and the known pipe size give the total flow from the event. Another function of the flow meter could be to automatically activate or signal a flow rate sampler when a flow-weighted sample must be taken.

8. A sampler is used to collect water samples from a flow stream. It consists of a pump, glass or plastic sample containers, and a controller. Samples can be taken at specific time intervals or after a set volume of water has passed the monitoring point. Another requirement may be to take a sample one time only (first flush) and keep it in a separate container. Regulations require that samples be taken from storm events ranging from ½ to 1½ times the average storm event.

In many cases, it is far less expensive and more desirable to train plant personnel to collect samples, rather than to rent and maintain automatic samplers and rain gauges. In order to do this, the following precautions must be observed:

1. Additional personnel must be available for the first flush grab sample. This is the most hectic time.
2. Collections should be made in pairs. Avoid collecting samples during thunderstorms and at night.
3. Extension rods should be used to take samples from manholes and ditches.
4. Holding times should be limited. Biological oxygen demand (BOD) tests must be made within 48 h and pH within 6 h. Preservatives in bottles are time limited.

POTABLE WATER SUPPLY

This section will discuss the source of potable water supply to a facility. This could be water obtained from a public utility, wells, or an approved surface water source. Included will be the distribution pipe extended from the source to a point just outside the building wall. Very often, the source will supply both the facility potable water and fire protection water. Potable water systems inside buildings are discussed in Chap. 9, Plumbing Systems.

Surface water sources are rarely used to supply potable water. This source of water is used most often for various process and production applications, cooling, and for fire protection. The use of water for these purposes is outside the scope of this book.

WATER SUPPLIED FROM A PUBLIC UTILITY

General

Public utilities are the most reliable sources of water supply. The utility company is required to provide water of sufficient purity to the facility, which relieves the facility of the requirements of monitoring and treating the water for impurities, thereby eliminating ongoing maintenance and technical problems. This service has a price, which must be balanced against the cost of electrical power, chemicals, and maintenance of supplying water from alternate sources.

When a single service is used to provide potable and fire protection water, the supply is considered a combined service and requires conformance to additional code requirements. It is recommended that separate supplies be provided for fire protection and potable water if possible. Some reasons for the separation are:

1. High pressures used for fire fighting may be detrimental to domestic piping and equipment. A pressure-reducing valve may be required on the domestic service.
2. Process use during an emergency may reduce the flow available for fire-fighting purposes.
3. System pressure may vary widely and be difficult to control.
4. Constant flow of water will increase corrosion of the combined main, in contrast to the relatively stagnant water in a dedicated fire main where much of the oxygen-causing corrosion is dissipated.

Codes and Standards

1. NFPA-24 for private water mains for fire service (if required)
2. AWWA codes for pipe materials, installation, and testing requirements
3. ASTM codes for pipe materials, installation, and testing requirements
4. Local plumbing and fire department requirements

5. Fire insurance carrier requirements

6. Authorities responsible for backflow prevention

System Components

In addition to the piping and valves, the system as a whole consists of backflow preventers, water meters, pipe joint restraints, fire hydrants, and air and vacuum release valves.

Connection to Utility Company Mains

Connection to a public main can be made either with the water shut off in the main or under pressure. Connections made with the water shut off are usually reserved for large building water services connected to large utility mains, but are also used if the main is too small to accept a new service line that is considered too large for a pressurized connection. A corporation cock that screws directly into a tapped hole made in the main is used for smaller connections. Very often, the utility company leaves branch connections in the main when the main is constructed. Their presence and location can be verified with the utility company.

Connection to a pressurized metallic main is done with proprietary methods using a tapping machine. For larger mains, the hole is made by the tapping machine through a tapping valve and sleeve that mates with the main and retains water after the drilling bit and pipe coupon are removed. The valve has an outlet flange for a future connection. For a detail of a typical tapping arrangement under pressure, refer to Fig. 6.43. For recommended minimum excavation dimensions, refer to Fig. 6.44.

For branch connections 2 in (50 mm) or smaller, a service clamp with a threaded outlet for connection to a corporation cock is used. The corporation cock is a fitting that has a male thread on one end which screws into the outlet on the clamp assembly, an integral shutoff, and on the other end a connection to match the future building service pipe material.

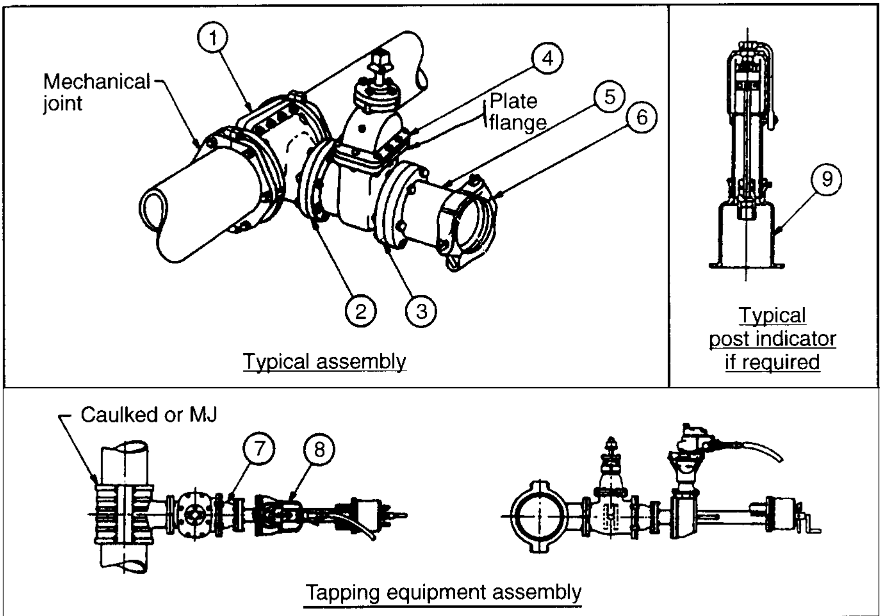
The best water main arrangement is in the form of a loop around the entire site within section valves. This will permit the flow of water in two directions, allowing service to most of the site if there is a break in any portion of the site main.

Valves

Valves in common use are gate and butterfly valves approved for underground service by the local authorities. Valves can be directly buried or installed in a valve pit. Installation of a valve in a pit permits easy access for repair and inspection, but is far more costly than direct burial.

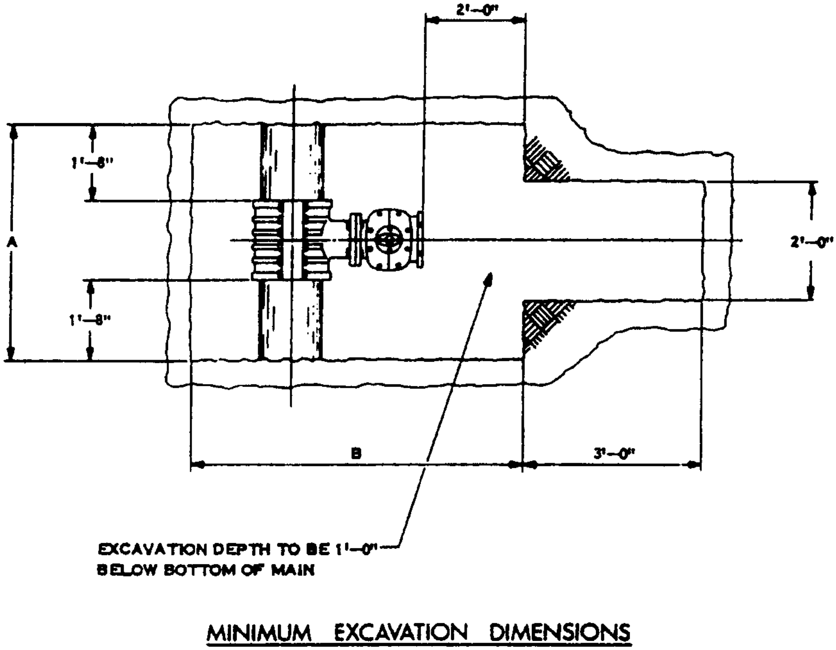
For valves directly buried, the operator can be provided with a nut instead of a handle or be extended above grade on a post called a *post indicator valve* (PIV), which indicates whether the valve is open or closed. Approval of the valve type and manufacturer by the insurance carrier or other authorities may be required.

The operating nut is made accessible through a valve box extending up to the surface. Operation is accomplished by use of a T handle wrench. Valves directly



1. Tapping sleeve, caulked type is usually available in line sizes 3" thru 48". Mechanical joint type is usually available in sizes 4" thru 12". Full line size outlets are not recommended. Tapping saddles may be used in place of tapping sleeves.
2. ANSI 125 lb flange inlet. Test tapping sleeve prior to installing tapping valve.
3. ANSI 125 lb outlet
4. Tapping valve. ANSI 125 lb flange inlet and flange outlet in sizes 2" thru 12". Conventional full-port gate valves may be used as tapping valves if hole is cut one size smaller than the valve size and stem threads are protected with a valve box.
5. Pipe–ANSI 125 lb flange one end, split coupling groove other end.
6. Split coupling in accordance with underground valve and piping specifications.
7. Adapter piece for attaching tapping equipment. Specify flange adapter (MJ is normally supplied).
8. Tapping machine.
9. Adjustable post indicator mounted on plate flange provided with tapping valve. Specify with valve and include depth of bury, centerline of valve to grade.

FIGURE 6.43 Pressure tapping of water main.



HEADER SIZE	A	B
3" THRU 8"	5' – 0"	5' – 6"
10" THRU 12"	5' – 6"	6' – 0"
14" THRU 24"	6' – 0"	7' – 0"

FIGURE 6.44 Pressure water main tapping. Excavation dimensions.

buried must have nonrising stems. The PIV valve operator extends above grade on a post that makes its location obvious, is easily accessible, and has an indicator to visually show if the valve is open or closed. A typical valve and valve box are illustrated in Fig. 6.45 and a typical PIV is illustrated in Fig. 6.46. If the PIV is located near a road, it is common practice to protect the valve with guard posts. Refer to Fig. 6.47 for a detail of typical guard posts. In general, the use of curb boxes instead of PIVs on a combined building service is discouraged for the following reasons:

1. They are difficult to locate and supervise, since they are often covered by dirt, snow, ice, or paving materials.
2. Constant care is required to prevent dirt and stones from entering the hole and preventing valve operation.
3. Delays may be encountered if the operating T wrench is misplaced or stored at a remote location.

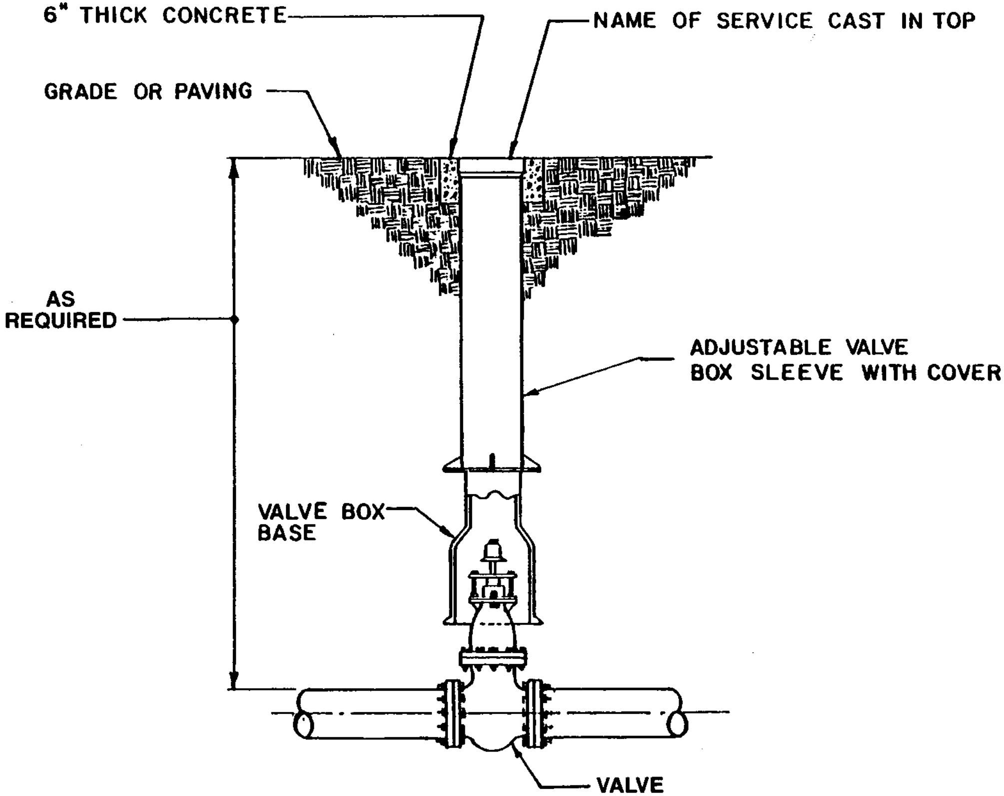


FIGURE 6.45 Valve box.

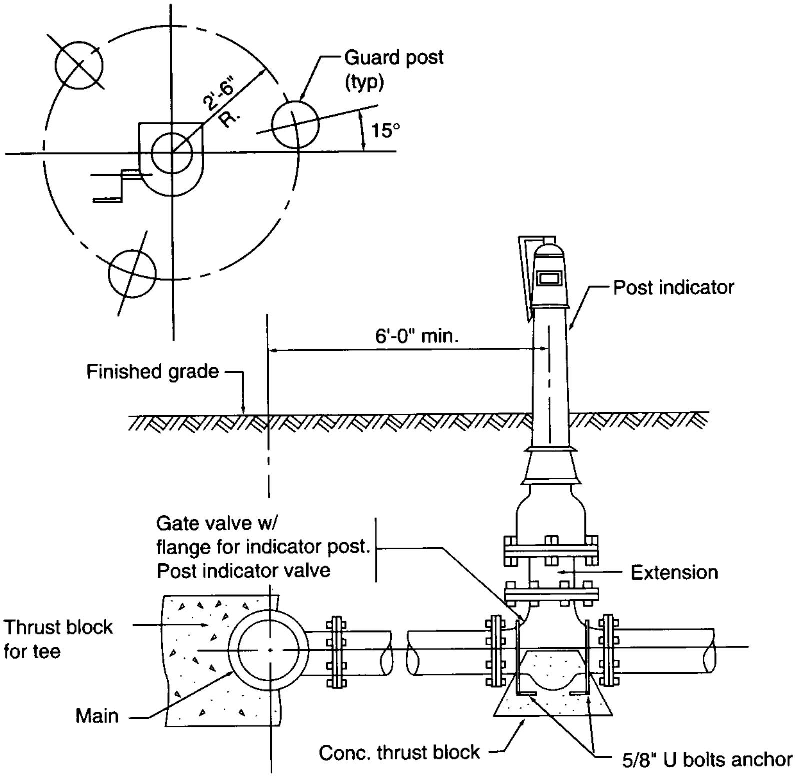


FIGURE 6.46 Detail of post indicator valve.

If the connection is for service to a site consisting of several buildings, the utility company requires that a shutoff or isolation valve be provided at the point of connection. Based on utility company requirements, a valve pit may be necessary or the valve could be directly buried.

On the site where water mains divide, it is good practice to provide an isolation valve, or section valves on all branch connections. For combined services, a post indicator valve is required.

Building Service Shutoff. All building water service branches must have a means of shutting off the water to the building from the site without the necessity of personnel having to gain access to the inside of a building. When the building service main is a combined service, a PIV located 50 ft (15.5 m) from the building wall is recommended. If this distance is not possible, the PIV shall be located as far away as practical. In urban locations, a shutoff valve is usually installed in a public street or adjacent sidewalk. This shutoff valve is commonly called a curb valve and is installed using a curb box, as shown in Fig. 6.45.

In addition to the shutoff valve outside the building, code requires that another shutoff valve inside the building must be provided as close to the water service entrance as space permits.

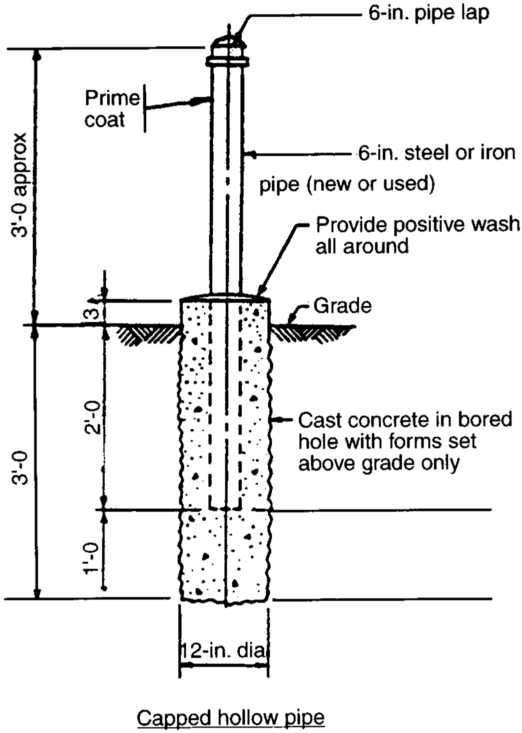


FIGURE 6.47 Detail of typical guard post.

Backflow Prevention

Because of concerns about contamination, the installation of backflow preventers on the facility service line is generally required by local authorities if they determine that a potential for contamination is present. External backflow prevention installed on a building or site service is different in concept from the internal backflow prevention required on or at equipment or devices inside buildings. The reasoning is that any possible contamination of public water mains that could originate from the building or facility must be prevented. This is achieved by isolating the facility service connection from the public main and containing any contamination within the facility service piping. Isolation and containment are accomplished by the installation of an approved device on the facility service pipe. The only acceptable backflow preventers (BFPs) are the reduced pressure zone (RPZ), the double check valve (DCV), and the air gap. The type of device used is based on the potential hazard. The types and categories of hazards vary widely and are known by various names in different localities.

There are two primary causes of contamination. The first is the fire department. If the service main is combined, the fire department may use a nearby source of water instead of the public water supply. A pumper may also have its own water supply for use immediately upon arrival to fight a fire until other apparatus arrives to make a connection to the public water supply. When the apparatus pumps contaminated water into the facility service, the water under high pressure must be stopped from entering the public mains. The other cause of contamination is the building itself, because if the public main breaks, it creates a vacuum which draws contaminated water from the facility into the public main.

For industrial facilities using a storage tank, an air gap must be provided in the water supply into the tank. This will provide the best possible protection of the public water supply.

Determining the Degree of Hazard. The choice of protective device is based on an evaluation of the degree of potential hazards. The following should be considered:

1. Use, toxicity, nature, and availability of contaminants
2. Availability of a supplemental supply of water
3. Fire-fighting system evaluation

Hazard Classifications. The facility rating or classification should be made only after consulting with the proper authorities. The categories mentioned cannot list every circumstance or facility type. Judgment must be used in the final selection. The following ratings are often used based on the use, toxicity, nature, and availability of contaminants:

1. *High* (*severe*) *hazard.* Any facility that uses chemicals considered toxic or has the potential for discharge of toxic waste is considered a high hazard. Typical facilities are hospitals, chemical processors and manufacturers, pharmaceutical processors and manufacturers, laboratories, food processors and manufacturers, industrial manufacturers and processors, and water and sewage treatment plants.

2. *Medium* (*moderate*) *hazard.* Commercial buildings and establishments, fire protection storage tanks and mains with no additives, and facilities that discharge water at higher than normal temperatures are medium hazards. The fire protection system will have only stagnant water present in the pipe.

3. *Nonhazardous* (*minor*) *hazard.* Private homes and commercial establishments without complex plumbing or fire protection systems are minor hazards.

The availability of a supplemental or auxiliary supply of water for fire-fighting purposes could create a situation that affects the hazard classification. If the facility is located close to a river or lake, the fire department could draw from this source to provide additional water during a fire and then this water of indeterminate toxicity could be pumped back through the combined facility service into the public main (if not protected). Also, well water of unsatisfactory purity that is used for any purpose could find its way back into the public water supply.

An evaluation of the fire protection system shall be made separately because of the number of variables involved. Factors such as whether the system is wet or dry, the presence of storage tanks and pumps, whether there is a direct connection to the public mains and interconnection with other water supplies must be studied in order to determine the degree of hazard. It is accepted practice to provide any water storage tank with an air gap between the pipe supplying the source of water and the highest possible level of water inside the tank. A gravity overflow two sizes larger than the supply pipe will provide sufficient protection from tank flooding.

Selection of the Appropriate BFP Device. Any facility classified as hazardous must be protected with either a reduced pressure zone (RPZ) BFP assembly or an approved air gap on the facility supply prior to connection to the public water main. For medium hazard, the selection of a double check valve (DCV) assembly is usually acceptable. For nonhazardous facilities, internal BFPs are usually sufficient, with no requirement for external BFP in the facility service line.

The above selections are generalities and must be approved and accepted by all responsible code officials and facility authorities.

Location and Installation of a BFP Device. For a site service main, the external BFP must be located at or very close to the property line of the facility. The installation requirements are obtained from the responsible authorities, which may be the local or state health department, the water utility, or the local building department. BFPs inside buildings must be located downstream of the meter or as close to the incoming service entrance as possible.

The installation of the BFP device depends on its type. RPZ BFPs must be installed above grade or above the level of the water main inside the building. This requirement is to ensure that the water released as a result of backflow will be freely discharged from the device with no restriction. The wastewater must flow away from the device into a drain and not form a pool where the device is located. The drain must discharge by gravity and be large enough to accept the full flow expected from the device. The drain cannot be flooded and must be screened to prevent the entrance of dirt and vermin. For most installations, this requires that the device be installed in an above-ground structure. The enclosure must be protected from freezing. A schematic illustration of an exterior RPZ BFP in a pit is shown in Fig. 6.48. A schematic illustration of an exterior RPZ BFP above grade is shown in Fig. 6.49. A schematic illustration of an interior RPZ BFP is shown in Fig. 6.50.

The DCV does not discharge water and can be installed in a pit below grade. It must be protected against freezing and access must be provided for operation of the test cocks. If the service main is installed below the frost line, this is considered acceptable freeze protection. A schematic illustration of an exterior DCV BFP in a pit is shown in Fig. 6.51.

The BFP must be installed in such a manner as to allow easy access for testing and maintenance.

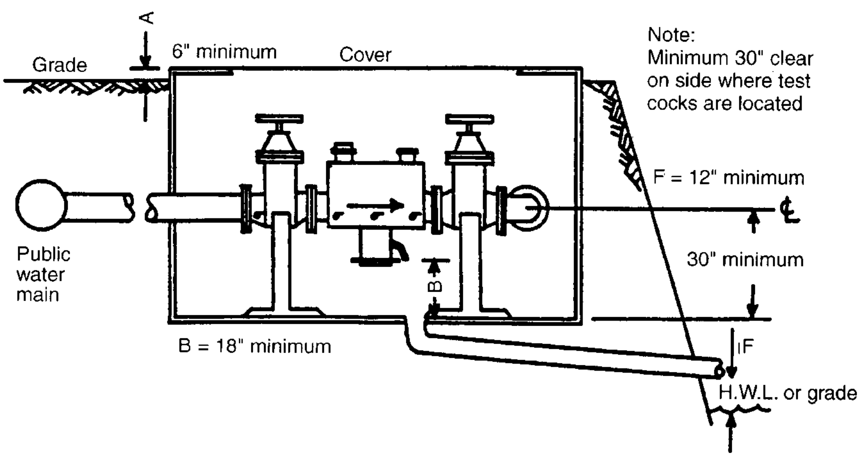


FIGURE 6.48 Detail of RPZ BFP in a pit.

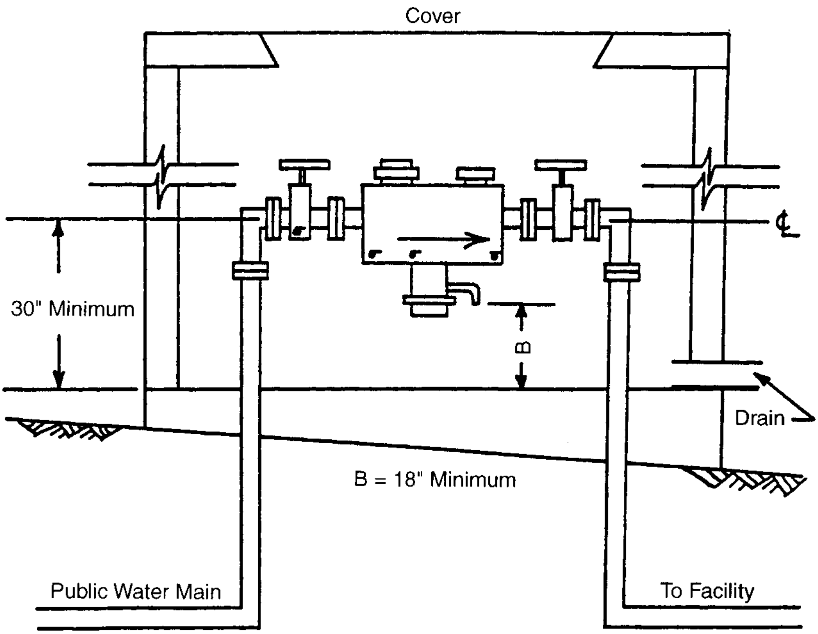


FIGURE 6.49 Detail of RPZ BFP above grade.

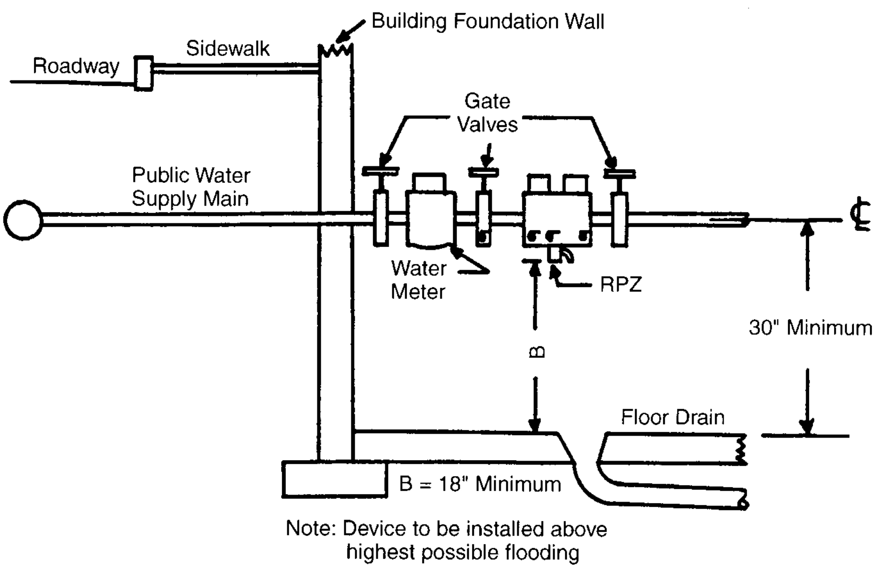


FIGURE 6.50 Interior detail of RPZ BFP.

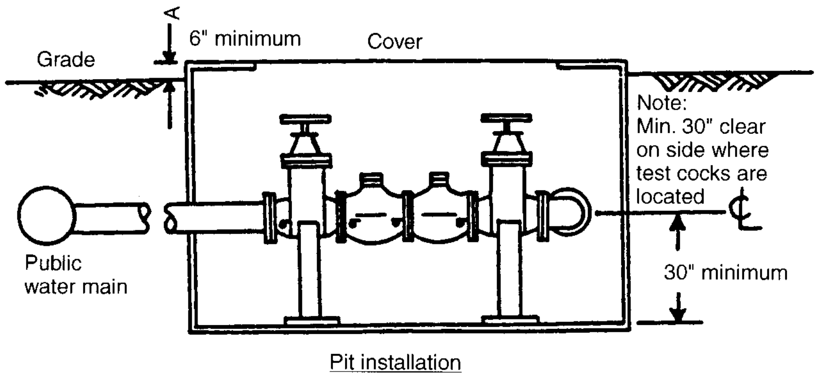


FIGURE 6.51 Detail of double check valve BFP in pit.

Water Meters

Since water obtained from a public utility must be paid for, a water meter is required to record the amount used. When there is no cost, meters are often installed by many facilities simply to record the amount of water used. Meters are selected by their function, accuracy, and pressure loss through the meter assembly. Registers on the meter are available to record usage in cubic feet or gallons (cubic meters or liters); remote reading and strip recording are usually available as options.

Meters can be installed at the property line, adjacent to a building, or inside a building. They can be installed aboveground or underground in a pit. The meter installation, called a meter assembly, usually includes valves, test tees, and a bypass around the meter to allow maintenance or replacement with no disruption to facility operation. The assembly arrangement is regulated by local authorities and the utility company.

Separate water meters are designed for fire protection or domestic water uses.

Fire Protection. Meters used for fire protection purposes are suitable only for large flows and minimal pressure loss. They are not very accurate. A "detector" function can be added to signal flow in a dedicated fire main. A check valve can also be an integral part of a fire water meter. The most popular meter is the detector-check type.

Domestic. The meters for domestic use should be very accurate and selected based on accuracy at designed maximum flow and pressure loss through the meter. Where wide extremes of flow are expected, a compound-type meter is used. This meter consists of two separate meters: one to accurately register low flow and another to register higher flow.

Where reasonably steady low to high flows are expected, turbine or disk meters are likely choices. Propeller-type meters are usually selected for large pipe sizes and very high, steady flows, or where low flows do not occur.

Smaller meters can be obtained to protect against frost with a breakaway bottom. If frozen, this is the only component that could have to be replaced. Special meters are required for hot water above 140°F (60°C).

Friction loss figures through meters at various flows are available from meter manufacturers.

Pipe Restraints

Mechanical joints and push-on joints seal when the elastomeric gasket becomes compressed in the space between the spigot end of the pipe and the bell. These joints are leaktight when installed on a straight line. When a fitting introduces a change of direction, the flowing water exerts a pressure on that fitting. (Pressures are listed in Table 6.29.) The joint is not capable of resisting this pressure, therefore a suitable means must be provided to restrain the joint from coming apart. The most common methods are integral joint restraint, clamps and tie rods, and thrust blocks.

Plastic piping with butt-fused joints has the same resistance as piping and does not require additional restraint. Split couplings with grooved pipe are also capable of resisting this pressure without restraint, if couplings designed for this purpose are used. This jointing method is generally recommended for new underground pipe installation.

Clamps and Tie Rods. Resistance is provided by clamps attached to the pipe at each side of the joint and rods connecting the clamps across each joint. Standard charts giving the required restraint for different size pipes, depth of bury, number of joints to be restrained, and different fittings are available.

Integral Joint Restraint. This type of joint uses integrally cast glands for mechanical joint pipe and internally locked, grooved, and keyed joints for push-on joints.

TABLE 6.29 Pressure Exerted at Joints by Flowing Water

Total pounds resultant thrust at fittings at 100 psi water pressure

Nom. pipe dia., in	Dead end	90° bend	45° bend	22½° bend	11¼° bend
4	1,810	2,559	1,385	706	355
6	3,739	5,288	2,862	1,459	733
8	6,433	9,097	4,923	2,510	1,261
10	9,677	13,685	7,406	3,776	1,897
12	13,685	19,353	10,474	5,340	2,683
14	18,385	26,001	14,072	7,174	3,604
16	23,779	33,628	18,199	9,278	4,661
18	29,865	42,235	22,858	11,653	5,855
20	36,644	51,822	28,046	14,298	7,183
24	52,279	73,934	40,013	20,398	10,249
30	80,425	113,738	61,554	31,380	15,766
36	115,209	162,931	88,177	44,952	22,585
42	155,528	219,950	119,036	60,684	30,489
48	202,683	286,637	155,127	79,083	39,733
54	256,072	362,140	195,989	99,914	50,199

Note: To determine thrust at pressures other than 100 psi, multiply the thrust obtained in the table by the ratio of the pressure to 100. For example, the thrust on a 12-in pipe with a 90° bend at 125 psi is $19{,}353 \times 125/100 = 24{,}191$ lb.

Thrust Blocks. Thrust blocks consist of cast-in-place concrete blocks, size and weight of which depend on the pipe size, water pressure (including surge loads), and safe soil-bearing load. Recommended locations for thrust blocks are illustrated in Fig. 6.52. These blocks must rest on undisturbed soil in order to resist the force exerted. Average safe soil-bearing loads are given in Table 6.30. These loads are general and must be verified for specific projects.

To determine the size of a thrust block, the thrust developed at the joint must be divided by the soil resistance. This will give the required area to be provided by the thrust block. General dimensions of thrust blocks for different fittings are given in Fig. 6.53.

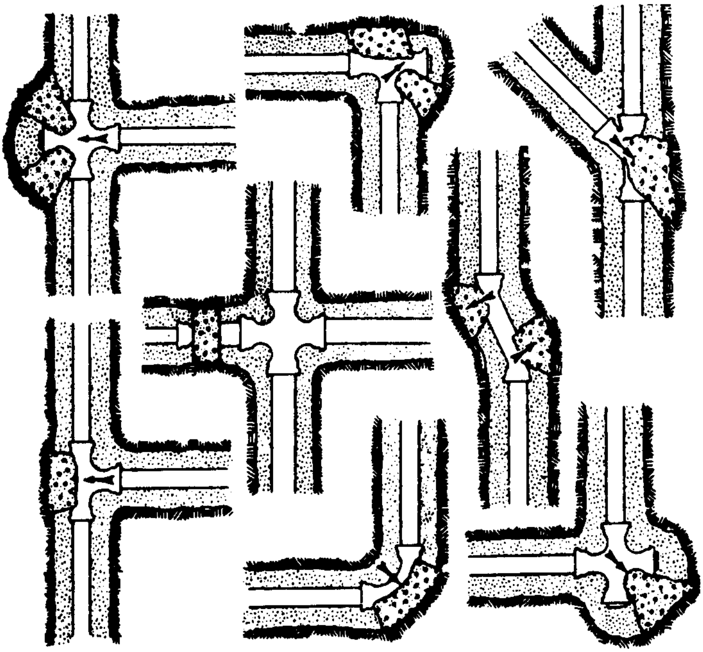
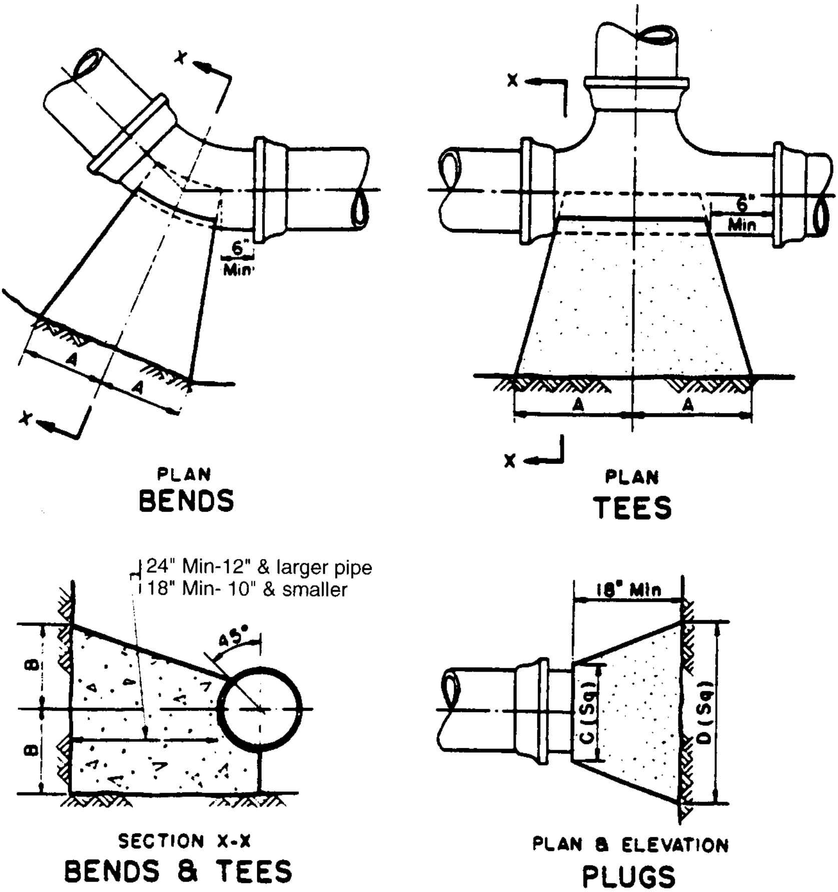


FIGURE 6.52 Locations for thrust blocks.

TABLE 6.30 Soil-Bearing Loads

Soil	Bearing load, lb/ft^2
Muck	0
Soft clay	1000
Silt	1500
Sandy silt	3000
Sand	4000
Sandy clay	6000
Hard clay	9000



TYPE	SIZE	1/4 BENDS		1/8 BENDS		1/16 BENDS		TEES		PLUGS	
		A	B	A	B	A	B	A	B	C	D
TYPE I 4000 PSF SOIL	6"	8"	10"	6"	8"	3"	8"	8"	8"	10"	15"
	8"	12"	12"	8"	10"	5"	9"	9"	12"	12"	20"
	10"	16"	14"	10"	12"	6"	10"	11"	14"	14"	25"
	12"	19"	16"	12"	14"	8"	11"	14"	16"	16"	30"
	14"	23"	18"	14"	16"	10"	12"	16"	18"	18"	34"
	16"	26"	20"	16"	18"	11"	13"	18"	20"	20"	38"
TYPE II 2000 PSF SOIL	6"	16"	10"	9"	10"	6"	8"	10"	12"	10"	21"
	8"	22"	13"	12"	13"	8"	10"	13"	16"	12"	29"
	10"	26"	17"	14"	17"	10"	13"	16"	20"	14"	36"
	12"	29"	21"	16"	21"	11"	16"	18"	24"	16"	41"
	14"	35"	24"	19"	24"	12"	20"	22"	27"	18"	48"
	16"	38"	27"	21"	27"	12"	24"	24"	30"	20"	54"

Note: Based on 100 p.s.i. static pressure plus AWWA water hammer
All bearing surfaces to be carried to undisturbed ground

FIGURE 6.53 Typical thrust block dimensions.

In general, thrust blocks are not recommended for the following reasons:

1. The blocks are comparatively large.
2. Often, undisturbed soil is not present.
3. High cost.
4. Space for the blocks may not be available due to the proximity of adjacent site utilities.

Air and Vacuum Relief Valve

When water is run in pipes, air pockets often develop at high points in the system. Air pockets reduce the area of the pipe thus lowering the flow rate and increasing the pressure loss. The device used to eliminate the air is called an *air release valve.* The following locations may require the installation of air release valves, but rarely at every location:

1. High points of the pipeline, in particular the hot water system
2. At aboveground locations, where the pipe rises up and then returns underground
3. Upstream from orifices, reducers, or other similar obstructions

A vacuum can be created when a pipeline is emptied. If a service line is intended to be emptied regularly, a vacuum relief valve should be installed. Also, for long runs of thin-walled pipe, a vacuum relief valve should be installed to avoid pipe collapse.

Combination air pressure and vacuum release valves are available from manufacturers. These devices are selected based on main pipe size, main pressure rating, main pipe material, and wall thickness.

Fire Hydrants

Fire hydrants are found only on combined services. A fire hydrant is a type of aboveground valve that provides a dedicated means for fire department apparatus to connect directly to a piped source of water.

There are two categories of fire hydrants, dry barrel to prevent freezing and wet barrel. The dry barrel has a device that allows the water in the barrel to drain down to the frost line after the valve that controls water to the outlets has been shut off. The dry type is recommended for all areas that have any remote possibility of freezing. Experience has shown that freezing only once in 25 years is sufficient reason to install dry-type hydrants. For a detail of a typical dry-type hydrant, refer to Fig. 6.54. Shutoff valves for the hydrant are optional. Check local code and client preference for installation requirements.

The sizes of connections available on hydrant barrels vary. All connections have a threaded end onto which fire hose is connected. These threads must be compatible with the local fire department and the facility on whose property the hydrant has been installed. The threaded ends are protected by a cap attached to the hydrant by a chain. A single internal valve, terminating in an operating nut on top of the hydrant, controls all of the connections. A special wrench is used to turn the nut. Special hydrants can be obtained with individual valves on each outlet. The most common sizes are nominal 2½ in (60 mm), 4 in (100 mm), and 6 in (150 mm). The two larger sizes are called pumper connections. The most common arrangement

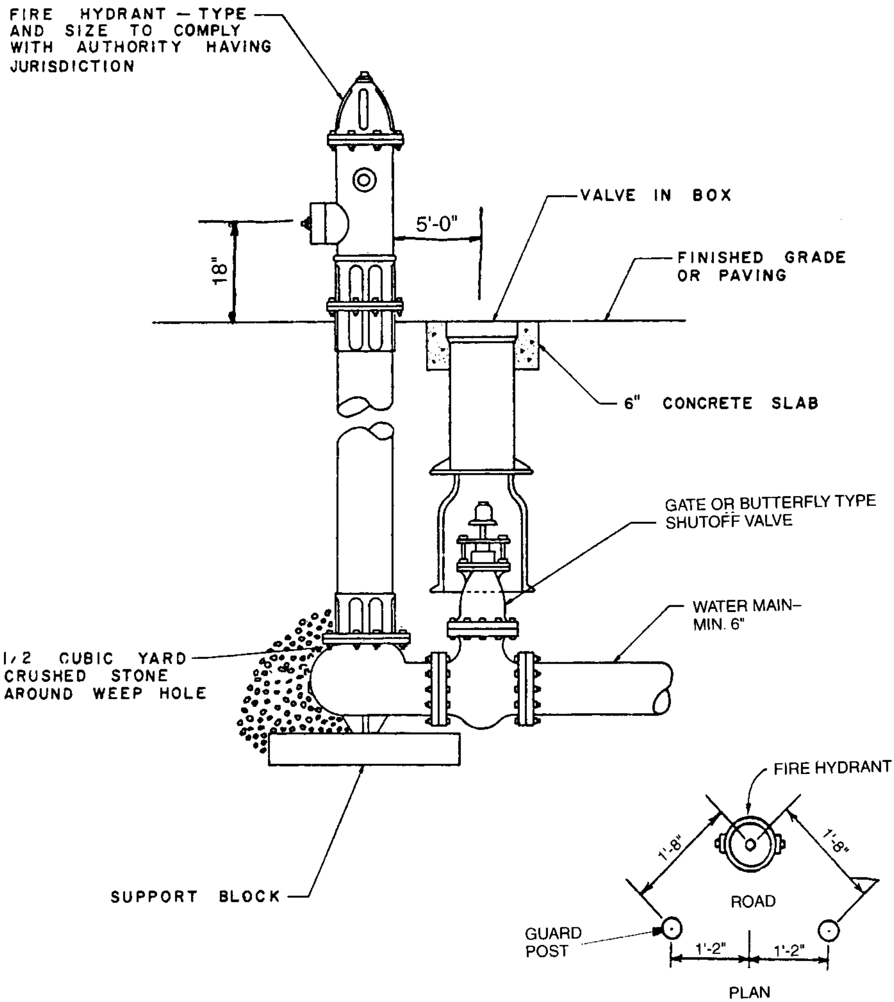


FIGURE 6.54 Typical fire hydrant.

consists of two 2½-in (60-mm) connections, mounted at a 90° angle, facing the road. When a larger size pumper connection is added, the pumper connection faces the road and the other, smaller connections, are arranged at 90° angles on either side. Generally accepted practice is not to provide a pumper connection unless specifically required by local code or client preference. Pumper connections are used when a cross-connection is needed between two water mains. Fire hydrants shall be rated for a minimum of 300 psig (2000 kPa).

Locating and Installing Hydrants. Hydrants are typically located 250 to 300 ft apart (78 to 90 m). For large sites, it is recommended that two hydrants be installed 200 ft (65 m) from any point where a significant hazard exists. They should be located about 40 ft (13 m) from a building wall under normal conditions, with a longer distance required if the building stores or uses explosives, flammable liquids, chemical products, petroleum products, or high piled storage. Excessive radiant heat and the danger from falling building walls are factors in considering longer distances. Hydrants are usually placed 2 to 10 ft (0.6 to 3 m) from center of hydrant to the curb of a road. Hydrant outlets are usually located from 1 ft, 3 in, to 2 ft (37 to 60 cm) above grade.

Hydrants are generally located adjacent to roads that will allow fire department apparatus to approach near enough to the hydrant to allow easy hookup. If this is not possible, a clear path over a lawn, for example, from a road shall be available.

A valve is often located between the water main and the hydrant to allow servicing of the hydrant without shutting down the water main. Opinion is divided about the need for this valve, since hydrant servicing is rarely necessary. The installation of the valve is a client preference.

Since the hydrant is located adjacent to roads where trucks travel, it is a generally accepted precaution to provide guard posts to protect the hydrant.

Fire hydrants are often painted different colors or marked to identify the flow capacity and/or pressure of the individual hydrant. This is regarded as a great help to fire department personnel and should be considered.

Monitors

A *monitor* is a permanently mounted fire protection nozzle assembly, connected to a water main and capable of being rotated and elevated. The nozzle can be a standalone unit or attached to a fire hydrant. A standalone monitor for a warm climate is illustrated in Fig. 6.55. A monitor installed on a hydrant is illustrated in Fig. 6.56. Monitors are usually installed as safeguards for significant hazards such as chemical and petroleum product storage. When mounted on towers, monitors can be used for vapor suppression from specific hazards.

The advantage of a monitor is that it can be placed into service very quickly, can be aimed in the proper direction, and, once set, can be left unattended.

Monitors are available in 500 to 2000 gpm (3150 to 12,600 lpm) models. They shall be located about 150 to 200 ft (45 to 62 m) apart, with all protected areas capable of being reached with two streams. A general requirement of 80 psig (550 kPa) minimum pressure is necessary for proper operation, which usually requires a fire pump. Consideration shall be given not to place the monitors too close to a hazard because of the potential heat generated by fire.

Pipe Material Selection

The pipe material selected depends first on whether the system is combined or potable only. Other factors are availability, pressure rating, water quality, soil conditions, depth of bury, pipe strength requirements, and approvals of various local authorities.

For potable water piping 2 in (50 mm) and smaller, the most commonly used metallic piping is soft temper K copper tubing and copper fittings. Plastic piping includes PP and HDPE plastic pipe and fittings. For sizes 2½ in and larger, the most commonly used metallic piping is cement-lined ductile iron pipe and fittings. Plastic piping includes FRP and PVC pipe and fittings.

For combined systems, the two piping materials used most often are ductile iron and PVC, approved for use as fire protection mains.

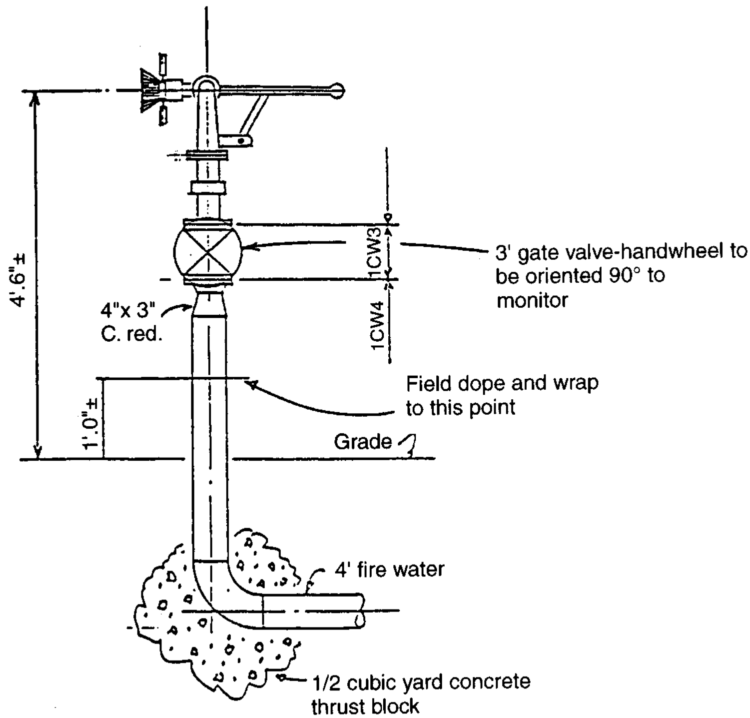


FIGURE 6.55 Typical standalone monitor for warm climate.

System Design Procedure

Preliminary Information Required

1. The location, size, material, and depth of the public water main available to the project under design must be known.

2. The water pressure must be determined for the specific elevation closest to the point where the proposed service will be connected. This requires that a flow test be conducted in order to be accurate. This test will give the static pressure (with no water flowing), the residual pressure (the water pressure observed with a flow rate), and the time of day when the test is conducted. Several flow rates and residual pressures should be recorded to allow plotting on a flow chart.

3. Determine the authority responsible for installation and approval of (and the need for) any BFP devices for the specific facility.

4. Obtain from the utility company or local authorities the type of water meter required, the location of the meter, and the extent of work provided by the utility company versus the amount of work required to be done by a contractor. Usually, the utility company has standard details for meter assembly installations and the names of specific devices required to be part of the meter assembly and their arrangement.

5. The frost depth must be found for the area as well as the minimum cover over the proposed water line.

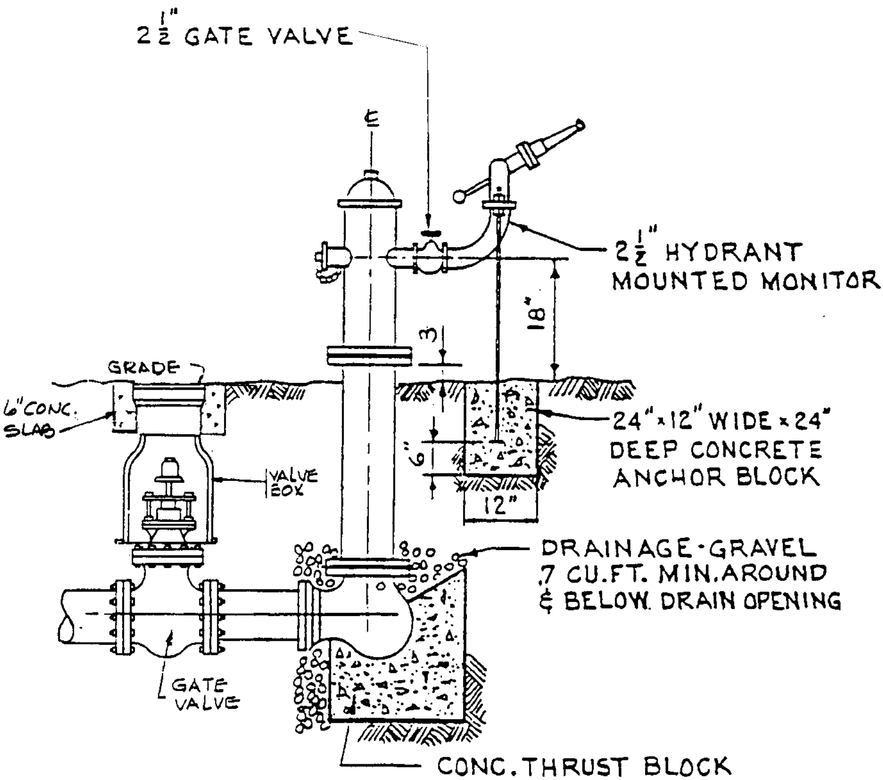


FIGURE 6.56 Typical hydrant-attached monitor.

6. Other special items or criteria that will affect the building service installation or add to the friction loss of the water service must be determined.

7. The maximum flow rate for the project must be calculated.

Calculation of the Pipe Size

1. Find the residual water pressure and the elevation at which the pressure was taken.

2. Calculate the maximum flow rate for the domestic water service.

3. Locate and lay out the service main from the building wall to the point of connection with the utility service or site main. Locate all devices (curb valve, BFP if required, meter, strainer, building valves, etc.).

4. Calculate the static pressure loss based on the elevation of the water main and the elevation of the service entrance into the building.

5. Select the pipe material.

6. Add up all the losses in the service main:

- Losses from valves, meters, BFP devices
- Pressure loss (or gain) from the difference in elevation (from step 4)
- Friction loss in service main to building entrance

Add an allowance of between 5 to 10 psig loss for future pressure drop in the public main. The pressure used shall reflect the possibility of other projects being built in the area. Size the pipe based on maximum flow rate and the above criteria, using the friction loss as a variable along with the velocity of the pipe size selected.

7. Determine if the available pressure is high enough for the selected building water distribution system.

WATER SUPPLIED FROM WELLS

A well is a vertical hole or shaft, excavated or bored through the earth for the purpose of bringing water to the surface from an aquifer. A major consideration is that construction of a well must protect the overall quality of the groundwater in the aquifer. Wells are often classified as deep or shallow. A shallow well is considered to be about 50 to 100 ft (15 to 30 m) deep. Deep wells are generally considered a superior source of water because the water is less susceptible to contamination and the depth of the aquifer usually fluctuates less than water in a shallow well.

Codes and Standards

1. The Safe Drinking Water Act
2. Local requirements

Water Well System Components

The total well system consists of the well pump and the well structure, which is made up of the borehole and casing, intake section, grout, and the distribution pipe that brings the water to the surface.

Well Pumps. The well pump is the means used to bring water in the aquifer to the surface. Three types of well pumps are generally used: vertical turbine, jet pumps, and submersible pumps.

Vertical turbine pumps are centrifugal pumps with the motor mounted on the surface over the borehole and the impeller suspended in the aquifer. The impeller is enclosed in a bowl and connected to the motor by a long shaft, positioned by bearings in the discharge pipe. This type of pump is well suited for larger flow rates, deep wells, and high discharge heads. A typical vertical pump installation is illustrated in Fig. 6.57.

Jet pumps are centrifugal pumps that use the flow of water through a special fitting to create a partial vacuum at the bottom of the well, which draws additional amounts of water into the discharge pipe. This is a two-pipe system—one pipe supplies water down into the well and the second discharges the water to the surface for use. The disadvantages of jet pumps are that since a portion of the discharge

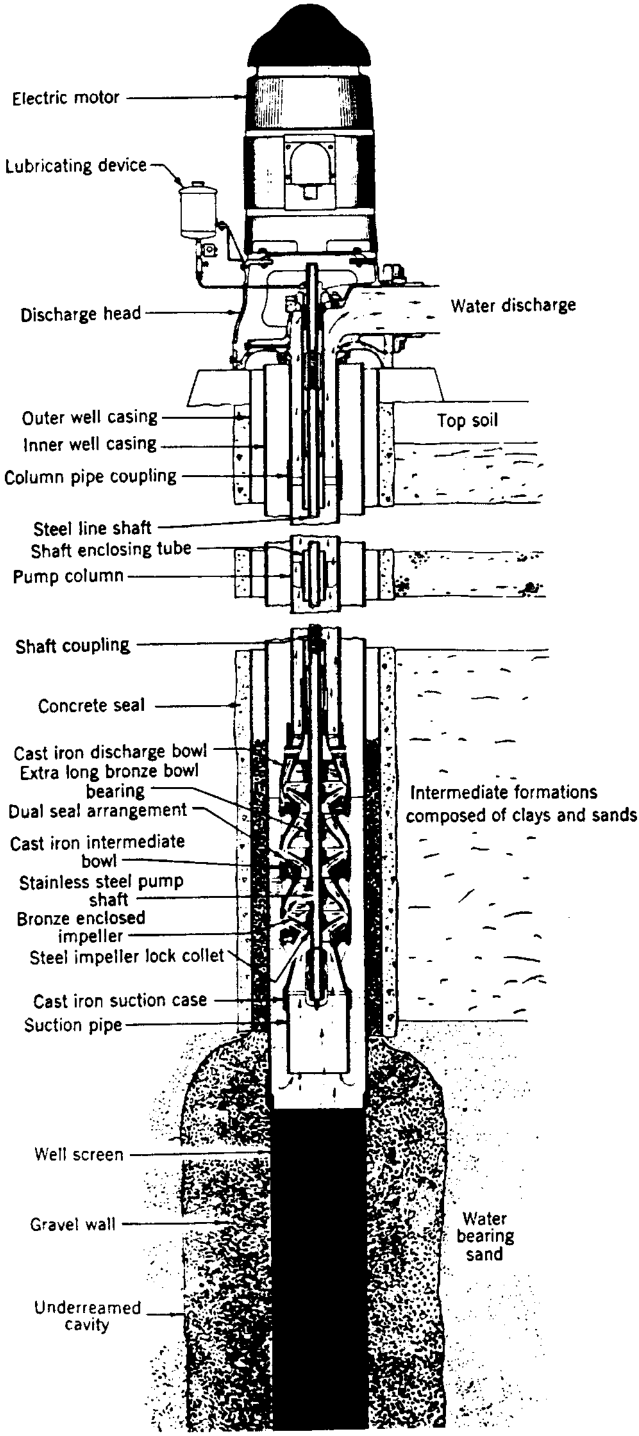


FIGURE 6.57 Typical vertical turbine installation.

must be pumped back down the well, the jet pump is generally used for smaller flow rates. In addition, a larger casing is usually required. The advantage is that it is located at the surface for ease of maintenance. The pump could be located over the well or in a remote location. A simplified, remote deep well jet pump installation is illustrated in Fig. 6.58.

The *submersible pump* system (Fig. 6.59) is a small diameter, totally self-contained centrifugal impeller in a housing that is close-coupled to an electric motor. It is placed inside the casing near the bottom of the well. The pump is supported by the discharge pipe and available in a wide range of flow rates and pressures. The electrical supply is attached to the discharge pipe. The submersible pump must be installed lower than the expected lowest drawdown level of the well. Advantages are avoidance of aboveground installation enclosures, installation in wells that have

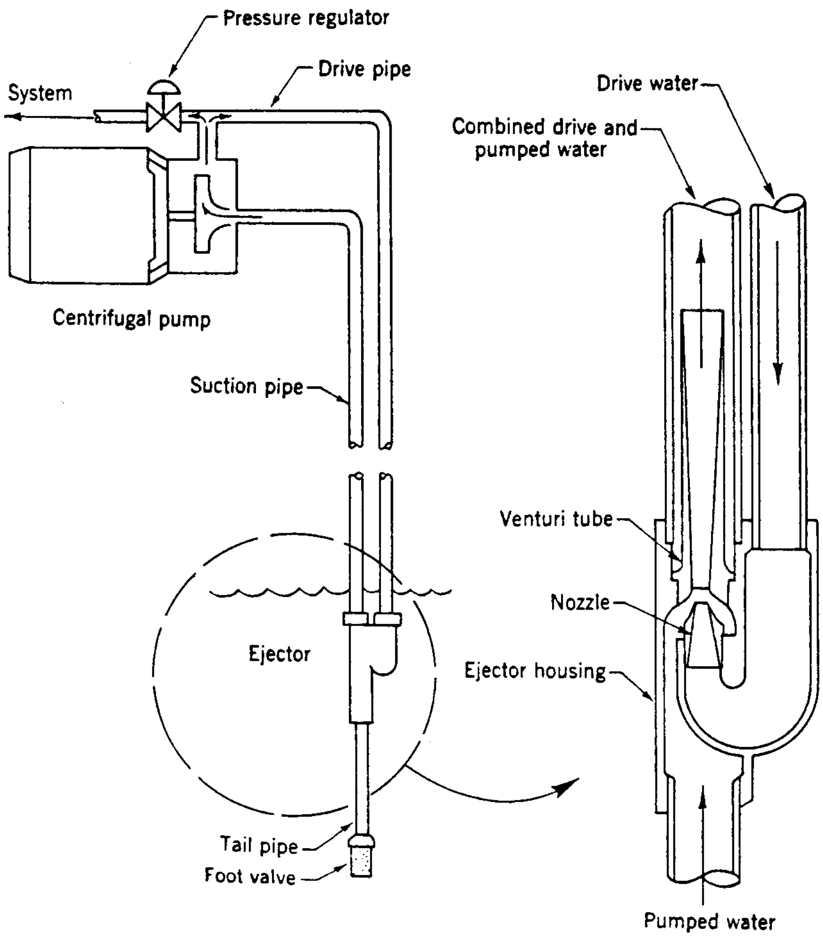


FIGURE 6.58 Simplified deep jet pump installation.

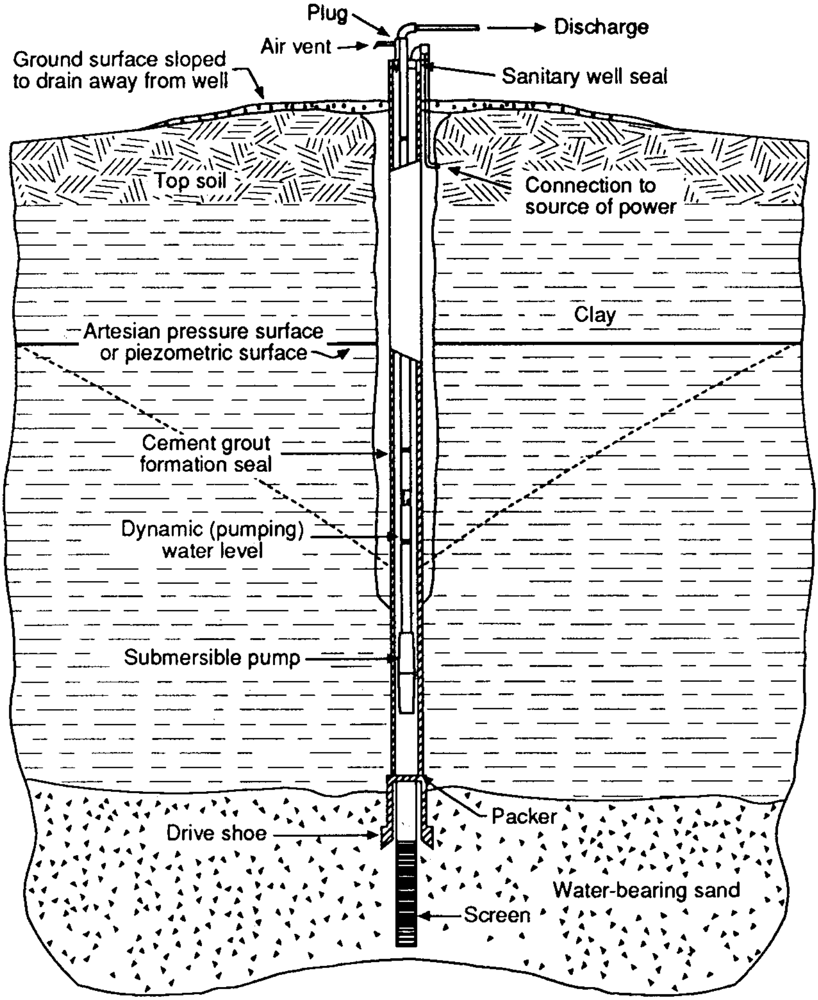


FIGURE 6.59 Typical submersible pump installation.

crooked casings, little noise, and ease of maintenance. An exploded view of a submersible pump is illustrated in Fig. 6.60.

Casing. The casing is a thin-walled cylinder placed in the borehole of the well, usually as it is drilled. It acts as a lining and extends from ground surface to the bottom of the well where the intake screen is located. It helps to withstand shifts in the earth and prevent cave-ins. It also helps to eliminate seepage of contaminants into the well from undesirable aquifers. Casing material is available in steel, fiberglass, or plastic.

Casing is subject to physical and chemical forces. The physical forces are tension, column loading, and collapse pressures. The chemical forces are those from

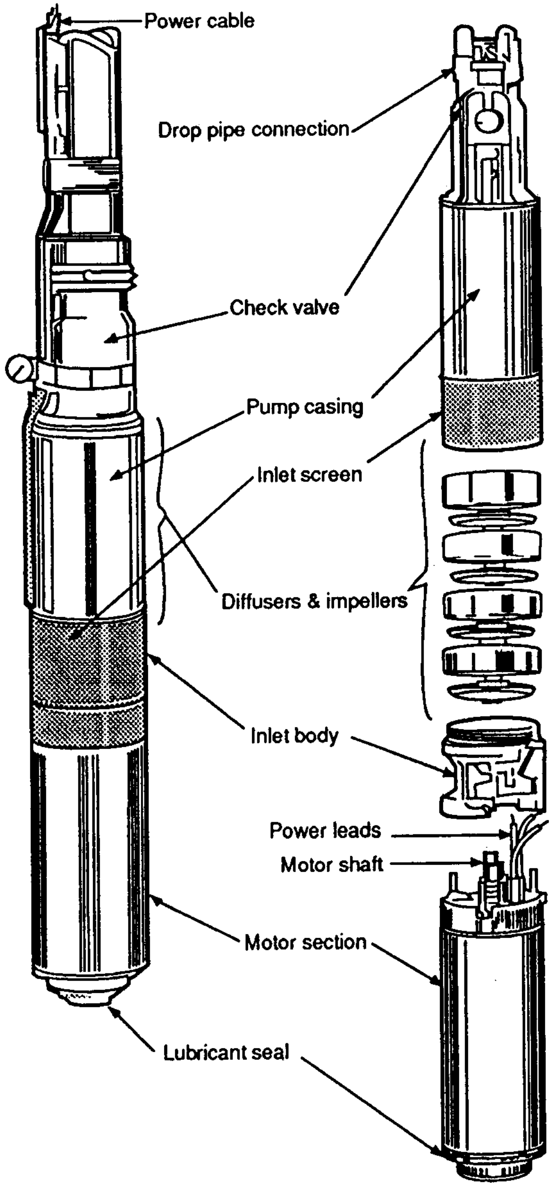


FIGURE 6.60 Exploded view of submersible pump.

contaminants in the soil and groundwater that cause corrosion and degradation of the casing.

Plastics are used primarily in short, small diameter wells, fiberglass is primarily used for highly corrosive waters, and steel, the strongest material, is used most often. Cable tool drilling requires the use of steel casing.

The casing diameter is generally based on yield. The recommended diameters of wells based on yield are listed in Table 6.31. Another consideration in selecting the casing size is to keep the maximum flow through the smallest part of the casing at or below a generally accepted velocity of 3 fps (1 mps). Another common practice is to use casing two sizes larger than the discharge pipe required by the pump selected.

Well Intakes. The intake is regarded by many as the heart of the well. Two of the most common intake constructions are the tube type, often called naturally developed, and the gravel-packed type.

The *well screen,* or intake, is a perforated device that allows water to move freely from the aquifer into the well while preventing sand from entering. It is connected to the end of the casing. Without the screen, many wells would not be capable of producing a useful flow of water. Typical well screens are slotted, louvered, and wire-wrapped. The wire-wrapped screen is well suited for large diameter, high yield wells. Wells drilled in consolidated rock are called *naturally developed* because the fractured rock acts as an intake and there is an absence of sand. There is little need for a screen intake in this case. Intakes are commonly manufactured from plastic, bronze, steel, stainless steel, and Monel metal. The length of the screen is determined by the type of aquifer (confined or unconfined), diameter of the casing, and the expected flow rate. For large flow rates, lengths of 30 to 40 ft are not uncommon. Generally accepted criteria limit the velocity of water passing through the screen to about 2 to 4 in/s (50 to 100 mm/s). It is recommended that the intake be located at the lowest practical portion of the aquifer so that the drawdown can be as great as possible.

Gravel packing is a common method of enhancing the flow of water by reducing the intake of sand particles at the well intake. This procedure replaces material around the screen with gravel or sand packed around the intake to maximize yield and prevent clogging of the intake openings by fine sand in the aquifer.

TABLE 6.31 Recommended Well Diameters

Anticipated well yield, gpm	Nominal pump bowl size, in	Optimum well casing size, in	Smallest well casing size, in
Less than 100	4	6 I.D.	5 I.D.
75 to 175	5	8 I.D.	6 I.D.
150 to 400	6	10 I.D.	8 I.D.
350 to 650	8	12 I.D.	10 I.D.
600 to 900	10	14 O.D.	12 I.D.
850 to 1300	12	16 O.D.	14 O.D.
1200 to 1800	14	20 O.D.	16 O.D.
1600 to 3000	16	24 O.D.	20 O.D.

Grouting. Grout is a material used to fill the annular space between the casing and the borehole and also to seal the well at the surface. This material must be able to be pumped far underground and capable of hardening sufficiently to form a seal. The material used most often is a slurry of plain concrete or a mixture of concrete and up to 8% bentonite. Other additives are available that resist water intrusion, add strength, and resist corrosion. The concrete/bentonite mixture has less shrinkage than plain concrete. Grouting and sealing are done for the following reasons:

1. To prevent seepage of polluted surface water down into the well along the outside of the casing
2. To seal out water of unsuitable quality from the strata above the aquifer
3. To stabilize and secure the casing in the drilled hole
4. To form a protective barrier around the casing, thereby increasing its life and protecting it against corrosion

It is not necessary to place grout down to the lowest level of the well, but only to the lowest level where it is necessary to keep out pollutants of concern.

Well Drilling

There are two primary methods used to drill a water well, cable tool drilling and rotary drilling. The choice of method depends on the expected borehole depth, access to the site, purity of the water required, and, often, the driller's preference.

Other methods, such as digging wells by hand, using water jets (jetted wells), and using an earth auger turned by hand or light machines (bored wells) are rarely used to obtain water for any facility, and are outside the scope of this book.

Cable Tool Drilling. The cable tool rig is often referred to as the ''yo-yo'' method. This system is based on the simple lifting and dropping action of the tool string in the borehole. Gravity, the weight of the drill bit, and the rotation of the drill string break up and cut through the underground formations. The *tool string* consists of the rope and connecting socket that connects the string to the surface engine, drilling jars (optional), the drill stem which serves as a guide to the string, and the drill bit. Water is added to produce a slurry mixture of the cuttings at the borehole bottom. All of this is connected at the surface to a derrick that lifts and drops the entire string.

Cuttings from the borehole must be removed periodically. This is done by means of a *bailer,* which is a 10- to 20-ft (3- to 6-m) section of pipe with a check valve that holds the cuttings along with the slurry. The tool string is raised clear of the hole and the bailer is lowered down for this purpose.

Although not used very often, the advantage of the cable tool method is the low cost of the entire rig as a whole, simplicity and sturdiness of the design and its ability to move over rough terrain. It is also able to collect qualitative data on the water-bearing characteristics of various strata as the casing is being inserted. Disadvantages are relatively slow drilling speed and the need for a casing to be inserted when drilling through unconsolidated formations.

Rotary Drilling. The rotary drilling method is used to drill approximately 80 percent of all water wells. The *rotary drilling* process bores a hole by means of a rapidly rotating drill bit that has a constant downward pressure exerted on it. The rotary drilling system consists of the rig, a pump to circulate or create the drilling fluid, the drill stem, and a drill bit. The rig is made up of a derrick and hoist, a motor, and a removable, rotating table placed directly over the borehole through which the drill stem passes. The drill stem is composed of the drill collar which connects the bit to the pipe; the drill pipe, which connects the bit to the surface; and the kelly, which is a fitting at the top of the pipe that imparts rotary motion to the stem from the table.

The use of drilling fluids is required in rotary drilling. Drilling liquid or air performs the following four primary functions:

1. Removes cuttings produced by the bit.
2. Transports cuttings up to the surface. The fluids can be interrupted for information about the geophysical features of the borehole.
3. Maintains borehole stability.
4. Cools the drill bit.

Rotary drilling is separated into two primary categories based on the method used to drill the borehole, liquid (or hydraulic) and air rotary drilling.

Hydraulic Rotary. This method can be used to drill in most formations. It functions by using a rotary drill along with the pumping of the drilling fluid which is generally drilling mud, referred to in the field simply as mud. This could be any type of mixture. The filter cake that forms around the length of the borehole wall from the mud flow assists in retaining soft formations and provides a protective layer that resists corrosive effects of the drilling mud.

Air Rotary. This method is mostly used to drill in consolidated formations. Often, mud or water is also used in the unconsolidated strata before bedrock is encountered. The same type of tools are used as for drilling. The difference is that compressed air, usually around 50 psig (340 kPa), is used to depths of about 250 ft (80 m).

Water Well Design and Construction

The design of a well includes criteria for present and future flow rates, the purpose of the well, and its expected useful life.

The drilling or construction of a well requires prior planning. Criteria such as probable depth of water-bearing formations and required permits and approvals of various agencies must be obtained. It must be understood that as the drilling of the well progresses, plans and methods may have to be altered or modified based on actual formations and drilling conditions encountered.

There are several issues that should be considered in the design of a well:

1. Minimum desired output (yield) of the well and the conditions under which the flow is desired.
2. Proposed diameter of the well casing.
3. Out-of-plumbness of the borehole. A common maximum figure is 1 well diameter per 100 ft (33 m).
4. Out-of-straightness. Generally accepted requirements should be limited to a dimension that would permit passage of a 33-ft (10-m) long blank, ½ in (13 mm)

less in diameter than the casing inside diameter. Straightness is more important than being plumb.

5. Screen characteristics, including size and type of openings, length, and net open area.
6. Casing material, type, and weight.
7. Cleaning and disinfection.
8. Development and testing.

During the drilling of the well, constant analysis of the drilling fluid to find an aquifer should be made as the bit passes through each formation. There are no set standards to recognize water-bearing strata. The experience of the well driller is important in this respect. Often, a pilot well drilled for this purpose is highly recommended.

When the borehole is complete, the casing shall be installed (if not previously done), the water level in the well accurately measured, gravel packing done, the intake connected, and the well grouted. The last step in the construction process is disinfection, where a strong chlorine solution is introduced into the well. It is also recommended that intermediate sterilization be performed during the drilling operation.

Well Development and Testing

Every type of drilling causes some kind of disturbance to the aquifer by clogging the pores of the formation where the aquifer exists. After the borehole is completed, the casing is inserted, the intake is attached and the gravel packing installed (if desired). The next step is to have it "developed." Development is the process that removes finer material from the natural formation around the intake, enlarging it and leaving only larger gravel and stones around the screen. The larger pores allow the water in the aquifer to flow into the intake at maximum capacity. Development is the last stage of construction and is regarded as much an art as a science. The experience of the driller is an important factor. After development, the well is capable of providing water.

Development. There is no single, effective method but rather several available, each with its own effectiveness. Most of the methods discussed are used in combination with others. The most common methods include:

1. *Overpumping.* This is a process in which the capacity of the well is greatly exceeded by pumping. All backflow preventing devices are in place. This process cleans out the fine materials and begins to collapse the aquifer strata material around the well. A separate pump is usually selected because one impeller would be excessively worn by the additional sand brought into the pump by the large volume of water.

2. *Rawhiding.* This is a variation of overpumping except that all backflow-preventing devices are removed. The pump is started and stopped, and when the water in the discharge pipe falls to the bottom of the borehole, it loosens the aquifer material adjacent to the well screen. Starting and stopping the pump are done many times.

3. *Surging.* This is a process that uses a plungerlike tool moved up and down inside the well and intake. A tight-fitting insert called a *surge block* creates an alternate vacuum and pressure that force water in and out of the intake.

4. *Air lifting.* This is a process in which an air rotary drill string is placed at the bottom of the well without the bit. Compressed air is pumped down the well allowing air pressure to loosen the material around the bottom of the borehole. This method often results in too great a flow out of the well. This method has many drawbacks and is used primarily in large production wells at the end of the development process.

5. *Air surging.* This method is a variation of air lifting that uses two types of air rotary drills, one within the other. The inner one carries air and the outer one carries water to the surface. A valve is alternately opened and closed after compressed air is allowed to enter the well, resulting in a surge of water that provides the desired results.

6. *High velocity jetting.* Generally regarded as the most effective method of development, it uses multiple nozzles to spray water at 150 to 300 fps (50 to 100 mps) through the intake while it is raised and lowered. Often, the pump will be turned on to take some water to the surface.

Testing for Yield. Following development of the well, the next procedure is testing the well by measuring the flow rate of pumping and the level of water at the various pumping stages. Testing will determine the quantity of water, or yield, that can be produced in a given period of time and the acceptable drawdown level. Testing will provide a basis to estimate the water supply quantity available from the well and to determine the type of pump to be used.

Water Level Measurement. The static water level is the level to which the water reaches in a well under atmospheric conditions when the well is not being pumped. The dynamic water level is the elevation to which water falls in the well during pumping at a given flow rate. The distance between the static level and the dynamic level is *drawdown.* This level will change depending on the pumping flow rate. When water is pumped from a well, the level of water adjacent to the intake creates an inverted cone of depression of the water level in the aquifer (see Fig. 6.61).

To determine drawdown, it is important that this level be checked often over a relatively long period of time at various flow rates. This test shall be recorded in a log. No calculation of capacity should be accepted unless it has been made for at least several hours at hourly intervals. For practical purposes, the pumping level is considered established when three test levels are the same for a minimum of 3 h. Often, a 24-h test is conducted. A common method used to describe the yield is to express the discharge capacity in relation to a distance of drawdown. This relationship is called the specific capacity of the well and is expressed as gpm per foot (m^3/m) of drawdown.

Yield. The character of the aquifer and the construction of the well each will affect the yield. In determining yield, the result that pumping has on the aquifer shall also be determined. The following are definitions of different types of yield:

1. *Safe yield* is the quantity of water that can be withdrawn annually without the ultimate depletion of the aquifer.
2. *Permissive sustained yield* is the maximum rate at which water can be withdrawn from an aquifer on a continuing basis without developing undesirable results.
3. *Maximum sustained yield* is the maximum rate at which water can be withdrawn from the aquifer on a continuing basis.

The yield can be effectively increased by lengthening the intake and changing the type or number of openings in the intake screen. Another method is to make

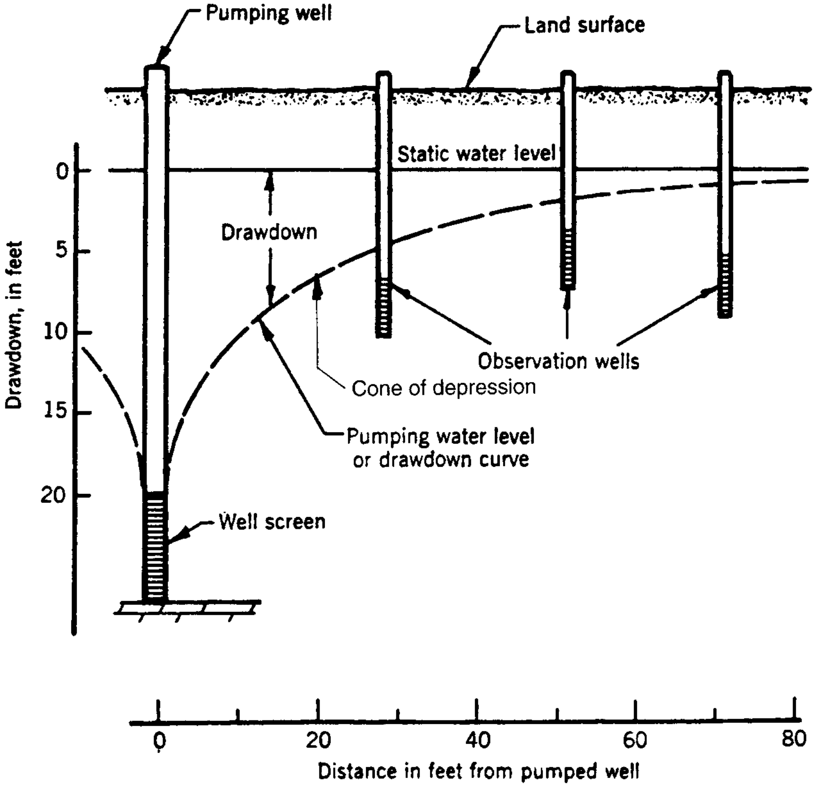


FIGURE 6.61 Cone of depression.

the diameter of the borehole larger, but studies have shown that doubling the diameter only increases the yield by 10 percent. In most cases, this is not an economical solution.

Water Pumps

The selection of the pump depends on many factors, including the following:

1. Flow rate and total head
2. Well diameter
3. Depth of the well
4. Straightness of the well
5. Presence of sand in the water
6. Duration of the pumping cycle

Well System Design Considerations

The total well system takes water from the well and distributes it in a manner that is useful to a facility, almost always as a supply to a water storage tank from which the water will be withdrawn. The purpose of the tank is to provide a storage capacity so that the well pump can be shut down for a period of time. If the tank is large, it can be filled by the well pump at a lower flow rate than the high instantaneous flow rate that might be required by the facility.

For very small systems, similar in capacity to private residence systems, the well water is routed into a small hydropneumatic tank. This tank has an air space that is compressed by the incoming water. When the volume of water in the tank compresses the air to a high enough level, the pump stops. For large systems, water should be supplied to ground-level or elevated water tanks in order to prevent the well pumps from running continuously.

The pump head, or total dynamic head, is calculated by adding the distance between the impeller and the surface, the friction loss in the pipe and fittings, and the height from 1 ft above the inlet into the elevated storage tank (if one is used) or the amount of pressure required in the hydropneumatic tank. If the system is used to directly supply a specific piece of equipment, the value for the pressure required by the equipment is used.

The discharge pipe material is the same as used for normal plumbing site piping.

WATER SUPPLIED FROM SURFACE WATER SOURCES

It is very rare that a surface water source is pure enough to serve as a source of potable water for individual facilities. If one is chosen, it must be approved by the local authorities.

PROJECT WATER SUPPLY

This section will describe the methods and criteria necessary to obtain the size and flow rate for potable water and fire protection water supply.

DOMESTIC WATER SERVICE

The first requirement is to find the water purveyor or utility company. The next requirement is to calculate the preliminary maximum instantaneous service requirements, in GPM. This is done from a preliminary (or final, if available) fixture count for the project under design and adding 10% to allow for estimated additional usage, such as cooling tower fill, boiler fill or other miscellaneous use of water in addition to the domestic use. The water supply fixture unit count can be obtained from Table 9.1. For specialized facilities, add a figure obtained from the owner for additional equipment flow rate.

Once this is calculated, a letter should be written to obtain the following information for your files. This cannot be overemphasized. The following should be requested:

1. A site plan from the utility company showing the water main

2. Other information

Other information shall be requested in a typical water utility letter, as follows:

To: Water Utility Company
Re: Project name
Lot and block number
Address
Dear sir or madam:

We are the engineers responsible for the design of the mechanical work for this proposed new [addition to an existing] project.

We are enclosing three site plans, which give a detailed location of the proposed structure and also our desired location for the point of entry into the building. At this time our preliminary water flow rate estimate for this project is a maximum instantaneous demand of ________ GPM domestic and ________ GPM fire protection flow, for a total of ________ GPM.

Based on the above data, we request the following information.

A. What are the size and location(s) of any existing water mains adjacent to the project site that can be utilized for both domestic and fire protection water service. If at present no such service exists, what is the expected date of completion for such services? Please mark up one copy of the enclosed site plan with this information.

B. What is the depth of bury of the water mains based on the datum taken from the enclosed site plans?

C. What is the static and residual pressure in these mains? If there is any cost involved for obtaining this information, or if you do not normally conduct such tests, please advise us. If weather conditions do not permit such tests at this time, please advise us when such tests would be conducted.

D. Please advise us about the requirements of, and any specific required locations for, both domestic and fire protection meter assembly installations. Is a separate meter required for both domestic and fire water service? Please include dimensions for all such meter assembly installations.

E. Are there any requirements or preferred piping materials and jointing methods for this service?

F. Please provide us with a breakdown of the work provided by you, the utility company, and all work required to be performed by the plumbing contractor relating to the domestic and fire protection service from the main into the building and the installation of the water meter assembly. Who is responsible for connecting to the public water main? If it is our contractor, please provide us with any rules and regulations regarding the installation.

G. Please provide us with any rules and regulations regarding the requirements for and installation of backflow preventing devices. If you are not responsible for such installations, please provide us with the name and address of the agency or department having jurisdiction in this matter. If there are none, please so state.

H. Based on your experience, would you please advise us of the minimum depth of bury acceptable for water mains in your jurisdiction?

I. At the present time, we expect construction to start on or about ______.

End of letter

After the utility company has answered this letter, they will have provided most of the information requested. Based on this information, the following major design items shall be determined.

1. Based on the available pressure, a decision regarding the method used to increase water pressure if required for the building will be decided. The space requirements for the necessary equipment shall be determined. The location inside the building shall be selected.

2. We have a fixture count and have calculated the maximum water supply flow rate.

3. The run into the building has been determined and the meter assembly has been selected and located.

A discussion follows for a typical domestic water service and all of the possible installed devices in detail. First will be a description of the various required information, installed components and their design parameters. Figure 6.62 is a typical sheet used to arrange and calculate all necessary data to determine the service water pressure.

1. *Difference in height* from the centerline of the main to the centerline of the service inside the building where it connects to the pressure increasing device or at the point where the distribution network begins for a street pressure system.

WATER SERVICE CALCULATION SHEET

SYSTEM: DOMESTIC WATER

SHEET 1

PROJECT:

PREPARED BY:

CHECKED BY:

DATE:

STATIC: ______ PSI

RESIDUAL: ______ PSI BASED ON ______ GPM FOR BLDG AT ELEV. ______

ACTUAL STREET PRESSURE ______________ PSI [FT] ①

SERVICE LOSSES

ST. PR. GPM _______________ ______

PUMP GPM _______________ ______

HVAC GPM _______________ ______

MISC. CONTINUOUS GPM _______________ ______

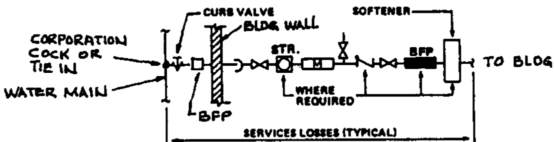
TOTAL GPM _______________ ______

SIZE OF SERVICE TO BLDG ______" VEL.* ______ FPS, MATERIAL __________

FRICT. LOSS / 100' ______ RUN INTO BLDG ______ FT. TOTAL FRICTION LOSS ______ FT.HD.

ALLOWANCE FOR FUTURE DROP IN ST. PR. ______ PSI ______________ FT. HD

DIFFERENCE IN ELEVATION ______ FT.



SERVICE LOSSES

QUANTITY	SIZE"	ITEM
		TIE IN @ ______ PSI = __________ FT.
		VALVES @ __________ = __________ FT.
		MISC FTGS @ __________ = __________ FT.

STRAINER LOSS @ ______ PSI x 2.31 = __________ FT.

METER LOSS @ ______ PSI x 2.31 = __________ FT.

BFP LOSS @ ______ PSI x 2.31 = __________ FT.

SOFTENER LOSS @ ______ PSI x 2.31 = __________ FT.

TOTAL SERVICE LOSSES [FT.] ②

AVAILABLE PRESSURE
ACTUAL PRESSURE − SERVICE LOSS [FT.] ③
① ②

FIGURE 6.62 Water service calculation sheet.

2. *Water pressure, both static and residual.* A hydrant flow test is the most often used method to obtain this information. It may be possible to obtain a map of the piping system of the area where the project is located in order to chose the hydrants where the flow test will be conducted.

This test consists of choosing two fire hydrants closest to the site where the project is located and taking three pressure readings. The first reading is the pressure from a pressure gauge connected to one hydrant with no water flowing. This is called the static pressure because there is no flow. The second reading uses both hydrants. The hydrant with the pressure gauge remains unchanged. The second hydrant is now opened and a velocity pressure reading is taken using a pitot tube held directly in the water stream. A pitot tube is illustrated in Fig. 6.63, and the method of taking the reading is illustrated in Fig. 6.64. At the same time, another reading is taken from the first hydrant while the second hydrant is flowing water. The actual flow rate must be known for this test to be meaningful. This is calculated by converting the velocity pressure into GPM by referring to Fig. 6.65, which is a relative discharge curve using the diameter of the nozzle from which the water is flowing and the pressure reading from the pitot tube held in the water stream.

The time of the year and time of day the test is taken are considerations that will affect the flow test data. There are seasonal variations, mainly in the summer, where the flow is generally regarded as greater than other times of the year in generally residential areas. Another factor is the time of day, which also often has large differences of flow, leading to lower residual pressures. Consult with the water utility company to decide if these items are important in deterring the actual pressure available to the project.

The static and residual pressure and the flow rate represent only two pieces of information. With these two points it is now possible to determine the pressure available at any flow rate. This is important since the flow rate for the project under design is almost certainly less than the flow rate at which the hydrant flow test was conducted. The method used is to plot these two points on hydraulic graph paper, illustrated in Fig. 6.66. The vertical axis has the pressure, in psi, and the horizontal axis has the flow rate, any one scale of

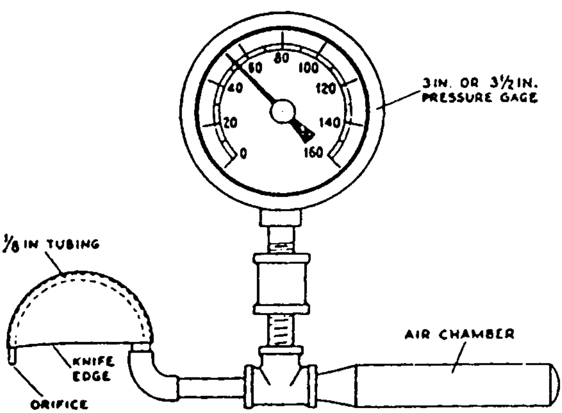


FIGURE 6.63 Pitot tube with gage and air chamber.

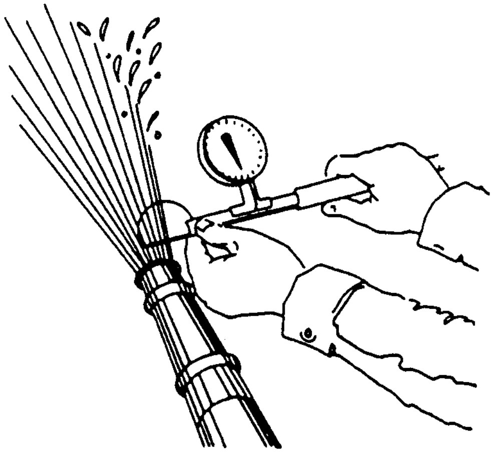


FIGURE 6.64 Taking nozzle pressure with a Pitot tube.

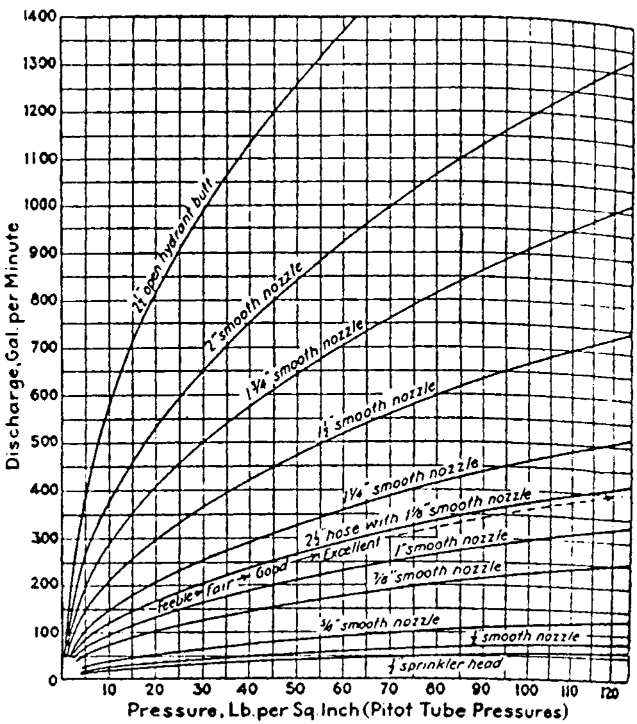


FIGURE 6.65 Relative discharge curves. (Chemical Engineers' Handbook, McGraw-Hill Book Co.)

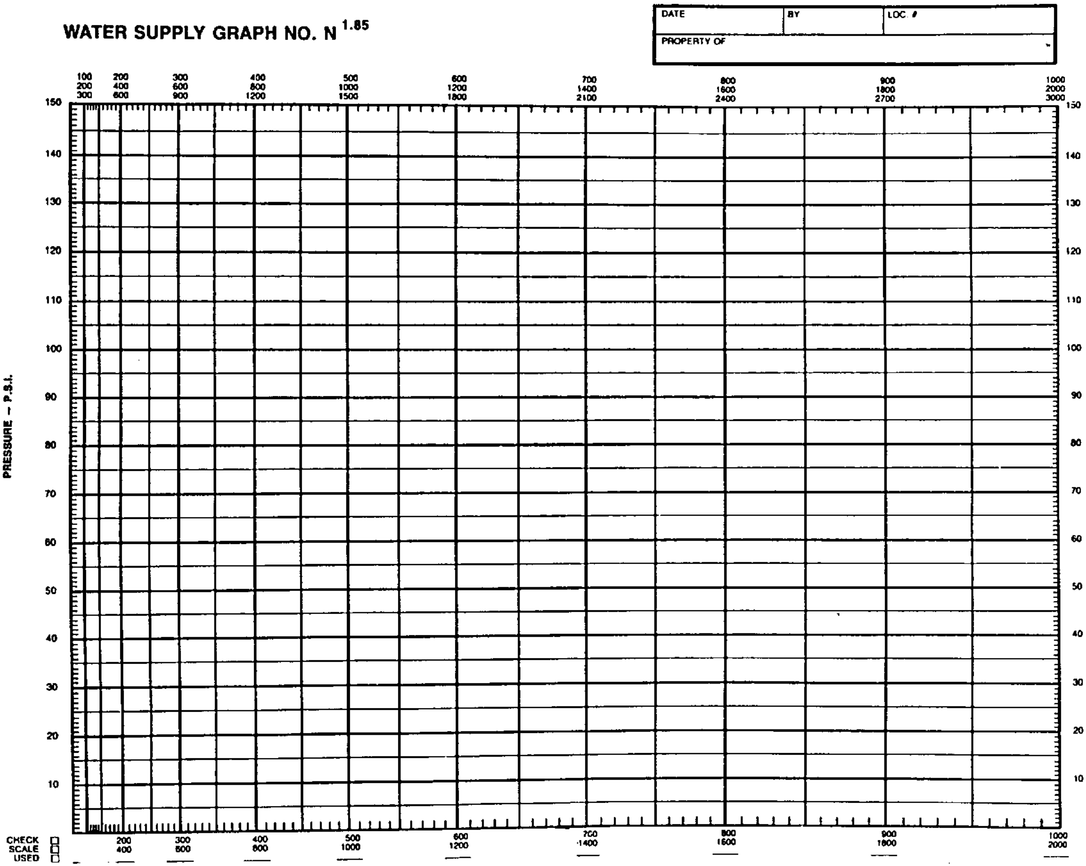


FIGURE 6.66 Water supply graph No. N1.85.

which can be selected for the flow rate established. By connecting the two points, any other point on the line will give the pressure using the flow rate desired.

3. *Pressure loss through ordinary taps* into the water main is given in Table 6.32. Pressure lost through larger wet tap is best obtained from the manufacturer.
4. *Pressure lost through valves and fittings* shall be obtained from Fig. 9.1.
5. *The total friction loss in the piping run* from the main to the last item in the water service assembly can be obtained from standard texts, such as Camerons Hydraulic Data.
6. *The difference in elevation* is the number of feet, either positive or negative, from the centerline of the water main to the centerline of the service to the last item in the water service assembly.
7. *Backflow preventers* (BFP). The water utility company is responsible for protecting the public water supply from any possibility of backflow contamination containing contaminants and pollutants. The most common reasons for the utility company to require a BFP is a close supply of unpure water (like from a stream) that could be used by a fire department to supply water that will be pumped into a building to fight a fire, and a potential source of contaminated water from a facility that could find its way back into a public main if there is a break in the main. For these reasons, a reduced pressure zone BFP will be required near the property line at the connection to the public main. Refer to Chapter 9, Potable Water Systems, for a further discussion of BFPs.

   At typical pressure loss through a RPZ BFP is 10 psi. A typical pressure

TABLE 6.32 Loss of Pressure through Taps and Tees in Pounds per Square Inch (psi)

	Size of trap or tee (inches)						
Gallons per minute	⅝	¾	1	1¼	1½	2	3
10	1.35	0.64	0.18	0.08			
20	5.38	2.54	0.77	0.31	0.14		
30	12.1	5.72	1.62	0.69	0.33	0.10	
40		10.2	3.07	1.23	0.58	0.18	
50		15.9	4.49	1.92	0.91	0.28	
60			6.46	2.76	1.31	0.40	
70			8.79	3.76	1.78	0.55	0.10
80			11.5	4.90	2.32	0.72	0.13
90			14.5	6.21	2.94	0.91	0.16
100			17.94	7.67	3.63	1.12	0.21
120			25.8	11.0	5.23	1.61	0.30
140			35.2	15.0	7.12	2.20	0.41
150				17.2	8.16	2.52	0.47
160				19.6	9.30	2.92	0.54
180				24.8	11.8	3.62	0.68
200				30.7	14.5	4.48	0.84
225				38.8	18.4	5.6	1.06
250				47.9	22.7	7.00	1.31
275					27.4	7.70	1.59
300					32.6	10.1	1.88

For SI: 1 inch = 25.4 mm, 1 psi = 6.895 kPa, 1 gpm = 3.785 L/m.

loss through a DCV BFP is 5 psi. Since there are many types of each BFP, check with the manufacturer regarding the exact amount of discharge and pressure loss. It varies with size. Testes at the independent Foundation for Cross-Connection Control and Hydraulic Research at the University of Southern California have established that various manufacturers do not represent their BFPs' pressure loss correctly. It would be appropriate for the design engineer to request the flow curves produced at the Foundation for the most accurate method of comparing various devices.

8. *Strainer losses* are given in Figs. 3.3 for "Y" strainers and 3.4 for basket strainers. Strainers are rarely used except for water with known problems.

9. *Meter losses* for typical meters shall conform to AWWA standards. Refer to manufacturers for exact information.

10. Water softener and other treatment systems shall be obtained from the manufacturer.

FIRE PROTECTION WATER SERVICE

The fire protection water supply to a building starts from connection from a source of water to a point above ground inside a building. This source is usually a public water main but could also include other sources. This building service includes water storage tanks, backflow preventers, meters, valves and other devices that may be required based on the nature of the water service, insurance company requirements and local regulations. This section is not intended to cover private fire service man. For private service mains, refer to NFPA-24 and Factory Mutual Loss Prevention Data Sheet 3-10.

Facility Water Supply

If the water supply to a facility is a combined service, that is both domestic and fire service, the main shall be considered a fire service until the domestic connection to the building service is made. If the source is from a well, an aquifer performance analysis and investigation of the history of adjacent wells shall be made. For many facilities, multiple water supplies from two separate public mains is a very desirable feature. This is to allow water to be supplied to the facility from either of two directions. If these connections are made to a single main, a sectionalizing valve installed in the source main somewhere between the connections will be necessary or this will not be possible. Another desirable feature would be a fire hydrant installed on the building supply adjacent to a hydrant on the public main. This would allow a fire department pumper to increase the volume and pressure to the building if necessary.

Contamination of the public water supply is a prime concern of the water company that supplies water to the project. The determination of the acceptable device for prevention of the fire protection water supply will be made by either the health department, the water purveyor or the plumbing sub-code official. The two most often used are the double check valve assembly or a reduced pressure zone back flow preventer. These devices are discussed in the domestic water supply section. The mains shall be buried below the frost line.

In many cases, it is not possible to supply the required water supply for fire fighting purposes from the public supply. For these situations, the use of a water storage tank is called for. These tanks can be either elevated or installed on the ground. The design of these tanks is outside the scope of this handbook. Connections from the water supply to a storage tank shall terminate 1 ft, 0 in over the overflow level. The overflow shall be two pipe sizes larger than the water supply pipe size. Where necessary, the tanks shall be protected against freezing.

Ancillary Devices

Fire Hydrants. Fire hydrants directly connected to the building supply shall be installed adjacent to roadways that will allow easy access by the fire department. If reasonable, they shall be located on all sides of the building being protected. A desired separation from the building wall is 50 ft, 0 in to provide some protection from building wall collapses. Recommended separation between hydrants would be 200 ft, 0 in apart at a building location. Hydrants located near a road shall be protected by guard posts. It is good practice to provide a shut-off valve on the branch line to a hydrant to allow east repair without having to shut down the main. For some industrial applications, fire hydrants with an attachment called a monitor shall be placed adjacent to specific hazards.

Guard Posts. Guard posts are necessary to protect any device that is installed above grade near roads.

Post Indicator Valve. A post indicator valve is used to shut off the supply of water. It is used as a section valve on water mains and also on building services. Since it is critical for emergency personnel to be certain that any control valve is open or closed, an indicator is positioned above grade with a window or some other method to allow easy observation of the valve position.

Joint Restraint. When run in a straight line, properly specified joints will provide a leak-proof installation. When there is any change in direction of the pipe, a great pressure is exerted by the flowing water on the fitting. The joints may not provide resistance to the force exerted by the pressure of the water against the fitting. In order to prevent joint failure due to the pressure exerted, most pipe joints must be restrained in some manner.

The preferred method of restraining joints is by means of the split ring couplings for grooved pipe. The use of properly selected joints will not require any additional method of restraint. If steel piping is used, welded joints do not require restraints.

Mechanical joints and push-on joints seal by a rubber type gasket compressed in a space between the spigot end of the pipe and the bell. These joints will not resist the pressure of the water on the joint. The method of restraint is with external clamps and rods. Friction clamps are bolted around the pipe on both sides of the joint or joints. These clamps engage steel tension rods, sometimes called tie rods, across the joint preventing it from separating. This will combine the resistance of these joints to prevent separation.

Another method is the use of integrally cast glands for mechanical joint pipe with internally laced grooved and keyed push on joints. This type of restraint is usually recommended only for repair of existing systems.

The most common but generally the least desirable method is to use a block of concrete contacting the fitting and poured against undisturbed soil. The size of the block varies with the water pressure (the higher the pressure, the larger the block), pipe size (the larger the pipe the larger the block), and bearing pressure of the soil (the less the soil bearing pressure the larger the block). The main problems are that on many project sites there is often no undisturbed soil against which to base the thrust block and the size of the concrete block prevents piping from being placed adjacent to the run of pipe being protected.

Sizing the Fire Protection Water Service

All of the information required for design has previously been obtained for the domestic water system. The fire marshal, the fire protection sub-code official and the insurance company shall be contacted for their installation requirements.

What constitutes an adequate water supply has generated much discussion over time. There are many factors that, when added together, will provide an adequate water supply. The water supply should be capable of supplying the largest demand, which is usually the largest sprinkler system and expected hose flow under reasonably adverse conditions. Other factors, such as building occupancy, yard storage, external structures that must be protected, exposure protection and a catastrophic hose demand will all add to the possible maximum flow rate.

Demand Flow Rate. The following figures are presented for discussion purposes only. They are not to be used for actual design, which should only be made on a specific project basis.

The flow requirements for the sprinkler system is the hydraulically calculated flow rate.

The base hose streams depend on the occupancy hazard. These are:

1. Light and ordinary hazard—500 gpm (1900 L/m)
2. Extra hazard, group 1—750 gpm (2900 L/m)
3. Extra hazard, group 2—1000 gpm (3800 L/m)
4. High piled storage—as required by insurance company

The hose demand shall be increased by 25% for the following conditions:

1. Combustible construction
2. Possible delay in response by public fire department
3. Minimum protection less than recommended by insurance company requirements
4. Limited access to remote interior sections

Additional requirements for monitors: allow 500 gpm for each monitor.

Storage Tank Capacity. If the gravity tank is the sole source of water, the tank shall be capable of being filled in 8 hours. Evaluation of the total capacity should consider the following storage capacity:

1. Light and ordinary hazard occupancies, group 1—2 hours
2. Ordinary hazard occupancies, group 2 & 3—3 hours
3. Extra hazard occupancies—4 hours

GROUNDWATER MONITORING WELLS

Monitoring wells are required to detect contamination in groundwater. Because the integrity of samples depends in large part on well construction, the drilling methods and materials used in the construction must have no influence on the quality of groundwater samples brought to the surface.

CODES AND STANDARDS

1. ASTM D 5092
2. ASTM, *Standard Guide for the Development of Groundwater Monitoring Wells in Granular Aquifers*

DRILLING METHODS

The most critical aspect of the drilling method selected is that it must not have any effect on groundwater quality near the borehole. The drilling methods must not use liquids. Cable tool, dual tube, and reverse circulation air rotary methods are acceptable. A typical monitoring well is illustrated in Fig. 6.67.

CASING

The material selected for casing material must be structurally strong, easy to handle, and durable enough to resist the long-term subsurface environment. Durability depends on groundwater and soil chemistry.

The casing material must be capable of resisting any degradation, adsorption, and reaction to the possible contamination products being measured. If it is for water only, it should not contribute any chemicals of its own.

PVC, PE, and other plastic materials are structurally strong and resistant to normal groundwater conditions, but should not be used when solvents might be encountered. Fluoropolymer materials such as Teflon are chemically inert, but their high cost and low tensile strength make them poor choices. PVC is used most often because solvents are rarely encountered. Stainless steel is also considered acceptable for many applications.

GROUT

The most common materials are straight (neat) cement or bentonite. A surface seal of cement must be placed so that surface runoff is prevented from running down the well.

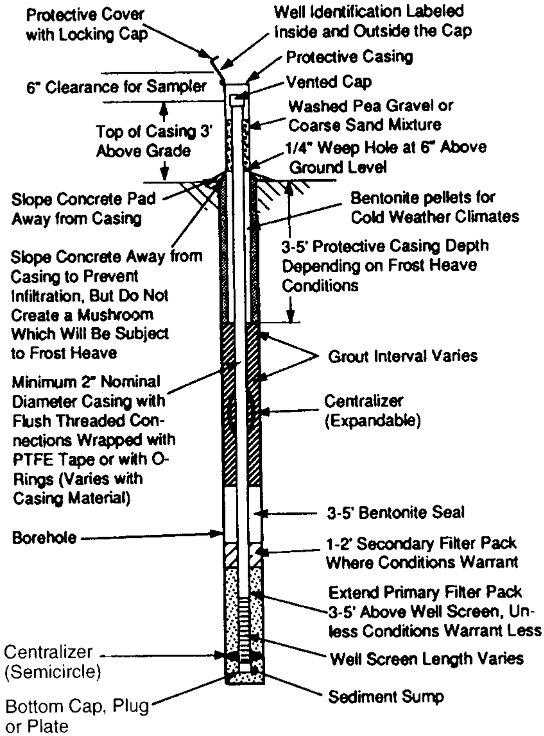


FIGURE 6.67 Typical monitoring well components.

INTAKES

If the intake screen and filter pack allow sediment into the well, increased sampling time and cost will result due to the additional filtering required. Factory-slotted casings and continuous slot wire wound intakes are preferred. For formations less than 50 ft, screen lengths are recommended. For larger formations, proportionally longer lengths can be used. Filter packs generally consist of medium sand to fine gravel with a thickness of 2 to 3 in surrounding the intake. They should extend about 3 ft above the intake to allow for settlement.

WELL DEVELOPMENT

The most effective and popular development technique is surging, although hydraulic jetting and overpumping can be used. Well development is not effective in clay formations or in wells that straddle the water table, such as those drilled to monitor petroleum hydrocarbons.

FINISHING THE MONITOR WELL

A surface seal of cement or concrete may be required by local codes to extend some distance beyond the borehole. Two types of well completion, sometimes called finishes, are common: the aboveground and flush-to-ground completions. In the aboveground type a protective casing is installed around the well casing. It should be either painted or manufactured from a corrosion-resistant material. It is important to drill a drainage hole in the protective casing to drain any water that might accumulate and possibly freeze in colder climates. If no enclosure is installed around the monitor well, bumper guards are recommended if the well is in an area well traveled by vehicles.

For a flush-to-ground completion, a small protective box the size of a small meter is set into the casing with the top flush with grade. This type is typically used in urban areas, parking lots, and streets where an aboveground completion is impractical. It must be watertight.

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