CHAPTER 25

WATER DISTRIBUTION

Gary Nickerson

Updated/Revised: Robert "Skip" Notte, PE, LEED AP

OVERVIEW OF A WATER SUPPLY SYSTEM

A water supply system includes the water supply, treatment facilities, pumping facilities, transmission lines, and local distribution network. The distribution network consists of pipes, fittings, valves, and other appurtenances designed to convey potable water at adequate pressures and discharges. Adequate discharges are necessary to cover the range of flows due to the fluctuation in demand. Adequate pressures are necessary for firefighting, general service to residents, and economic considerations such as leakage and energy losses. The pressures within a water supply system must be high enough to overcome the energy losses in the system, yet maintained below the point at which damage to fittings and other appurtenances occurs. Due to the size and varying topography within many systems, different pressure zones may be generated to ensure reliability in meeting the fluctuation in demands. These pressure zones are typically divided through booster pumping stations and storage facilities. Pressures and discharges are a function of the hydraulic characteristics of the system as well as the system's service characteristics. Service characteristics include such items as demand as it relates to present and projected population, economic base, fire flow, and climate. Hydraulic characteristics include the length, size, and condition of the pipe; types and sizes of fittings and appurtenances; and variations in elevation of the system. Since service and hydraulic characteristics constantly change, the design must incorporate allowances for these changes. Dynamic demographics of the users and deterioration of the system affect how and when the changes are made. The changes that can be made include relocating or replacing the existing waterline, increasing the pipe sizes, adding auxiliary pumps and storage, or adding more distribution lines to accommodate new development.

The water utility company is responsible for the water quality and operation of the distribution system. A water utility company can either be a public entity that, like many other public bureaus, exists for the health, safety, and welfare of the public, or it may be a privately owned utility providing water for profit. In some of the larger metropolitan and suburban areas, the water utility company heavily controls the design of a water distribution system. Personnel within the water company perform most of the design, analysis, and layout. In this case the site designers of the land development project perform only the lesser waterworks design aspects and incorporate their analyses, along with the water company's information, into the construction drawing set. In smaller locales, the analysis and design may be performed entirely by the private consultant working for the developer. Typically, the water supply company reviews the private consultant's design to ensure conformance with local standards. Regardless of whether the water supply company is public or private, the design must conform to the state health department's criteria as well as those of any other controlling public agencies. Design parameters and regulations for waterworks are available from the state board of health and the local city/county health departments. Additional information on water quality, as well as material and construction standards, are available from the Environmental Protection Agency (EPA), the American Water Works Association (AWWA), and the American National Standards Institute (ANSI).

The overall waterworks system consists of the following elements:

- The *water source,* usually a lake, river, or aquifer that serves as the municipality's main supply. Larger municipalities may have more than one source. Lakes and reservoirs are located in outlying areas where there is less pollution and the advantage of runoff from large catchments. Along rivers, water is usually extracted on the upstream side of the population centers. The minimum supply volume at the source must coincide with the present and projected demand, which it is to serve.

- A *treatment facility* to purify the water for safe use. Since pumping clean water is less expensive than pumping sediment-laden water, the location of the filtration plant should be near the source. The treatment facility treats and disinfects the water to meet the water quality standards set by government regulations. The law mandates that the water supply company provide quality potable water, and consumers expect and rely on the water company to do so.

- *Transmission lines* that convey the water leading from the source to the treatment facility as well as from the treatment facility to the distribution network. For moderate to large population areas, transmission lines can be 6 feet in diameter (and larger) and operate at pressures in excess of 200 lb/in^2. They serve as the link between the source, the treatment facility, and the distribution network. Therefore, transmission lines have limited branches and service taps.

- *Pumping facilities* that provide necessary energy to move the potable water to consumers. Pumping facilities are needed to convey the water through transmission lines to the distribution system. These pumping facilities can be simple, such as the well pumps in small systems, or they can be complex, high-capacity pumping stations needed for large municipalities. Pumping stations within the distribution network, called *booster stations*, are used to maintain required minimum pressures.

- *Intermediate storage facilities* (e.g., water tanks) that are located near the distribution network. Intermediate storage facilities stabilize the line pressures, serve as reserve for peak demand periods, and provide storage for fire flow requirements. All domestic waterline systems operate under pressure. In times of high demand, the pressure in the network is decreased, requiring the water supply to be supplemented from the storage tanks along with the water from the treatment plant. During low-demand periods, when supply pressure in the distribution lines is high, the storage tanks refill. Discounting fire flow storage, water storage volumes fluctuate 40 percent to 70 percent on a daily basis.

- *Distribution lines* that are used to link the intermediate storage facilities and the feeder lines that connect the residential, commercial, and industrial service areas. Typically, distribution networks are laid out as interconnecting loops. The looped (grid) layout allows for bidirectional flow of water. Since water travels toward the area with the lowest pressure, which is often the area of highest demand, looping delivers twice the volume of water for fire flows and other heavy demands. Additionally, grid layout allows for isolation of small areas during repairs. A branching type of configuration consists of a main feeder line with single dead-end branches to service areas. In comparison to the grid system, the branch system is not as economical for two reasons: closing down a branch for maintenance disrupts service to a larger area, and poor water quality resulting from stagnation and sedimentation in the branch ends means periodic flushing at the ends. Although dead ends cannot be avoided, they should be minimized. Fire hydrants, flushing hydrants, or blowoff valves at the dead ends allow for such periodic flushing.

- *Appurtenances* such as fire hydrants, valves, and auxiliary pumps, and fittings such as wyes, elbows, crosses, and tees that augment the operation of the distribution system. The arrangement of the appurtenances and fittings is specific to the local area they serve. Valves allow the system to isolate small service areas when repairs are needed. Other fittings are used to change flow direction and provide economic flexibility in sizing pipes.

The design of the entire system requires an evaluation of numerous factors. Location and size of main water supplies depend on general characteristics of the region such as climate, hydrology, geology, and topography. Design of treatment facilities, transmission, and distribution lines depends on existing water quality at the source, existing and projected population, and spatial distribution. Additionally, as the municipality grows, the increasing population and expanding development inevitably influence hydrologic and topographic factors as well as others that, in turn, affect water demand.

WATER USE AND DEMAND

Demand Forecasting

Water use is categorized as either consumptive use or nonconsumptive use. Consumptive use includes municipal, agricultural, industrial, and mining, whereas nonconsumptive use refers to water used for hydropower, transportation, and recreation. Water demand is the quantity that consumers use per unit of time. Land development projects involve analysis of municipal water uses such as residential, commercial, institutional, industrial, firefighting, swimming pools, and lawn and park watering.

Demand forecasting projects the future water use based on previous water use, socioeconomic trends, climatic factors, and other parameters. Various types of forecasting models may be used to estimate water demand. The least complex is a single coefficient method through which the projected demand is based on one factor such as per capita or per connection or on land use. Complex models may factor in water pricing, income, and even incorporate statistical analysis. The selection of the model depends on the type and size of the project and the availability of the data.

Part of the design of a water distribution system includes estimating the amount of water needed for a service area. Water demand for an area depends on population, climate, industry, and economic factors. Additionally, the design analysis must include fire flow requirements. Although the overall volume of water used for fighting fires is quite low relative to most other uses, the amount of water required to fight a fire, even if only for a few hours, puts a heavy strain on the system. Since fire flow requirements are so high relative to other uses, they are usually the controlling criteria in the design of distribution piping and storage. In part, the waterworks system design is based on an estimated average demand and maximum daily demand. However, variations in demand must also be included as part of the design analysis.

Variation in water use becomes apparent by considering hourly and daily individual personal water use. Each individual drinks, bathes, and generally uses varying daily amounts of water. On an hourly basis, this variation fluctuates substantially. Extend the individual fluctuations in hourly and daily use over a population of tens of thousands and it becomes apparent why water demand projection is difficult. Fortunately, individual daily use generally has an average value that exhibits only slight variations over longer periods. Variations in short-term use are further reduced as population and development stabilize. However, the capricious variables inherent in commercial, manufacturing, and agricultural industries (e.g., climate, demand for consumer goods and services) can add to wide variations in use and complicate demand forecasting.

The types of use for water demand include domestic, commercial, industrial, mining, agricultural, thermoelectric power, public use, and many others depending on where the interest is. Collecting and sorting the data to determine average use are difficult, cumbersome, and inexact. Water supply companies keep numerous records of water withdrawals and metered use. Inconsistencies in how each supplier defines various uses, or categorizes the various uses, leads to the inexactness in determining average use figures when compared over statewide areas. The point here is that it is difficult to confidently say a certain value is applicable as a blanket design value for all areas of the United States. For preliminary estimates a blanket design value is appropriate, but, for specific design, regional data is more appropriate.

An additional component of demand projection, which requires significant consideration, is the understanding and implementation of sustainability or green design goals. Water is a limited resource and all design concepts should be aware of industry trends that help to conserve this valuable commodity. Some conservation techniques include developments that do not install or permit the use of permanent landscape irrigation systems, or residential, commercial, and industrial facilities that utilize water reduction fixtures in bathrooms and kitchens. Knowledge of sustainability goals and techniques for incorporating them into design supports the conservation of this resource.

Residential Demand

Domestic water use refers to the water used for household purposes, including water for drinking, cooking, sanitary needs, landscape watering, swimming pool maintenance, street cleaning, firefighting, leakage, and system maintenance. Residential water demand widely fluctuates on both a daily and a yearly basis. Water demand has two daily peaks: once in the morning hours of 7 to 11 A.M. and another peak in the evening between 4 and 8 P.M. Although the peaks remain fixed for the time periods, the amount varies according to season and yearly rainfall. Average daily demand for domestic use ranges from 60 gallons per capita per day (gpcd) to 130 gpcd. However, studies have shown that average water use decreases as population increases.

The U.S. Geological Survey (USGS) presents data on water use in the report, "Estimated Use of Water in the United States in 1995." The data from this report shows publicly supplied domestic water use ranging from 53 gpcd in Wisconsin to 212 gpcd in Nebraska, with the average water use for all states, as well as Washington D.C., at 179 gpcd.

Commercial Demand

Commercial water use refers to the water used in motels, hotels, restaurants, office buildings, shopping centers, and other commercial establishments. Compared to residential demand, commercial demand is considerably less than residential peak demands and not as varied. Note that most of the water use for office buildings is for air-conditioning purposes. Restaurants, hotels, and motels use water mainly for cooking and cleaning. Water use estimates for design purposes are usually based on the square feet of space, potential number of occupants, or the number of water fixture units in the building. Utilizing data available in the previously noted USGS report, it can be deduced that commercial use, relative to total use, ranges anywhere from 1 percent in Iowa to nearly 41 percent in Alaska. Table 25.1 provides values for water use for various types of uses. At first glance, the values might appear to be high relative to what an individual perceives as actual use. However, these values account for unmetered water use such as firefighting, maintenance (e.g., flushing the system), leakage, and illegal connections.

Industrial Use

Industrial use accounts for the water used for fabrication, processing, washing, and cooling associated with such industries as food processing, steel production, chemical processing, mining, and petroleum refining. Note that some industry plants install their own water supply systems rather than rely on the municipal water supply. Depending on the industry and the capacity of the municipal water supply, it may be necessary for the industry to furnish its own water supply. The amount of water used varies greatly with the type of industry. A manufacturer of woolens uses 140,000 gallons per ton of woolens produced, whereas a ton of tanned leather requires 16,000 gallons. The USGS data indi-

TABLE 25.1 Daily Water Consumption Rates

Type of Land Use	Water Consumption (gal/day)
Dwellings, per person	100
High schools with showers, per person	16
Elementary schools without showers, per person	10
Community colleges per student and faculty	15
Motels @ 65 gals/person, minimum per room	120
Trailer courts @ 3 persons/trailer, per trailer	300
Restaurants, per seat	50
Interstate or through highway restaurants, per seat	180
Interstate rest areas	5
Shopping centers, per 1000 ft^2 of ultimate floor space	200–300
Theaters, auditorium type, per seat	5
Hospitals, per bed	300
Laundromats, 9 to 12 machines, per machine	500
Factories per person per 8-hour shift	15–35

(*Source:* Virginia Dept. of Health. 1993. Waterworks Regulations.)

cates that the state with the highest industrial water use (as a percent of total use) is Kentucky at 41 percent.

The wide fluctuations in water use for these three categories are evident from the data listed in Tables 25.2 and 25.3.

Other Uses

Water use can be identified with other types of manufacturing and industries. Depending on the region, the water use associated with mining, irrigation, livestock, and thermoelectric power may heavily influence design considerations for a water system. This section of the book will be concerned only with residential and commercial demand. If, however, you wish to obtain additional information on the other types of water demand, these other special uses depend on the characteristics of specific localities.

Peak Demand

Distribution system design must account for the peak periods of daily use. The maximum daily use for domestic water ranges between 1.5 and 3 times greater than the average daily use, whereas the maximum hour for domestic water use is 2 to 5 times greater than the average annual use. Since peak factors are a function of present population, population growth rate, and land use, the older stabilized areas tend to have peak factors that are 2 to 3 times lower than rapidly expanding areas. Table 25.4 shows the relationship of population on peaking factors.

TABLE 25.2 States with Highest and Lowest Water Use

	3 Highest States (GPCD)			3 Lowest States (GPCD)		
Domestic	Nevada (212)	Utah (184)	Idaho (181)	Wisconsin (53)	Ohio (54)	Pennsylvania (62)
Commercial	D.C. (90)	Nevada (81)	Utah (62)	Kentucky (7)	Texas (7)	Missouri (14)
Industrial	Alabama (62)	Kentucky (59)	Wisconsin (42)	D.C. (1)	Nevada (2)	Montana (2)
Total	Nevada (324)	Utah (269)	Washington (266)	Massachusetts (130)	Rhode Island (132)	W. VA (134)

TABLE 25.3 Public Supply Freshwater Use by State (from USGS Circular 1200)

State	Domestic (GPCD)	Commercial (GPCD)	Industrial (GPCD)	Public Use and Losses (GPCD)	Total Per Capita Use (GPCD)
Alabama	117.7	35.6	62.1	27.4	236.7
Alaska	99.7	60.3	31.5	21.0	211.6
Arizona	134.2	34.4	16.8	20.7	206.1
Arkansas	96.5	29.0	28.5	36.5	190.5
California	121.6	32.6	9.3	20.6	184.1
Colorado	141.9	29.8	5.6	29.6	203.8
Connecticut	75.5	35.2	16.6	27.7	154.9
Delaware	76.2	35.5	28.4	19.5	159.6
D.C.	171.5	90.3	1.3	0	263.1
Florida	103.3	31.6	8.4	25.6	168.9
Georgia	106.6	28.5	32.9	27.3	195.3
Hawaii	117.0	42.0	5.0	27.7	191.7
Idaho	180.8	23.1	8.6	29.5	241.9
Illinois	90.0	42.3	11.4	31.2	174.8
Indiana	76.2	27.8	29.2	23.1	156.3
Iowa	64.7	30.2	36.3	40.9	172.1
Kansas	82.3	28.9	16.0	32.0	159.2
Kentucky	69.9	6.9	58.6	12.5	147.9
Louisiana	121.6	14.3	9.1	20.8	165.7
Maine	65.0	35.3	19.8	19.8	139.8
Maryland	103.8	20.4	10.6	65.0	199.8
Massachusetts	64.9	33.7	15.4	15.8	129.8
Michigan	90.3	36.7	39.1	22.3	188.4
Minnesota	71.6	30.8	12.3	30.8	145.5
Mississippi	109.7	14.6	8.9	17.7	150.9
Missouri	86.4	13.6	32.3	28.9	161.2
Montana	119.4	40.3	1.6	60.5	221.7
Nebraska	120.2	61.3	20.2	20.2	221.7
Nevada	212.5	80.6	1.5	29.2	323.8
New Hampshire	81.8	30.1	18.7	9.6	140.2
New Jersey	77.6	25.8	13.1	29.3	145.9

TABLE 25.3 (Continued)					
State	**Domestic (GPCD)**	**Commercial (GPCD)**	**Industrial (GPCD)**	**Public Use and Losses (GPCD)**	**Total Per Capita Use (GPCD)**
New Mexico	136.2	56.5	10.9	21.7	225.4
New York	111.7	25.3	22.0	26.2	185.1
North Carolina	69.9	29.1	40.6	22.1	161.7
North Dakota	81.8	30.7	5.1	30.7	148.3
Ohio	53.6	38.3	38.3	23.0	153.2
Oklahoma	82.3	58.0	41.6	11.6	193.5
Oregon	135.8	36.7	33.0	28.8	234.4
Pennsylvania	61.8	24.1	21.3	63.4	170.6
Rhode Island	64.9	22.8	13.7	29.6	131.0
South Carolina	135.3	18.4	16.2	29.8	199.6
South Dakota	86.4	34.9	13.1	11.8	146.2
Tennessee	80.3	48.2	29.4	17.7	175.8
Texas	139.2	7.4	15.2	23.4	185.2
Utah	183.8	62.2	9.2	13.5	268.7
Vermont	82.6	24.4	24.4	17.5	148.9
Virginia	85.5	30.7	17.7	24.4	158.3
Washington	127.6	36.3	74.7	27.5	266.1
West Virginia	72.7	17.4	10.6	33.3	134.1
Wisconsin	53.1	31.2	42.4	41.6	168.3
Wyoming	157.0	46.5	7.0	49.4	259.9
AVERAGE USE	101.8	30.0	21.4	26.2	179.4

(*Source:* U.S. Geological Survey. 2000. Estimated Water Use in 1995.)
The authors of this handbook understand that more recent data has been released for USGS Circular 1200 data from 2000 that may be more relevant as to the actual usage in different areas of the country. However, due to modifications in the collection and reporting methodology, it is difficult to decipher usage specific for the purposes of this handbook. Therefore, the 1995 data is utilized to provide an understanding of demands and completion of design examples. All professionals are encouraged to contact the local jurisdictional agency to gain an understanding of project-specific usage when completing actual design computations and projections.

Typically, distribution lines are sized for design flows of maximum daily consumption plus the required fire flow. However, if the maximum daily consumption plus fire flow is less than the estimated maximum hourly amount, the design may be based on the latter value. Local regulations and state waterworks design manuals dictate minimum design flows.

Since the estimated water use design values are based on immediate as well as future expectations in growth and development, economic assessments are made to determine the long-range plan of the system. The decision may be to design the system for the short-term future and improve the system later rather than size and build the system for long-range future projections.

DESIGN REQUIREMENTS

Pressure Requirements

Pressure requirements in the system depend on the combined normal service requirements and fire flow demand. Pressures must be high enough to overcome energy losses within the distribution and service lines as well as the losses incurred from hydrants, nozzles, hoses, and other firefight-

TABLE 25.4 Estimated Peak Ratios for Residential Water Systems

POPULATION	RATIO OF MAXIMUM DAY TO AVERAGE ANNUAL USE	RATIO OF MAXIMUM HOUR TO AVERAGE ANNUAL USE
0–500	3.0	4.50
500–1000	2.75	4.13
1000–2000	2.50	3.75
2000–3000	2.25	3.38
3000–10,000	1.90	2.85
10,000–25,000	1.80	2.85
25,000–50,000	1.80	2.70
50,000–75,000	1.75	2.62
75,000–150,000	1.65	2.48
over 150,000	1.50	2.25

Reprinted by permission of R.V. Anderson Associates Limited, Willowdale, Ontario.
(*Source:* Recommended Guidelines for Residential Servicing in Ontario. 1973. Ontario Housing Advisory Committee.)

ing equipment. Additionally, pressure requirements are a function of the topography. Since most waterlines are installed at minimum depth, they closely follow the ground surface topography. Consequently, pressures in the system must be increased in the hilly service areas to overcome elevation differences. A balance must be struck between the pressures required for normal service operations and the pressures required for short-term high-demand and emergency occasions. Although the water supply system can be designed to operate at higher pressures, sustained excessive pressures add to system costs by increasing leakage volumes and potential damage to fixtures. To address fluctuations in pressure due to extreme topography changes and demands over a large area, some water distribution systems divide into different pressure zones to maintain reliability with the supply. These pressure zones allow the system to provide varying pressure ranges in different locations, but require some form of mechanical equipment, such as booster pumps, to move the water supply between the different zones.

A municipality's exact pressure requirements are established by a state agency, such as the health department, with supplemental requirements imposed by agencies within the municipality such as the fire marshal. Typically, street water main pressures of 20 lb/in^2 are the minimum when providing either maximum daily demand (including fire flow requirements) or the peak hour demand. This is a precautionary measure, designed to prevent potential cross-connection backflows from house fixtures. This minimum pressure also represents the minimum pressure required to overcome frictional losses in the service line and push the water up three stories, the upper height limit typical in residential areas.

Preferable normal pressures in the street should be between 40 lb/in^2 and 60 lb/in^2 in residential areas. In most cases, a pressure of about 80 lb/in^2 is recommended as the upper limit. Above 80 lb/in^2, special pressure-reducing valves may be necessary to prevent residential fixture damage. Typically, commercial areas require a minimum pressure of 75 lb/in^2. Booster pumps for tall buildings (e.g., more than 10 stories) eliminate the need for excessively high pressures in the street mains.

Fire Flow Requirements

Each municipality establishes its own parameters for fire flow requirements based on local conditions, although the municipality may instead refer to the fire flow recommendations provided by the Insurance Services Office (ISO). The ISO recommends criteria for establishing insurance rates and for classifying municipalities with reference to their fire defenses and physical conditions. The criteria provide a means for rating a municipality based on type of firefighting equipment, proximity of equipment, water distribution flows, storage, and other factors. One such document by the ISO is the "Guide for Determination of Required Fire Flow." Although the guide only recommends fire flows, many areas defer to it for fire flow considerations while others have established their own requirements. Minimum fire flows vary widely among jurisdictions. The design engineer should consult with the local water authority and fire marshal's office for specific criteria.

Estimating Fire Flow

The ISO uses the following equation to estimate the required fire flow:

$$F = 18 \times C \times \sqrt{A} \qquad (25.1)$$

where F = required fire flow in gpm, C = coefficient related to the type of construction, and A = total floor area in ft² including all stories but excluding basements.

For fire-resistive buildings, consider the six largest successive floor areas if the vertical openings are unprotected; if the vertical openings are properly protected, consider only the three largest successive floor areas. Coefficient $C = 1.5$ for wood frame construction, 1.0 for ordinary construction, 0.9 for heavy timber type buildings, 0.8 for noncombustible construction, and 0.6 for fire-resistive construction. Coefficients should not be greater than 1.5 nor less than 0.6 and may be determined by interpolation. Such interpolation should be between the consecutive types of construction just noted. Additionally, these guides recommend a minimum fire flow of 500 gpm and maximum fire flows under the following conditions: 8000 gpm for wood frame construction, ordinary, and heavy timber construction; 6000 gpm for noncombustible construction and fire-resistive construction. For normal one-story building of any type construction, the fire flow should not exceed 6000 gpm. Table 25.5 identifies required fire flows for areas of single-family and small two-family dwellings not exceeding two stories in height.

Adjustments to recommended fire flow values might be applicable for certain occupancy conditions. Equation 25.1 may be reduced up to 25 percent for occupancies having a low fire hazard or increased up to 25 percent for occupancies having a high fire hazard. Examples of occupancies considered low fire hazard are residential dwellings, churches, schools, hotels, hospitals, and other public buildings. Occupancy of high fire hazard are manufacturing and processing plants using explosives and high combustibles such as oil refineries, paint shops, and aircraft hangers. Other modifications that affect the required fire flow rate include the existence of sprinkler systems, noncombustible construction, and building separation. However, the range of adjustment to the initial fire flow estimates cannot be greater than 75 percent and the final adjusted flow cannot be less than 500 gpm nor exceed 12,000 gpm.

TABLE 25.5 Required Fire Flows for Single-Family and Small Two-Family Dwellings

Distance between buildings (ft)	Suggested required fire flow (gpm)
Over 100	500
31–100	750–1000
11–30	1000–1500
10 or less	1500–2000*

*If the buildings are continuous use a minimum of 2500 gpm. Where wood shingle roofs could contribute to spreading fires, add 500 gpm.
(From the publication "Guide for Determination of Required Fire Flow," 2nd ed. Reprinted with the permission of Insurance Services Office, Inc. Copyright 1974.)

Fire flow estimates resulting from applying these criteria are only a guide and warrant the judgment of experienced and knowledgeable persons in fire protection. Typically, local jurisdictions require fire flow approval by controlling agencies such as the fire marshal's office, public water supply agency, and health department. Exact requirements by local jurisdictions for fire flows may not be the same as what is recommended by the ISO. In fact, there can be a wide range of fire flow requirements even among adjacent jurisdictions.

Fire Flow Tests

Before new projects ultimately get approval for construction by the fire marshal's office, fire flow tests are performed to determine the fire flow capacity and adequacy of the existing system. Fire flow tests are used to determine available flow rates and pressures at various locations for firefighting purposes. Although the water utility company has extensive data on the existing water system in the larger municipalities, uncertainties in the hydraulic variables and service variables require further data to corroborate existing information. On commercial sites and small subdivisions, the test is performed on hydrants near the point where the proposed water main will connect to the existing water main. On larger sites, where the project is built in phases, fire flow tests may be required near the connection points at each subsequent phase of the development.

The following discussion on testing procedures and analysis is extracted from the National Fire Protection Association (NFPA). The exact method and analysis may deviate according to local regulations.

The test consists of discharging water at a measured flow rate from one or more hydrants and observing the resulting pressure drop in the main through another fire hydrant. When delivering fire flows, a minimum residual pressure of 20 lb/in² in the main is typically required for two basic reasons: First, fire department pumpers require 20 lb/in² in order to operate effectively, and second, a minimum residual pressure is needed to prevent a vacuum (negative pressures) from developing in the system that can cause collapse of pipes and inadvertent back siphonage of polluted water.

For the test, one base point hydrant is selected as the residual hydrant. The hydrants to be flowed are between the residual hydrant and the larger mains. Typical patterns for selecting the test hydrants are illustrated in Figure 25.1. The residual pressure at this location is measured before other hydrants are opened. The pressure at this location is then determined when the other hydrants are fully opened. The test is made during a period of ordinary demand such that the pressure drop in the residual hydrant is 25 percent or less of what it would be at the total demand necessary for firefighting purposes.

A pressure gauge is placed on one of the hose outlets on the residual hydrant, and the hydrant valve is fully opened.

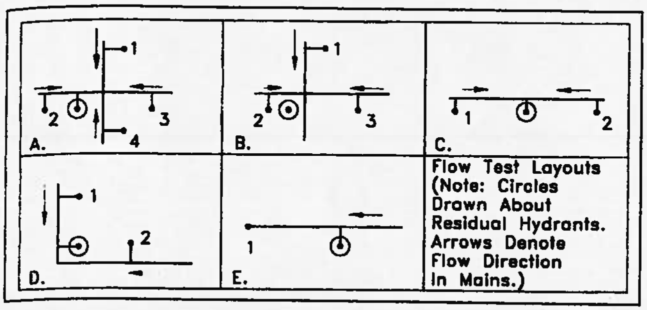


Figure 25.1 Suggested test layout for hydrants. (From the publication "Grading Schedule for Municipal Fire Protection." Reprinted with the permission of Insurance Services Office, Inc. Copyright 1974)

After the barrel air is exhausted, the pressure is recorded. The other hydrants are opened full in succession, one at a time. After each hydrant is opened and the flow stabilized, the pressure is recorded at the residual hydrant and the discharge at each open hydrant is measured using a pitot gage. The discharge Q (gpm) is determined from

$$Q = 29.83\, cd^2 \sqrt{p} \tag{25.2}$$

where c is the coefficient of discharge of the pitot tube (depends on the manufacturer specifications, typically around 0.9), d is the diameter of the outlet hose connection in inches, and p is the velocity pressure in lb/in^2 (i.e., the pitot tube reading).

After all observations are recorded, the discharge (in gpm) at the specified residual pressure or for any pressure drop is

$$Q_R = Q_F \times \left(\frac{h_r}{h_f}\right)^{0.54} \tag{25.3}$$

where Q_R = flow predicted at the desired residual pressure, Q_F = total flow measured during the test, h_r = pressure drop to the desired residual pressure, and h_f = pressure drop measured during test.

As the equation is written, any consistent units for the Q's and h's will work.

Fire Duration Requirements

In addition to providing adequate discharges and pressures for normal consumption, a water supply system must be designed to provide water for sustained periods for firefighting purposes. Since the magnitude, duration, location, and frequency of a fire are quite unpredictable, the design of a system should be based on a reasonable worst-case scenario.

The ISO defines the average daily consumption as the total amount of water used each day during a 1-year period considered as a rate for a 24-hour period. The maximum daily consumption is defined as the maximum total amount used during any 24-hour period in a three-year period, disregarding any unusually high circumstantial amounts caused by filling storage tanks. An adequate water supply, according to the ISO, can deliver the required fire flow for the duration given in Table 25.6 at the maximum daily rate.

Distribution Storage

Distribution storage, which consists of small reservoirs located near service areas, acts as service storage to compensate for the widely fluctuating demands and provides storage for firefighting and for emergency reserve. Distribution storage is used to normalize operating pressures, eliminate the need for nonuniform pumping, reduce the need for constructing large mains at the periphery of the system, and provide increased reliability for fire protection and other emergencies.

Whether ground storage or elevated storage tanks are used depends on such factors as topography, size of service area, reliability of main water supply, and economics. Auxiliary pumps at the ground storage facility or higher water surface elevations in the elevated storage reservoir can be utilized to equalize pressures during high-demand periods. Since short and frequent high-demand periods can require pumps to turn on and off regularly to maintain system pressures, one advantage to elevated storage tanks (over auxiliary pumps) is the ability to supply a sustained pressure without the cyclic pump operations. However, the selection of the type of distribution storage should be considered on a case-by-case basis.

Although the need for design and construction of storage facilities is infrequent in moderately sized land development projects, very large projects or areas of unanticipated rapid growth may warrant distribution storage facilities. Nonetheless, the project engineer should be aware of the major factors governing the design of the facilities. In many cases, an engineer specializing in the design of waterworks will be

TABLE 25.6 Required Fire Flow Duration

Required Fire Flow (gpm)	Required Duration (hr)
10,000 and greater	10
9500–9000	9
8500–8000	8
7500–7000	7
6500–6000	6
5500–5000	5
4500–4000	4
3500–3000	3
2500 and less	2

(*Source:* Virginia Dept. of Health, 1993, Waterworks Regulations.)

called on to perform the specific design. In the projects where such facilities are required, the developer and jurisdiction may enter into some kind of joint venture where the costs would be shared by the two. This practice is common in instances where users beyond the project boundaries would share the facility.

The size of a storage tank depends on the characteristics of the service zone, fire demand requirements, and the local standards governing such design. A general rule-of-thumb storage requirement is given as one day's supply plus fire flow. The one-day supply is based on the maximum day usage, while the fire flow storage is based on the duration as suggested by the ISO for the design fire flow (see Tables 25.5 and 25.6). Note that local design standards may supersede this method of storage demand design. Preferably, the tank should be sized to provide two to three days' supply, although economic and risk analysis will dictate the upper limit of the size.

Another consideration for storage tank sizing deals with the retention time of the water in the tank. Since a storage tank is designed to supplement the added demand during peak periods and refill during off-peak periods, a properly designed tank under most operating conditions never completely empties. However, due to this fluctuation, there is a somewhat continuous exchange of water within the tank. With the residual effects of disinfectant chlorine limited to six to ten days, the maximum residence time of water in the tank should be less than eight days to prevent stagnation and other health problems.

Formal design of a storage facility should incorporate the hourly use hydrograph or mass diagram for the maximum day. A description of the design can be found in most basic water resource textbooks. One way to estimate storage requirements based on land use follows.

Example 1

Table 25.7 shows the land use for a new service area and the corresponding demand for the average day for the type of land use. For this service area, fire flow requirements are 4 hours' duration for a flow of 2500 gpm. Local ordinance requires a reserve of 20 percent of the maximum day demand, and the maximum day demand is assumed to be equal to 1.6 times the average day demand. Use a t factor of safety = 1.25. What are the storage requirements, given these parameters?

Storage Required:

$$\text{4-hour fire flow @ 2500 gpm} \times 60\ \text{min/hr} = 600{,}000\ \text{gal} \quad (25.4)$$

$$2{,}068{,}910\ \text{gal} \times 1.6 \times 0.20 = 662{,}051\ \text{gal}$$

$$\text{Subtotal} = 1{,}262{,}051\ \text{gal}$$

$$\begin{aligned}\text{Storage} &= 1{,}262{,}051 \times (1.25)\\ &= 1{,}577{,}564\ \text{gal}\end{aligned}$$

Further considerations in this analysis would be to determine whether one or several facilities would supply the required storage, the size of each facility, and the location.

TABLE 25.7 Demand Requirements for New Service Area (Example 1)

Land use	Unit of measurement	No. of units	Average demand (gpd)	Total demand (gpd)
Single-family detached	per dwelling	1682	350	588,700
Town house	per dwelling	519	350	181,650
School	per student	750	16	12,000
Daycare center	per student	200	16	3200
Recreation center	per person	1528	10	15,280
Hotel	per room	120	168	20,160
Restaurant	per seat	800	50	40,000
High-rise commercial	per acre	89.1	2000	1,782,000
Low-rise commercial	per acre	362	1500	543,000
			Subtotal =	1,582,190
Existing demand				486,720
			TOTAL =	2,068,910

PUMPING FACILITIES

Pumping facilities are necessary to transport treated water through the transmission and distribution lines to fill storage tanks and maintain system pressure. For small water utilities, the well pump or the single finished water pumping arrangement at a treatment plant may be all that is required. For large municipal distribution systems, a complex network of pumping stations, monitored and controlled from a central point, may be needed.

Pump Selection

Small water utilities commonly employ three types of pumps for water distribution service: centrifugal, positive displacement, and jet (ejector).

Centrifugal Pumps. Centrifugal pumps are the most commonly used of the three. A centrifugal pump involves a casing containing a rotating impeller mounted on a shaft turned by a motor power source. Water that enters the suction side of the rotating impeller is thrown at high velocity against the casing to convert velocity head into pressure. Centrifugal pumps can be designed to provide stages in series, where each impeller and matching casing constitutes a stage to realize pressures not attainable with a single-stage pump. Pumps having more than one stage are called multistage pumps, of which there are two types—submersible pumps and turbine pumps.

The submersible pump consists of one or more pump stages driven by a closely coupled motor designed for submerged operation (see Figure 25.2). They are commonly used in wells, but may also be used in finished water clear wells at treatment facilities.

Turbine pumps consist of one or more centrifugal pump stages driven by a vertical shaft, connecting the pumping assembly to a motor mounted at the surface. Figure 25.3 shows a turbine pump used for well service. Turbine pumps in this configuration are called vertical turbine pumps. Turbine pumps can also be mounted on treatment plant clear wells for distribution of finished water.

Positive Displacement Pumps. Positive displacement pumps, as the name implies, displace a set volume of water with each turn of the pump. The pumping rate varies with the speed of the pump. Although these pumps are not that common in water distribution service, especially in high-volume applications, they are well suited for intermittent pumping at high pressure. Positive displacements pumps are available in several configurations:

Reciprocating pumps: This pump consists of a plunger driven back and forth in a closely fitted cylinder with check valves at both the inlet and the pump outlet.

Helical rotor pump: The helical pump consists of a spiral rotor that rotates in a sleeve. As the spiral rotor turns, it traps water between the rotor and the sleeve, forcing it to the outlet end of the sleeve.

Rotary gear pump: This type of pump uses two meshing gears housed in a sealed casing that traps water in cavities between them. As the gears mesh, they carry water from the inlet to outlet port.

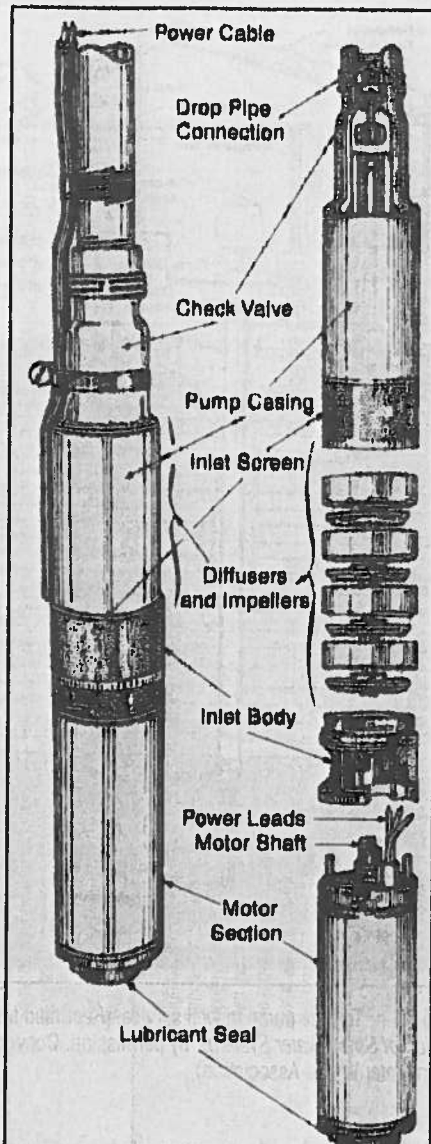


FIGURE 25.2 View of a multistage submersible pump. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, American Water Works Association).

Jet (Ejector) Pumps. Jet pumps are actually a combination of centrifugal and ejector pumps (see Figure 25.4). A portion of the centrifugal pump discharge is diverted through a nozzle and venturi tube near the water level of the source. This creates a low-pressure zone that draws flow upward toward the surface, where the centrifugal pump suction can further lift it into the distribution system. These jet installations are economical for low-volume facilities, and are commonly used in wells.

The fundamental considerations for pump selection within a particular installation are as follows:

- Yield of the well, or other water source
- Total daily needs and instantaneous demand of the system

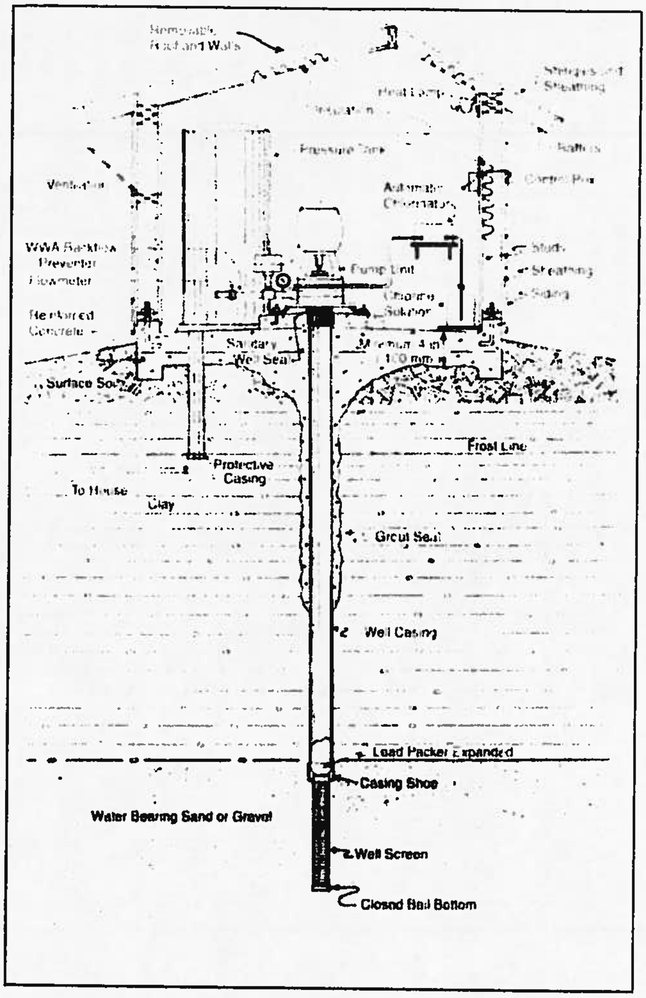


FIGURE 25.3 Turbine pump in well service. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, American Water Works Association).

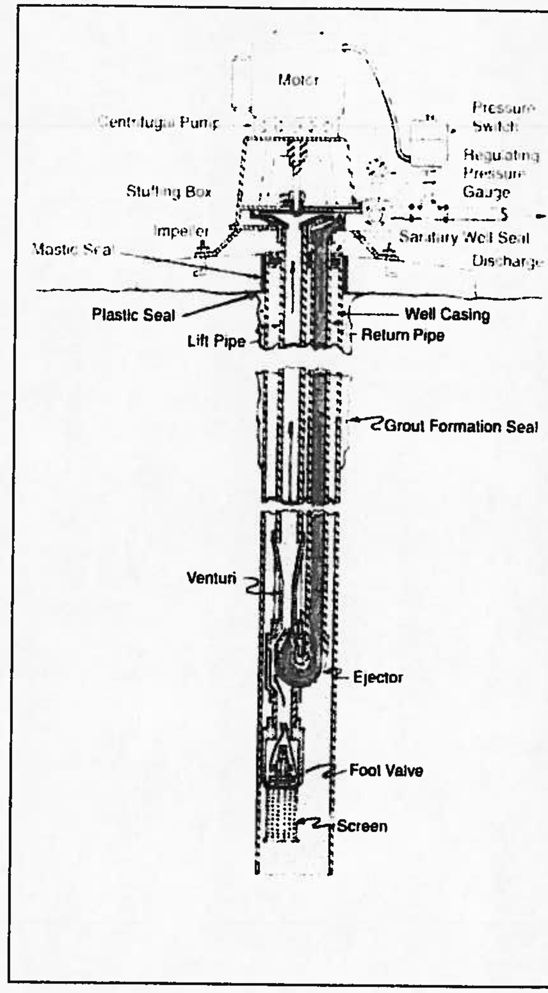


FIGURE 25.4 Combined centrifugal and ejector pump. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, American Water Works Association)

- Total operating head pressure of the pumps at normal delivery rate, including lift and all friction losses
- Elevation difference between the ground level and the water level in the well during pumping
- Availability of power
- Ease of maintenance and availability of parts
- Initial and operations costs
- Reliability of pumping equipment

Table 25.8 provides information useful for selecting the type of pump needed for typical small system applications. Selecting the proper pumping unit size must be done by carefully evaluating the peak demand, available storage in the system, and capacity of the water source. Increased storage capacity is needed when source yield is low in comparison with peak demand (as often happens with well sources). This is necessary to avoid starting and stopping pump motors more than four or five times per hour in order to maintain optimum service life of the equipment.

System Head Curve

The system head curve is an essential tool for use in determining the pump's type, size, and number of units to be applied for the most economically feasible installation. It graphically presents hydraulic head requirements for various flow characteristics of a particular system, making it easier to determine the appropriate configuration. Figure 25.5 shows an example of system head curve including a typical centrifugal pump curve.

The concept of total dynamic head must be understood to construct a proper system head curve for a potable water distribution pumping system. Figure 25.6 is a graphical representation of total dynamic head for a well pump installation. Static head, friction head, and pressure head are the three components of total dynamic head. Static head is the difference in elevation of the source water and the point of discharge. Friction head is the loss that occurs due to valves, fittings, and piping (discussed later in this chapter under "System Analysis, Minor Losses"). The pressure head includes the velocity head as well as the minimal residual pressure required in the system by state health departments (usually no less than 25 lb/in^2 at the outlet fixture).

TABLE 25.8 Specifications for Various Types of Pumps

Type of Pump	Practical Suction Lift*	Usual Well-Pumping Depth	Usual Pressure Heads	Advantages	Disadvantages	Remarks
Reciprocating: Shallow well	22–25 ft (7–8 m)	22–25 ft (7–8 m)	100–200 ft (30–60 m)	Positive action Discharge against variable heads	Pulsating discharge Subject to vibration and noise	Best suited for capacities of 5–25 gpm (19–95 L/min) against moderate to high heads
Deep well	22–15 ft (7–8 m)	Up to 600 ft (183 m)	Up to 600 ft (183 m) above cylinder	Pumps water containing sand and silt Especially adapted to low capacity and high lifts	Maintenance cost may be high May cause destructive pressure if operated against closed valve	Adaptable to hand operation Can be installed in very small diameter wells (2-in [51-mm] casing)
Centrifugal Shallow well Straight centrifugal	20 ft (6 m) maximum	10–20 ft (3–6 m)	100–150 ft 30–46 m)	Smooth, even flow Pumps water containing sand and silt Pressure on system is even and free from shock Low starting torque Usually reliable; good service life	Loses prime easily Efficiency depends on operating under design heads and speed	Very efficient pump for capacities above 60 gpm (227 L/min) and heads up to about 150 ft (46 m)
Regenerative vane turbine type (single-impeller)	28 ft (7 m) maximum	28 ft (7 m)	100–200 ft (30–60 m)	Same as straight centrifugal except not suitable for pumping water containing sand or silt Self-priming	Same as straight centrifugal, except maintains priming easily	Reduction in pressure with increased capacity not as severe as with straight centrifugal
Deep well Vertical line shaft turbine (multistage)	Impeller submerged	50–300 ft (15–91 m)	100–800 ft (30–244 m)	Same as shallow-well turbine All electrical components are accessible aboveground	Efficiency depends on operating under design head and speed Requires straight well large enough for turbine bowls and housing Lubrication and alignment of shaft critical Abrasion from sand	
Submersible turbine (multi-stage)	Pump and motor submerged	50–400 ft (15–122 m)	50–400 ft (15–122 m)	Same as shallow-well turbine Easy to frostproof installation Short pump shaft to motor Quiet operation Well straightness not critical	Repair to motor or pump requires pulling from well Sealing of electrical equipment from water vapor critical Abrasion from sand	3500 RPM models, although popular because of smaller diameters or greater capacities, are more vulnerable to wear and failure from sand and other causes
Jet Shallow-well	15–20 ft (5–6 m) below ejector	Up to 14–20 ft (5–6 m) below ejector	80–150 ft (24–46 m)	High capacity at low heads Simple operation Does not have to be installed over the well No moving parts in well	Capacity reduces as lift increases Air in suction or return line will stop pumping	
Deep Well	15–20 ft (5–6 m) below ejector	25–120 ft (8–37 m) 200 ft (61 m) maximum	80–150 ft (24–46 m)	Same as shallow-well jet Well straightness not critical	Same as shallow-well jet Lower efficiency, especially at greater lifts	The amount of water returned to ejector increases with increased lift—50% of total water pumped at 50-ft (15 m) lift and 75% at 100-ft (30 m) lift
Rotary Shallow-well (gear type)	22 ft (7 m)	22 ft (7 m)	50–250 ft (15–76 m)	Positive action Discharge constant under variable heads Efficient operation	Subject to rapid wear if water contains sand or silt Wear of gears reduces efficiency	
Deep-well (helical rotary type)	Usually submerged	50–500 ft (15–152 m)	100–500 ft (30–152 m)	Same as shallow well rotary Only one moving pump device in well	Same as shallow-well rotary, except no gear wear	A cutless rubber stator increases life of pump Flexible drive coupling has been weak point in pump Best adapted for low capacity and high heads

*Practical suction lift at sea level. Reduce lift 1 ft (0.3 m) for each 1,000 ft (305 m) above sea level.

(Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999. American Water Works Association)

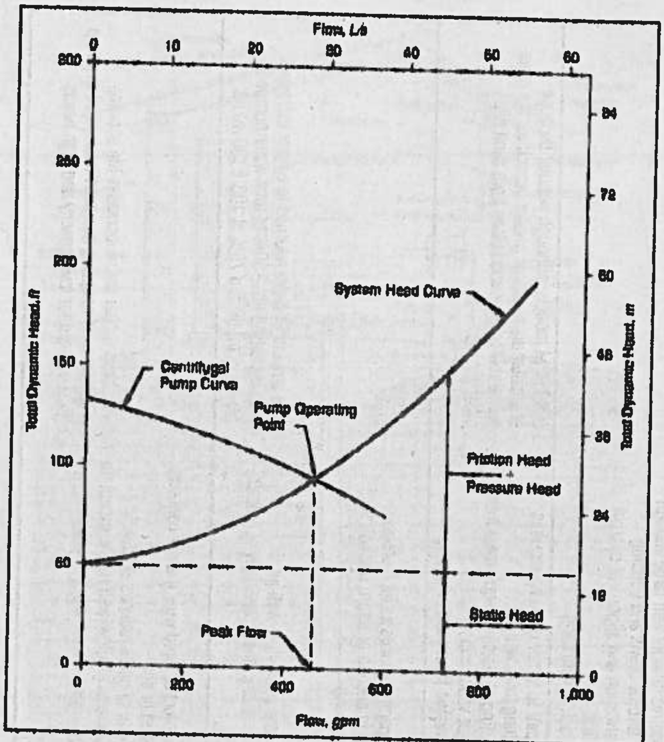


FIGURE 25.5 System head curve. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, American Water Works Association)

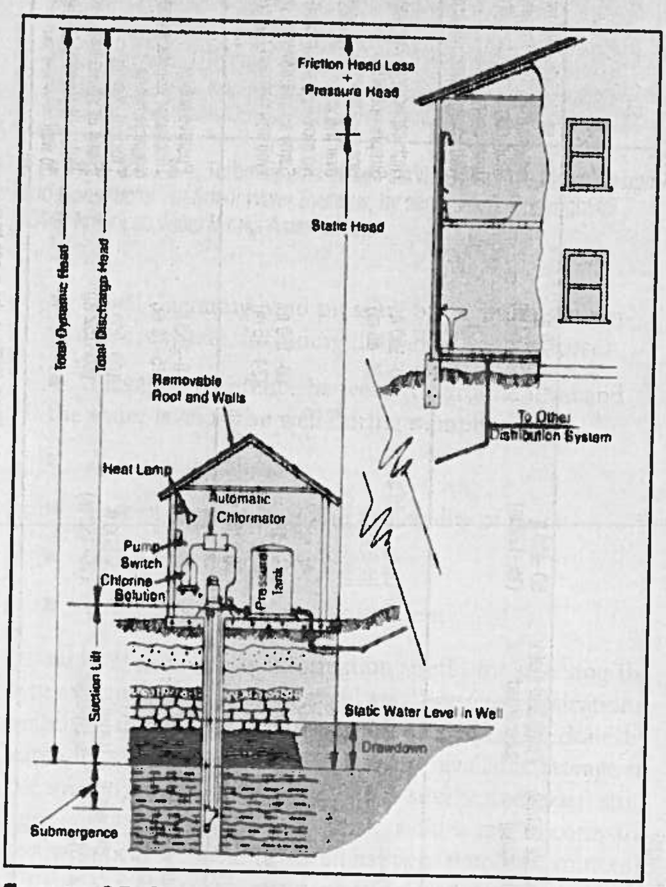


FIGURE 25.6 Components of total dynamic head in well pump. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, American Water Works Association)

Water hammer, or hydraulic surge, occurs when water flowing in a pipe comes to an abrupt stop. A common example of this is when a pump shuts off suddenly due to a power failure. On such an occasion, the resulting surge can cause system pressure that is many times the normal working pressure. These high pressures, if not adequately accounted for in the design, can damage piping, valves, and pumping equipment. Lack of proper surge protection can result in catastrophic failure, requiring several days of repair while the utility customers are without water service. Surge protection must be carefully designed, even for small water utilities. Common surge protection methods include surge relief valves and pressure surge tanks located on the discharge main near the pumps and vacuum relief valves located at key points on long forcemains.

Pump Drives

Pumps in the water utility industry are normally driven by alternating current (AC) motors. The power source for AC motors can be single-phase or three-phase. However, single-phase is generally limited to small motors of 10 hp (0.75 kW) or less. On a first cost basis, establishing a three-phase power source for a pumping facility is more expensive than it is with single-phase, although the three-phase is more efficient.

AC motors can be provided as squirrel-cage, synchronous wound, or wound-rotor induction types. However, squirrel-cage inductor types are most commonly used because they are less expensive than the other two and they have proven operation. In general, lower-speed motors are larger and more expensive than higher-speed motors, but they have a longer service life. Normally, the needs of small water utilities can be met through properly designed constant-speed pumping arrangements of multiple pumps. However, two-speed squirrel-cage motors are available.

Note that fairly recent improvements in technology, reliability, and costs of variable-frequency drives (VFDs) make them worthy of consideration for small utility service. VFDs work by converting AC to DC and then back to AC but at a different frequency. Since the speed of a synchronous motor is directly proportional to frequency, varying the frequency varies the motor speed. Because of this conversion process, VFDs produce considerable heat that must be properly vented or even addressed by the use of air conditioning.

Packaged Booster Stations

Packaged booster pumping stations are very popular with small water utilities because of their economy and ease of installation. These units, which are delivered to the construction site already assembled, complete with controls and electrical, can be installed with minimal labor and time. Although these packaged pumping units can be used for primary finished water service at treatment plants, they are more often used as booster stations out in the distribution system. These packaged stations can be ordered for either above- or below-ground-level installations, as shown in Figures 25.7 and 25.8, respectively.

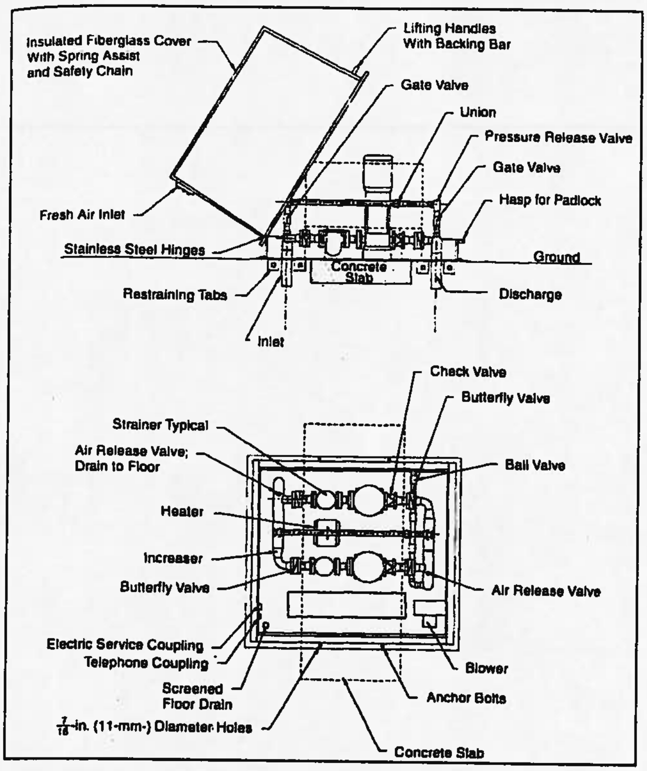


FIGURE 25.7 Aboveground packaged booster pump station. (Reprinted with permission of Universal Sanitary Equipment Manufacturing Co., Tumah, WI)

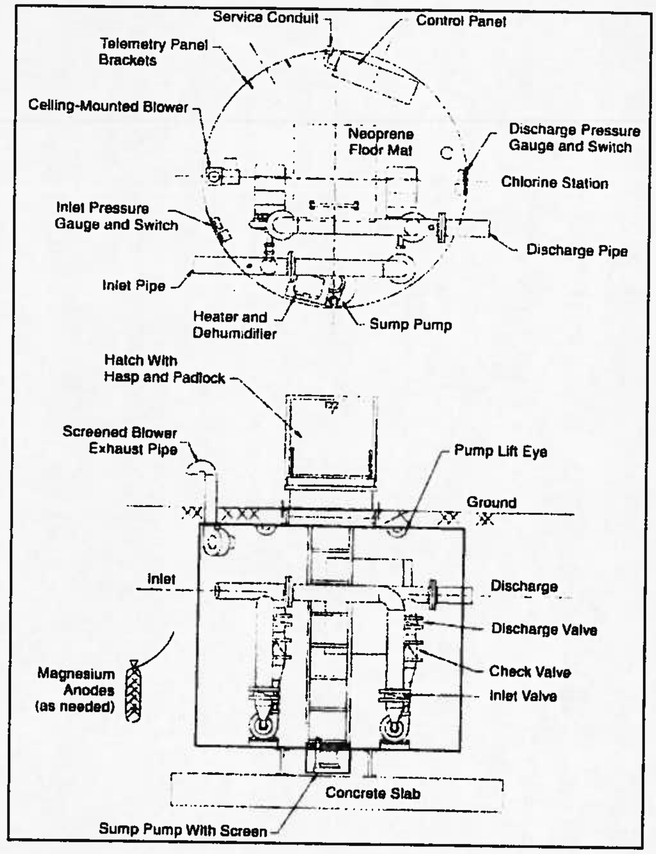


FIGURE 25.8 Below-ground packaged booster pump station. (Reprinted with permission of Universal Sanitary Equipment Manufacturing Co., Tumah, WI)

Emergency Power

Electrical power to a treatment facility or pumping station can be disrupted by a storm or other natural disaster for many hours or even for days. Many state health departments require that critical waterworks and pumping stations be protected against power failure by a standby power source. Normally, the standby power source is a connection to a second independent public power source (served by a separate power grid that serves the primary source) or a standby generator set.

As a minimum, generator sets should be sized to provide power for the average system demand plus basic lighting and heating. Generator sets must be maintained and regularly exercised to ensure that they will operate properly when called upon during an emergency. Although these sets normally operate on diesel fuel, units can be obtained that operate on propane, liquefied natural gas, or natural gas, if available. Both the generator set and its auxiliary fuel tanks must be properly secured against vandalism, and the fuel tank and its feed lines must be contained to prevent contamination from leakage.

There are several available options for securing and containing the generator set and its fuel tank. The first option is a custom-designed, built-in-place building. The advantages of such a building are that it can match the aesthetics of nearby buildings and surroundings and can provide ideal conditions for maintenance and repair of the set. Care must be taken to properly ventilate the building to prevent air starving of the operating generator set. This is typically done with properly sized louver systems that are opened either by the force of the air being pulled toward the generator set or by electric motors that activate when the generator set is started.

Generator sets can also be provided by the manufacturer with a sound-attenuated enclosure made of steel or fiberglass, with removable panels to enable repairs (see Figure 25.9). The disadvantage of this is that the repair personnel must

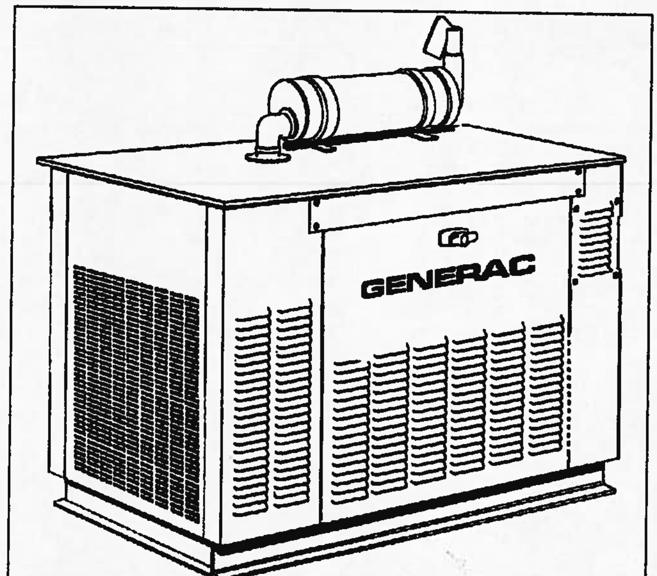


FIGURE 25.9 Sound attenuated walkin enclosure. (Reprinted with permission of Copyright © Tramont Corporation 2001)

stand in the elements to do their work. To facilitate repair, the generator manufacturer can also provide a sound-attenuated shelter to cover the set and provide space for repairs to be done under protected conditions. These shelters are essentially small prefabricated buildings made of steel or fiberglass that fit compactly over the set, with clearances to allow repairs out of the weather. They are generally less costly than custom-designed buildings, but are more costly than a simple generator set enclosure (see Figure 25.10).

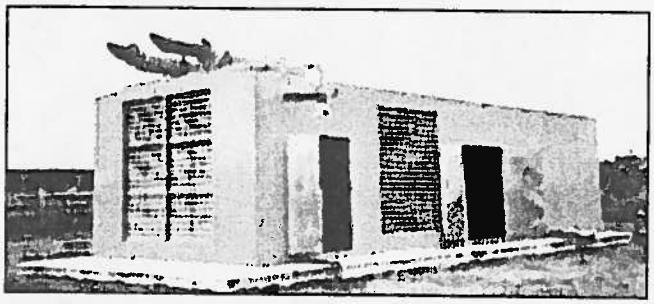


FIGURE 25.10 Typical generator enclosure. (Reprinted with permission of General Power Systems, Wakesha, WI)

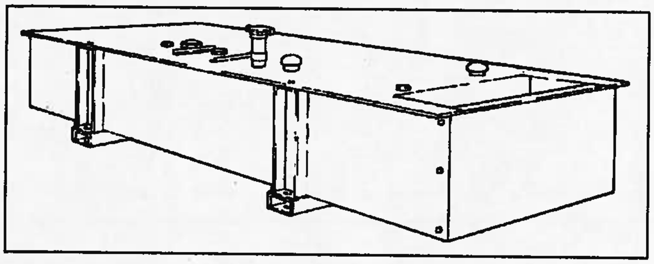


FIGURE 25.11 Skid-mounted fuel tank for location beneath a generator set. (Reprinted with permission of Cummins Power Generation)

Until recently, the normal arrangement for fuel tanks serving generator sets was to use buried tanks. However, as with buried tanks serving gasoline stations, there have been many instances when failure of the tanks and their feed lines causes contamination of the surrounding soils and ground water. Buried tanks are still a viable option, but they must meet much higher standards concerning leak monitoring and double-walled containment for the tank and buried lines (state and local regulatory agencies can provide detailed standards for buried tank installations). Because this has greatly increased the cost of buried tanks, most public utilities are choosing other options.

The other available options for nonburied fuel tanks are aboveground tanks in buildings that are otherwise protected. One approach is to use a protected skid-mounted tank that is

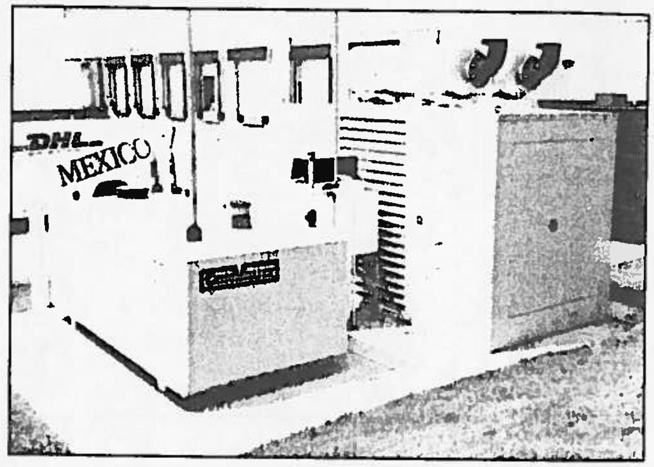


FIGURE 25.12 Aboveground storage tank and generator set. (Reprinted with permission of Core Engineered Solutions)

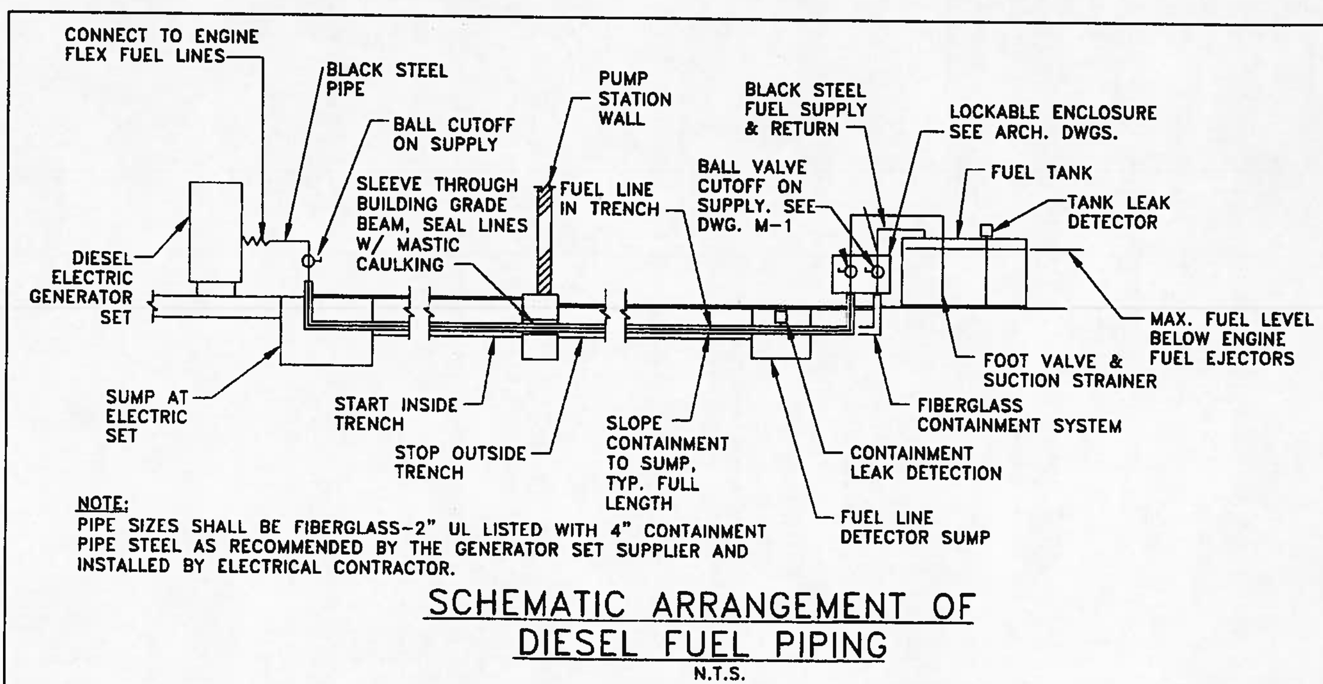


FIGURE 25.13 Schematic arrangement of diesel fuel piping.

designed to fit securely under a skid-mounted generator set. These tanks are double-walled for containment, and are protected by the generator shelter or enclosure and the skid framing. Their feed lines are inside the generator enclosure and are therefore also protected (see Figure 25.11).

Another option for an aboveground tank is a concrete-enclosed tank such as that provided by Convault (see Figure 25.12). The feed lines from the protected tank must also be protected. If buried, the lines need to be double-walled for containment. Some utilities also require the buried double-walled lines to be installed in a lined trench (see Figure 25.13).

SYSTEM COMPONENTS

Essential to a distribution system are such components as valves, fittings, fire hydrants, thrust blocks, service lines, and meters to control, direct, and measure the flow. The location and size of these components impacts the system's hydraulic efficiency and cost effectiveness as well as the health, safety, and welfare of the public.

Valves

Approximately 16.7 percent of all valves in use are for water and sewerage purposes (Merrick, 1991). In a waterworks system, valves are used to control flow directions, regulate flow rates, control pressure, isolate flows, and suppress transient waves. Typically, faucets, bibs, cocks, stoppers, and plugs are types of valves used in the water supply lines within a building, while gate valves, butterfly valves, and check valves are those typically used in water distribution lines serving the buildings.

A valve is completely closed when some type of operating mechanism, such as a wheel, forces the gasket or plug tightly against the fixed seat. The operating mechanism—that is, actuator—for a valve is either manual or automatic. Manual actuators include wheels and levers; automatic actuators can be electric, pneumatic, or hydraulic. Whereas automatic valves are more commonly used at the treatment facilities or pump stations, most valves used in the distribution pipe network are manual.

In addition to their function, valves are also described by their numerous characteristics such as size, material, fitting end, pressure rating, and actuator. The five most common types of valves used in waterlines are the ball valve, the plug valve, the gate valve, the butterfly valve, and the globe valve.

A *ball valve* (Figure 25.14) has a slotted spherical or section of a spherical closure element that rotates within the casing. The valve is opened when the opening is parallel to the flow direction. A ¼-turn of the opening closes the valve. Typically, ball valves are used for on-off service and throttling the flow.

A *plug valve* (Figure 25.15) is similar to the ball valve in that a cone or cylinder attached to a shaft, with a rectangular slot or circular orifice, opens and closes the flow by rotating the slot parallel to the direction of flow. Eccentric plug valves use a seat that consists of only part of the cone or cylinder to close the valve. The shaft rotates the seat over the pipe opening to regulate the rate of flow. Typically, plug valves are used for control and isolation purposes. Smaller plug valves are used on service connections and are referred to as service or corporation cocks. Plug valves have low head losses and typically cost more than gate, globe, and butterfly valves.

Gate valves, a type of the broader classification of slide valves, are the most frequently used when trying to isolate flows in the pipe network (see Figure 25.16). The gate valve (shutoff valve) is used to completely stop the flow through the pipeline. They operate by raising or lowering a plate or disc into the flow path. A shutoff valve's intended use is to operate in the full open or full closed positions only, and not as a throttling valve that is partially opened. Considerable wear and tear on the mechanism occurs in the partially opened position, and the head loss, at this position, is very high. Typically, gate valves are placed in valve boxes that extend to the ground surface to permit access. The valve is operated with an extension wrench that reaches down

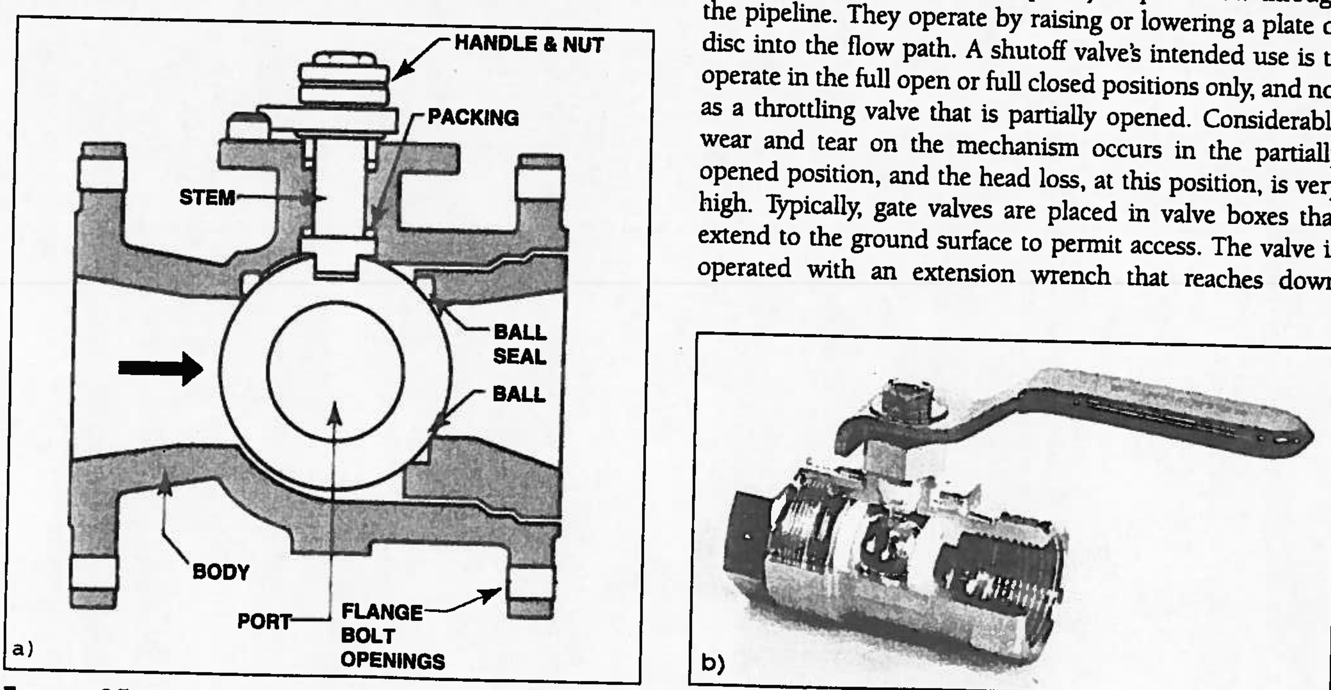
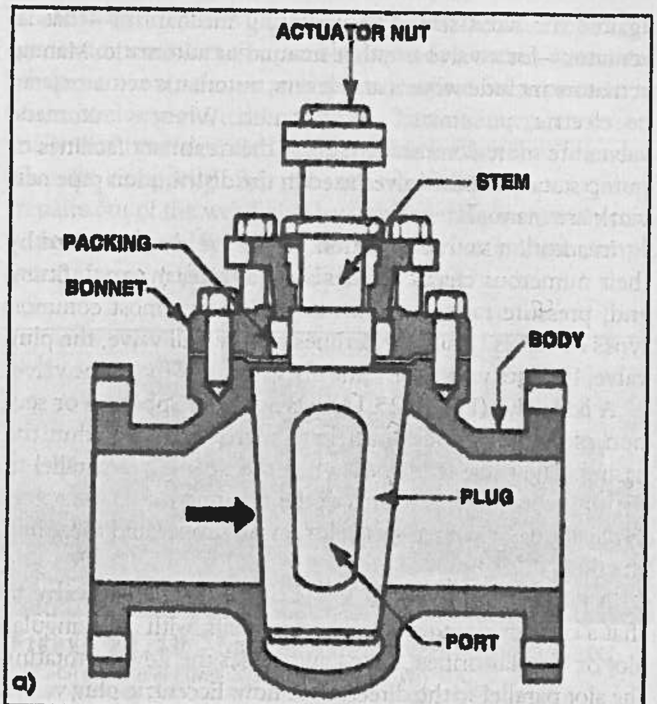


FIGURE 25.14 Ball valve: (*a*) schematic; (*b*) photo diagram. (Courtesy of Flowserve Corp.)



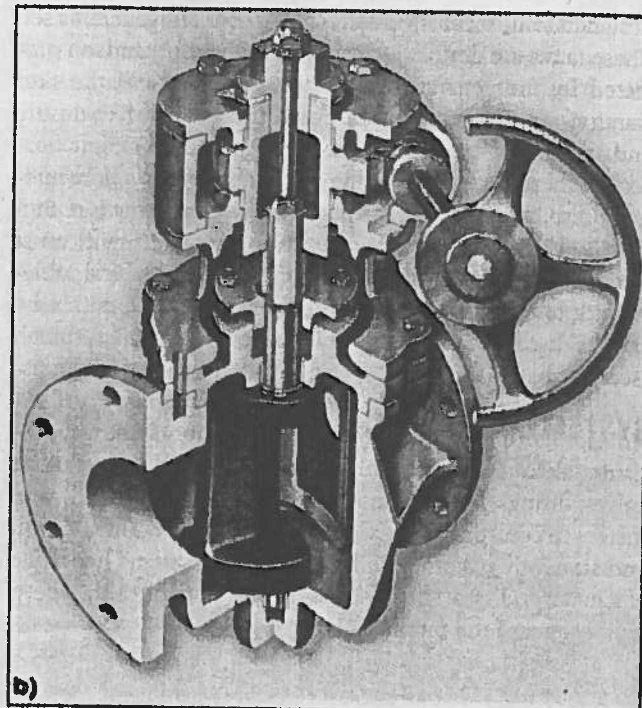
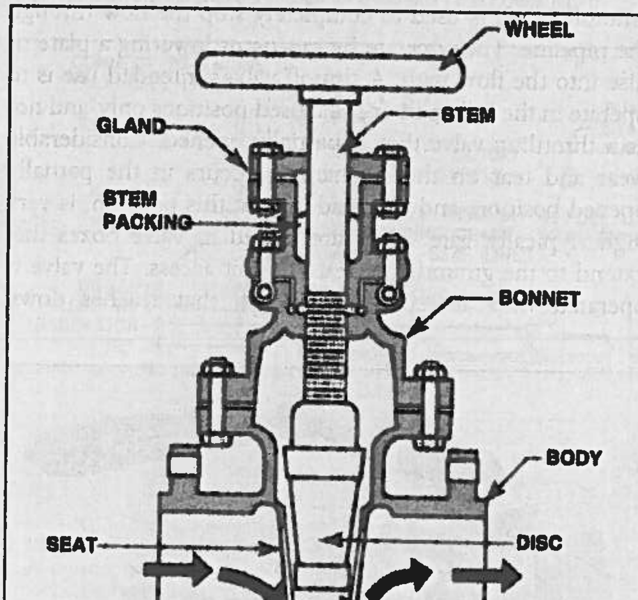


FIGURE 25.15 (*a*) Schematic diagram of a plug valve; (*b*) eccentric plug valve. (Reprinted with permission of DeZurik, Sartell, MN)

through the box to turn the operating nut on top of the valve stem.

As shown in Figure 25.17, a *butterfly valve*, also a type of rotary valve, consists of a thin disc that rotates about a thin shaft. When the face of the disc is parallel to the flow directions, the valve is fully opened. Butterfly valves can be used



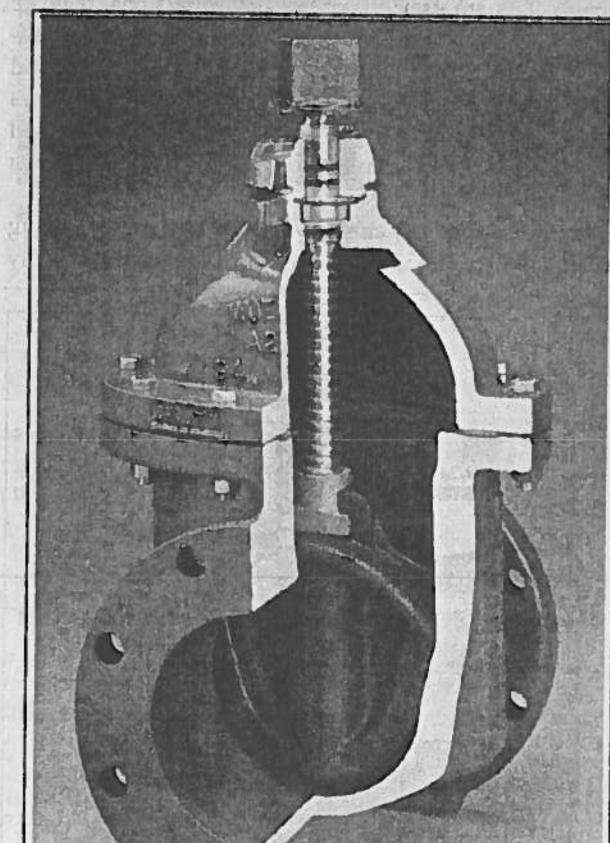


FIGURE 25.16 Gate valve: (*a*) schematic diagram; (*b*) resilient seat gate valve. (Reprinted with permission of Mueller Co., Decatur, IL)

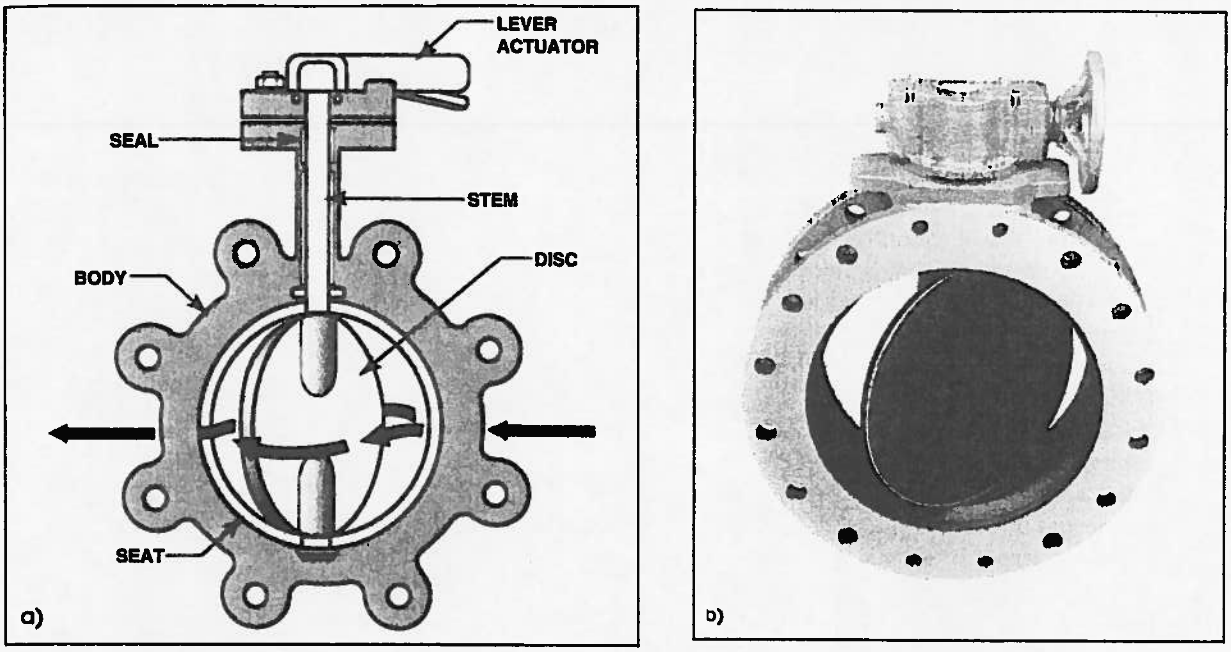


FIGURE 25.17 (*a*) Butterfly valve; (*b*) AWWA butterfly valve. (Reprinted with permission of DeZurik, Sartell, MN)

for either shutoff or throttling purposes. Although these types of valves have relatively low head loss, they present problems when cleaning since the disc is in the flow stream.

The *globe valve,* which is another type of slide valve, consists of a flexible disc attached to a screw-operated stem. A schematic diagram of a globe valve is shown in Figure 25.18. Raising and lowering the disc onto a horizontal seat controls the flow. Use of globe valves is most common in smaller domestic waterlines; a familiar globe valve is the household faucet. High-pressure losses are typical of most globe valves.

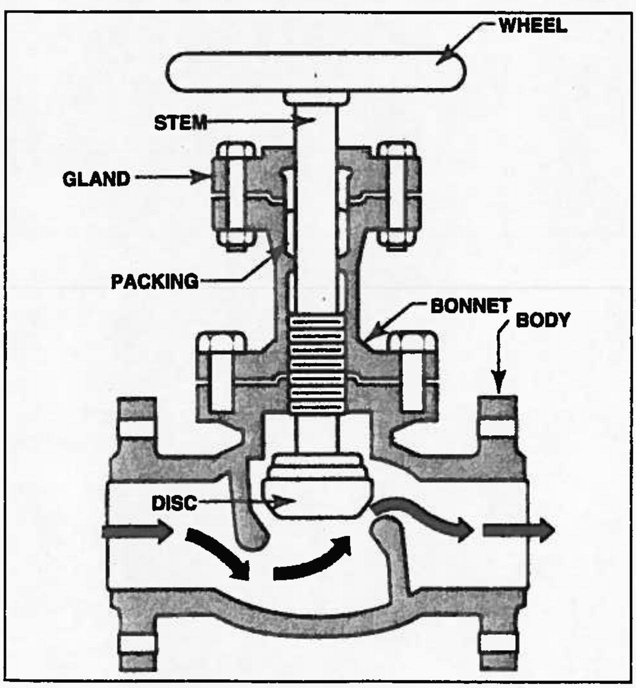


FIGURE 25.18 Schematic diagram of a globe valve.

A summary of the important characteristics of these particular valves appears in Table 25.9. In addition to the basic valves just described, there are other specialty valves used in waterworks systems.

Check valves are pressure-activated valves. Typical water main systems are bidirectional in that water flows according to the pressure differential in the pipes. Check valves are inserted to restrict flows to one direction. For example, a check valve on the discharge end of a pump or treatment plant prevents the water from reversing directions and flooding out the facility when the facility shuts down operation. Check valves are also used to control flows at the boundary of different water supply districts and service areas of different pressures.

Pressure-reducing valves (PRV) are automatic valves that protect against excess pressure in the waterlines. Pressure-reducing valves reduce pressure by controlling the flow and intentionally causing high head losses. Large pressure-reducing valves are operated by a smaller pilot valve. A wheel on the pilot valve sets the internal spring tension, which in turn sets the activating pressure. Larger PRVs are placed on lines between service areas of different elevations. Smaller PRV (less than 2-inch) valves are used to protect residential plumbing from excessive pressures in the main line. The valve acts as the transition between high-pressure zones and low-pressure zones. For example, if line pressures are greater than 70 lb/in^2, a pressure-regulating valve installed on a domestic service line reduces the service line to pressures so that the pressure will not damage household fixtures.

Altitude valves are automatically activated according to the pressure differential on either side of the seat. This type of

TABLE 25.9 Valve Summary (Water Distribution Operator Training HB, AWWA, 1976)

Valve Type	Common Size Range (in.)	Particularly Adapted to	Main Advantages	Main Disadvantages
Gate	½–24	Isolation services in distribution grids	Low cost in small sizes Low friction Good service life Ease of installation	High cost in large sizes Large sizes are quite heavy Poor for throttling; should not be used where frequent operation is necessary
Butterfly	3 & up	Isolation and automatic control	Low cost in larger sizes for normal service pressures Some types have very short lengths Ease of operation	Higher friction loss than gate valve Difficult to open in lines where differential pressures exist May cause problems when relining pipe Leaks because of seat damage
Globe	½–24	Isolation in smaller sizes Flow control in larger sizes Pressure control	Simple construction Dependable, can be used for throttling Good for pressure control Sediment or material unlikely to prevent complete closing	High friction loss Very heavy and expensive in large sizes
Ball and Plug	½–3 ½–12	Isolation and throttling	Dependable Very low friction loss Slow shut-off characteristic minimizes closing surges Ease of operation Resistant to erosion Long life	High cost in large sizes Very heavy

valve is usually used to control the flow to and from an elevated storage tank. The valve opens, allowing the tank to refill when the pressure in the distribution line is greater than the static pressure of the water in the tank. In times of high demand, when the pressure in the main decreases, the valve opens to allow the water in the storage tank to equalize the distribution system.

Blowoff valves and *drain valves* are used to dewater lines for repairs or remove accumulated sediment. These valves are most commonly used at the end of branches and at low points of the water main (see Figure 25.19).

Air-release valves. Water at standard conditions contains about 2 to 3 percent dissolved air by volume. The amount of dissolved air in a waterline system is governed by the turbulence before it enters the system and the pressure and temperature within the system. Pressurized distribution systems change elevations as the pipe follows the terrain. In effect, the changes in elevation change the pressure in the system. Consequently, the air dissociates from the water and collects at the higher points in the pipeline. Since the return of air back into the water does not occur as readily as it leaves the water, air pockets form and add 10 to 15 percent more resistance to the flow. Additionally, these pockets can lead to air lock of the system, which can completely stop the flow. Typically, air-release valves are installed at the local peaks in the pipe system. As shown in Figure 25.20, a peak is any section of pipe that slopes up toward the hydraulic grade line or runs parallel to it. Air-release valves should also be considered in sections where there is an increase in the downward slope or decrease in the upward slope[1] of the pipe.

Hydraulic surge (water hammer) can occur when water flowing in a long pipeline suddenly stops. Surge can actu-

[1]Golden Anderson Automatic Valve Reference Manual, Golden Anderson Industries, Inc., PA.

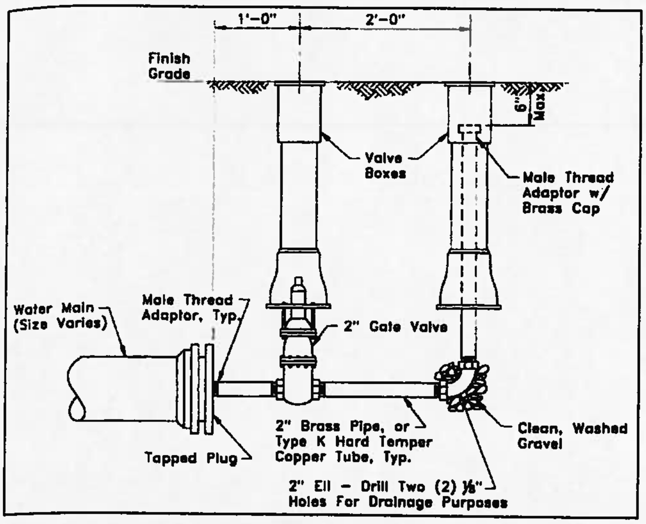


Figure 25.19 Two-inch blowoff valve detail.

ally cause a water column separation at high points, or peaks, on the main. This, at first, exerts a tremendous negative pressure (or vacuum) on the main that can cause it to collapse. If the system does not collapse, the vacuum will be followed immediately by a pressure spike that can rupture the pipe as the separated water columns reunite. To prevent this, a combination of air-release and vacuum relief valves are used at points on water mains where analysis indicates water column separation might occur. The vacuum relief valve lets air into the pipeline during hydraulic surge to both relieve the vacuum and provide a volume of air that cushions the pressure spike caused by the two water columns coming together again. Note that hydraulic surge analysis is a specialized field and must be done by qualified personnel.

Pipes, Joints, and Fittings

Typically, pipes used in waterworks systems are classified according to their design working pressures. The four most common classes of working pressures are 100, 150, 200, and 250 lb/in^2. Other parameters for identifying pipe strength are the bursting strength rating, which identifies the strength against internal forces caused by hydrostatic and hydrodynamic pressures such as pressure surges and water hammer, and the crushing strength, which deals with the external forces related to soil, vehicles, and impact loadings.

Typical standard pipe diameters used in distribution systems range from 6 inches through 20 inches in 2-inch increments. Then, beginning with 24 inches, diameters increase in 6-inch increments. For residential plumbing pipe, diameters begin at ½ inch and increase in ¼-inch increments. Since manufacturers and pipe material dictate the available pipe diameters, diameters other than those mentioned here might be available. The supplier should be contacted to verify the pipe diameters that are available and whether they are in stock.

Although ductile iron pipe is the most popular cast-iron pipe, other common pipe materials for water distribution lines include gray cast iron, steel, plastic, and polyethylene. Note that this same type of piping is used for water service lines larger than 2 inches. Copper or plastic tubing is commonly used for service lines smaller than 2 inches.

Plastic Pipe. Distribution lines for small water systems are normally constructed of PVC pipe because of its ease of construction and economy. Plastic pipe used for drinking applications should be certified by an acceptable testing laboratory (such as the National Sanitation Foundation) as nontoxic and not producing objectionable taste. Standards for PVC pipe (up to 12 inches) for underground service are ASTM D1785, ASTM D2241, and ANSI/AWWA C900. ASTM D1785 pipe is available in Schedule 40, 80, and 120 thickness. ASTM D2241 pipe has pressure ratings up to 250 lb/in^2, while AWWA C900 pipe is available in pressure ratings up to 200 lb/in^2. AWWA C900 pipe is manufactured to ductile-iron pipe outside dimensions, and both ASTM D2241 and ASTM D1785 are manufactured to steel pipe outside dimensions. Joining these pipes requires a transition gasket used with standard ductile-iron fittings.

Ductile-Iron Pipe (DIP). DIP is standardized in ANSI/AWWA C150/A21.50-96, and is available in sizes from 3 to 64 inches in various pressure classes. Push-on or mechanical joints are used on DIP for underground service. For water service, DIP is normally lined with cement-mortar. Soft water will leach the cement-mortar lining if not properly conditioned. Depending on soil conditions, DIP is often installed in polyethylene wrapping to aid in corrosion protection.

Steel Pipe. Steel pipe is normally used for large-diameter installations beyond the sizes of which DIP is available and is not often used for small water systems. However, it is available in various sizes and thickness classes. It is covered under AWWA C200 standard. Note that buried steel pipe is more susceptible to corrosion than DIP and must be adequately protected. It is usually installed underground with a tape-wrap coating as a minimum, and the pipe interior is normally specified to be provided with a cement mortar lining for drinking water service.

Polyethylene Pipe (PE). PE is becoming popular for small water service, mainly because it is economical to purchase and install. PE is available in sizes larger than 4 inches and the standard for PE is AWWA C906.

High-Density Polyethylene Pipe (HDPE). HDPE is popular in directional drilling applications. The pipe is fused together with a machine that provides a jointless connection. The pipe is flexible and can be installed around a radius without the use of joints. The pipe is covered under ASTM D1248, ASTM 3350, AWWA C901, AWWA C906, and NSF Standards 14 and 61. It can be used in a direct burial application or with a directional drilling machine that bores a hole through the ground for a specified distance and then pulls the pipe back into the void. The directional drilling application can be used to install the pipe in numerous situations such as under environmentally sensitive areas, railroad rights-of-way, and road rights-of-way.

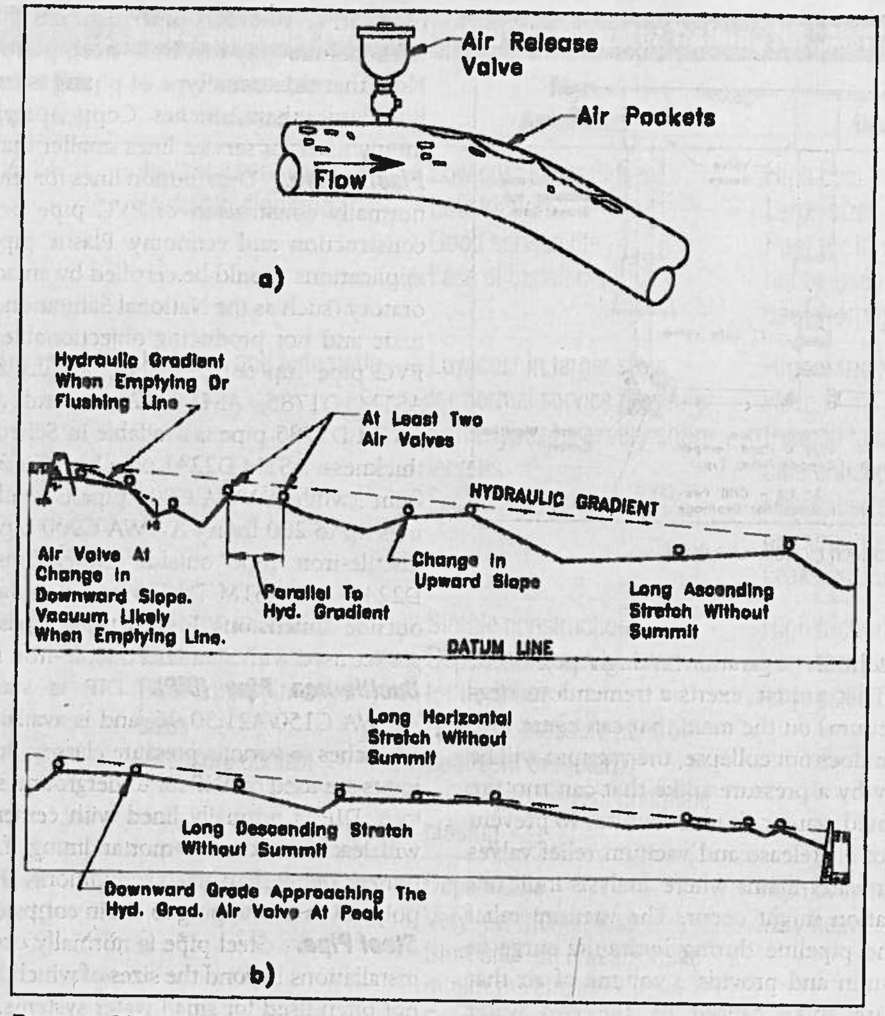


FIGURE 25.20 (*a*) Typical bubble behavior; (*b*) suggested air-release valve locations. (Reprinted with permission of GA Industries, Inc., Cranberry Township, PA)

Additionally, it is important to note that ductile-iron pipe (DIP) and prestressed concrete cylinder pipe (PCCP) are typically used for larger-diameter waterlines such as transmission lines from treatment facilities to the distribution network.

Joints. Distribution pipes are connected by either of the several different types of joints shown in Figure 25.21.

The *push-on joint* is a compression joint that creates a pressure-tight seal when a gasket inside the bell end is compressed by the plain end of the pipe. This type of joint allows for fast and easy installation and is low in cost, which probably accounts for its popularity. However, this type of joint does not restrain the individual lengths of piping from separating under pressure, so concrete thrust blocks are cast at elbows and other turns to restrain the pipe. Soil friction on the pipe also acts to prevent joint separation. The push-on joint is not suitable for aboveground use in pressure piping.

For the *mechanical joint*, the pipes fit together as a plain end inside a bell-shaped end. A pressure-tight seal is created by a gasket tightened between the flange on the bell end and the gland, a flangelike collar that slips over the plain end. The flange and gland are held together with bolts. The mechanical joint is considered a semirestrained joint because friction in the highly compressed gasket provides some restraint against pipe separation under pressure. Where joint restraint is not provided by thrust blocking, as for the push-on joint, a restraining collar with locking bolts is used to hold the joint together. The mechanical joint is easy to assemble and is intermediate in cost between the push-on joint and the more costly ball joint, flanged joint, or threaded joint.

The *ball joint* is used for crossing waterways, as a restrained joint, and where free-turning deflections are needed. The gasket is inserted into the bell and compressed by the entering ball. The joint is secured by locking the bayonet-type retainer over the lugs on the bell. A retainer lock inserted between the lugs prevents the retainer from rotating.

The *flanged joint* is used for the larger sizes of aboveground distribution piping, such as in pump stations. This joint is seldom used for underground distribution piping because of its cost and difficulty of installation. The flanged joint has a flat-faced or flat-faced with gasket groove bolt collar or flange on each end of the pipe. A suitable gasket is placed between the flange faces and compressed by the bolts to form a pressure-tight joint. The flange joint is the strongest of the joints. The chief disadvantage of the flange

FIGURE 25.21 Joint connections for cast-iron pipe. (Reprinted with permission of McWane Cast Iron Pipe Company, Birmingham, AL)

joint is that usually some type of length-compensating joint must be included in the pipe run because, as the flange bolts are drawn up, the overall length of the pipe run is shortened by the compression of the gaskets.

The *threaded or screwed joint* is formed by cutting threads into the end of the pipe on a tapered diameter, in other words, pipe threads. When the tapered threads are screwed together, they create an interference fit, which makes a pressure tight joint with the aid of a mastic pipe sealing compound on the threads. The threaded joint is not commonly used for distribution piping, since it is difficult to make in larger pipe sizes, and, due to the loss of metal in the pipe wall at the threads, it is weaker than the flanged joint.

The ball, flanged, and threaded joints are considered restrained joints because they will hold the individual lengths of pipe together under system operating pressure and are, therefore, suitable for aboveground use.

Fittings. Fittings are used to connect same- or different-size pipes, change the flow direction, split the flow into several directions, and stop the flow. Examples of fittings commonly used in the distribution network are crosses, tees, wyes, and bends. Standard bends are 90° or full bends, 45° or ½° bends, 22½° or ¼° bends, and 11¼° or ⅛° bends. Fittings are readily available in standard sizes. Nonstandard sizes are special-order items from a manufacturer. Because the special-order items cost significantly more and may delay construction while on order, their use and necessity in design should be avoided except in the most extreme cases. Typically, the AWWA, ANSI, and ASTM designate acceptable standards and specifications for various pipes and fixtures for supply water systems. Examples of various fittings are shown in Figure 25.22.

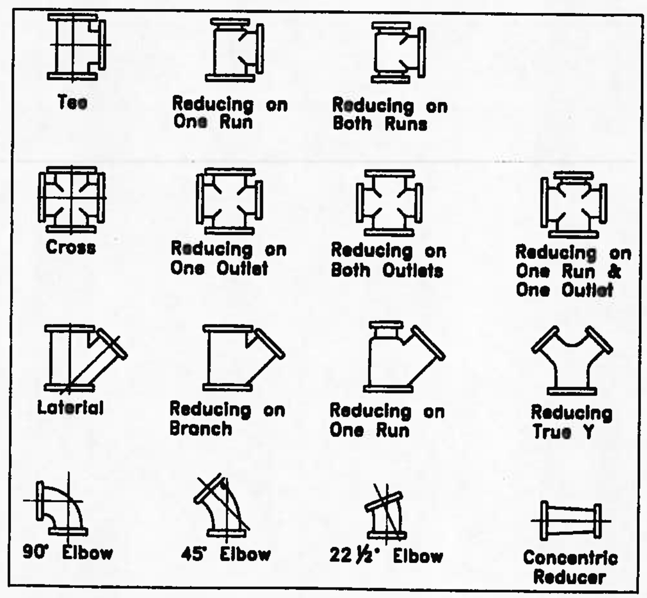


FIGURE 25.22 Various types of fittings.

Fire Hydrants

A fire hydrant consists of the aboveground barrel that extends below grade to the water supply line. The supply line connecting the barrel to the water main is a minimum 6-inch-diameter pipe. Typically, a gate valve located in an underground valve box allows for shutoff and repair. Maximum recommended friction losses are 2.5 lb/in^2 in the hydrant and 5 lb/in^2 in the water supply line, from the main to the hydrant, for flows of 600 gpm. Hydrants have various nozzle arrangements, but the most common hydrant has two 2.5-inch hose connections and one 4.5-inch pumper connection. The bend directly below the barrel connects the barrel to the supply line. A thrust block prevents movement from lateral forces and a concrete block underneath the bend prevents excessive settling. After use of the hydrant, the barrel is drained by way of a drain near the base of the barrel. A gravel bed surrounds the thrust block or bend to allow for draining of the barrel.

Meters

Meters are commonly used for measuring water for setting cost, collecting payments from customers, and determining fair distribution of water delivery costs. Other uses for meters include measuring flow to and from treatment plants and reservoirs, blending water and chemicals, and measuring water sold to other jurisdictions.

Meters used to measure usage for residential dwellings and commercial use are located in areas easily accessible to the water utility company. They should not interfere with public safety nor act as a hazardous obstacle. Most meters are located below ground surface levels for safety and to reduce damage due to weather and tampering. Occasionally, meters will be located inside the building with a remote recorder mounted on the outside.

Meters are available in the same sizes as pipe diameters. Usually the meter is sized one diameter size less than the pipe for reasons associated with accuracy, head loss, and cost. Unlike pipe, where the cost differential between successive pipe sizes may be on the order of 10 percent to 25 percent, the cost differential between successive meter sizes can be significantly more. Downsizing the meter also downsizes succeeding appurtenances and fixtures, thereby reducing costs. The increase in accuracy in using the smaller-size meter is counterbalanced by an increase in the head loss. Usually the cost savings and the accuracy offset the drawback incurred from the increased head loss.

Thrust Restraint

Two basic forces associated with flowing water under pressure are the hydrostatic forces and hydrodynamic forces. Simply explained, hydrostatic forces are due to the pressure exerted by a depth of fluid, as shown in Figure 25.23. This pressure force acts perpendicular to a surface. Thus, for a pipe with full flow, the pressure acts radially outward and also along the longitudinal axis of the pipe.

Consider a pipe connected to a tank and plugged at the other end, as shown in Figure 25.23. From basic hydrostatics, the pressure on the plugged end of the pipe is proportional to the depth of water H, as shown by the hydrostatic pressure distribution diagram. The pressure, p, is equal to γH, where γ is the specific weight of the fluid. The magnitude of the force acting at the end of the pipe is equal to the product of the pressure and the cross-sectional area of the pipe $F_p = \gamma HA$, where A is the cross-sectional area of the pipe.

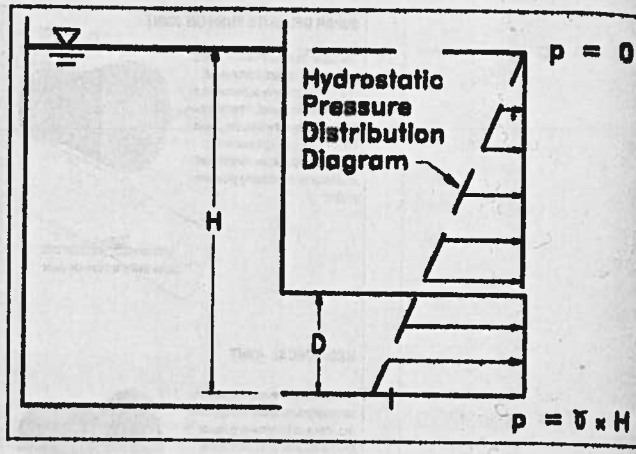


FIGURE 25.23 Hydrostatic pressure in a pipe.

Hydrodynamic forces are the result of changes in momentum of the moving fluid. Any change in direction or magnitude of flow velocity results in a change in momentum of fluid. Fittings such as bends, tees, and wyes change the direction of flow; nozzles, valves, and reducers change the cross-sectional area of the flow path. Each of these fixtures has force acting on it due to hydrostatics and hydrodynamics.

The impulse-momentum equation (assuming incompressible flow) determines the resultant hydrodynamic force of the water on the fitting. The impulse-momentum equation, in vector component form, is:

$$\sum F_x = \rho Q(\Delta V_x) \qquad (25.5)$$

$$\sum F_y = \rho Q(\Delta V_y)$$

where Q is the discharge, ρ is the density, and V_x and V_y represent the x and y components of velocity. Consider the horizontal bend of Figure 25.24*a*. The force of the water on the bend is the result of the pressure forces pA acting on the water within the bend, as illustrated in Figure 25.24*b*. To counter these forces, an equal and opposite force is produced by the longitudinal stresses in the pipe wall (Figure 25.24). Since common pipe joints cannot resist these longitudinal forces, the developed forces must be reduced or eliminated by additional anchors, either in the form of thrust blocks or with supplemental clamps and collars at the joints.

As an example of the magnitude of such forces, consider a typical water distribution pipe system. Typically, the operating pressure ranges from 40 to 150 lb/in^2 with velocities approximately 3 to 10 fps. Within these ranges, hydrostatic forces are far greater than hydrodynamic forces and, for the purposes of design, can be ignored in most cases. To illustrate this point, consider a 12-inch diameter, 45° horizontal bend. Assume line pressure of 50 lb/in^2 and flow velocity of

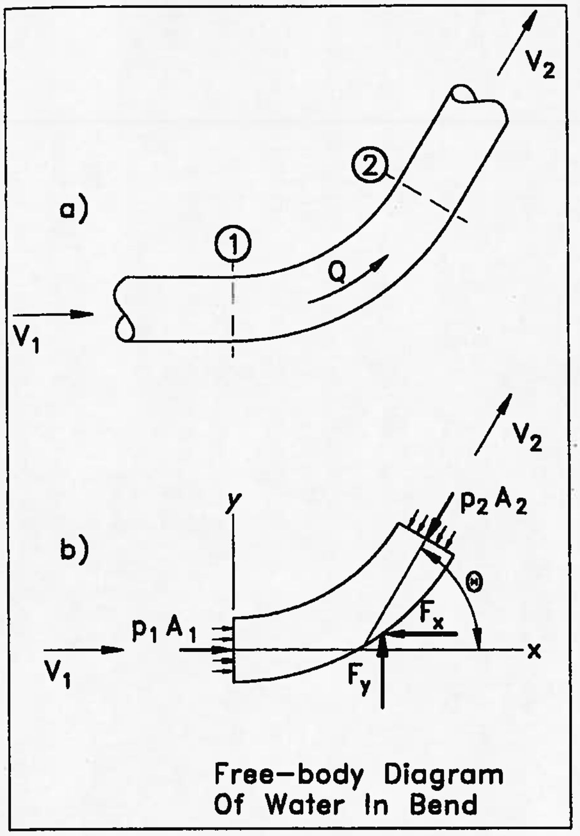


FIGURE 25.24 Forces for flow through a horizontal bend.

5 fps. The right side of the momentum equation (Equation 25.5) is the force due to the change in momentum of the fluid. The left side represents all forces acting on the fluid such as gravity, pressure forces, and shear forces. For the conditions given, let F_x and F_y represent the forces of the bend on the fluid. Expanding Equation 25.5 gives:

$$p_1A_1 - F_x - p_2A_2 \cos 45° = \rho Q(V_2 \cos 45° - V_1) \quad (25.6)$$

$$F_y - p_2A_2 \sin 45° = \rho Q V_2 \sin 45°$$

Rearranging terms and substituting the appropriate values,

$$\left(50 \text{ psi} * 144 \frac{\text{in}^2}{\text{psi}}\right)(0.79 \text{ ft}^2) \quad (25.7)$$

$$-\left(50 \text{ psi} * 144 \frac{\text{in}^2}{\text{psi}}\right)(0.79 \text{ ft}^2) \cos 45°$$

$$-1.94 \text{ slugs} * 3.9 \frac{\text{ft}^3}{\text{sec}}$$

$$\times\left(5 \frac{\text{ft}}{\text{sec}} \cos 45° - 5 \frac{\text{ft}}{\text{sec}}\right) = F_x$$

or

$$1666 \text{ lbs} - (-11 \text{ lbs}) = 1677 \text{ lbs} = F_x \quad (25.8)$$

Similarly the y-component of the force is:

$$\left(50 \frac{\text{lbs}}{\text{in}^2} * 144 \frac{\text{in}^2}{\text{ft}^2} * 0.79 \text{ ft}^2 \sin 45°\right) \quad (25.9)$$

$$+ (19.4 \text{ slugs}) * \left(3.9 \frac{\text{ft}^3}{\text{sec}} \sin 45°\right) = F_y$$

or

$$5688 \text{ lbs} + 5.4 \text{ lbs} = 5693 \text{ lbs} = F_y \quad (25.10)$$

The preceding computations show that, for conditions typical of water distribution systems, the components of the momentum force (i.e., 11 lb and 5.4 lb) are substantially less than the x and y components of the pressure forces, namely 1666 lb and 5688 lb, respectively.

The types of joints commonly used to connect ductile-iron pipes are push-on or mechanical joints. Although leakproof and quite effective for normal radial stresses, these joints are not capable of resisting the hydrostatic and hydrodynamic forces at the components in the longitudinal direction. Added measures are necessary to reinforce longitudinal separation at the joints. Thrust blocks are used to transfer the fluid pressure forces to undisturbed soil. Other measures include collars and clamps around the joints to provide additional restraint.

The design concept of thrust blocks is similar to foundation design. The intent is to transfer internal hydrostatic forces to the undisturbed soil mass through a mass of concrete. For a given hydraulic design, the size of the thrust block depends on the bearing capacity of the soil and the hydrostatic forces within the waterline component. Figure 25.25 shows the magnitude and line of action of hydrostatic forces for various types of waterline components. The location of the thrust block depends on the type of pipe fittings. Parameters considered in the design of thrust blocks include pipe size, hydrostatic pressures, type of component, and soil properties.

The Ductile Iron Pipe Research Association (DIPRA) recommends the schematic shown in Figure 25.26 for thrust block design.

Bearing surface of the block should be placed against undisturbed soil. Alternatively, when this is not possible, the fill soil between the block and the undisturbed earth must be compacted to minimum 90 percent Standard Proctor density. The required bearing block area is

$$A_b = hb = \frac{TS_f}{S_B} \quad (25.11)$$

where:

A_b = the bearing surface area of the block
h = height of the block
b = width of the block
T = the resultant hydrostatic thrust force on the component. The hydrostatic forces (PA terms) corresponding to T are found from Figure 25.25.
S_f = factor of safety (usually 1.5 for thrust block design)
S_B = bearing strength of the soil (psf)

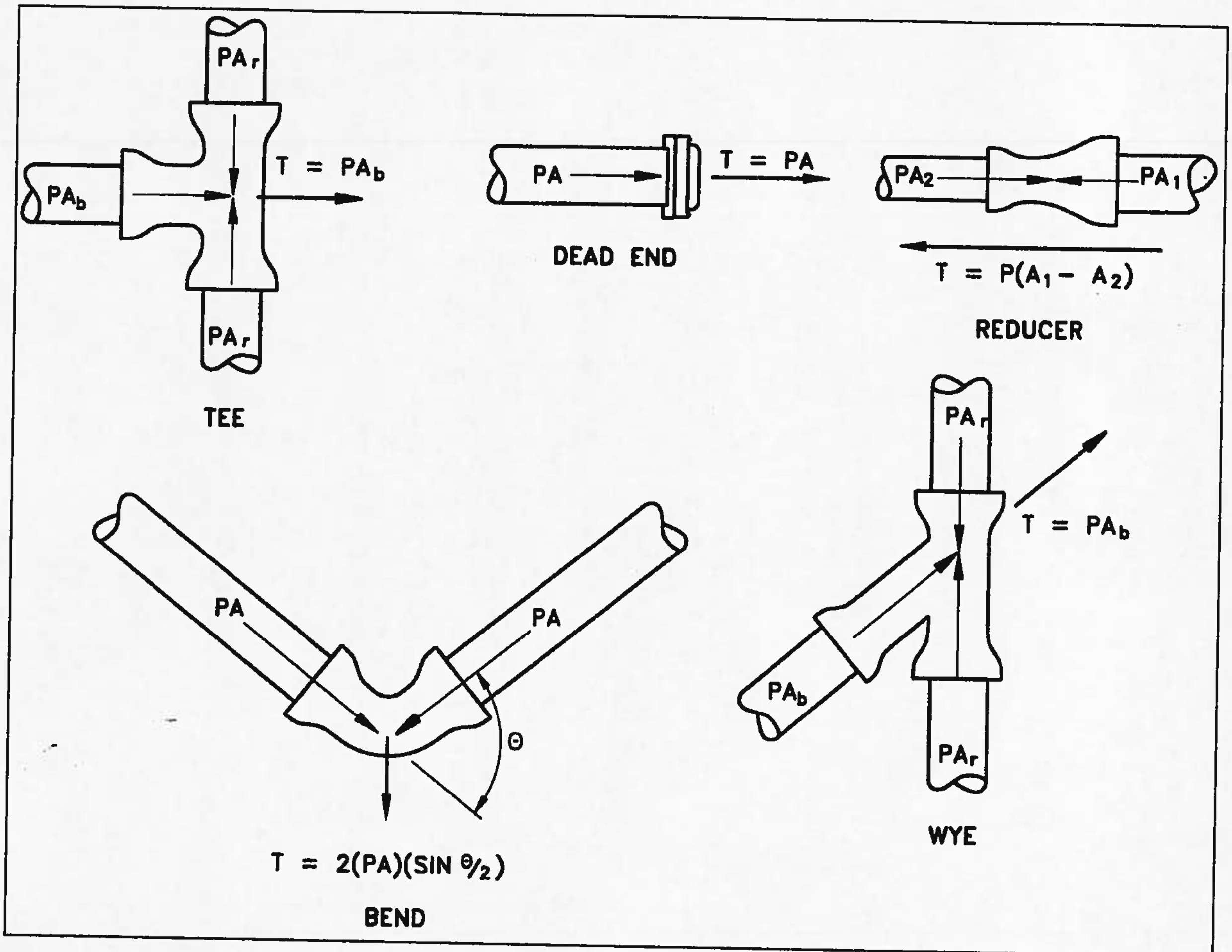


FIGURE 25.25 Thrust forces and line of action for selected fittings. (Reprinted with permission of Ductile Iron Pipe Research Association, Birmingham, AL)

The block height h should be equal to or less than one-half the total depth to the bottom of the block H_T, but not less than the pipe diameter D.

Block height h should be chosen such that the calculated block width b varies between one and two times the height.

Table 25.10 provides bearing strengths for generalized soil that can be used whenever actual bearing strengths are not available. The engineer is responsible for proper selection of the bearing strength based on proper soil identification. The DIPRA suggests that, in lieu of the values for soil-bearing strength shown in Table 25.10, a designer may choose to use calculated Rankine passive pressure or other determination of soil-bearing strength based on actual soil properties.

On horizontal bends, the placement of the thrust block is against the outer edge of the bend. On vertical down bends, similar placement would put the thrust block on top of the pipe. Besides the danger of crushing the pipe, the fill dirt on top of the block would provide marginal bearing support. Instead, the vertical down bend is stabilized with a gravity thrust block attached to the underside of the bend (see Figure 25.21b). The vertical component (T_y = PA sin θ) of the thrust force is countered by the weight of the block itself. The required volume of the block is

$$V_G = \frac{S_f PA\,(\sin\theta)}{W_M} \tag{25.12}$$

where V_G = the volume of the block and W_M = density of the block material. The horizontal component of the thrust force $T_X = PA\,(1 - \cos\Theta)$ is transferred to the soil by the right side–bearing area of the block.

In situations where thrust blocks cannot be used, another common method for thrust resistance is the use of restrained joints. A restrained joint is a specially designed joint that provides longitudinal restraint. The thrust force at the component is transferred to the surrounding soil through frictional resistance and bearing from a predetermined length of pipe.

Other types of thrust resistance include steel straps attached to boulders and bedrock (not very common), and tie rods connecting collars on either side of a joint.

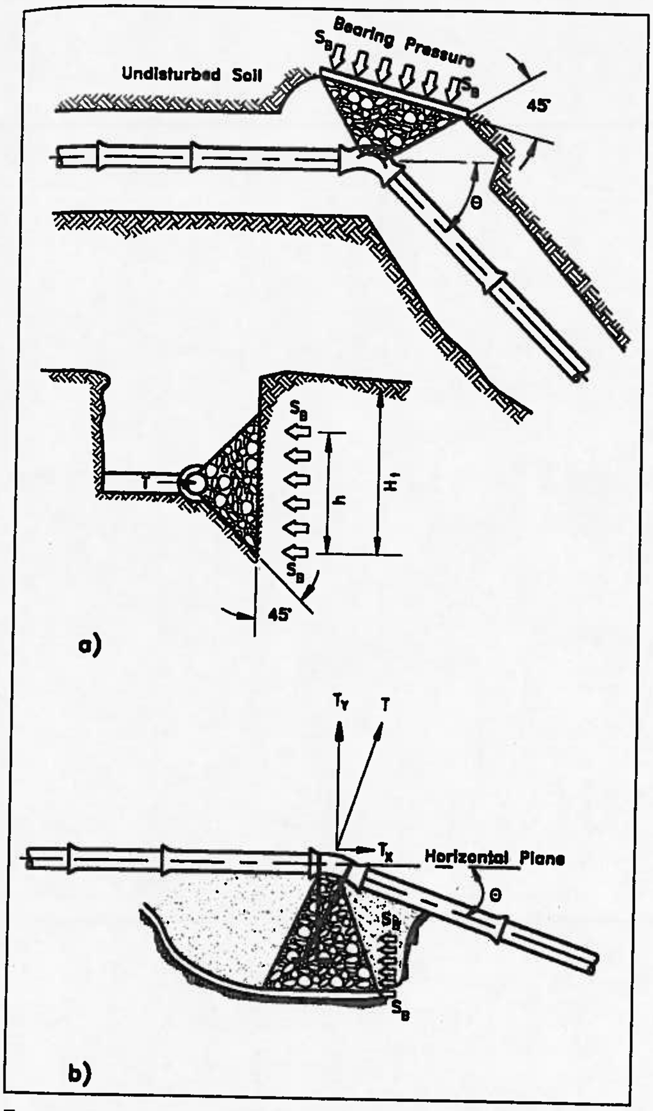


FIGURE 25.26 Schematic for (*a*) bearing block; (*b*) gravity block. (Reprinted with permission of Ductile Iron Pipe Research Association, Birmingham, AL)

TABLE 25.10 Bearing Strength of Generalized Soils

SOIL	STRENGTH (S_B)* (LB/FT^2)
Muck	0
Soft clay	1000
Silt	1500
Sandy silt	3000
Sand	4000
Sandy clay	6000
Hard clay	9000

*Although the above bearing strength values have been used successfully in the design of thrust blocks and are considered to be conservative, their accuracy is totally dependent on accurate soil identification and evaluation. The ultimate responsibility for selecting the proper bearing strength of a particular soil type must rest with the design engineer.
(Reprinted with permission of Ductile Iron Pipe Research Association, Birmingham, Alabama.)

SYSTEM LAYOUT CONSIDERATIONS

Pipe Network

In residential areas, distribution lines typically follow the street network and are usually located within the public right-of-way. Some localities require the location of the waterline to be a fixed distance from the centerline, curb line, or right-of-way line and on the same side of the street for uniformity. For instance, in some parts of the United States, the waterline is laid on the north and east side of the centerline. Whether the water main is located under the pavement or behind the curb line is a matter of preference set by the local standards.

Although the grid layout is preferable to the branching layout, dead-end lines cannot always be avoided. Therefore, most distribution systems are a combination of loops and branches with the branch lines kept to a minimum. Many branch lines are temporary and are built for the convenience of future connections needed when new developments are added and streets are extended. Cul-de-sac streets, common in residential developments, make looping of distribution lines difficult since easements are required to extend the waterline through private property. The water supply company, in its desire to minimize dead-end waterlines, may require the developer to provide easements through the project for interconnecting the waterline. The easements should preferably run through outparcels and common land rather than individual lots. However, if the waterline must pass through a lot, the preferred location is along lot lines, with the lot lines coincident with the edge of the easement. Note that this method of locating the easement must comply with other subdivision and zoning constraints; for example, the waterline must meet the criteria for minimum distance to dwellings and other structures.

Location of water mains in commercial areas and high-density residential areas (e.g., town houses and condominiums) is dictated, in part, by other utilities, their structures, location of fire prevention and control devices for the internal parts of the building (e.g., fire hose connections, sprinkler systems), location of fire hydrants, and type of metering system. Generally, these sites contain a more complex maze of underground utilities within the travelways, and this complexity presents obstacles in the layout. To illustrate, the travelways are narrower than public street rights-of-way; the sites contain higher impervious areas, which translates to more storm sewer pipe and inlets; and the higher density usually means more sanitary connections. Since waterlines are normally nongravity dependent, the waterline can be adapted to weave over, under, and around the other gravity-dependent utilities.

Although water mains are usually no smaller than 6 inches in diameter, 4-inch lines may be acceptable for special situations such as stub lines serving fire hydrants and lines that serve only a few residential units. Typical pipe sizes for water mains are 4-, 6-, 8-, 12-, 16-, 18-, and 24-inch diameters. The size varies according to the specifications of the jurisdiction and the availability of pipe materials. Since the velocity increases as the diameter decreases, smaller-diameter pipes result in higher head losses for the same discharge. Velocities of 3 fps to 6 fps at normal working conditions are preferred, although higher velocities in short lengths of pipe for short periods are tolerable. Note sustained high-discharge velocities scour the interior of pipes.

For health safety reasons the horizontal distance between a waterline and a sanitary sewer line or manhole should be a minimum of 10 feet. Pipes and pipe joints may develop leaks due to deterioration, improper construction, and excessive pressures. Contamination of the water supply from the sanitary wastewater system presents a potential for creating a major health problem as well as a major cost in locating the point of contamination and repairing it. When the water and sanitary sewer line are essentially parallel and the horizontal distance is less than 10 feet, special provisions are made to ensure no contamination. When the water main is in the vicinity of a sanitary sewer, the water main should be above the sanitary line with a minimum distance of 18 inches between the invert of the waterline and the crown of the sanitary pipe. In addition, pressure pipes should be used for the sanitary sewer with watertight joints to provide additional protection from contamination. Sanitary sewer manholes in the vicinity of water mains must also be of watertight construction. As an alternative, design standards may require concrete encasement of the waterline when it falls in close proximity to sanitary lines. In the extreme cases, when the sanitary line must be above the water main, the sanitary sewer should have adequate support (e.g., concrete cradle) to prevent excessive deflection, settling on, and breaking the waterline. If the sanitary sewer crosses the water main, not only are the upgraded materials used, but the water main section of pipe should be centered at the point of crossing to put the water main's pipe joints as far from the sanitary pipe as possible.

Waterlines are set at a depth below the frost line. The engineer profiles the waterline after the horizontal layout is complete to show how the waterline weaves over and under other utilities in the development. Minimum clearance between water main and storm sewer is approximately 1 foot. When the water main passes under a storm sewer pipe, additional structural support to the storm pipe should be considered to prevent crushing of the water main. When water mains pass over utilities at less than minimum depth, consideration for freezing is necessary.

Waterlines can also be installed using directional drilling machines that facilitate "trenchless" installation. This type of installation is beneficial in several different scenarios. For example, when the waterline must be extended across waterways such as streams to serve another section of the distribution system, a directional drilling machine allows the line to be installed without impacting the stream or other environmentally sensitive areas such as wetlands. A directional drilling machine can also be used to extend a water distribution line across a congested roadway in a manner that avoids significant disturbance to everyday vehicular traffic that occurs with typical open trench excavation.

Valve Locations

Isolation valves are placed on the line for shutting down a section of the system for repairs and maintenance. Recommended valve spacing ranges from 500- to 1200-foot intervals. Since the purpose of isolation valves is to shut down a section for repairs, the objective in placing the valves is to balance the economics of using as few valves as possible with the inconvenience of the smallest number of customers when a section is shut down.

Most water main systems follow a community's public road. In moderate- to high-density (i.e., more than three du/ac) residential and commercial areas, the frequency of street intersections is typically less than 1000 feet and in many developments 500 feet and less. At these intersections, tees and crosses divide the waterline into other feeder lines. Included with these tees and crosses are valves that allow for various ways to segment small service areas. Generally, valves are placed on two sides of a tee and three sides of a cross. At the tees, one valve is placed on the minor line. Likewise, with crosses, valves are placed at each side of the minor connection (see Figure 25.27*a*). The location of the remaining valves for the cross and tee depends on the location of the valves at the next junction upstream and downstream from the point under consideration. Referring to Figure 25.27*a*, recognize that two valves at each tee can accomplish the same results as three valves. The water main segments of B-D, D-F, and C-E can be isolated without the need for a valve at P.

The exception to the rule occurs on tees where the stub side connects to either a reasonably large commercial building or a fire hydrant. In these cases, valves are placed on all three sides of the tee, as shown in Figure 25.27*b*. The two valves along the distribution line allow flows to the building or hydrant, although part of the nearby distribution line is shut down. The third valve on the stub side allows the building or hydrant to be isolated without disrupting flows along the water main.

Valves are also commonly located where the distribution line has been downsized. As the number of service connections decreases on the line—for instance, at a terminating street—the distribution line's diameter can be reduced because of the reduced demand. A reducer is a component that allows for the connection of a large pipe to a smaller pipe. As an economic consideration, valves are typically placed after the reducer on the side with the smaller diameter.

The number and location of valves in a development project are a function of the density. In the low- and moderate-

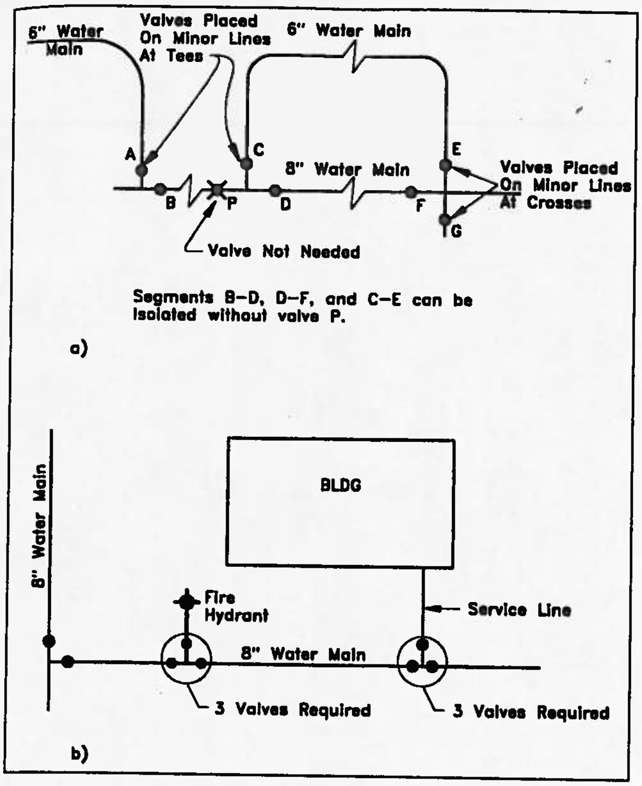


FIGURE 25.27 Valve placement.

density areas, the placement of valves every ±500 feet is inherent due to the street layout. In areas of higher densities (e.g., 10 du/ac or apartment complexes), the placement of valves at 500-foot intervals would mean inconveniencing numerous households when a section is shut down. In such cases the location of valves depends on the number of service lines that can comfortably be shut down. A reasonable range is 15 to 25 service lines between isolation valves. To do this requires intermediate valves placed on the line to limit the number of households to a reasonable level.

Valves are installed in cast-iron or PVC valve boxes with a sleeve placed over the valve that prevents backfilling and allows access from the ground level. The valve box and its lid present a nuisance when located in lawns. When possible, the valve should be placed in accessible areas, but not where the valve box would be covered with soil and overgrown vegetation (e.g., wooded areas). Although locating valves in such areas should be avoided, this is not always possible. In such instances, the valve is enclosed within a vault. The vault may be concrete or another permanent enclosure that protrudes above the ground surface at least 6 inches. A vent stack may also be connected to the vault. The protruding vault and vent stack allow the valve to be located much more easily.

Placement of valves for isolation purposes can be very subjective. Occasionally, the water supply company dictates the location of valves and other controlling components. The water supply company has immediate access to information not readily available to the project engineer. The project engineer can then only make the best guess using the foregoing concepts and personal experience for showing waterline and component location on the plan sheets.

Fire Hydrant Location

Although aesthetic value of a development has a high priority, there is very little compromise with safety and fire protection. Fire hydrants need to be conspicuously located and highly accessible. Firefighters in the throes of fighting a blaze need to locate a hydrant by quick glances in obvious directions. Typically, fire hydrants are located 2 to 3 feet behind the curb or edge of pavement. The hydrant is oriented so that the pumper connection faces the street or travelway and is set far enough behind the face of the curb to avoid damage from vehicles. Hydrants are placed to be accessible for hose connections and the pumper truck and, therefore, should not be placed high on embankments, in depressions or ditches, or near trees, poles, or walls, or where there is insufficient clearance for hose lines (Figure 25.28).

In high-density residential areas (e.g., town house and condominium sites), fire hydrants are frequently placed on parking islands near buildings. Consideration is given to the minimum distance between the fire hydrant and the closest building it is intended to serve. Local codes prescribe the safe distance for which the firefighters can access a hydrant without sustaining injury from the burning building (approximately 50 to 75 feet). Therefore, several hydrants may be needed throughout high-density and commercial areas to protect the building. Another consideration when locating hydrants is the placement of fire hoses. For example, Building Officials Code Administrators (BOCA) fire codes do not recommend placement of unprotected fire hoses across travelways. Placement of fire hydrants on both sides of the street to protect the building opposite the fire hydrant will not suffice because of the BOCA restrictions on placement of fire hoses.

On commercial sites, hydrant spacing is governed by the estimated coverage as determined by fire flow requirements, the course of the unrolled fire hose, and the location of fire hose connections on the building (in addition to any other local requirements). Since fire hoses are unrolled along the travelways and grassy areas, the distance between hydrants is not measured along a straight line connecting the hydrants. Subsequently, for streets and travelways with many short-radius curves and obstructions (e.g., fences and walls) more fire hydrants will be needed. Commercial buildings may be required to have an exterior fire hose manifold (e.g., standpipe, siamese connections). The location of this manifold on the building must be near a fire hydrant, probably within 100 feet.

The International Standards Organization (ISO) recommends hydrant distribution according to the coverage presented in Table 25.11. Rule-of-thumb hydrant spacing ranges from 300-foot to 1000-foot intervals. Actual spacing considers the type and size of the dwelling, land use, risk

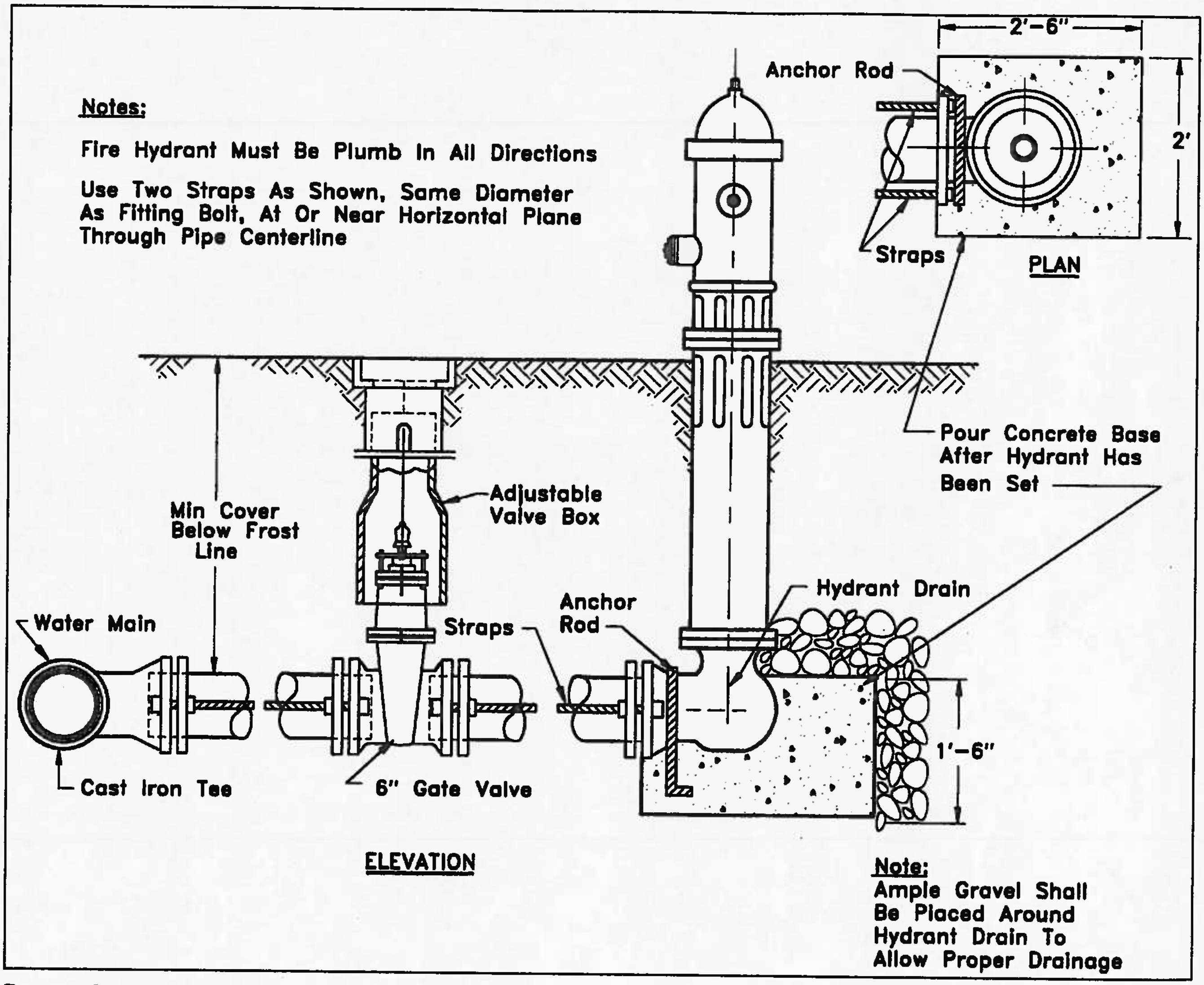


FIGURE 25.28 Typical hydrant settings.

(e.g., a high-value district in terms of property cost and public safety), and local controlling ordinances.

To the extent possible, placement of hydrants should not detract from the streetscape. The fire hydrant needs to be visible and accessible to the firefighters and yet not be obtrusive. For example, in single-family detached subdivisions a hydrant placed behind the curb at the side property line would serve both purposes. A fire hydrant in the middle of the front property line may be unattractive, especially if the house sits close to the front property line.

In placing fire hydrants in a residential subdivision, a systematic method would be to start on the cul-de-sac streets. Determine the fire flow coverage or the maximum distance from fire hydrant to the farthest buildings as prescribed by local codes. Using this distance, place the fire hydrant as far away from the houses within the cul-de-sac as this distance allows. Proceed along the cul-de-sac street toward the through street, placing fire hydrants at the appropriate distances and considering hydrant spacing and prescribed maximum distances to dwellings. After all cul-de-sac streets have the appropriate number of hydrants, the hydrants along through streets are located to provide fire protection not covered by the cul-de-sac fire hydrants or any existing fire hydrants.

A map showing hydrant and valve locations, such as the one in Figure 25.29, might be available from the water utility company or Department of Public Works. A copy of a map such as this for the project area should be obtained prior to beginning the design. This provides information for assessing water availability and hydrant location during the site analysis phase (see Chapter 5).

Pipe Curvature

Gradual change of direction with pipe is accomplished by using bends or simulating curvature through succeeding deflection of pipe joints. The approximate radius of curvature depends on the type of pipe joint, length of pipe, and pipe diameter. Figure 25.30 shows joint deflections for mechanical and push-on pipe joints.

TABLE 25.11 Standard Hydrant Distribution

Fire Flow Required (gpm)	Average Area per Hydrant (ft²)
1000 or less	160,000
1500	150,000
2000	140,000
2500	130,000
3000	120,000
3500	110,000
4000	100,000
4500	95,000
5000	90,000
5500	85,000
6000	80,000
6500	75,000
7000	70,000
7500	65,000
8000	60,000
8500	57,500
9000	55,000
10,000	50,000
11,000	45,000
12,000	40,000

Connections

During construction, building service connections are extended from the water main to the right-of-way line. Whereas the water main may be a 6- to 12-inch diameter pipe, the service connections for residential dwellings are typically ¾- to 1-inch diameter. A corporation stop assembly (see Figure 25.31) connects the service line to the water main at a point above the horizontal center of the water main. This location does not draw off any sediment from the bottom of the water main, is an easily accessible location for repairs, and allows trapped air to escape from the water main and be expelled through water fixtures. However, the specific location of the connection depends on the policy of the utility company.

FIGURE 25.29 Municipal map showing waterline, valve, and hydrant location in residential area.

The service connection ends at the meter box or curb box. The meter box contains the shutoff service valve, a yoke, and the water meter. The yoke is a special fitting to hold the service pipe stubs at the proper alignment for connecting the meter. The service box contains only the service valve. In some areas, the building contractor is responsible for installation of the meter and the service line leading to the house and, in others, the water utility company manages the installation of the water main and service lines. The stubbed service lines are installed during construction of the water main. This allows the house connections to be made, as the houses are built without cutting into new pavement.

Connecting to an Existing Waterline

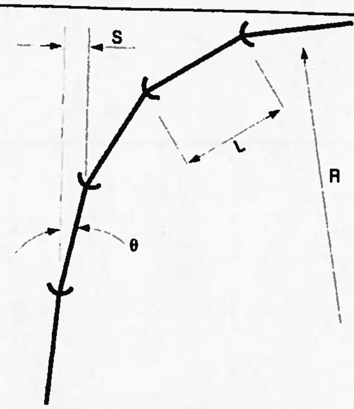
Special types of construction methods are used to connect the proposed water main to an existing water main when the existing service cannot be shut down—even for the short time needed to make the connection. The type of method selected depends on the specific site conditions and the locality.

One common method uses a tapping sleeve and valve. As illustrated in Figure 25.32, the tapping sleeve and valve consists of two half sections of pipe that are strapped over the existing line. Special equipment designed to fit over a valve cuts a hole in the existing water main through the opened valve fixture. After the hole is cut, the equipment is withdrawn and the valve is closed. The water main is extended

Pipeline Curve Geometry

θ = deflection angle
S = joint deflection offset
L = laying length
R = radius of curve

$$R = \frac{L}{2\tan\frac{\theta}{2}}$$



Maximum Joint Deflection Full-Length Pipe—Mechanical-Joint Pipe

Nominal Pipe Size *in.*	Deflection Angle–θ *deg*	Maximum Offset—S *in. (m)* L = 18 ft (5.5 m)		Maximum Offset—S *in. (m)* L = 20 ft (6.1 m)		Approx. Radius of Curve—R Produced by Succession of Joints *ft. (m)* L = 18 ft (5.5 m)		Approx. Radius of Curve—R Produced by Succession of Joints *ft. (m)* L = 20 ft (6.1 m)	
3	8–18	31	(0.79)	35	(0.89)	125	(38)	140	(43)
4	8–18	31	(0.79)	35	(0.89)	125	(38)	140	(43)
6	7–07	27	(0.69)	30	(0.76)	145	(44)	160	(49)
8	5–21	20	(0.51)	22	(0.56)	195	(59)	220	(67)
10	5–21	20	(0.51)	22	(0.56)	195	(59)	220	(67)
12	5–21	20	(0.51)	22	(0.56)	195	(59)	220	(67)
14	3–35	13.5	(0.34)	15	(0.38)	285	(87)	320	(98)
16	3–35	13.5	(0.34)	15	(0.38)	285	(87)	320	(98)
18	3–00	11	(0.28)	12	(0.30)	340	(104)	380	(116)
20	3–00	11	(0.28)	12	(0.30)	340	(104)	380	(116)
24	2–23	9	(0.23)	10	(0.25)	450	(137)	500	(152)
30	2–23	9	(0.23)	10	(0.25)	450	(137)	500	(152)
36	2–05	8	(0.20)	9	(0.23)	500	(152)	550	(167)
42	2–00	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)
48	2–00	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)

Maximum Joint Deflection* Full-Length Pipe—Push-On Type Joint Pipe

Nominal Pipe Size *in.*	Deflection Angle–θ *deg*	Maximum Offset—S *in. (m)* L = 18 ft (5.5 m)		Maximum Offset—S *in. (m)* L = 20 ft (6.1 m)		Approx. Radius of Curve—R Produced by Succession of Joints *ft. (m)* L = 18 ft (5.5 m)		Approx. Radius of Curve—R Produced by Succession of Joints *ft. (m)* L = 20 ft (6.1 m)	
3	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
4	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
6	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
8	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
10	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
12	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
14	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
16	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
18	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
20	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
24	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
30	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
36	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
42	2*	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)
48	2*	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)
54	1 1/2*	5.5	(0.14)	6	(0.15)	680	(207)	760	(232)

* For 14-in. and larger push-on joints, maximum deflection angle may be larger than shown above. Consult the manufacturer.

FIGURE 25.30 Joint deflections for mechanical and push-on joints. (Reprinted by permission from American Water Works Association Standard for Ductile Iron Water Mains and Their Appurtencances. Copyright © 1993, American Water Works Association)

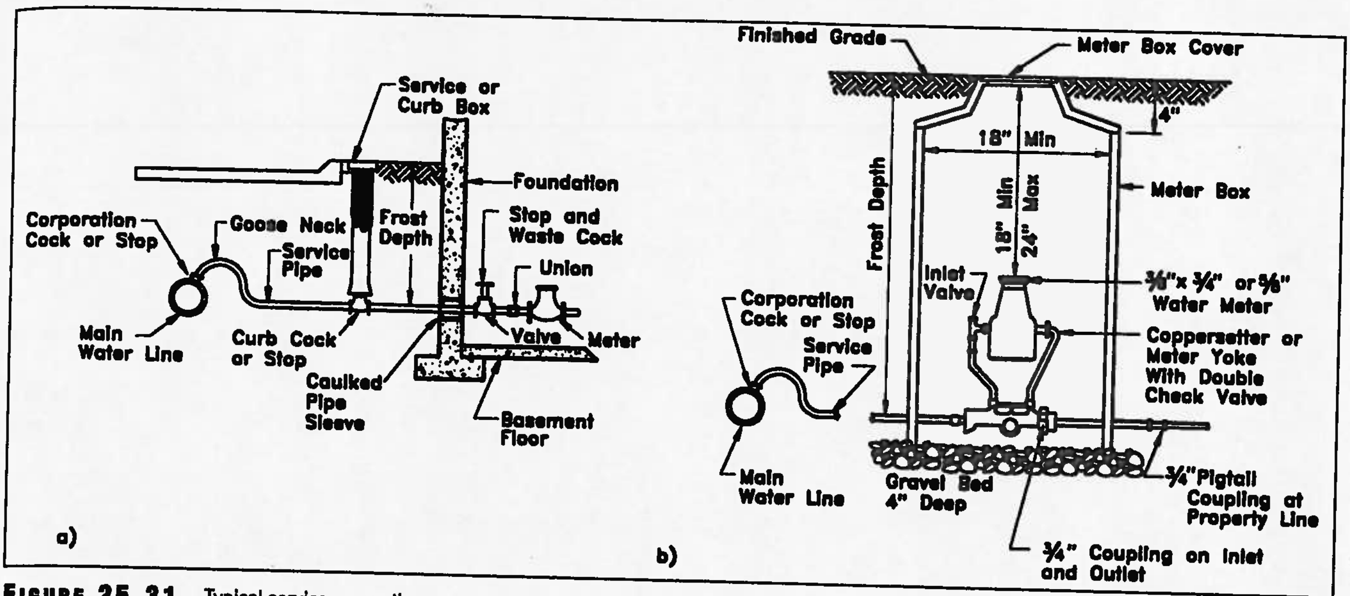


FIGURE 25.31 Typical service connections.

and the valve is opened when the extended water main has been approved for service. The advantage of the tapping sleeve and valve is that it does not require existing service to be shut down while the connection is made. However, this type of connection, sometimes referred to as a wet tap, can be expensive.

If isolation valves are located in close proximity to the proposed connection, they can be closed while the connection is made. While the valves are closed, the existing pipe can be cut. The cut section is removed, and a tee and sleeve is inserted in its place. Note that this method requires inconveniencing the customers while the construction is performed.

Another method, similar to the wet tap method, consists of inserting valves onto the existing line in the area where the connection is to be made. The valves are closed and the existing waterline is cut and replaced by the fixture. However, like the wet tap, this method can be expensive.

Many connections of appurtenances require additional stability due to pressure forces. Besides using thrust blocks, stability is attained by strapping the appurtenances to other stable parts. Strapping consists of connecting one fixture to another fixture or pipe by using several rods attached at the flanges.

Cross Connections

A cross connection is a link through which contaminated water enters the potable water supply. The most obvious situation in which this occurs is when the potable water supply is directly connected to a contaminated source, either through an error during construction or poor hydraulic design. Although the system of checks and balances that exists during the design and approval process minimizes the chances of such a connection occurring, the repercussions of even one occurrence are costly both in human safety and monetary damages. Even if not physically connected to the water supply system, cross connections can occur when back siphonage and backflows of contaminated water enter the potable water system. Back siphonage, a type of backflow, results from negative pressures within the distributing pipes of the potable water supply, which draw water from the surrounding area into the system. Back siphonage can occur from inadequate pressure at the suction side of a booster pump, a water main break causing significantly reduced pressure in the system, and reduced pressure in the system caused during repairs.

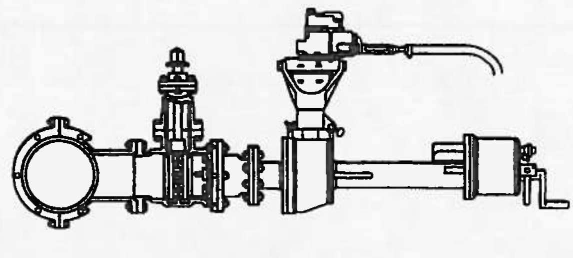
An actual situation where contamination of the water supply resulted from backflow occurred in 1971. As reported by the EPA:

> A contractor using a tank truck with a rig designed to pump and spray a mixture of water, fertilizer, grass seed and wood pulp was working on the grounds of a subdivision. The contractor was using a direct connection to a fire hydrant to fill the tank with water, which was then mixed with the fertilizer, etc. A high pressure pump then sprayed the mixture onto the ground. As the wood pulp circulated through the tank piping system, it plugged one of the lines while the pump continued to run creating a very high pressure in the tank. This pressure was higher than the water supply system pressure and it forced the solution of fertilizer into the water system. Several people in the subdivision became ill after drinking the water, but the contamination was discovered and quick action in flushing and disinfecting the lines eliminated the danger.[2]

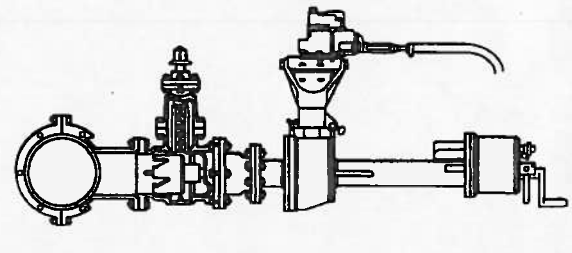
Although certain precautionary measures can be used to reduce the potential for cross connection during the design, the potential will always exist as long as the human element is present in design and construction. The most direct and

[2]Cross Connection Control Manual, USEPA 1975.

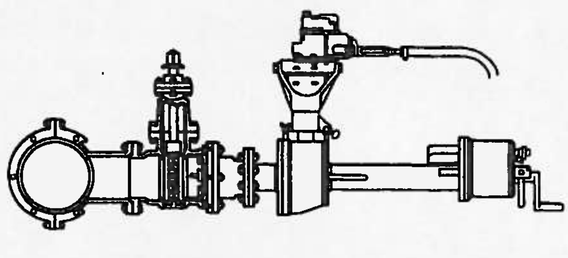
The MUELLER method for making a lateral connection using the MUELLER CL-12 Drilling Machine



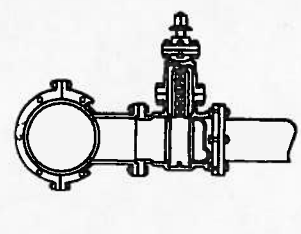
1 The tapping sleeve and valve are first attached to the main and tested. Then the drilling machine, with a shell cutter attached to the boring bar, is attached to the tapping valve using an adapter. The assembly should be pressure tested prior to making the cut.



2 With the tapping valve open, the shell cutter and boring bar are advanced to cut the main.



3 The boring bar is retracted and the tapping valve closed to contain the water pressure.



4 With the machine removed, the lateral is connected and the tapping valve opened to pressurize the lateral and place it in service.

Large drilling machine selection guide

Machine	Tapping valve size range	Maximum pressure	Operation	Recommended uses
CL-12	2"-12"	250 psig (1724 kPa)	Hand or power	For making lateral connections or inserting valves under pressure
C1-36-99002	2"-24"	500 psig (3447 kPa)	Hand or power	
CC-25	2"-12"	500 psig (3447 kPa)	Hand	

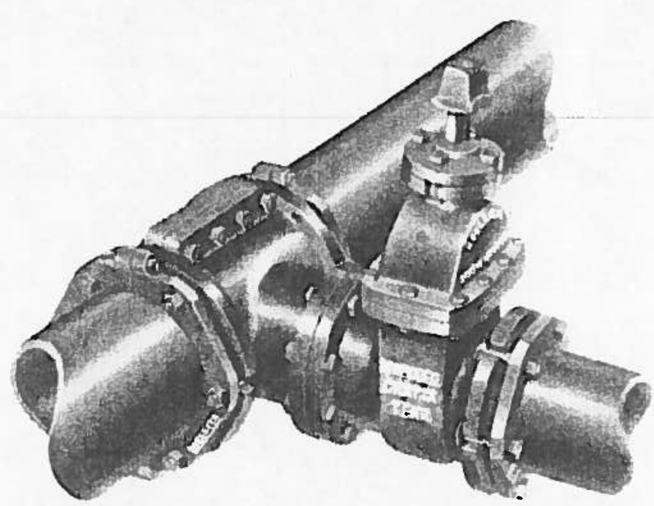


FIGURE 25.32 Tapping sleeve and valve. (Reprinted with permission of Mueller Co., Decatur, IL)

easiest method for backflow prevention is the inclusion of an air space between the free-flowing discharge end of the pipe and the flood-level rim of the fixture. Other mechanical methods, used mainly in the smaller distribution and service lines, include vacuum breakers, double check valve, and reduced-pressure-principle device.

SYSTEM ANALYSIS

Analysis and design of a water distribution system is a complex process. As a result of the availability of computers and the myriad software programs available to analyze complex systems, detailed longhand computations have become virtually obsolete. This does not de-emphasize the importance of the fundamental knowledge of hydraulics. Very often, the computer and software are used as a "black box" to generate answers from the input data. Although it is not imperative that the program users fully understand the program and the numerical methods used within the program, it is imperative that the users understand the limitations of the program and the reliability of the input data. To intuitively get a feel for the correctness of the output, the engineer still needs to understand the fundamental methods typically used in the software program. The results from a computer software program are only as good as the input data and the numerical methods used within the program.

Pipe Flow

In general, flow through a pipe conduit is affected by its hydraulic and geometric properties. Length and shape are the significant geometric properties; hydraulic properties include pipe material (for roughness), type and number of components (valves, reducers, bends, tees, etc.), and pressure. Analysis of fluid flow is a matter of determining the total hydraulic energy of the fluid. To do so requires the conversion of the fluid's known energy-related parameters—pressure, velocity, and relative elevation—to units of energy.

Most water distribution systems function as pressure flow. Mathematically, system pressure is converted to energy by changing the units of pressure to an equivalent height of fluid. This is done by dividing the pressure by the specific weight of the fluid. Although specific weight is a function of temperature, the value of 62.4 pounds per cubic foot is ordinarily assumed constant for the conditions encountered in water distribution systems. Equation 25.13 converts pressure to pressure head—the depth of fluid that creates a pressure identical to the pressure in the pipe.

$$\frac{p}{\gamma} = h \tag{25.13}$$

where p is the pipe pressure in lb/ft^2, γ is the specific weight of the fluid (lb/ft^3), and h is a depth in ft. This is also referred to as the flow energy of the fluid. The kinetic energy of a moving fluid, typically referred to as the velocity head, is

$$h_v = \frac{v^2}{2g} \tag{25.14}$$

where h_v = velocity head, v = average velocity in the cross section of flow, and g = gravitational acceleration.

Fluid within the distribution system also has potential energy. This is the energy associated with a fluid's height relative to a reference datum. The datum is arbitrarily selected for the convenience of analyzing the system or identifying other parameters associated with the system. The variable z is used in hydraulic nomenclature to reference the potential energy of the system.

Analysis of a system, or components within specific confines of the system, usually requires the comparison of the total energy of the fluid at two locations. From the conservation of energy principle, equating the energy at one location to the fluid's energy at the second location produces

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 + \Sigma h_l = \frac{p_2}{\gamma} + \frac{v_1^2}{2g} + z_2 + \Sigma h_L \tag{25.15}$$

where the Σh_L terms represent energy lost by the fluid in going from location one to location two. Energy lost or taken from the system is attributed to pipe friction, components such as valves and reducers, and sudden enlargements and contractions.

In the energy equation (Equation 25.13), the $p/\gamma + z$ terms represent the hydraulic grade line (HGL). This is the locus of values above the centerline of the pipe that designate the height to which the water level would rise due to the pressure head if it were not confined within the pipe. Along straight sections of pipe, energy losses in the system cause the HGL to drop in the direction of flow. However, increases in pipe velocity in the downstream direction will cause the HGL to rise. Adding the $v^2/2g$ term to the HGL represents the energy grade line. This represents the fluid's available energy and, similar to the HGL, decreases in the downstream direction when no external energy is put into the system.

Discharge through pipes is represented by the continuity equation, $Q = VA$. Unless there is a leak or the fluid is highly compressible, the continuity equation shows that the amount of water flowing in a length of pipe is constant; velocity adjusts proportionally to any change in cross-sectional area to keep the discharge constant. Inherent in pipe flow is the energy loss from the effects of the roughness of the pipe material. The pipe's surface roughness affects the velocity across the cross-sectional flow area, with a net result of retarding the flow. Therefore, the equations for determining the discharge in pipes must account for this head (i.e., energy) loss.

The Darcy-Weisbach equation is one of the most popular methods for determining the head loss in pipe flow:

$$h_L = f\frac{L}{D}\frac{V^2}{2g} \tag{25.16}$$

where h_L = head loss, f = dimensionless friction factor, L = length of pipe, D = diameter of the pipe, V = average velocity in the pipe, and g = gravitational constant. Experimental verification shows the following to be true:

- The head loss varies directly with the length of pipe and inversely with the diameter.
- The head loss varies with the square of the velocity.
- The head loss depends on the roughness of the interior pipe wall.
- The head loss depends on fluid properties of density and viscosity and is independent of pressure.

The last two items are evident from the significance of the parameter f. The value of f is dependent on the Reynolds number, the ratio of the inertia forces to the viscous forces of the fluid, and the relative roughness of the pipe. These are the only two significant forces of the fluid in a completely filled conduit. For a circular conduit the Reynolds number (R) is

$$R = \frac{\rho DV}{\mu} \tag{25.17}$$

where ρ is the density and μ is the absolute viscosity of the fluid. For any consistent system of units, R is a dimensionless number.

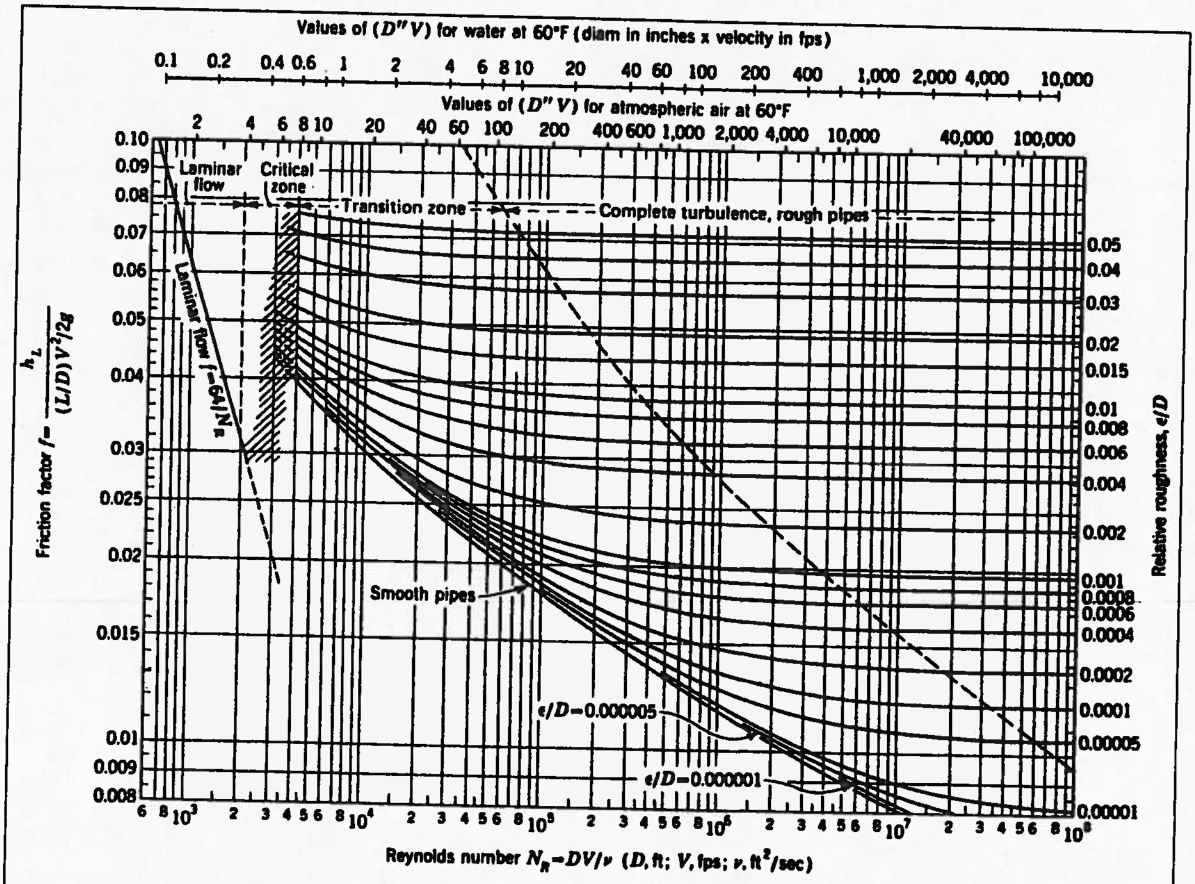
For laminar flows (i.e., $R < 2000$),

$$f = \frac{64}{R} \tag{25.18}$$

For turbulent flows, f has been found through experimentation to be related to the ratio of ε/D, where ε is the absolute roughness, a parameter that measures the size of the irregularities of the pipe material. The Moody diagram (Figure 25.33) shows the relationship between f, ε, and R. This ratio depends on the pipe material and the pipe diameter. Figure 25.34 shows this ratio for various commercial pipes.

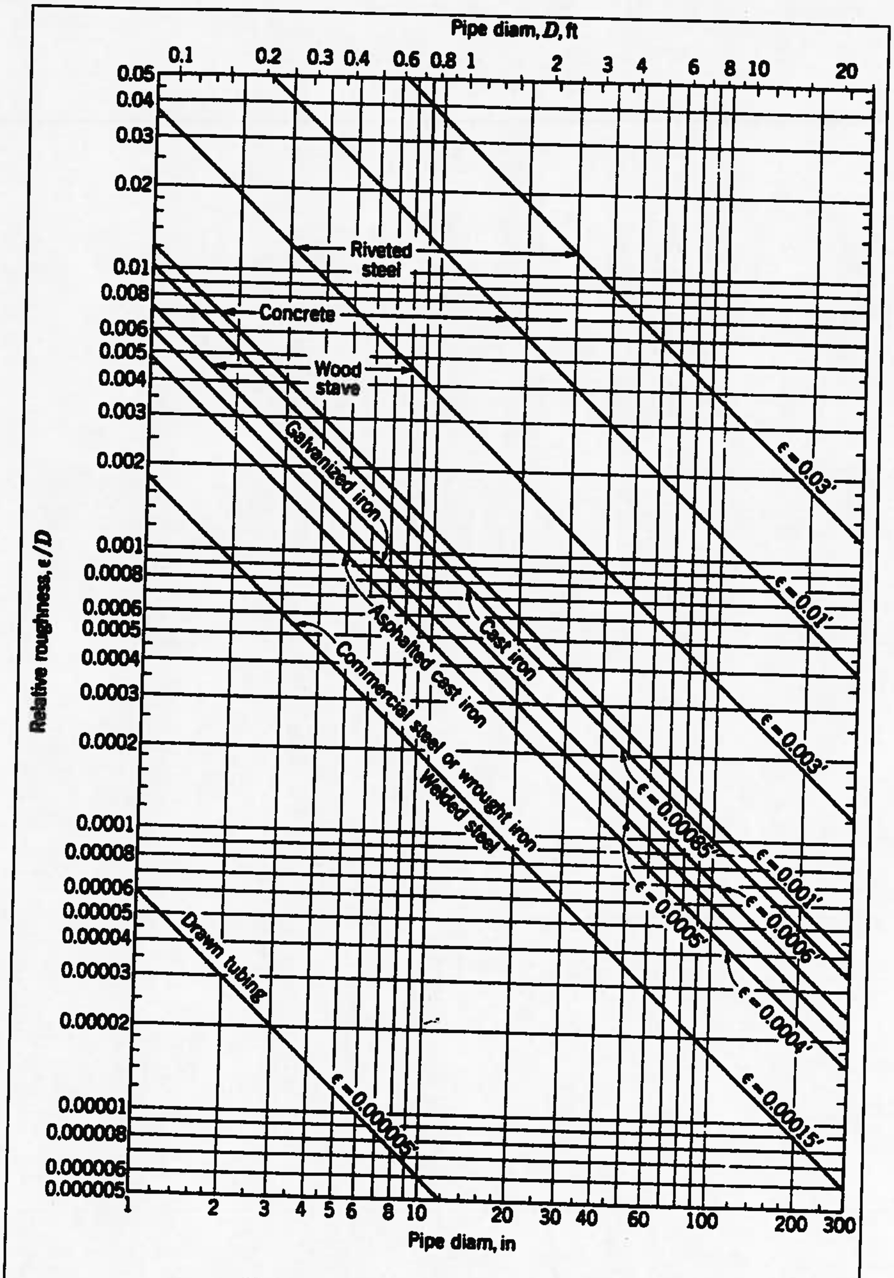
The energy loss h_L for a known discharge and pipe can be found by the following process:

1. Determine the Reynolds number (Equation 25.17).
2. Determine ε from Figure 25.34.
3. Determine the ε/D ratio and enter R and ε/D on the Moody diagram (Figure 25.34) and read the corresponding value of f.
4. Use the Darcy-Weisbach (Equation 25.16) to find h_L.
5. Use the Bernoulli equation and/or the continuity equation to determine other design parameters.



Friction factor for pipes.

FIGURE 25.33 Moody diagram. (ASME Transactions, vol. 66, 1944. Reprinted by permission of American Society of Mechanical Engineers)



Roughness factors (expressed in feet) for commercial pipes.

FIGURE 25.34 Roughness factors for commercial pipes. (ASME Transactions, vol. 66, 1944. Reprinted by permission of American Society of Mechanical Engineers)

In lieu of the Moody diagram, Equation 25.19 can be used to approximate f for the given constraints

$$f = \frac{1.325}{[\ln(\varepsilon / 3.7\,D + 5.74/R^{0.9})]^2} \tag{25.19}$$

for

$$10^{-6} \leq \frac{\varepsilon}{D} \leq 10^{-2}$$

$$5000 \leq R \leq 10^{8}$$

Pipe analysis problems with incompressible flow require the determination of the variables Q, L, D, h_f, and V. Recognize that V, D, and f are interrelated. Hence, the solution to solving problems where these parameters are unknown (and therefore R is unknown) requires a trial-and-error procedure. Since the value of f changes very slowly at the higher R values, begin the trial-and-error procedure by assuming a value for either f or R and obtain a final solution for Q or V. Compute an f (or R) using the final solution, and use the Moody diagram to find R (or f) and compare this to the assumed value. The procedure ends when the computed value is within an acceptable tolerance of the assumed value. For example, if D is known, the trial-and-error procedure begins by assuming f and using the Darcy-Weisbach equation to find the velocity. Use this velocity to compute R and find the corresponding f value from the Moody diagram. If the two values are within an acceptable tolerance, the procedure ends and the design parameters are known; otherwise, assume another value for f and repeat the procedure.

Another popular method used to find pipe discharges and other design parameters is the Hazen-Williams equation, shown for use with English units as

$$V = 1.318\, C\, R^{0.63}\, S^{0.54} \tag{25.20}$$

where V = velocity, C = coefficient associated with pipe roughness, R = hydraulic radius (= $D/4$ for circular pipes flowing full), and S = energy gradient (h_l/L).

Although not dimensionally correct, as many empirical equations tend to be, the Hazen-Williams equation provides generally acceptable discharge estimates. In most problems for design purposes, C is taken to be 100 but, for aged pipe C, can be as low as 70. Table 25.12 provides a range of C values for different pipe materials. The advantage in using this equation, over the Darcy-Weisbach equation; is the independence of C from the Reynolds number. Because this equation does not include any terms relating to the physical properties of the fluid, the Hazen-Williams equation is used for water discharges for turbulent conditions only. Additionally, this nondependence on the physical properties of the water makes the Hazen-Williams equation the preferred equation for pipe flow analysis. Figure 25.35 is the nomograph for the Hazen-Williams equation. Given the value of two of the variables in the equation, the nomograph provides the unknown values of the remaining two variables.

Minor Losses

Components in the pipe network contribute to the system's head losses. These are considered as minor losses and, depending on the section of the system under analysis, may or may not need to be incorporated into the design computations. Energy lost through friction is significantly larger than the combined minor losses when pipe lengths exceed 1000± diameters. In these situations, the minor energy losses can be ignored without appreciable effect on the results (due to the uncertainty in the C coefficient).

The loss coefficient method uses a constant of proportionality based on the configuration of the component. This constant is applied to the velocity head of the water entering or exiting the component. This method computes the minor losses using the form

$$h_v = k\frac{v^2}{2g} \tag{25.21}$$

where k is the constant of proportionality. Table 25.13 shows k factors for various fittings and other waterline appurtenances. For additional information on head losses through fittings as well as other components, the manufacturer's specifications should be consulted.

Equivalent Pipe Lengths

A pipe network typically consists of loops and branches, with fittings and pipes of various diameters. In order to simplify pipe network computations, it may be advantageous to convert the fittings and various pipe lengths into equivalent lengths of a uniform diameter. The equivalent pipe length represents a length of pipe of known diameter that has either the same head loss as a length of pipe of different diameter with the same discharge or the same head loss as a fitting for the given discharge. Using the Darcy-Weisbach equation, an equivalent-length pipe for a fitting with a given head loss is

$$f\left(\frac{L_e}{D}\right)\frac{V^2}{2g} = K\left(\frac{V^2}{2g}\right) \tag{25.22}$$

$$L_e = K\frac{D}{f}$$

where the terms are as defined earlier and L_e represents the length of pipe of diameter D that corresponds to the head loss through the component. The nomograph of Figure 24.30 can be used in lieu of the equations to convert various fixtures to equivalent lengths.

To convert a pipe of one diameter to a pipe of different diameter for a given discharge, either the Darcy-Weisbach equation or the Hazen-Williams equation can be used. After rearranging terms of the Darcy-Weisbach equation by converting one pipe of known diameter D to another diameter D_e of equivalent length, L_e is found by

$$L_e = L\left(\frac{f}{f_e}\right)\left(\frac{D_e}{D}\right)^5 \tag{25.23}$$

where f and f_e are, respectively, the friction factors for the given pipe and equivalent-length pipe.

Similarly, the Hazen-Williams equation is rearranged to find an equivalent-length pipe by using the diameters and the coefficients of pipe roughness for the equivalent-length pipe C_e and the coefficient of roughness for the given pipe C:

$$L_e = L\left(\frac{D_e}{D}\right)^{4.87}\left(\frac{C_e}{C}\right)^{1.85} \tag{25.24}$$

As an example, find the equivalent length of 12-inch diameter ductile iron pipe (C = 100) for 500 feet of 10-inch-diameter pipe (C = 100) attached with a globe valve at one end (Q = 5 cfs).

The velocity in the 10-inch pipe is:

$$V_{10} = \left(\frac{Q}{A_{10}}\right) = \frac{5}{\frac{\Pi}{4}\left(\frac{10}{12}\right)^2} = 9.2 \text{ fps} \tag{25.25}$$

Using the same equation, the velocity in the 12-inch diameter pipe is 6.4 fps.

From Table 25.13 the head loss coefficient for a globe

TABLE 25.12 Relative Roughness and Hazen-Williams Constants for Various Pipe Materials

	ε(FT)		C		
TYPE OF PIPE OR SURFACE	RANGE	DESIGN	RANGE	CLEAN	DESIGN
STEEL					
welded and seamless	0.0001–0.0003	0.0002	150–80	140	100
interior riveted, no projecting rivets				139	100
projecting girth rivets				130	100
projecting girth and horizontal rivets				115	100
vitrified, spiral-riveted, flow with lap				110	100
vitrified, spiral-riveted, flow against lap				100	90
corrugated				60	60
MINERAL					
concrete	0.001–0.01	0.004	152–85	120	100
cement-asbestos			160–140	150	140
vitrified clays					110
brick sewer					100
IRON					
cast, plain	0.0004–0.002	0.0008	150–80	130	100
cast, tar (asphalt) coated	0.0002–0.0006	0.0004	145–50	130	100
cast, cement-lined	0.000008	0.000008		150	140
cast, bituminous-lined	0.000008	0.000008	160–130	148	140
cast, centrifugally spun	0.00001	0.00001			
galvanized, plain	0.0002–0.0008	0.0005			
wrought, plain	0.0001–0.0003	0.0002	150–80	130	100
MISCELLANEOUS					
fiber				150	140
copper and brass	0.000005	0.000005	150–120	140	130
wood stave	0.0006–0.003	0.002	145–110	120	110
transite	0.000008	0.00008			
lead, tin, glass		0.000005	150–120	140	130
plastic (PVC and ABS)		0.000005	150–120	140	130

(Reprinted with permission from Professional Publications, Inc., *Civil Engineering Reference Manual*, 10th ed., by Michael R. Lindeburg, copyright © 2006 by Professional Publications, Inc.)

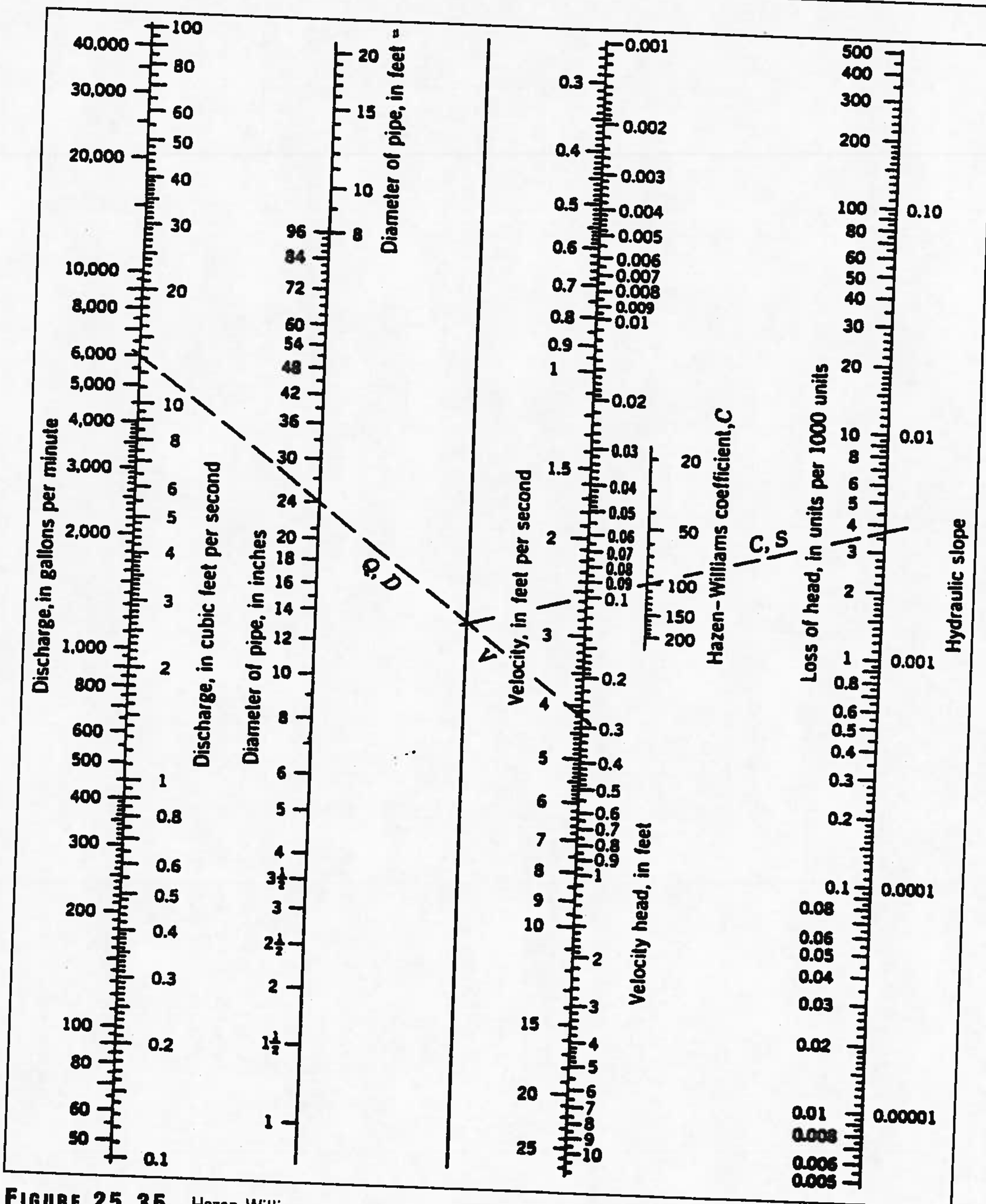


FIGURE 25.35 Hazen-Williams nomograph. Design and construction of sanitary and storm sewers (1969). (American Society of Civil Engineers and the Water Pollution Control Federation. Reprinted with permission of ASCE, New York)

valve is 10 and the corresponding head loss through the globe valve is:

$$h_l = 10\left(\frac{9.2^2}{2g}\right) = 13.1\text{ ft} \qquad (25.26)$$

Using the Hazen-Williams equation, the equivalent length of 12-inch diameter pipe for the globe valve is:

$$L_{12} = \left(\frac{1.318CR^{0.63}h_L^{0.54}}{V}\right)^{1.85} \qquad (25.27)$$

$$= \frac{1.318(100)\left(\frac{1}{4}\right)^{0.63} 13.1^{0.54}}{6.4} = 700\text{ ft}$$

From Equation 25.24, the equivalent length of 12-inch-diameter pipe for 500 feet of 10-inch-diameter pipe is

$$L_{12} = 500\left(\frac{1}{\left(\frac{10}{12}\right)}\right)^{4.87}\left(\frac{100}{100}\right)^{1.85} = 1215\text{ ft} \qquad (25.28)$$

The total equivalent length of 12-inch diameter pipe is 700 + 1215 = 1915 feet.

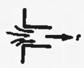
Fixture Unit Method

Occasionally, the project engineer may be required to size the water service line to a building. This normally occurs on commercial or other high-rise/high-density projects. For the most part, however, the analysis and sizing of the internal

TABLE 25.13 Table of Local Loss Coefficients

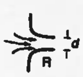
Use the equation $h_v = kv^v/2g$ unless otherwise indicated. Energy loss E_L equals h_v head loss in feet.

Perpendicular square entrance:



$k = 0.50$ if edge is sharp

Perpendicular rounded entrance:



$R/d =$	0.05	0.1	0.2	0.3	0.4
$k =$	0.25	0.17	0.08	0.05	0.04

Additional loss due to skewed entrance:

$k = 0.505 + 0.303 \sin \alpha + 0.226 \sin^2 \alpha$

Strainer bucket:

$k = 10$ with foot valve
$k = 5.5$ without foot valve

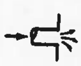
Standard tee, entrance to minor line:

$k = 1.8$

Confusor outlet:

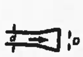
$d/D =$	0.5	0.6	0.8	0.9
$k =$	5.5	4	2.55	1.1

Exit from pipe into reservoir:



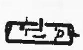
$k = 1.0$

Diffusor outlet for D/d > 2:



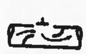
$\alpha° =$	8	15	30	45
$k =$	0.05	0.18	0.5	0.6

Gate valve:



$e/D =$	0	1/4	3/8	1/2	5/8	3/4	7/8
$k =$	0.15	0.26	0.81	2.06	5.52	17.0	97.8

Globe valve:

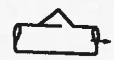


$k = 10$ when fully open

Rotary valve:

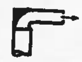
$\alpha° =$	5	10	20	30	40	50	60	70	80
$k =$	0.05	0.29	1.56	5.47	17.3	52.6	206	485	∞

Check valves:



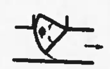
Swing type $k = 2.5$ when fully open
Ball type $k = 70.0$
Lift type $k = 12.0$

Angle valve:



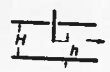
$k = 5.0$ if fully open

Segment gate in rectangular conduit:



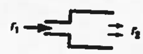
$k = 0.3 + 1.3\,[(^1/_n)]^2$
where $n = \phi/\phi_0 =$ the rate of opening with respect to the central angle

Sluice gate in rectangular conduit:



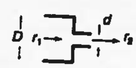
$k = 0.3 + 1.9\,[(^1/_n) - n]^2$
where $n = h/H$.

Sudden expansion:



$E_L = \left(1 - \frac{V_2}{V_1}\right)^2 \frac{V_1^2}{2g}$ or $E_L = \left(\frac{V_1}{V_2} - 1\right)^2 \frac{V_2^2}{2g}$

Sudden contraction:

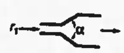


$(d/D)^2 =$	0.01	0.1	0.2	0.4	0.6	0.8
$k =$	0.5	0.5	0.42	0.33	0.25	0.15

use v_2 in Equation 13.13

Diffusor:

$E_L = k(v_1^2 - v_2^2)/2g$



$\alpha° =$	20	40	60	80
$k =$	0.20	.028	0.32	0.35

Confusor:

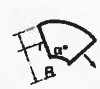
$E_L = k(v_1^2 - v_2^2)/2g$

$\alpha° =$	6	10	20	40	60	80	100	120	140
$D = 3d$	0.12	0.16	0.39	0.80	1.0	1.06	1.04	1.04	1.04
D-$1.5d$	0.12	0.16	0.39	0.96	1.22	1.16	1.10	1.06	1.04

Sharp elbow:

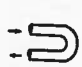
$k = 67.6 \times 10^{-6}(\alpha°)^{2.17}$

Bends:



$k = (0.13 + 1.85(r/R)^{3.5})\sqrt{\alpha°/180°}$

Close return bend:



$k = 2.2$

(Courtesy of Simon, Andrew L. *Hydraulics.* 1986. Adopted by permission of Prentice Hall, Inc., Englewood Cliffs, NJ.)

distribution system are typically done by mechanical/plumbing engineers. The size of the water supply line to the building depends on the demand within the building, which may be related to the number and type of plumbing fixtures in the building or perhaps the building's use. Since there are several methods available to estimate the demand, the one used depends on the building codes of that jurisdiction.

The simplest method used to estimate demand is one that relates the demand to the building use. For example, water demand for an office building could be based on the square footage of the floor area or the number of employees, while for apartments the demand may be based on the number of units in the building. If this type of method is used, the demand table for various building uses will be prescribed in the applicable building code.

Another method estimates the demand based on the number and type of plumbing fixtures in the building. However, the size of the water supply line is based on maximum probable demand rather than the sum of the maximum flows of all of the fixtures. It is highly unlikely that every plumbing fixture would be operating at maximum discharge simultaneously. Under this assumption, the demand is related to the number and type of fixtures, the rates required by these fixtures, and the probable simultaneous operation of the fixtures. The unit of measurement used to correlate these flow characteristics is the water supply fixture unit (WSFU).

The fixture unit is a measure of the hydraulic demand of a fixture. It is the average discharge during use of an arbitrarily selected plumbing fixture. It takes into account the anticipated discharge, the average duration of flow when the fixture is in use, and the frequency with which the fixture is likely to be used. The WSFU is a factor chosen so that the load-producing effects of different kinds of plumbing fixtures, as well as their conditions of service, can be expressed as multiples of that factor. For example, if a bathtub with a discharge of 7.5 gpm is selected as the arbitrary fixture, a fixture with a discharge of 22 gpm is assigned a fixture unit of 3.

Typical values of fixture units are given in Table 25.14. By knowing the type and number of water supply fixtures, the total WSFU for a building can be determined and then converted to flow rates using Figure 25.36.[3]

After estimating the peak demand, the service line is sized using head loss and velocity parameters for the pipe material selected. Consideration is given to the available pressure in the water main to determine whether the pressure is adequate for the calculated losses through the building. Booster pumps may be required when pressures are inadequate at the upper-level floors.

[3]This curve, frequently referred to as the Hunter curve, was first proposed by Roy B. Hunter of the National Bureau of Standards in 1923. Since then it has been modified based on other research data.

EXAMPLE 2

Size the water supply line for a three-story apartment building having five apartment units on each level. Each apartment has one bathroom and a kitchen with a dishwasher. There are four (16-lb) laundry machines in the basement. The elevation of the 8-in water main located in the street is 165.5 ft. The water meter, where the service line connects to the building, is 160 ft away from the water main and is at an elevation of 175.5 ft. The service line has two 90° elbows and one angle valve. Minimum pressure in the water main is 45 lb/in² (see Table 25.15).

1. Tabulate the fixture units.

2. Using Figure 25.36, the total demand for 151 WSFU is 60 gpm.

3. Using Figure 25.37, the demand discharge of 60 gpm and a design velocity of 6 fps indicates that a 2-in pipe is required. Note that the corresponding pressure drop (i.e., head loss) is 2.8 lb/in² per 100 ft.

4. Convert the elbows and angle valve to equivalent lengths of pipe. From Figure 24.30, the angle valve is 28 ft of 2-in pipe and an elbow is 5 ft of 2-in pipe.

5. The pressure at the inlet side of the water meter is the minimum pressure in the water main minus the losses in the valves and piping and the elevation difference (i.e., static lift) between the water main and the outlet connection:

$$\text{Pressure @ water meter} = 45\ \text{lb/in}^2 - \left[\frac{2.8\ \text{lb/in}^2}{100\ \text{ft}}(160 + 28 + 2(5)) - (174.5 - 164.5)\right] = 29.5\ \text{lb/in}^2 \tag{25.29}$$

Hardy-Cross

Analysis of a water distribution system can be extremely complex due to the varying pipe sizes, pump storage facilities, fluctuations in demand, and fire flow requirements that must be considered. Many methods used for analysis are iterative processes that incrementally adjust the flows or head losses until convergence. Several iterative techniques include the linear theory method, the Newton-Raphson method, and the ever popular Hardy-Cross method. One of the most widely used and accepted computer models, KYPIPE, is based on the linear theory method.

The simplicity of the Hardy-Cross method, a form of the Newton-Raphson method, lends itself to hand calculations. Although the personal computer has made laborious hand calculations nearly obsolete, an understanding of even the simplest method helps in assessing the validity of the final values of the analysis. The Hardy-Cross method uses an equation of the form

$$h_l = KQ^n \tag{25.30}$$

TABLE 25.14 Demand Load of Fixtures*

Fixture	Occupancy	Type of supply control	Load values assigned, water supply fixture units: Cold	Hot	Total
Water closet	Public	Flush valve	10		10
Water closet	Public	Flush tank	5		5
Urinal	Public	1″ (25.4 mm) flush valve	10		10
Urinal	Public	¾″ (19 mm) flush valve	5		5
Urinal	Public	Flush tank	3		3
Lavatory	Public	Faucet	1.5	1.5	2
Bathtub	Public	Faucet	3	3	4
Showerhead	Public	Mixing valve	3	3	4
Service sink	Offices, etc.	Faucet	2.25	2.25	3
Kitchen sink	Hotel, restaurant	Faucet	3	3	4
Drinking fountain	Offices, etc.	⅜″ (9.52 mm) valve	0.25		0.25
Water closet	Private	Flush valve	6		6
Water closet	Private	Flush tank	3		3
Lavatory	Private	Faucet	0.75	0.75	1
Bathtub	Private	Faucet	1.5	1.5	2
Shower stall	Private	Mixing valve	1.5	1.5	2
Kitchen sink	Private	Faucet	1.5	1.5	2
Laundry trays (1 to 3)	Private	Faucet	2.25	2.25	3
Combination fixture	Private	Faucet	2.25	2.25	3
Dishwashing machine	Private	Automatic		1	1
Laundry machine [8 lb (3.6 kg)]	Private	Automatic	1.5	1.5	2
Laundry machine [8 lb (3.6 kg)]	Public or general	Automatic	2.25	2.25	3
Laundry machine [16 lb (7.3 kg)]	Public or general	Automatic	3	3	4

*For fixtures not listed, loads should be assumed by comparing the fixture with one listed using water in similar quantities and at similar rates. The assigned loads for fixtures with both hot and cold water supplies are given for separate hot and cold water loads and for total load. The separate hot and cold water loads are three-fourths of the total load for the fixture in each case.
(From Nielson, L. 1981. *Standard Plumbing Engineerings Design*, 2nd ed. New York: McGraw-Hill.)

where h_f is the friction head loss in a section of pipe, K is a constant, and Q is the discharge.

The value of the exponent n depends on the governing equation selected for solving the problem. As an example, the Hazen-Williams equation (equation 25.20) can be written as

$$Q = 1.318C\left(\frac{D}{4}\right)^{0.63} S^{0.54}\left(\frac{\pi D^2}{4}\right) \qquad (25.31)$$

For a given pipe of known material and diameter flowing full the equation simplifies to:

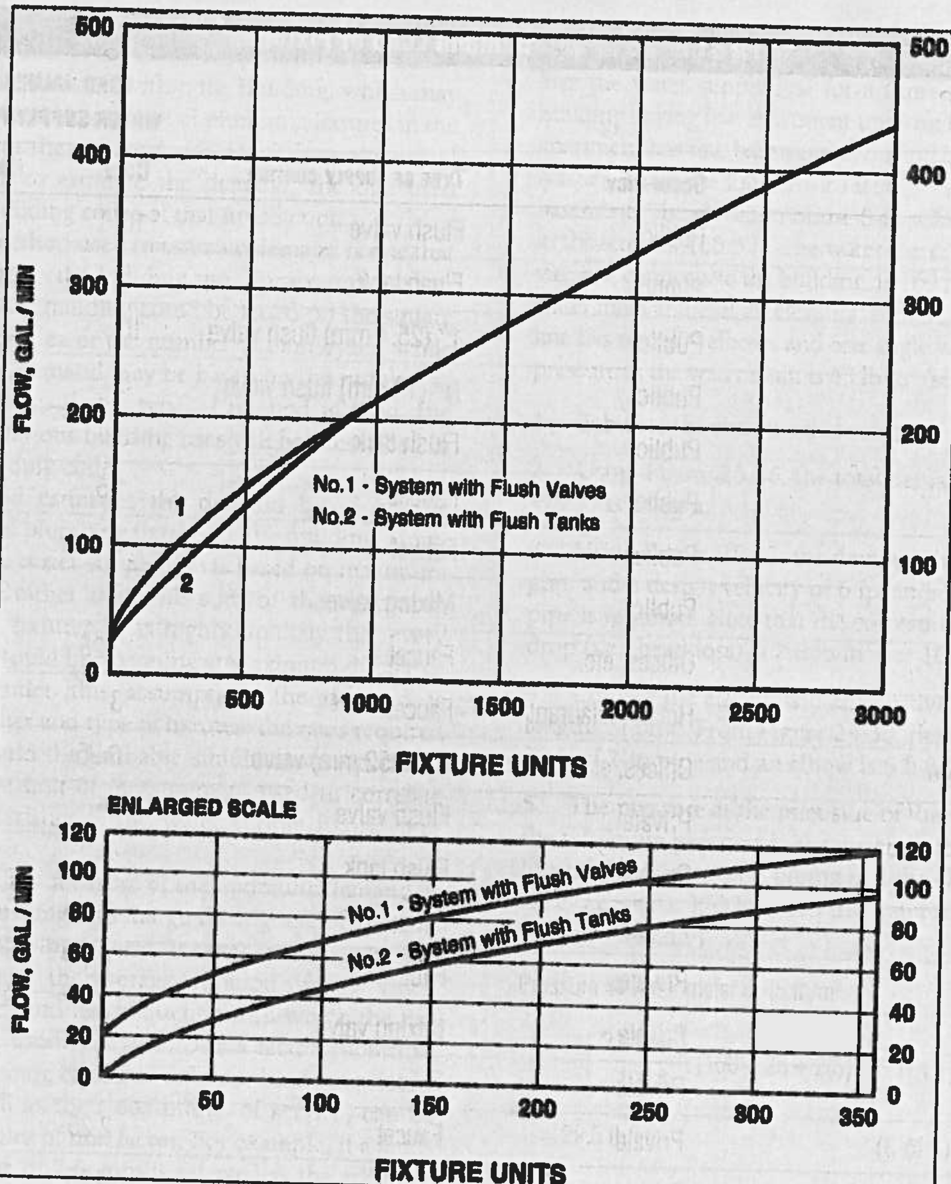


FIGURE 25.36 Curves for estimating demand load.

$$Q = KS^{0.54} \tag{25.32}$$

where the constant K incorporates the coefficient 1.318, the diameter, the C factor, n, and other constants. Substituting for S, the equation is:

$$h_f = KQ^{1.85} \tag{25.33}$$

There are two main underlying assumptions in the Hardy-Cross method. The first is that continuity must be preserved (the total flow into a junction equals the total flow exiting the junction). The second is that pressure at any junction is single valued (the summation of the head loss around a loop is zero).

With reference to Figure 25.38, if continuity is preserved, the flow at point B is equal to the flow at point C. For the pressure at point C to be single valued, the head loss in the upper loop must equal the head loss in the lower loop. In all likelihood the pipe lengths and diameters and possibly even the pipe material in each loop are different. The discharge will divide at point B in such a way that the head loss through the two sections of loop is equal. As a result, if a clockwise direction of flow is assumed to be positive, the summation of the head loss from BC_{up} around CB_{low} will be zero.

The solution is determined through a series of iterations, with each iteration producing a loop correction factor. Some value of flow for Q_{up} and Q_{low} is assumed. For these assumed flows, if $h_{f(BC\text{-}up)}$ is greater than $h_{f(BC\text{-}low)}$, then the discharge in the upper loop must be reduced and the discharge in the lower loop must be increased. That is, $Q'_{up} = Q_{up} + \Delta Q$ and $Q'_{low} = Q_{low} - \Delta Q$, where ΔQ is the loop correction factor.

TABLE 25.15 Fixture Unit Count for Example Problem

Fixture	Total Units	WSFU	Total WSFU
Kitchen sink	15	2	30
Dishwasher	15	1	15
Lavatory (sink)	15	1	15
Water closet (flush tank)	15	3	45
Bathtub	15	2	30
Laundry machine	4	4	16
			Total = 151

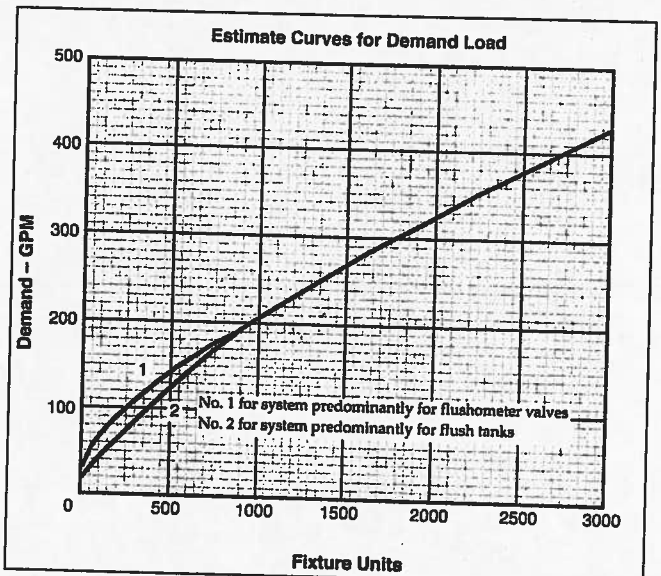
Eventually the discharges in each loop are adjusted so that the summation of the head loss around the loop converges to zero. The flow correction factor for the loop is:

$$\Delta Q = \frac{h_{f_{BD\text{-}up}} - h_{f_{BC\text{-}low}}}{1.85\left(\frac{h_{f_{BC\text{-}up}}}{Q_{up}} + \frac{h_{f_{BC\text{-}low}}}{Q_{low}}\right)} \qquad (25.34)$$

For a more complex multiloop problem, the initial task is to set up the following parameters for the pipe system:

- Pipe sizes
- Pipe lengths and materials
- All locations of flows entering and leaving the system
- A sign convention for the direction of flow

In the initial assumption of pipe flows, it is imperative for the sum of the flows at any junction to be zero. Using



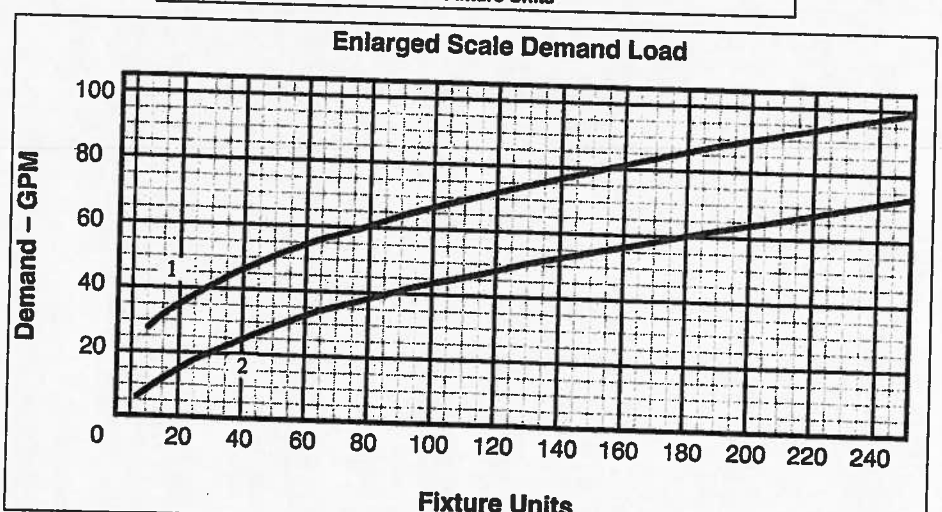


FIGURE 25.37 Chart for determination of flow in pipes. (Reprinted with the permission of the International Association of Plumbing and Mechanical Officials)

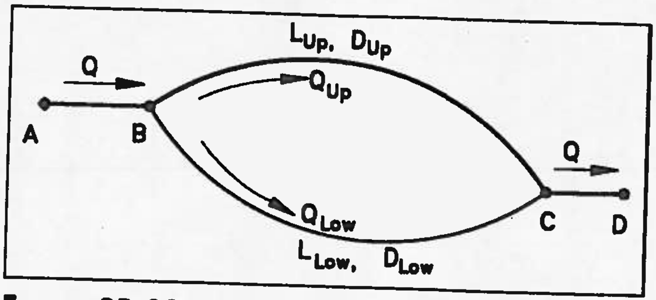


FIGURE 25.38 Simple loop for Hardy-Cross solution.

these initial conditions, the head loss for each pipe is computed as:

$$\Delta Q = -\left(\sum_{i=1}^{n} (H_f)_i\right) \times \sum_{i=1}^{n}\left(\left|\frac{(H_f)_i}{Q_i}\right|\right) \tag{25.35}$$

where $(H_f)_i$ represents the head loss in the ith pipe of the loop and Q_i is the discharge in that pipe, which was used to calculate H_f.

The sign convention applies to the H_f term—all flows in the negative direction will have negative H_f terms.

Figure 25.39 shows a simple three-loop system with pipe information and assumed flows. Table 25.16 summarizes the information shown on Figure 25.39. Note the algebraic sign attached to the discharge: negative if the flow is counterclockwise, positive for clockwise direction. At each junction the $\Sigma Q = 0$. In most systems there are valves, pumps, storage tanks, and other components that impact the calculations. Additionally, systems are analyzed for maximum fire flow demands at several locations to ensure adequate pressures everywhere. This simple system serves only to illustrate the fundamental procedure for Hardy-Cross calculations.

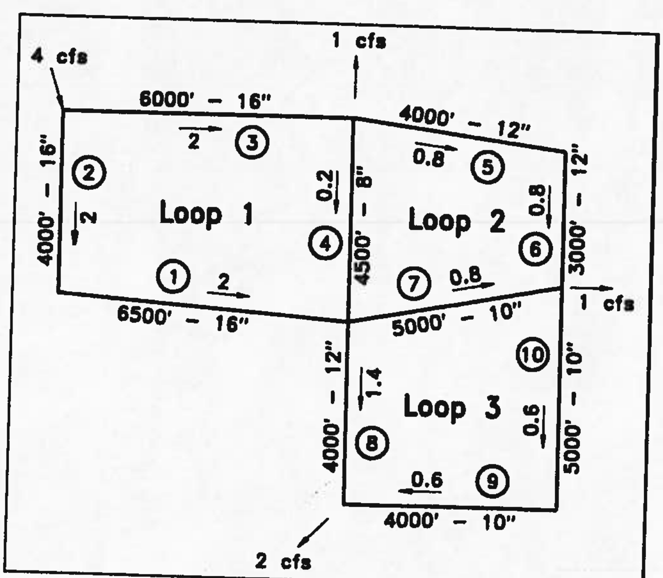


FIGURE 25.39 Schematic diagram for Hardy-Cross example.

TABLE 25.16 Initial Assumptions for Flows and Directions

INPUT VALUES

LOOP NO.	PIPE NO.	DIAMETER (IN.)	LENGTH (FT)	Q (CFS)
1	1	16	6500	–2.00
	2	16	4000	–2.00
	3	16	6000	2.00
	4	8	4500	.20
2	4	8	4500	–.20
	5	12	4000	.80
	6	12	3000	.80
	7	10	5000	–.80
3	7	10	5000	.80
	8	12	4000	–1.40
	9	10	4000	.60
	10	10	5000	.60

Tables 25.17 and 25.18 show how loop flows are adjusted by a ΔQ correction. Note how the adjustments are determined for pipes common to more than one loop. For example, pipe 4 is common to loops 1 and 2. In loop 1, the first iteration ΔQ is +0.082, while the ΔQ for loop 2 is 0.113. The new flow in pipe 4 is then 0.20 + 0.82 – 0.113 = 0.17 cfs. The initial flow of 0.2 is adjusted simultaneously by the correction of the first loop and the negative of the correction of the second loop. The iterative process continues until the sum of the absolute values of the ΔQ's is less than a prescribed minimum tolerance. The acceptable tolerance depends on the precision of the known values and the degree of accuracy required.

DISTRIBUTION SYSTEM CONSTRUCTION PLANS

Preliminary Layout and System Modeling

Waterline analysis and design are done using computers and commercial software, such as KYPIPE, CYBERNET, WaterCAD, and others. To model the system accurately, data is needed on pipe geometry, system demand, flow, and pressure for the existing water services at the points where the proposed waterline connects to the existing waterline. Pressure and flows at these connection points serve as the boundary conditions for the proposed waterline. Additionally, data may be needed on storage tank capacities, water surface elevations in the storage tank at the time of the flow tests, and pumping station and well pump curves.

TABLE 25.17 First Iteration Values

LOOP No.	PIPE No.	DIAMETER (IN.)	LENGTH (FT)	Q (CFS)	H_L	$1.85*H_L/Q$	Q' (CFS)
1	1	16	6500	–2.00	–5.47	5.06	–1.92
	2	16	4000	–2.00	–3.37	3.11	–1.92
	3	16	6000	–2.00	5.05	4.67	2.08
	4	8	4500	.20	1.56	14.43	.17
					–2.23	27.27	
					$\Delta Q = -(-2.23/27.27) = .082$		
2	4	8	4500	–.20	–1.56	14.43	–.17
	5	12	4000	.80	2.51	5.80	.91
	6	12	3000	.80	1.88	4.34	.91
	7	10	5000	–.80	–7.61	17.60	–.52
					–4.78	42.17	
					$\Delta Q = -(-4.78/42.17) = 0.113$		
3	7	10	5000	.80	7.61	17.60	.52
	8	12	4000	–1.40	–7.06	9.33	–1.57
	9	10	4000	.60	3.57	11.00	.43
	10	10	5000	.60	4.47	51.71	.43
					8.59	51.73	
					$\Delta Q = -(8.59/51.73) = -0.166$		

If the data is not available directly from the utility company, flow tests will have to be conducted at nearby hydrants to obtain static and residual pressures for several different flow conditions. If the municipality or water utility company has the computational abilities and resources, they may dictate the location and size of the water supply system components. Some larger utilities now use system modeling software that is integrated with a geographic information system (GIS) for the management of their water infrastructure. For small systems, the consulting engineer will normally do the hydraulic analysis and design of the additional distribution system needed for the new development project.

The following criteria are considered when designing the water supply system:

- In most cases the flows in the water distribution system are designed to supply the amount of water needed to meet the maximum daily potable demand plus fire flows or the peak hour flow, whichever is greater (the AWWA defines the maximum daily demand as the maximum amount of water used during one 24-hour period occurring during the latest three-year period).
- The minimum residual pressure in the system at the average daily demand should be around 40 to 50 lb/in². According to the AWWA, the average daily demand is the average amount of water used each day during a one-year period for the entire system. If the water pressure in the main is 60 lb/in² or greater, the engineer should consider incorporating pressure-reducing valves on the domestic service lines.
- The minimum residual system pressure during the peak hour demand on the maximum day is typically required to be 30 lb/in² or greater. Additionally, the pressure in the water main should be greater than 20 lb/in² during the maximum daily demand plus fire flow.

The preceding considerations are only guidelines for designing the water distribution system. The regulating

TABLE 25.18 Second Iteration Values

LOOP NO.	PIPE NO.	DIAMETER (IN.)	LENGTH (FT)	Q (CFS)	H_L	$1.85{*}H_L/Q$	Q' (CFS)
1	1	16	6500	–1.92	–5.07	4.89	–1.85
	2	16	4000	–1.92	–3.12	3.01	–1.85
	3	16	6000	2.08	5.44	4.84	2.15
	4	8	4500	.17	1.13	12.30	.26
					–1.62	25.04	
					$\Delta Q = -(-1.62/25.04) = 0.064$		
2	4	8	4500	–.17	–1.13	12.30	–.26
	5	12	4000	.91	3.20	6.51	.88
	6	12	3000	.91	2.40	4.88	.88
	7	10	5000	–.52	–3.43	12.20	–.57
					–1.04	35.89	
					$\Delta Q = -(1.04/35.89) = 0.029$		
3	7	10	5000	.52	3.43	12.20	.57
	8	12	4000	–1.57	–8.68	10.23	–1.55
	9	10	4000	.43	1.96	8.43	.45
	10	10	5000	.43	2.45	10.54	.45
					–0.84	41.40	
					$\Delta Q = -(-.84/41.40) = -0.20$		

authorities should be consulted for specific line pressures, demand flows, and other design criteria.

After estimating the demand and determining the peaking factors, lay out the pipe network, including fire hydrants, valves, tees, and crosses, on a plan view of the project. The layout also should include any stubouts needed to provide water service to future developments. In laying out the waterline, the engineer follows the local criteria for setting the waterline in public rights-of-way. Based on the number of dwelling units being served, the engineer estimates the pipe size and labels them on the plan view. In most moderately sized developments, pipe diameters typically range from 6 inches on short cul-de-sacs to 12 inches for the main feeder line. Typically, the size requirement results from fire flow demand and not because of the domestic demand. In all likelihood, the distribution lines through the proposed development will have sizes similar to the existing lines in surrounding similar developments.

Input this data into the model and examine the output for high velocities, excessive head losses, and unacceptable pressures. Adjust the pipe sizes to meet demand and local standards criteria. When the model analysis is complete, finish the plan and profile of the waterline for the construction drawings.

The Water Main Plan and Profile

The plan and profile of the waterline identifies the size and type of pipe and the location of the fittings. To the contractor, it shows the material needed, the volume of excavating, and potential construction problems. To the review agencies, the plan and profile shows that the waterline is in conformance with the applicable building and health codes.

The waterline location is accurately drawn on the plans as per local criteria. For stakeout and construction purposes, the location of a waterline is identified by stationing. Since most waterlines in a development project are located in the street and typically run near parallel with the centerline, the street stationing can be used for stationing the waterline. When the course of a waterline cannot be tied to the street, an arbitrary stationing system on the waterline itself serves the same pur-

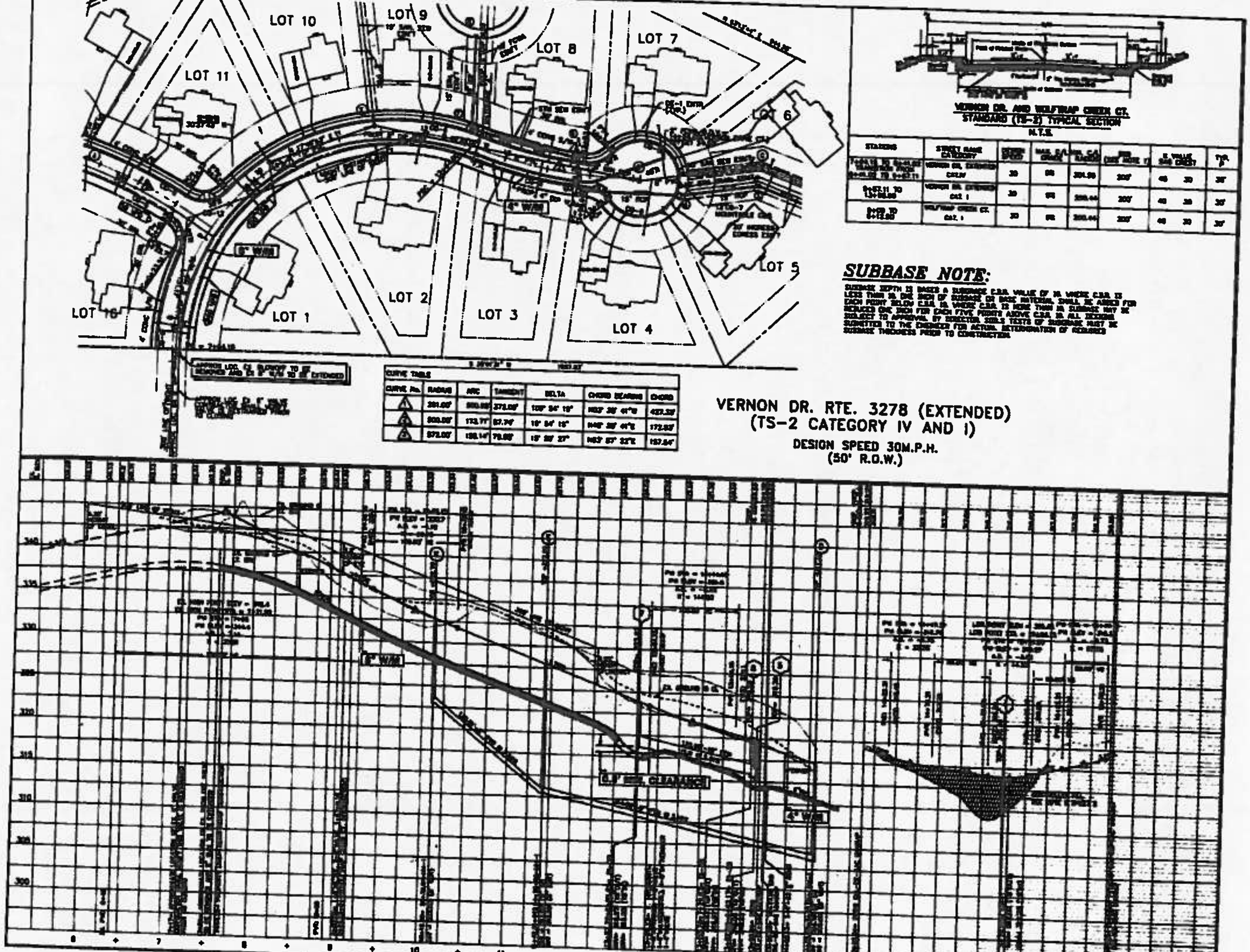


FIGURE 25.40 Plan and profile view of a typical waterline.

pose. Although stationing of the pipe system is done as a matter of convenience and should reasonably reflect the length of the pipe used, it does not need to start with 0 + 00. In the figures showing the plan and profile of a typical waterline, the waterline stationing coincides with the street stationing.

Although some localities allow pipes to be laid with curves, other standards require that pipe direction changes be made with elbow fittings. On commercial sites, the designer needs to know where the fire hose connections at the building are located. In addition, some locales have minimum spacing for fire hydrants in commercial areas. Both of these items affect the placement of the hydrants. In residential areas, placement of fire hydrants may be based on minimum spacing or based on the area of coverage for the required fire flow. The engineer should ensure that the location of the hydrants shown on the worksheet complies with appropriate design criteria. After the waterline location and stationing are established in the plan view, the waterline profile is drawn using the same stationing system. In the profile view, pipe characteristics (e.g., diameter, material, and bearing strength) are labeled. The profile view identifies the station of fitting, hydrants, and other pertinent components. The profiled waterline shows how it will weave around other utilities. As discussed previously, special construction techniques may be necessary when the waterline is placed either near a sanitary sewer or at less than minimum depth. The engineer should ensure that all utility crossings are shown in the profile view and that minimum clearance (as per applicable standards) is maintained.

At the point where the proposed waterline connects to the existing water main, the plans should identify the method for location of existing line and type of connection (e.g., wet tap with tapping sleeve and valve). Although the water utility company probably has records to show where the existing line should be, test holes are dug to find the exact location. In many instances, where the waterline is terminated from a prior development, a blowoff valve is set to easily connect a future waterline (see Figure 25.40).

REFERENCES

American Water Works Association. 1976. *Water Distribution Operator Training Handbook.*

Building Officials and Construction Administrators. 1990. Country Club Hills, IL: National Plumbing Code.

Babbitt, H.E., J.J. Doland, and J.L. Cleasby. 1962. *Water Supply Engineering,* 6th ed. New York: McGraw-Hill.

Cooley, R.L., J.F. Harsh, and D.C. Lewis. 1972. *Hydrologic Engineering Methods for Water Resources Development, Vol. 10, Principles of Ground Water Hydrology.* Davis, CA: Hydrologic Engineering Center.

Cross-Connection Control Manual. 1973. U.S. Environmental Protection Agency. Washington, DC: U.S. Government Printing Office.

Engineers Automatic Valve Reference. Pittsburgh, PA: Golden Anderson Industries Inc.

Fire Suppression Rating Schedule. 1980. New York: Insurance Services Office.

Guide For Determination of Required Fire Flow. 1974. New York: Insurance Services Office.

Grading Schedule for Municipal Fire Protection. 1974. New York: Insurance Service Office.

Lehr, Jay, et al. 1988. *Design and Construction of Water Wells.* New York: Van Nostrand Reinhold.

Merrick, Ronald, C. 1991. *Valve Selection and Specification Guide.* New York: Van Nostrand Reinhold.

National Fire Codes Vol. 10, Recommended Practice for Fire Flow Testing and Marking of Hydrants. 1988. Quincy, MA: National Fire Protection Association.

Nielsen, Louis S., PE. 1982. *Standard Plumbing Engineering Design,* 2nd ed. New York: McGraw-Hill.

Ponce, Victor Miguel. 1989. *Engineering Hydrology.* Englewood Cliffs, NJ: Prentice Hall.

Recommended Standards for Water Works. 1982. Great Lakes Upper Mississippi River Board of State Sanitary Engineers.

Roscoe Moss Company. 1990. *Handbook of Ground Water Development.* New York: John Wiley & Sons.

Thrust Restraint Design for Ductile Iron Pipe. 1984. Birmingham, AL: Ductile Iron Pipe Research Association.

U.S. Department of the Interior, U.S. Geological Survey. 1985. *Estimated Use of Water in the United States in 1985.* Circular 1004. Washington, DC: U.S. Government Printing Office.

U.S. Green Building Council (USGBC). 2005. *LEED-NC for New Construction: Reference Guide,* 1st ed. Washington, DC: USGBC.

Viessman, Warren, Jr., and Mark J. Hammer. 1985. *Water Supply and Pollution Control,* 4th ed. Cambridge: Harper & Row.

Water Distribution Systems Land Development Standards. 1974. Rockville, MD: National Association of Home Builders Research Foundation.

Waterworks Regulations, Commonwealth of Virginia/State Board of Health. 1982. Richmond, VA: Bureau of Water Supply, Department of Health.