$\boxed{\text{Water-Resources Engineering, Second Edition}}$

David A. Chin

Contents

Chapter 2

Flow in Closed Conduits

2.1 Introduction

Flow in closed conduits includes all cases where the flowing fluid completely fills the conduit. The cross-sections of closed conduits can be of any shape or size and can be made of a variety of materials. Engineering applications of the principles of flow in closed conduits include the design of municipal water-supply systems and transmission lines. The basic equations governing the flow of fluids in closed conduits are the continuity, momentum, and energy equations. The most useful forms of these equations for application to pipe flow problems are derived in this chapter. The governing equations are presented in forms that are applicable to any fluid flowing in a closed conduit, but particular attention is given to the flow of water.

The computation of flows in pipe networks is a natural extension of the flows in single pipelines, and methods of calculating flows and pressure distributions in pipeline systems are also described here. These methods are particularly applicable to the analysis and design of municipal water distribution systems, where the engineer is frequently interested in assessing the effects of various modifications to the system. Because transmission of water in closed conduits is typically accomplished using pumps, the fundamentals of pump operation and performance are also presented in this chapter. A sound understanding of pumps is important in selecting the appropriate pump to achieve the desired operational characteristics in water transmission systems. The design protocol for municipal water distribution systems is presented as an example of the application of the principles of flow in closed conduits. Methods for estimating water demand, design of the functional components of distribution systems, network analysis, and the operational criteria for municipal water distribution systems are all covered.

2.2 Single Pipelines

The governing equations for flows in pipelines are derived from the conservation laws of mass, momentum, and energy; and the forms of these equations that are most useful for application to closed-conduit flow are derived in the following sections.

2.2.1 Continuity Equation

Consider the application of the continuity equation to the control volume illustrated in Figure 2.1. Fluid enters and leaves the control volume normal to the control surfaces, with the inflow velocity

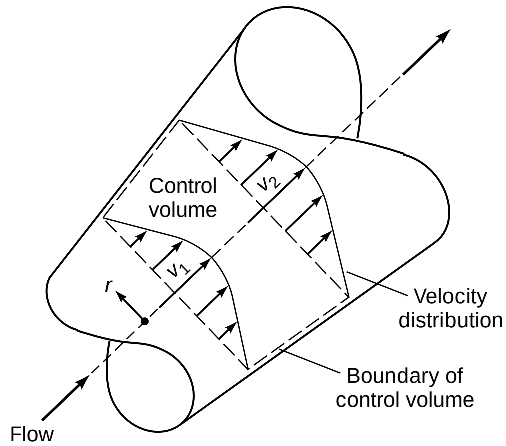


Figure 2.1: Flow Through Closed Conduit

denoted by $v_1(\mathbf{r})$ and the outflow velocity by $v_2(\mathbf{r})$. Both the inflow and outflow velocities vary across the control surface. The steady-state continuity equation for an incompressible fluid can be written as

$$\int_{A_1} v_1\, dA = \int_{A_2} v_2\, dA \tag{2.1}$$

Defining V_1 and V_2 as the average velocities across A_1 and A_2, respectively, where

$$V_1 = \frac{1}{A_1}\int_{A_1} v_1\, dA \tag{2.2}$$

and

$$V_2 = \frac{1}{A_2}\int_{A_2} v_2\, dA \tag{2.3}$$

the steady-state continuity equation becomes

$$\boxed{V_1 A_1 = V_2 A_2 (= Q)} \tag{2.4}$$

The terms on each side of Equation 2.4 are equal to the volumetric flowrate, Q. The steady-state continuity equation simply states that the volumetric flowrate across any surface normal to the flow is a constant.

Example 2.1.
Water enters a pump through a 150-mm diameter intake pipe and leaves the pump through a 200-mm diameter discharge pipe. If the average velocity in the intake pipeline is 1 m/s, calculate the average velocity in the discharge pipeline. What is the flowrate through the pump?

Solution.
In the intake pipeline, $V_1 = 1$ m/s, $D_1 = 0.15$ m and

$$A_1 = \frac{\pi}{4}D_1^2 = \frac{\pi}{4}(0.15)^2 = 0.0177 \text{ m}^2$$

In the discharge pipeline, $D_2 = 0.20$ m and

$$A_2 = \frac{\pi}{4}D_2^2 = \frac{\pi}{4}(0.20)^2 = 0.0314 \text{ m}^2$$

According to the continuity equation,

$$V_1A_1 = V_2A_2$$

Therefore,

$$V_2 = V_1\left(\frac{A_1}{A_2}\right) = (1)\left(\frac{0.0177}{0.0314}\right) = 0.56 \text{ m/s}$$

The flowrate, Q, is given by

$$Q = A_1V_1 = (0.0177)(1) = 0.0177 \text{ m}^3\text{/s}$$

The average velocity in the discharge pipeline is 0.56 m/s, and the flowrate through the pump is 0.0177 m^3/s.

2.2.2 Momentum Equation

Consider the application of the momentum equation to the control volume illustrated in Figure 2.1. Under steady-state conditions, the component of the momentum equation in the direction of flow (x-direction) can be written as

$$\sum F_x = \int_A \rho v_x \mathbf{v}\cdot\mathbf{n}\; dA \tag{2.5}$$

where $\sum F_x$ is the sum of the x-components of the forces acting on the fluid in the control volume, ρ is the density of the fluid, v_x is the flow velocity in the x-direction, and $\mathbf{v}\cdot\mathbf{n}$ is the component of the flow velocity normal to the control surface. Since the unit normal vector, $\mathbf{n}$, in Equation 2.5 is directed outward from the control volume, then the momentum equation for an incompressible fluid ($\rho =$ constant) can be written as

$$\sum F_x = \rho\int_{A_2} v_2^2\, dA - \rho\int_{A_1} v_1^2\, dA \tag{2.6}$$

where the integral terms depend on the velocity distributions across the inflow and outflow control surfaces. The velocity distribution across each control surface is generally accounted for by the *momentum correction coefficient*, β, defined by the relation

$$\boxed{\beta = \frac{1}{AV^2}\int_A v^2\, dA} \tag{2.7}$$

where A is the area of the control surface and V is the average velocity over the control surface. The momentum coefficients for the inflow and outflow control surfaces, A_1 and A_2, are then given by β_1 and β_2, where

$$\beta_1 = \frac{1}{A_1V_1^2}\int_{A_1} v_1^2\, dA \tag{2.8}$$

$$\beta_2 = \frac{1}{A_2V_2^2}\int_{A_2} v_2^2\, dA \tag{2.9}$$

Substituting Equations 2.8 and 2.9 into Equation 2.6 leads to the following form of the momentum equation

$$\sum F_x = \rho\beta_2V_2^2A_2 - \rho\beta_1V_1^2A_1 \tag{2.10}$$

Recalling that the continuity equation states that the volumetric flowrate, Q, is the same across both the inflow and outflow control surfaces, where

$$Q = V_1 A_1 = V_2 A_2 \tag{2.11}$$

then combining Equations 2.10 and 2.11 leads to the following form of the momentum equation

$$\sum F_x = \rho \beta_2 Q V_2 - \rho \beta_1 Q V_1 \tag{2.12}$$

or

$$\sum F_x = \rho Q(\beta_2 V_2 - \beta_1 V_1) \tag{2.13}$$

In many cases of practical interest, the velocity distribution across the cross-section of the closed conduit is approximately uniform, in which case the momentum coefficients, β_1 and β_2, are approximately equal to unity and the momentum equation becomes

$$\sum F_x = \rho Q(V_2 - V_1) \tag{2.14}$$

Consider the common case of flow in a straight pipe with a uniform circular cross-section illustrated in Figure 2.2, where the average velocity remains constant at each cross section,

$$V_1 = V_2 = V \tag{2.15}$$

then the momentum equation becomes

$$\sum F_x = 0 \tag{2.16}$$

The forces that act on the fluid in a control volume of uniform cross-section are illustrated in Figure 2.2. At Section 1, the average pressure over the control surface is equal to p_1 and the elevation

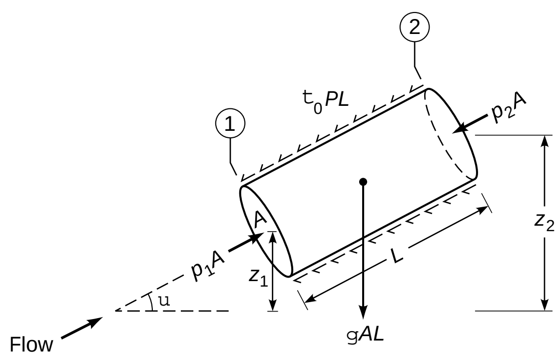


Figure 2.2: Forces on Flow in Closed Conduit

of the midpoint of the section relative to a defined datum is equal to z_1, at Section 2, located a distance L downstream from Section 1, the pressure is p_2, and the elevation of the midpoint of the section is z_2. The average shear stress exerted on the fluid by the pipe surface is equal to τ_o, and the total shear force opposing flow is $\tau_o PL$, where P is the perimeter of the pipe. The fluid weight acts vertically downward and is equal to γAL, where γ is the specific weight of the fluid and A is the cross-sectional area of the pipe. The forces acting on the fluid system that have components in the direction of flow are the shear force, $\tau_o PL$; the weight of the fluid in the control volume,

γAL; and the pressure forces on the upstream and downstream faces, p_1A and p_2A, respectively. Substituting the expressions for the forces into the momentum equation, Equation 2.16, yields

$$p_1A - p_2A - \tau_o PL - \gamma AL \sin\theta = 0 \tag{2.17}$$

where θ is the angle that the pipe makes with the horizontal and is given by the relation

$$\sin\theta = \frac{z_2 - z_1}{L} \tag{2.18}$$

Combining Equations 2.17 and 2.18 yields

$$\frac{p_1}{\gamma} - \frac{p_2}{\gamma} - z_2 + z_1 = \frac{\tau_o PL}{\gamma A} \tag{2.19}$$

Defining the *total head*, or energy per unit weight, at Sections 1 and 2 as h_1 and h_2, where

$$h_1 = \frac{p_1}{\gamma} + \frac{V^2}{2g} + z_1 \tag{2.20}$$

and

$$h_2 = \frac{p_2}{\gamma} + \frac{V^2}{2g} + z_2 \tag{2.21}$$

then the *head loss* between Sections 1 and 2, Δh, is given by

$$\Delta h = h_1 - h_2 = \left(\frac{p_1}{\gamma} + z_1\right) - \left(\frac{p_2}{\gamma} + z_2\right) \tag{2.22}$$

Combining Equations 2.19 and 2.22 leads to the following expression for head loss

$$\Delta h = \frac{\tau_o PL}{\gamma A} \tag{2.23}$$

In this case, the head loss, Δh, is entirely due to pipe friction and is commonly denoted by h_f. In the case of pipes with circular cross-sections, Equation 2.23 can be written as

$$h_f = \frac{\tau_o(\pi D)L}{\gamma(\pi D^2/4)} = \frac{4\tau_o L}{\gamma D} \tag{2.24}$$

where D is the diameter of the pipe. The ratio of the cross-sectional area, A, to the perimeter, P, is defined as the *hydraulic radius*, R, where

$$R = \frac{A}{P} \tag{2.25}$$

and the head loss can be written in terms of the hydraulic radius as

$$h_f = \frac{\tau_o L}{\gamma R} \tag{2.26}$$

The form of the momentum equation given by Equation 2.26 is of limited utility in that the head loss, h_f, is expressed in terms of the boundary shear stress, τ_o, which is not a measurable quantity.

However, the boundary shear stress, τ_o, can be expressed in terms of measurable flow variables using dimensional analysis, where τ_o can be taken as a function of the mean flow velocity, V; density of the fluid, ρ; dynamic viscosity of the fluid, μ; diameter of the pipe, D; characteristic size of roughness projections, ϵ; characteristic spacing of the roughness projections, ϵ'; and a (dimensionless) form factor, m, that depends on the shape of the roughness elements on the surface of the conduit. This functional relationship can be expressed as

$$\tau_o = f_1(V, \rho, \mu, D, \epsilon, \epsilon', m) \tag{2.27}$$

According to the Buckingham pi theorem, this relationship between eight variables in three fundamental dimensions can also be expressed as a relationship between five nondimensional groups. The following relation is proposed

$$\frac{\tau_o}{\rho V^2} = f_2\left(\text{Re}, \frac{\epsilon}{D}, \frac{\epsilon'}{D}, m\right) \tag{2.28}$$

where Re is the Reynolds number defined by

$$\text{Re} = \frac{\rho V D}{\mu} \tag{2.29}$$

The relationship given by Equation 2.28 is as far as dimensional analysis goes, and experiments are necessary to determine an empirical relationship between the nondimensional groups. Nikuradse (1932; 1933) conducted a series of experiments in pipes in which the inner surfaces were roughened with sand grains of uniform diameter, ϵ. In these experiments, the spacing, ϵ', and shape, m, of the roughness elements (sand grains) were constant and Nikuradse's experimental data fitted to the following functional relation

$$\frac{\tau_o}{\rho V^2} = f_3\left(\text{Re}, \frac{\epsilon}{D}\right) \tag{2.30}$$

It is convenient for subsequent analysis to introduce a factor of 8 into this relationship, which can then be written as

$$\frac{\tau_o}{\rho V^2} = \frac{1}{8} f\left(\text{Re}, \frac{\epsilon}{D}\right) \tag{2.31}$$

or simply

$$\frac{\tau_o}{\rho V^2} = \frac{f}{8} \tag{2.32}$$

where the dependence of the *friction factor*, f, on the Reynolds number, Re, and relative roughness, ϵ/D, is understood. Combining Equations 2.32 and 2.24 leads to the following form of the momentum equation for flows in circular pipes

$$\boxed{h_f = \frac{fL}{D}\frac{V^2}{2g}} \tag{2.33}$$

This equation, called the *Darcy-Weisbach equation*,* expresses the frictional head loss, h_f, of the fluid over a length L of pipe in terms of measurable parameters, including the pipe diameter (D),

*Henry Darcy (1803–1858) was a nineteenth-century French engineer; Julius Weisbach (1806 – 1871) was a German engineer of the same era. Weisbach proposed the use of a dimensionless resistance coefficient, and Darcy carried out the tests on water pipes.

average flow velocity (V), and the friction factor (f) that characterizes the shear stress of the fluid on the pipe. Some references name Equation 2.33 simply as the Darcy equation, however this is inappropriate since it was Julius Weisbach who first proposed the exact form of Equation 2.33 in 1845, with Darcy's contribution on the functional dependence of f on V and D in 1857 (Brown, 2002; Rouse and Ince, 1957). The occurrence and differences between laminar and turbulent flow was later quantified by Osbourne Reynolds† in 1883 (Reynolds, 1883).

Based on Nikuradse's (1932, 1933) experiments on sand-roughened pipes, Prandl and von Kármán established the following empirical formulae for estimating the friction factor in turbulent pipe flows

$$\boxed{\begin{array}{ll} \text{Smooth pipe}\left(\frac{k}{D}\approx 0\right): & \frac{1}{\sqrt{f}} = -2\log\left(\frac{2.51}{\mathrm{Re}\sqrt{f}}\right) \\[2ex] \text{Rough pipe}\left(\frac{k}{D}\gg 0\right): & \frac{1}{\sqrt{f}} = -2\log\left(\frac{k/D}{3.7}\right) \end{array}} \tag{2.34}$$

where k is the roughness height of the sand grains on the surface of the pipe. Turbulent flow in pipes is generally present when Re > 4,000; transition to turbulent flow begins at about Re = 2,300. The pipe behaves like a *smooth pipe* when the friction factor does not depend on the height of the roughness projections on the wall of the pipe and therefore depends only on the Reynolds number. In *rough pipes*, the friction factor is determined by the relative roughness, k/D, and becomes independent of the Reynolds number. The smooth pipe case generally occurs at lower Reynolds numbers, when the roughness projections are submerged within the viscous boundary layer. At higher values of the Reynolds number, the thickness of the viscous boundary layer decreases and eventually the roughness projections protrude sufficiently far outside the viscous boundary layer that the shear stress of the pipe boundary is dominated by the hydrodynamic drag associated with the roughness projections into the main body of the flow. Under these circumstances, the flow in the pipe becomes *fully turbulent*, the friction factor is independent of the Reynolds number, and the pipe is considered to be (hydraulically) rough. The flow is actually turbulent under both smooth-pipe and rough-pipe conditions, but the flow is termed *fully turbulent* when the friction factor is independent of the Reynolds number. Between the smooth- and rough-pipe conditions, there is a transition region in which the friction factor depends on both the Reynolds number and the relative roughness. Colebrook (1939) developed the following relationship that asymptotes to the Prandl and von Kármán relations

$$\boxed{\frac{1}{\sqrt{f}} = -2\log\left(\frac{k/D}{3.7} + \frac{2.51}{\mathrm{Re}\sqrt{f}}\right)} \tag{2.35}$$

This equation is commonly referred to as the *Colebrook equation* or*Colebrook-White* equation. Equation 2.35 can be applied in the transition region between smooth-pipe and rough-pipe conditions, and values of friction factor, f, predicted by the Colebrook equation are generally accurate to within 10–15% of experimental data (Finnemore and Franzini, 2002; Alexandrou, 2001).

Commercial pipes differ from Nikuradse's experimental pipes in that the heights of the roughness projections are not uniform and are not uniformly distributed. In commercial pipes, an *equivalent sand roughness*, k_s, is defined as the diameter of Nikuradse's sand grains that would cause the same head loss as in the commercial pipe. The equivalent sand roughness, k_s, of several commercial pipe materials are given in Table 2.1. These values of k_s apply to clean new pipe only; pipe that has been

†Osbourne Reynolds (1842 to 1912).

Table 2.1: Typical Equivalent Sand Roughness for Various New Materials

Material	Equivalent sand roughness, k_s (mm)
Asbestos cement:	
Coated	0.038
Uncoated	0.076
Brass	0.0015–0.003
Brick	0.6
Concrete:	
General	0.3–3.0
Steel forms	0.18
Wooden forms	0.6
Centrifugally spun	0.13–0.36
Copper	0.0015–0.003
Corrugated metal	45
Glass	0.0015–0.003
Iron:	
Cast iron	0.19–0.26
Ductile iron:	
Lined with bitumen	0.12–0.03
Lined with spun concrete	0.030–038
Galvanized iron	0.013–0.15
Wrought iron	0.046 – 0.06
Lead	0.0015
Plastic (PVC)	0.0015–0.03
Steel	
Coal-tar enamel	0.0048
New unlined	0.045–0.076
Riveted	0.9–9.0
Wood stave	0.18

Sources: Haestad Methods, Inc. (2002), Moody (1944), Sanks (1998)).

in service for a long time usually experiences corrosion or scale buildup that results in values of k_s that are orders of magnitude larger than the values given in Table 2.1 (Echávez, 1997; Gerhart et al., 1992). The rate of increase of k_s with time depends primarily on the quality of the water being transported, and the roughness coefficients for older water mains are usually determined through field testing (AWWA, 1992). The expression for the friction factor derived by Colebrook (Equation 2.35) was plotted by Moody (1944) in what is today commonly referred to as the *Moody diagram*,* reproduced in Figure 2.3. The Moody diagram indicates that for Re $\leq$ 2,000, the flow is laminar

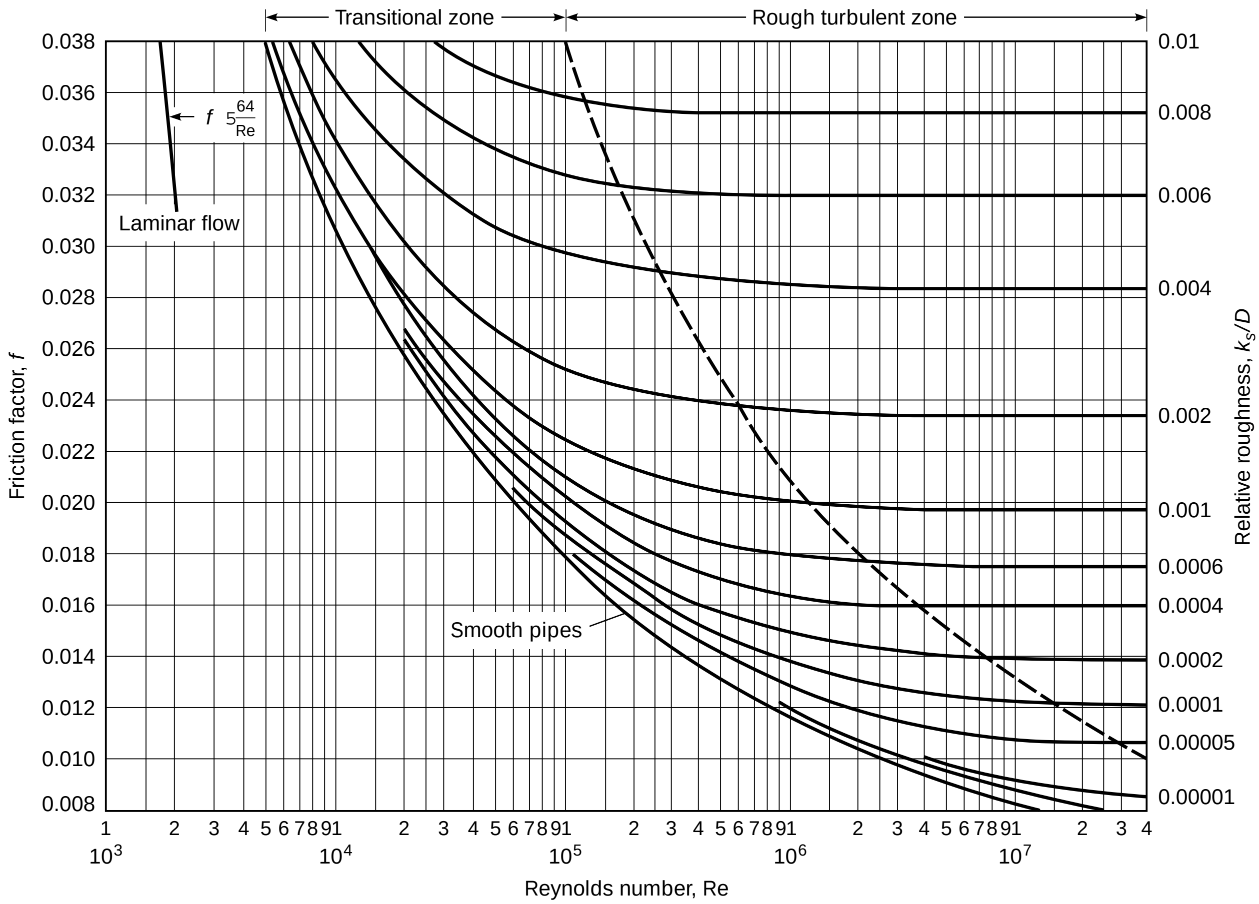


Figure 2.3: Moody Diagram

Source: Moody, L. F. "Friction Factors for Pipe Flow" 66(8), 1944, ASME, New York.

and the friction factor is given by

$$\boxed{f = \frac{64}{\text{Re}}} \tag{2.36}$$

which can be derived theoretically based on the assumption of laminar flow of a Newtonian fluid (Daily and Harleman, 1966). For 2000 < Re $\leq$ 4000 there is no fixed relationship between the friction factor and the Reynolds number or relative roughness, and flow conditions are generally uncertain (Wilkes, 1999). Beyond a Reynolds number of 4000, the flow is turbulent and the friction

*This type of diagram was originally suggested by Blasius in 1913 and Stanton in 1914 (Stanton and Pannell, 1914). The Moody diagram is sometimes called the *Stanton diagram* (Finnemore and Franzini (2002).

factor is controlled by the thickness of the laminar boundary layer relative to the height of the roughness projections on the surface of the pipe. The dashed line in Figure 2.3 indicates the boundary between the fully turbulent flow regime, where f is independent of Re, and the transition regime, where f depends on both Re and the relative roughness, k_s/D. The equation of this dashed line is given by Mott (1994) as

$$\boxed{\frac{1}{\sqrt{f}} = \frac{\text{Re}}{200(D/k_s)}} \tag{2.37}$$

The line in the Moody diagram corresponding to a relative roughness of zero describes the friction factor for pipes that are hydraulically smooth.

Although the Colebrook equation (Equation 2.35) can be used to calculate the friction factor in lieu of the Moody diagram, this equation has the drawback that it is an *implicit equation* for the friction factor and must be solved iteratively. This minor inconvenience was circumvented by Jain (1976), who suggested the following explicit equation for the friction factor

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{k_s/D}{3.7} + \frac{5.74}{\text{Re}^{0.9}}\right), \qquad 10^{-6} \leq \frac{k_s}{D} \leq 10^{-2},\ 5{,}000 \leq \text{Re} \leq 10^8 \tag{2.38}$$

where, according to Jain (1976), Equation 2.38 deviates by less than 1% from the Colebrook equation within the entire turbulent flow regime, provided that the restrictions on k_s/D and Re are honored. According to Franzini and Finnemore (1997) and Granger (1985), values of the friction factor calculated using the Colebrook equation are generally accurate to within 10% to 15% of experimental data, while Potter and Wiggert (2001) put the accuracy of the Moody diagram at no more than 5%. The Jain equation (Equation 2.38) can be more conveniently written as

$$\boxed{f = \frac{0.25}{\left[\log\left(\frac{k_s}{3.7D} + \frac{5.74}{\text{Re}^{0.9}}\right)\right]^2}, \qquad 10^{-6} \leq \frac{k_s}{D} \leq 10^{-2},\ 5{,}000 \leq \text{Re} \leq 10^8} \tag{2.39}$$

Uncertainties in relative roughness and in the data used to produce the Colebrook equation make the use of several-place accuracy in pipe flow problems unjustified. As a rule of thumb, an accuracy of 10% in calculating friction losses in pipes is to be expected (Munson et al., 1994; Gerhart et al., 1992).

Example 2.2.
Water from a treatment plant is pumped into a distribution system at a rate of 4.38 m^3/s, a pressure of 480 kPa, and a temperature of 20°C. The diameter of the pipe is 750 mm and is made of ductile iron. Estimate the pressure 200 m downstream of the treatment plant if the pipeline remains horizontal. Compare the friction factor estimated using the Colebrook equation to the friction factor estimated using the Jain equation. After 20 years in operation, scale buildup is expected to cause the equivalent sand roughness of the pipe to increase by a factor of 10. Determine the effect on the water pressure 200 m downstream of the treatment plant.

Solution.
According to the Darcy-Weisbach equation, the difference in total head, Δh, between the upstream section (at exit from treatment plant) and the downstream section (200 m downstream from the upstream section) is given by

$$\Delta h = \frac{fL}{D}\frac{V^2}{2g}$$

where f is the friction factor, L is the pipe length between the upstream and downstream sections (= 200 m), D is the pipe diameter (= 750 mm), and V is the velocity in the pipe. The velocity, V, is given by

$$V = \frac{Q}{A}$$

where Q is the flowrate in the pipe ($= 4.38$ m^3/s) and A is the area of the pipe cross-section given by

$$A = \frac{\pi}{4}D^2 = \frac{\pi}{4}(0.75)^2 = 0.442 \text{ m}^2$$

The pipeline velocity is therefore

$$V = \frac{Q}{A} = \frac{4.38}{0.442} = 9.91 \text{ m/s}$$

The friction factor, f, in the Darcy-Weisbach equation is calculated using the Colebrook equation

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{k_s}{3.7D} + \frac{2.51}{\mathrm{Re}\sqrt{f}}\right]$$

where Re is the Reynolds number and k_s is the equivalent sand roughness of ductile iron ($= 0.26$ mm). The Reynolds number is given by

$$\mathrm{Re} = \frac{VD}{\nu}$$

where ν is the kinematic viscosity of water at 20°C, which is equal to 1.00×10^{-6} m^2/s. Therefore

$$\mathrm{Re} = \frac{VD}{\nu} = \frac{(9.91)(0.75)}{1.00 \times 10^{-6}} = 7.43 \times 10^6$$

Substituting into the Colebrook equation leads to

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{0.26}{(3.7)(750)} + \frac{2.51}{7.43 \times 10^6\sqrt{f}}\right]$$

or

$$\frac{1}{\sqrt{f}} = -2\log\left[9.37 \times 10^{-5} + \frac{3.38 \times 10^{-7}}{\sqrt{f}}\right]$$

This is an implicit equation for f, and by trial and error the solution is

$$f = 0.016$$

The head loss, Δh, between the upstream and downstream sections can now be calculated using the Darcy-Weisbach equation as

$$\Delta h = \frac{fL}{D}\frac{V^2}{2g} = \frac{(0.016)(200)}{0.75}\frac{(9.91)^2}{(2)(9.81)} = 21.4 \text{ m}$$

Using the definition of head loss, Δh,

$$\Delta h = \left(\frac{p_1}{\gamma} + z_1\right) - \left(\frac{p_2}{\gamma} + z_2\right)$$

where p_1 and p_2 are the upstream and downstream pressures, γ is the specific weight of water, and z_1 and z_2 are the upstream and downstream pipe elevations. Since the pipe is horizontal, $z_1 = z_2$ and Δh can be written in terms of the pressures at the upstream and downstream sections as

$$\Delta h = \frac{p_1}{\gamma} - \frac{p_2}{\gamma}$$

In this case, $p_1 = 480$ kPa, $\gamma = 9.79$ kN/m^3, and therefore

$$21.4 = \frac{480}{9.79} - \frac{p_2}{9.79}$$

which yields

$$p_2 = 270 \text{ kPa}$$

Therefore, the pressure 200 m downstream of the treatment plant is 270 kPa. The Colebrook equation required that f be determined iteratively, but the explicit Jain approximation for f is given by

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{k_s}{3.7D} + \frac{5.74}{\mathrm{Re}^{0.9}}\right]$$

Substituting for k_s, D, and Re gives

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{0.26}{(3.7)(750)} + \frac{5.74}{(7.43\times 10^6)^{0.9}}\right]$$

which leads to

$$f = 0.016$$

This is the same friction factor obtained using the Colebrook equation within an accuracy of two significant digits.

After 20 years, the equivalent sand roughness, k_s, of the pipe is 2.6 mm, the (previously calculated) Reynolds number is 7.43×10^6, and the Colebrook equation gives

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{2.6}{(3.7)(750)} + \frac{2.51}{7.43\times 10^6\sqrt{f}}\right]$$

or

$$\frac{1}{\sqrt{f}} = -2\log\left[9.37\times 10^{-4} + \frac{3.38\times 10^{-7}}{\sqrt{f}}\right]$$

which yields

$$f = 0.027$$

The head loss, Δh, between the upstream and downstream sections is given by the Darcy-Weisbach equation as

$$\Delta h = \frac{fL}{D}\frac{V^2}{2g} = \frac{(0.027)(200)}{0.75}\frac{(9.91)^2}{(2)(9.81)} = 36.0 \text{ m}$$

Hence the pressure, p_2, 200 m downstream of the treatment plant is given by the relation

$$\Delta h = \frac{p_1}{\gamma} - \frac{p_2}{\gamma}$$

where $p_1 = 480$ kPa, $\gamma = 9.79$ kN/m^3, and therefore

$$36.0 = \frac{480}{9.79} - \frac{p_2}{9.79}$$

which yields

$$p_2 = 128 \text{ kPa}$$

Therefore, pipe aging over 20 years will cause the pressure 200 m downstream of the treatment plant to decrease from 270 kPa to 128 kPa. This is quite a significant drop and shows why velocities of 9.91 m/s are not used in these pipelines, even for short lengths of pipe.

The problem in Example 3.2 illustrates the case where the flowrate through a pipe is known and the objective is to calculate the head loss and pressure drop over a given length of pipe. The approach is summarized as follows: (1) calculate the Reynolds number, Re, and the relative roughness, k_s/D, from the given data; (2) use the Colebrook equation (Equation 2.35) or Jain equation (Equation 2.38) to calculate f; and (3) use the calculated value of f to calculate the head loss from the Darcy-Weisbach equation (Equation 2.33), and the corresponding pressure drop from Equation 2.22.

Flowrate for a Given Head Loss. In many cases, the flowrate through a pipe is not controlled but attains a level that matches the pressure drop available. For example, the flowrate through faucets in home plumbing is determined by the gage pressure in the water main, which is relatively insensitive to the flow through the faucet. A useful approach to this problem that utilizes the

Colebrook equation has been suggested by Fay (1994), where the first step is to calculate $\text{Re}\sqrt{f}$ using the rearranged Darcy-Weisbach equation

$$\text{Re}\sqrt{f} = \left(\frac{2gh_f D^3}{\nu^2 L}\right)^{\frac{1}{2}} \tag{2.40}$$

Using this value of $\text{Re}\sqrt{f}$, solve for Re using the rearranged Colebrook equation

$$\text{Re} = -2.0(\text{Re}\sqrt{f})\log\left(\frac{k_s/D}{3.7} + \frac{2.51}{\text{Re}\sqrt{f}}\right) \tag{2.41}$$

Using this value of Re, the flowrate, Q, can then be calculated by

$$Q = \frac{1}{4}\pi D^2 V = \frac{1}{4}\pi D\nu\text{Re} \tag{2.42}$$

This approach must necessarily be validated by verifying that Re > 2,300, which is required for the application of the Colebrook equation. Swamee and Jain (1976) combine Equations 2.40 to 2.42 to yield

$$\boxed{Q = -0.965D^2\sqrt{\frac{gDh_f}{L}}\ln\left(\frac{k_s/D}{3.7} + \frac{1.784\nu}{D\sqrt{gDh_f/L}}\right)} \tag{2.43}$$

Example 2.3.
A 50-mm diameter galvanized iron service pipe is connected to a water main in which the pressure is 450 kPa gage. If the length of the service pipe to a faucet is 40 m and the faucet is 1.2 m above the main, estimate the flowrate when the faucet is fully open.

Solution.
The head loss, h_f, in the pipe is estimated by

$$h_f = \left(\frac{p_{\text{main}}}{\gamma} + z_{\text{main}}\right) - \left(\frac{p_{\text{outlet}}}{\gamma} + z_{\text{outlet}}\right)$$

where $p_{\text{main}} = 450$ kPa, $z_{\text{main}} = 0$ m, $p_{\text{outlet}} = 0$ kPa, and $z_{\text{outlet}} = 1.2$ m. Therefore, taking $\gamma = 9.79$ kn/m^3 (at 20°C) gives

$$h_f = \left(\frac{450}{9.79} + 0\right) - (0 + 1.2) = 44.8 \text{ m}$$

Also, since $D = 50$ mm, $L = 40$ m, $k_s = 0.15$ mm (from Table 2.1), $\nu = 1.00 \times 10^{-6}$ m^2/s (at 20°C), the Swamee-Jain equation (Equation 2.43) yields

$$\begin{aligned}
Q &= -0.965D^2\sqrt{\frac{gDh_f}{L}}\ln\left(\frac{k_s/D}{3.7} + \frac{1.784\nu}{D\sqrt{gDh_f/L}}\right) \\
&= -0.965(0.05)^2\sqrt{\frac{(9.81)(0.05)(44.8)}{40}}\ln\left[\frac{0.15/50}{3.7} + \frac{1.784(1.00\times10^{-6})}{(0.05)\sqrt{(9.81)(0.05)(44.8)/40}}\right] \\
&= 0.0126 \text{ m}^3\text{/s} = 12.6 \text{ L/s}
\end{aligned}$$

The faucet can therefore be expected to deliver 12.6 L/s when fully open.

Diameter for a Given Flowrate and Head Loss. In many cases, an engineer must select a size of pipe to provide a given level of service. For example, the maximum flowrate and maximum allowable pressure drop may be specified for a water delivery pipe, and the engineer is required to calculate the minimum diameter pipe that will satisfy these design constraints. Solution of this problem necessarily requires an iterative procedure. Streeter and Wylie (1985) have suggested the following steps

1. Assume a value of f.

2. Calculate D from the rearranged Darcy-Weisbach equation,

$$D = \sqrt[5]{\left(\frac{8LQ^2}{h_f g \pi^2} f\right)} \tag{2.44}$$

   where the term in parentheses can be calculated from given data.

3. Calculate Re from

$$\mathrm{Re} = \frac{VD}{\nu} = \left(\frac{4Q}{\pi\nu}\right)\frac{1}{D} \tag{2.45}$$

   where the term in parentheses can be calculated from given data.

4. Calculate k_s/D.

5. Use Re and k_s/D to calculate f from the Colebrook equation.

6. Using the new f, repeat the procedure until the new f agrees with the old f to the first two significant digits.

Example 2.4.
A galvanized iron service pipe from a water main is required to deliver 200 L/s during a fire. If the length of the service pipe is 35 m and the head loss in the pipe is not to exceed 50 m, calculate the minimum pipe diameter that can be used.

Solution.

Step 1: Assume $f = 0.03$

Step 2: Since $Q = 0.2$ m^3/s, $L = 35$ m, and $h_f = 50$ m, then

$$D = \sqrt[5]{\left[\frac{8LQ^2}{h_f g \pi^2}\right] f} = \sqrt[5]{\left[\frac{8(35)(0.2)^2}{(50)(9.81)\pi^2}\right](0.03)} = 0.147 \text{ m}$$

Step 3: Since $\nu = 1.00 \times 10^{-6}$ m^2/s (at 20°C), then

$$\mathrm{Re} = \left[\frac{4Q}{\pi\nu}\right]\frac{1}{D} = \left[\frac{4(0.2)}{\pi(1.00 \times 10^{-6})}\right]\frac{1}{0.147} = 1.73 \times 10^6$$

Step 4: Since $k_s = 0.15$ mm (from Table 2.1, for new pipe), then

$$\frac{k_s}{D} = \frac{1.5 \times 10^{-4}}{0.147} = 0.00102$$

Step 5: Using the Colebrook equation (Equation 2.35) gives

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{k_s/D}{3.7} + \frac{2.51}{\mathrm{Re}\sqrt{f}}\right) = -2\log\left(\frac{0.00102}{3.7} + \frac{2.51}{1.73\times 10^6\sqrt{f}}\right)$$

which leads to

$$f = 0.020$$

Step 6: $f = 0.020$ differs from the assumed f $(= 0.03)$, so repeat the procedure with $f = 0.020$.

Step 2: For $f = 0.020$, $D = 0.136$ m

Step 3: For $D = 0.136$, $\mathrm{Re} = 1.87 \times 10^6$

Step 4: For $D = 0.136$, $k_s/D = 0.00110$

Step 5: $f = 0.020$

Step 6: The calculated f $(= 0.020)$ is equal to the assumed f. The required pipe diameter is therefore equal to 0.136 m or 136 mm. A commercially available pipe with the closest diameter larger than 136 mm should be used.

The iterative procedure demonstrated in the previous example converges fairly quickly, and does not pose any computational difficulty. Swamee and Jain (1976) have suggested the following explicit formula for calculating the pipe diameter, D,

$$\boxed{\begin{array}{c} D = 0.66\left[k_s^{1.25}\left(\dfrac{LQ^2}{gh_f}\right)^{4.75} + \nu Q^{9.4}\left(\dfrac{L}{gh_f}\right)^{5.2}\right]^{0.04}, \\ 3{,}000 \leq \mathrm{Re} \leq 3\times 10^8,\; 10^{-6} \leq \dfrac{k_s}{D} \leq 2\times 10^{-2} \end{array}} \tag{2.46}$$

Equation 2.46 will yield a D within 5% of the value obtained by the method using the Colebrook equation. This method is illustrated by repeating the previous example.

Example 2.5.
A galvanized iron service pipe from a water main is required to deliver 200 L/s during a fire. If the length of the service pipe is 35 m, and the head loss in the pipe is not to exceed 50 m, use the Swamee-Jain equation to calculate the minimum pipe diameter that can be used.

Solution.
Since $k_s = 0.15$ mm, $L = 35$ m, $Q = 0.2$ m^3/s, $h_f = 50$ m, $\nu = 1.00 \times 10^{-6}$ m^2/s, the Swamee-Jain equation gives

$$\begin{aligned} D &= 0.66\left[k_s^{1.25}\left(\frac{LQ^2}{gh_f}\right)^{4.75} + \nu Q^{9.4}\left(\frac{L}{gh_f}\right)^{5.2}\right]^{0.04} \\ &= 0.66\left\{(0.00015)^{1.25}\left[\frac{(35)(0.2)^2}{(9.81)(50)}\right]^{4.75} + (1.00\times 10^{-6})(0.2)^{9.4}\left[\frac{35}{(9.81)(50)}\right]^{5.2}\right\}^{0.04} \\ &= 0.140 \text{ m} \end{aligned}$$

The calculated pipe diameter (140 mm) is about 3% higher than calculated by the Colebrook equation (136 mm).

2.2.3 Energy Equation

The steady-state energy equation for the control volume illustrated in Figure 2.4 is given by

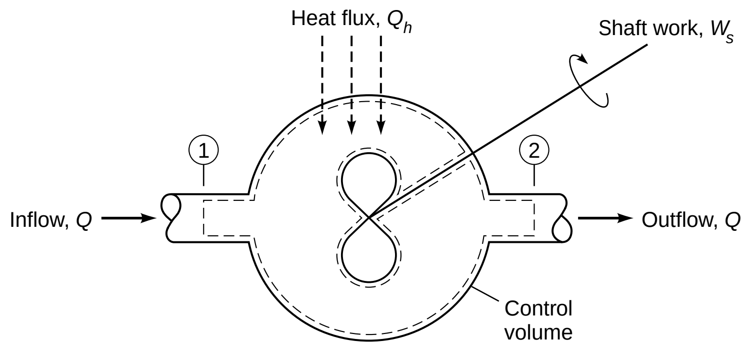


Figure 2.4: Energy Balance in Closed Conduit

$$\frac{dQ_h}{dt} - \frac{dW}{dt} = \int_A \rho e \; \mathbf{v} \cdot \mathbf{n} \; dA \tag{2.47}$$

where Q_h is the heat added to the fluid in the control volume, W is the work done by the fluid in the control volume, A is the surface area of the control volume, ρ is the density of the fluid in the control volume, and e is the internal energy per unit mass of fluid in the control volume given by

$$e = gz + \frac{v^2}{2} + u \tag{2.48}$$

where z is the elevation of the fluid mass having a velocity v and internal energy u. By convention, the heat added to a system and the work done by a system are positive quantities. The normal stresses on the inflow and outflow boundaries of the control volume are equal to the pressure, p, with shear stresses tangential to the boundaries of the control volume. As the fluid moves across the control surface with velocity $\mathbf{v}$, the power (= rate of doing work) expended by the fluid against the external pressure forces is given by

$$\frac{dW_p}{dt} = \int_A p\mathbf{v} \cdot \mathbf{n} \; dA \tag{2.49}$$

where W_p is the work done against external pressure forces. The work done by a fluid in the control volume is typically separated into work done against external pressure forces, W_p, plus work done against rotating surfaces, W_s, commonly referred to as the *shaft work*. The rotating element is called a *rotor* in a gas or steam turbine, an *impeller* in a pump, and a *runner* in a hydraulic turbine. The rate at which work is done by a fluid system, dW/dt, can therefore be written as

$$\frac{dW}{dt} = \frac{dW_p}{dt} + \frac{dW_s}{dt} = \int_A p\mathbf{v} \cdot \mathbf{n} \; dA + \frac{dW_s}{dt} \tag{2.50}$$

Combining Equation 2.50 with the steady-state energy equation (Equation 2.47) leads to

$$\frac{dQ_h}{dt} - \frac{dW_s}{dt} = \int_A \rho \left(\frac{p}{\rho} + e \right) \mathbf{v} \cdot \mathbf{n} \; dA \tag{2.51}$$

Substituting the definition of the internal energy, e, given by Equation 2.48 into Equation 2.51 yields

$$\frac{dQ_h}{dt} - \frac{dW_s}{dt} = \int_A \rho \left(h + gz + \frac{v^2}{2} \right) \mathbf{v} \cdot \mathbf{n} \; dA \tag{2.52}$$

where h is the enthalpy of the fluid defined by

$$h = \frac{p}{\rho} + u \tag{2.53}$$

Denoting the rate at which heat is being added to the fluid system by $\dot{Q}$, and the rate at which work is being done against moving impervious boundaries (shaft work) by $\dot{W}_s$, then the energy equation can be written in the form

$$\dot{Q} - \dot{W}_s = \int_A \rho\left(h + gz + \frac{v^2}{2}\right) \mathbf{v}\cdot\mathbf{n}\, dA \tag{2.54}$$

Considering the terms $h + gz$, where

$$h + gz = \frac{p}{\rho} + u + gz = g\left(\frac{p}{\gamma} + z\right) + u \tag{2.55}$$

and γ is the specific weight of the fluid, then Equation 2.55 indicates that $h + gz$ can be assumed to be constant across the inflow and outflow openings illustrated in Figure 2.4, since a hydrostatic pressure distribution across the inflow/outflow boundaries guarantees that $p/\gamma + z$ is constant across the inflow/outflow boundaries normal to the flow direction, and the internal energy, u, depends only on the temperature, which can be assumed constant across each boundary. Since $\mathbf{v}\cdot\mathbf{n}$ is equal to zero over the impervious boundaries in contact with the fluid system, Equation 2.54 can be integrated to yield

$$\begin{aligned}
\dot{Q} - \dot{W}_s &= (h_1 + gz_1)\int_{A_1} \rho\mathbf{v}\cdot\mathbf{n}\, dA + \int_{A_1} \rho\frac{v^2}{2}\mathbf{v}\cdot\mathbf{n}\, dA + (h_2 + gz_2)\int_{A_2} \rho\mathbf{v}\cdot\mathbf{n}\, dA \\
&\quad + \int_{A_2} \rho\frac{v^2}{2}\mathbf{v}\cdot\mathbf{n}\, dA \\
&= -(h_1 + gz_1)\int_{A_1} \rho v_1\, dA - \int_{A_1} \rho\frac{v_1^3}{2}\, dA + (h_2 + gz_2)\int_{A_2} \rho v_2\, dA \\
&\quad + \int_{A_2} \rho\frac{v_2^3}{2}\, dA
\end{aligned} \tag{2.56}$$

where the subscripts 1 and 2 refer to the inflow and outflow boundaries, respectively, and the negative signs result from the fact that the unit normal points out of the control volume, causing $\mathbf{v}\cdot\mathbf{n}$ to be negative on the inflow boundary and positive on the outflow boundary.

Equation 2.56 can be simplified by noting that the assumption of steady flow requires that rate of mass inflow to the control volume is equal to the mass outflow rate and, denoting the mass flow rate by $\dot{m}$, the continuity equation requires that

$$\dot{m} = \int_{A_1} \rho v_1\, dA = \int_{A_2} \rho v_2\, dA \tag{2.57}$$

Furthermore, the constants α_1 and α_2 can be defined by the equations

$$\int_{A_1} \rho\frac{v^3}{2}\, dA = \alpha_1\rho\frac{V_1^3}{2}A_1 \tag{2.58}$$

$$\int_{A_2} \rho\frac{v^3}{2}\, dA = \alpha_2\rho\frac{V_2^3}{2}A_2 \tag{2.59}$$

where A_1 and A_2 are the areas of the inflow and outflow boundaries, respectively, and V_1 and V_2 are the corresponding mean velocities across these boundaries. The constants α_1 and α_2 are determined by the velocity profile across the flow boundaries, and these constants are called *kinetic energy correction factors*. If the velocity is constant across a flow boundary, then it is clear from Equation 2.58 that the kinetic energy correction factor for that boundary is equal to unity; for any other velocity distribution, the kinetic energy factor is greater than unity. Combining Equations 2.56 to 2.59 leads to

$$\dot{Q} - \dot{W}_s = -(h_1 + gz_1)\dot{m} - \alpha_1 \rho \frac{V_1^3}{2} A_1 + (h_2 + gz_2)\dot{m} + \alpha_2 \rho \frac{V_2^3}{2} A_2 \tag{2.60}$$

Invoking the continuity equation requires that

$$\rho V_1 A_1 = \rho V_2 A_2 = \dot{m} \tag{2.61}$$

and combining Equations 2.60 and 2.61 leads to

$$\dot{Q} - \dot{W}_s = \dot{m} \left[\left(h_2 + gz_2 + \alpha_2 \frac{V_2^2}{2} \right) - \left(h_1 + gz_1 + \alpha_1 \frac{V_1^2}{2} \right) \right] \tag{2.62}$$

which can be put in the form

$$\frac{\dot{Q}}{\dot{m}g} - \frac{\dot{W}_s}{\dot{m}g} = \left(\frac{p_2}{\gamma} + \frac{u_2}{g} + z_2 + \alpha_2 \frac{V_2^2}{2g} \right) - \left(\frac{p_1}{\gamma} + \frac{u_1}{g} + z_1 + \alpha_1 \frac{V_1^2}{2g} \right) \tag{2.63}$$

and can be further rearranged into the useful form

$$\left(\frac{p_1}{\gamma} + \alpha_1 \frac{V_1^2}{2g} + z_1 \right) = \left(\frac{p_2}{\gamma} + \alpha_2 \frac{V_2^2}{2g} + z_2 \right) + \left[\frac{1}{g}(u_2 - u_1) - \frac{\dot{Q}}{\dot{m}g} \right] + \left[\frac{\dot{W}_s}{\dot{m}g} \right] \tag{2.64}$$

Two key terms can be identified in Equation 2.64: the (shaft) work done by the fluid per unit weight, h_s, defined by the relation

$$h_s = \frac{\dot{W}_s}{\dot{m}g} \tag{2.65}$$

and the energy loss per unit weight, commonly called the head loss, h_L, defined by the relation

$$h_L = \frac{1}{g}(u_2 - u_1) - \frac{\dot{Q}}{\dot{m}g} \tag{2.66}$$

Combining Equations 2.64 to 2.66 leads to the most common form of the *energy equation*

$$\boxed{\left(\frac{p_1}{\gamma} + \alpha_1 \frac{V_1^2}{2g} + z_1 \right) = \left(\frac{p_2}{\gamma} + \alpha_2 \frac{V_2^2}{2g} + z_2 \right) + h_L + h_s} \tag{2.67}$$

where a positive head loss indicates an increase in internal energy, manifested by an increase in temperature or a loss of heat, and a positive value of h_s is associated with work being done by the fluid, such as in moving a turbine runner. Many practitioners incorrectly refer to Equation 2.67 as the *Bernoulli equation*, which bears some resemblance to Equation 2.67 but is different in several important respects. Fundamental differences between the energy equation and the Bernoulli equation are that the Bernoulli equation is derived from the momentum equation, which is independent of the energy equation, and the Bernoulli equation does not account for fluid friction.

Energy and Hydraulic Grade Lines. The *total head*, h, of a fluid at any cross-section of a pipe is defined by

$$\boxed{h = \frac{p}{\gamma} + \alpha\frac{V^2}{2g} + z} \tag{2.68}$$

where p is the pressure in the fluid at the centroid of the cross-section, γ is the specific weight of the fluid, α is the kinetic energy correction factor, V is the average velocity across the pipe, and z is the elevation of the centroid of the pipe. The total head measures the average energy per unit weight of the fluid flowing across a pipe cross-section. The energy equation, Equation 2.67, states that changes in the total head along the pipe are described by

$$h(x + \Delta x) = h(x) - (h_L + h_s) \tag{2.69}$$

where x is the coordinate measured along the pipe centerline, Δx is the distance between two cross-sections in the pipe, h_L is the head loss, and h_s is the shaft work done by the fluid over the distance Δx. The practical application of Equation 2.69 is illustrated in Figure 2.5, where the head loss, h_L, between two sections a distance Δx apart is indicated. At each cross-section, the total

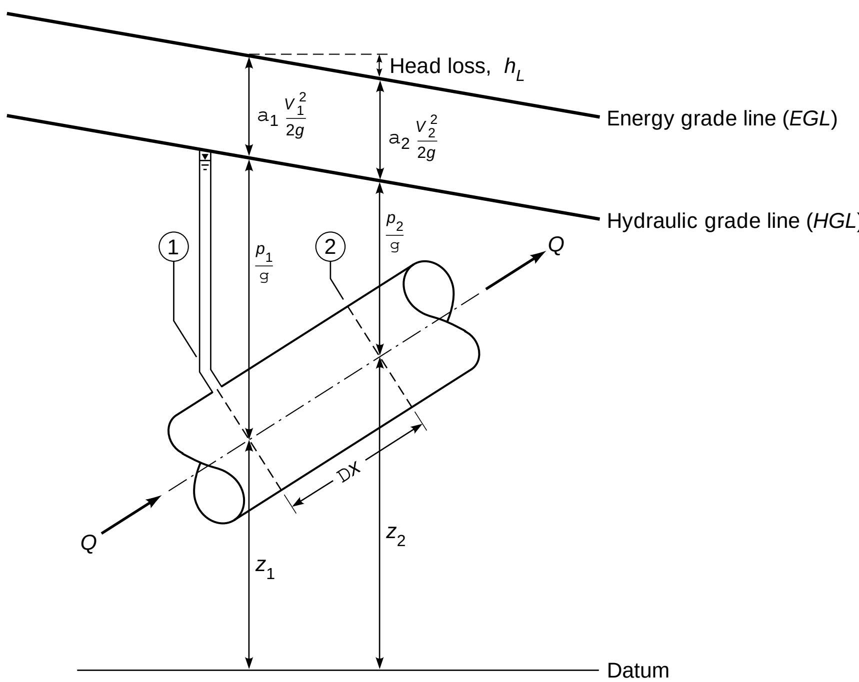


Figure 2.5: Head Loss Along Pipe

energy, h, is plotted relative to a defined datum, and the locus of these points is called the *energy grade line*. The energy grade line at each pipe cross-section is located a distance $p/\gamma + \alpha V^2/2g$ vertically above the centroid of the cross-section, and between any two cross-sections the elevation of the energy grade line falls by a vertical distance equal to the head loss caused by pipe friction, h_L, plus the shaft work, h_s, done by the fluid. The *hydraulic grade line* measures the hydraulic head $p/\gamma + z$ at each pipe cross-section. It is located a distance p/γ above the pipe centerline and indicates the elevation to which the fluid would rise in an open tube connected to the wall of the pipe section. The hydraulic grade line is therefore located a distance $\alpha V^2/2g$ below the energy grade line. In most water-supply applications the velocity heads are negligible and the hydraulic grade line closely approximates the energy grade line.

Both the hydraulic grade line and the energy grade line are useful in visualizing the state of the fluid as it flows along the pipe and are frequently used in assessing the performance of fluid delivery systems. Most fluid delivery systems, for example, require that the fluid pressure remain positive, in which case the hydraulic grade line must remain above the pipe. In circumstances where additional energy is required to maintain acceptable pressures in pipelines, a pump is installed along the pipeline to elevate the energy grade line by an amount h_s, which also elevates the hydraulic grade line by the same amount. This condition is illustrated in Figure 2.6. In cases where the

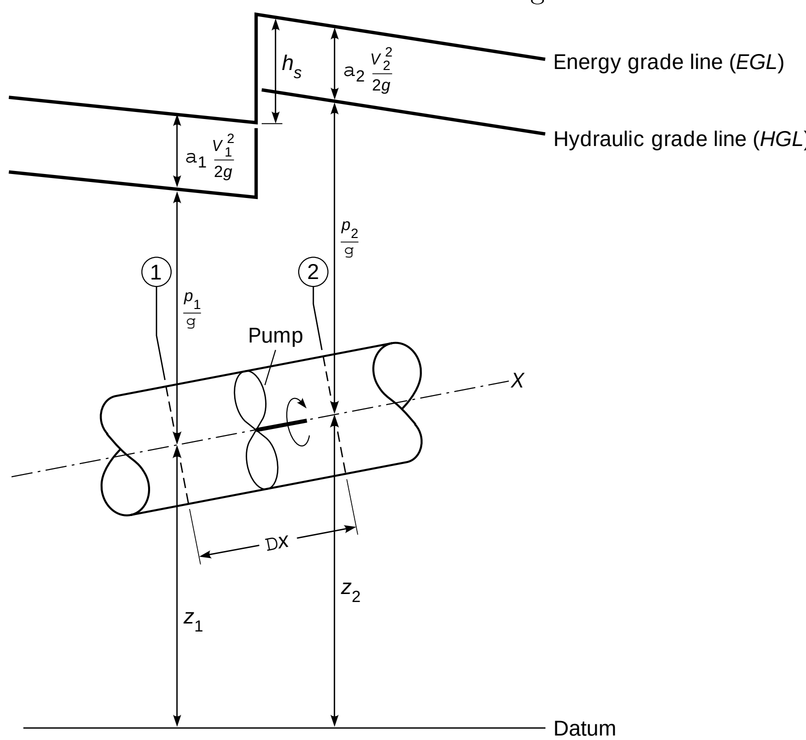


Figure 2.6: Pump Effect on Flow in Pipeline

pipeline upstream and downstream of the pump are of the same diameter, then the velocity heads $\alpha V^2/2g$ both upstream and downstream of the pump are the same, and the head added by the pump, h_s, goes entirely to increase the pressure head, p/γ, of the fluid. It should also be clear from Figure 2.5 that the pressure head in a pipeline can be increased by simply increasing the pipeline diameter, which reduces the velocity head, $\alpha V^2/2g$, and thereby increases the pressure head, p/γ, to maintain the same total energy at the pipe section.

Velocity Profile. The momentum and energy correction factors, α and β, depend on the cross-sectional velocity distribution. The velocity profile in both smooth and rough pipes of circular cross-section can be estimated by the semi-empirical equation

$$\boxed{v(r) = \left[(1 + 1.326\sqrt{f}) - 2.04\sqrt{f}\log\left(\frac{R}{R-r}\right)\right] V} \tag{2.70}$$

where $v(r)$ is the velocity at a radial distance r from the centerline of the pipe, R is the radius of the pipe, f is the friction factor, and V is the average velocity across the pipe.

The velocity distribution given by Equation 2.70 agrees well with velocity measurements in both smooth and rough pipes. This equation, however, is not applicable within the small region close to the centerline of the pipe and is also not applicable in the small region close to the pipe boundary. This is apparent since at the axis of the pipe dv/dr must be equal to zero, but Equation 2.70 does not have a zero slope at $r = 0$. The pipe boundary v must also be equal to zero, but Equation 2.70 gives a velocity of zero at a small distance from the wall, with a velocity of $-\infty$ at $r = R$. The energy and momentum correction factors, α and β, derived from the velocity profile are (Moody, 1950)

$$\alpha = 1 + 2.7f \tag{2.71}$$
$$\beta = 1 + 0.98f \tag{2.72}$$

Another commonly used equation to describe the velocity distribution in turbulent pipe flow is the empirical *power law* equation given by

$$\boxed{v(r) = V_o\left(1 - \frac{r}{R}\right)^{\frac{1}{n}}} \tag{2.73}$$

where V_o is the centerline velocity and n is a function of the Reynolds number, Re. Values of n typically range between 6 and 10 and can be approximated by (Fox and McDonald, 1992; Schlichting, 1979)

$$n = 1.83 \log \mathrm{Re} - 1.86 \tag{2.74}$$

The power law is not applicable within $0.04R$ of the wall, since the power law gives an infinite velocity gradient at the wall. Although the profile fits the data close to the centerline of the pipe, it does not give zero slope at the centerline. The kinetic energy coefficient, α, derived from the power law equation is given by

$$\alpha = \frac{(1+n)^3(1+2n)^3}{4n^4(3+n)(3+2n)} \tag{2.75}$$

For n between 6 and 10, α varies from 1.08 to 1.03. In most engineering applications, α and β are taken as unity (see Problem 2.14).

Head Losses in Transitions and Fittings. The head losses in straight pipes of constant diameter are caused by friction between the moving fluid and the pipe boundary and are estimated using the Darcy-Weisbach equation. Flow through pipe fittings, around bends, and through changes in pipeline geometry cause additional head losses, h_o, that are quantified by an equation of the form

$$\boxed{h_o = \sum K \frac{V^2}{2g}} \tag{2.76}$$

where K is a loss coefficient that is specific to each fitting and transition and V is the average velocity at a defined location within the transition or fitting. The loss coefficients for several fittings and transitions are shown in Figure 2.7. Head losses in transitions and fittings are also called *local head losses* or *minor head losses*. The latter term should be avoided, however, since in some cases

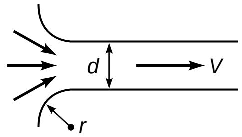
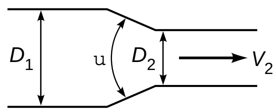
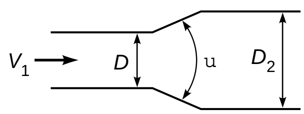
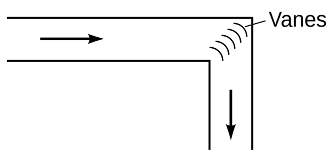
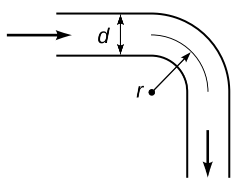
Description	Sketch	Additional Data	K	K
Pipe entrance		*r/d*	*K*	
		0.0	0.50	
		0.1	0.12	
		>0.2	0.03	
Contraction		D_2/D_1	*K* u 5 60°	*K* u 5 180°
		0.0	0.08	0.50
		0.20	0.08	0.49
		0.40	0.07	0.42
		0.60	0.06	0.27
		0.80	0.06	0.20
		0.90	0.06	0.10
Expansion		D_1/D_2	*K* u 5 20°	*K* u 5 180°
		0.0		1.00
		0.20	0.30	0.87
		0.40	0.25	0.70
		0.60	0.15	0.41
		0.80	0.10	0.15
90° miter bend		Without vanes	*K* 5 1.1	
		With vanes	*K* 5 0.2	
90° smooth bend		*r/d*	*K* u 5 90°	
		1	0.35	
		2	0.19	
		4	0.16	
		6	0.21	
Threaded pipe fittings			*K*	
	Globe valve — wide open		10.0	
	Angle valve — wide open		5.0	
	Gate valve — wide open		0.2	
	Gate valve — half open		5.6	
	Return bend		2.2	
	Tee			
	straight-through flow		0.4	
	side-outlet flow		1.8	
	90° elbow		0.9	
	45° elbow		0.4	

Figure 2.7: Loss Coefficients for Transitions and Fittings

Source: Roberson, John A. and Crowe, Clayton T., *Engineering Fluid Mechanics*. Copyright © 1997 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

these head losses are a significant portion of the total head loss in a pipe. Detailed descriptions of local head losses in various valve geometries can be found in Mott (1994), and additional data on local head losses in pipeline systems can be found in Brater and colleagues (1996).

Example 2.6.
A pump is to be selected that will pump water from a well into a storage reservoir. In order to fill the reservoir in a timely manner, the pump is required to deliver 5 L/s when the water level in the reservoir is 5 m above the water level in the well. Find the head that must be added by the pump. The pipeline system is shown in Figure 2.8. Assume

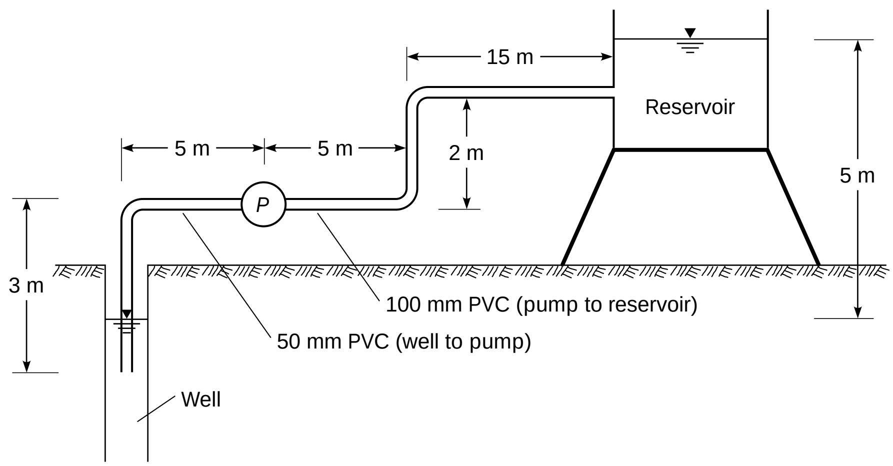


Figure 2.8: Pipeline System

that the minor loss coefficient for each of the bends is equal to 0.25 and that the temperature of the water is 20°C.

Solution.
Taking the elevation of the water surface in the well to be equal to 0 m, and proceeding from the well to the storage reservoir (where the head is equal to 5 m), the energy equation (Equation 2.67) can be written as

$$0 - \frac{V_1^2}{2g} - \frac{f_1 L_1}{D_1}\frac{V_1^2}{2g} - K_1\frac{V_1^2}{2g} + h_p - \frac{f_2 L_2}{D_2}\frac{V_2^2}{2g} - (K_2 + K_3)\frac{V_2^2}{2g} - \frac{V_2^2}{2g} = 5$$

where V_1 and V_2 are the velocities in the 50-mm ($= D_1$) and 100-mm ($= D_2$) pipes, respectively; L_1 and L_2 are the corresponding pipe lengths; f_1 and f_2 are the corresponding friction factors; K_1, K_2, and K_3 are the loss coefficients for each of the three bends; and h_p is the head added by the pump. The cross-sectional areas of each of the pipes, A_1 and A_2, are given by

$$\begin{aligned} A_1 &= \frac{\pi}{4}D_1^2 = \frac{\pi}{4}(0.05)^2 = 0.001963 \text{ m}^2 \\ A_2 &= \frac{\pi}{4}D_2^2 = \frac{\pi}{4}(0.10)^2 = 0.007854 \text{ m}^2 \end{aligned}$$

When the flowrate, Q, is 5 L/s, the velocities V_1 and V_2 are given by

$$\begin{aligned} V_1 &= \frac{Q}{A_1} = \frac{0.005}{0.001963} = 2.54 \text{ m/s} \\ V_2 &= \frac{Q}{A_2} = \frac{0.005}{0.007854} = 0.637 \text{ m/s} \end{aligned}$$

PVC pipe is considered smooth ($k_s \approx 0$) and therefore the friction factor, f, can be estimated using the Jain equation

$$f = \frac{0.25}{\left[\log_{10} \frac{5.74}{\text{Re}^{0.9}}\right]^2}$$

where Re is the Reynolds number. At 20°C, the kinematic viscosity, ν, is equal to 1.00×10^{-6} m^2/s and for the 50-mm pipe

$$\mathrm{Re}_1 = \frac{V_1 D_1}{\nu} = \frac{(2.51)(0.05)}{1.00 \times 10^{-6}} = 1.27 \times 10^5$$

which leads to

$$f_1 = \frac{0.25}{\left[\log_{10} \dfrac{5.74}{(1.27 \times 10^5)^{0.9}}\right]^2} = 0.0170$$

and for the 100-mm pipe

$$\mathrm{Re}_2 = \frac{V_2 D_2}{\nu} = \frac{(0.637)(0.10)}{1.00 \times 10^{-6}} = 6.37 \times 10^4$$

which leads to

$$f_2 = \frac{0.25}{\left[\log_{10} \dfrac{5.74}{(6.37 \times 10^4)^{0.9}}\right]^2} = 0.0197$$

Substituting the values of these parameters into the energy equation yields

$$\begin{aligned} 0 \quad & - \quad \left[1 + \frac{(0.0170)(8)}{0.05} + 0.25\right] \frac{2.54^2}{(2)(9.81)} + h_p \\ & - \quad \left[\frac{(0.0197)(22)}{0.10} + 0.25 + 0.25 + 1\right] \frac{0.637^2}{(2)(9.81)} = 5 \end{aligned}$$

which leads to

$$h_p = 6.43 \text{ m}$$

Therefore the head to be added by the pump is 6.43 m.

Minor losses are frequently neglected in the analysis of pipeline systems. As a general rule, neglecting minor losses is justified when, on average, there is a length of 1,000 diameters between each minor loss (Streeter et al., 1998).

Head Losses in Noncircular Conduits. Most pipelines are of circular cross-section, but flow of water in noncircular conduits is commonly encountered in cases such as rectangular culverts flowing full. The hydraulic radius, R, of a conduit of any shape is defined by the relation

$$R = \frac{A}{P} \tag{2.77}$$

where A is the cross-sectional area of the conduit and P is the wetted perimeter. For circular conduits of diameter D, the hydraulic radius is given by

$$R = \frac{\pi D^2/4}{\pi D} = \frac{D}{4} \tag{2.78}$$

or

$$D = 4R \tag{2.79}$$

Using the hydraulic radius, R, as the length scale of a closed conduit instead of D, the frictional head losses, h_f, in noncircular conduits can be estimated using the Darcy-Weisbach equation for circular conduits by simply replacing D by $4R$, which yields

$$\boxed{h_f = \frac{fL}{4R}\frac{V^2}{2g}} \tag{2.80}$$

where the friction factor, f, is calculated using a Reynolds number, Re, defined by

$$\mathrm{Re} = \frac{\rho V(4R)}{\mu} \tag{2.81}$$

and a relative roughness defined by $k_s/4R$.

Characterizing a noncircular conduit by the hydraulic radius, R, is necessarily approximate since conduits of arbitrary cross-section cannot be described with a single parameter. Secondary currents that are generated across a noncircular conduit cross-section to redistribute the shears are another reason why noncircular conduits cannot be completely characterized by the hydraulic radius (Liggett, 1994). However, according to Munson and colleagues (1994) and White (1994), using the hydraulic radius as a basis for calculating frictional head losses in noncircular conduits is usually accurate to within 15% for turbulent flow. This approximation is much less accurate for laminar flows, where the accuracy is on the order of ±40% (White, 1994). Olson and Wright (1990) state that this approach can be used for rectangular conduits where the ratio of sides, called the *aspect ratio*, does not exceed about 8. Potter and Wiggert (2001) state that aspect ratios must be less than 4:1.

Example 2.7.
Water flows through a rectangular concrete culvert of width 2 m and depth 1 m. If the length of the culvert is 10 m and the flowrate is 6 m^3/s, estimate the head loss through the culvert. Assume that the culvert flows full.

Solution.
The head loss can be calculated using Equation 2.80. The hydraulic radius, R, is given by

$$R = \frac{A}{P} = \frac{(2)(1)}{2(2+1)} = 0.333 \text{ m}$$

and the mean velocity, V, is given by

$$V = \frac{Q}{A} = \frac{6}{(2)(1)} = 3 \text{ m/s}$$

At 20°C, $\nu = 1.00 \times 10^{-6}$ m^2/s, and therefore the Reynolds number, Re, is given by

$$\mathrm{Re} = \frac{V(4R)}{\nu} = \frac{(3)(4 \times 0.333)}{1.00 \times 10^{-6}} = 4.00 \times 10^6$$

A median equivalent sand roughness for concrete can be taken as $k_s = 1.6$ mm (Table 2.1), and therefore the relative roughness, $k_s/4R$, is given by

$$\frac{k_s}{4R} = \frac{1.6 \times 10^{-3}}{4(0.333)} = 0.00120$$

Substituting Re and $k_s/4R$ into the Jain equation (Equation 2.38) for the friction factor yields

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{k_s/4R}{3.7} + \frac{5.74}{\mathrm{Re}^{0.9}}\right] = -2\log\left[\frac{0.00120}{3.7} + \frac{5.74}{(4.00 \times 10^6)^{0.9}}\right] = 6.96$$

which yields

$$f = 0.0206$$

The frictional head loss in the culvert, h_f, is therefore given by the Darcy-Weisbach equation as

$$h_f = \frac{fL}{4R}\frac{V^2}{2g} = \frac{(0.0206)(10)}{(4 \times 0.333)}\frac{3^2}{2(9.81)} = 0.0709 \text{ m}$$

The head loss in the culvert can therefore be estimated as 7.1 cm.

Empirical Friction-Loss Formulae. Friction losses in pipelines should generally be calculated using the Darcy-Weisbach equation. However, a minor inconvenience in using the Darcy-Weisbach equation to relate the friction loss to the flow velocity results from the dependence of the friction factor on the flow velocity; therefore, the Darcy-Weisbach equation must be solved simultaneously with the Colebrook equation. In modern engineering practice, computer hardware and software make this a very minor inconvenience. In earlier years, however, this was considered a real problem, and various empirical head-loss formulae were developed to relate the head loss directly to the flow velocity. The most commonly used empirical formulae are the *Hazen-Williams formula* and the *Manning formula*.

The *Hazen-Williams formula* (Williams and Hazen, 1920) is applicable only to the flow of water in pipes and is given by

$$\boxed{V = 0.849 C_H R^{0.63} S_f^{0.54}} \tag{2.82}$$

where V is the flow velocity (in m/s), C_H is the Hazen-Williams roughness coefficient, R is the hydraulic radius (in m), and S_f is the slope of the energy grade line, defined by

$$S_f = \frac{h_f}{L} \tag{2.83}$$

where h_f is the head loss due to friction over a length L of pipe. Values of C_H for a variety of commonly used pipe materials are given in Table 2.2. Solving Equations 2.82 and 2.83 yields the

Table 2.2: Pipe Roughness Coefficients

	C_H		n	
Pipe material	Range	Typical	Range	Typical
Ductile and cast iron:				
New, unlined	120–140	130	—	0.013
Old, unlined	40–100	80	—	0.025
Cement lined and seal coated	100–140	120	0.011–0.015	0.013
Steel:				
Welded and seamless	80–150	120	—	0.012
Riveted	—	110	0.012–0.018	0.015
Mortar lining	120 – 145	130	—	—
Asbestos cement		140	—	0.011
Concrete	100–140	120	0.011–0.015	0.012
Vitrified clay pipe (VCP)	—	110	0.012–0.014	—
Polyvinyl chloride (PVC)	135 – 150	140	0.007–0.011	0.009
Corrugated metal pipe (CMP)	—	—	—	0.025

Source: Velon and Johnson (1993); Wurbs and James (2002).

following expression for the frictional head loss,

$$h_f = 6.82 \frac{L}{D^{1.17}} \left(\frac{V}{C_H} \right)^{1.85} \tag{2.84}$$

where D is the diameter of the pipe. The Hazen-Williams equation is applicable to the flow of water at 16°C in pipes with diameters between 50 mm and 1850 mm, and flow velocities less than 3 m/s (Mott, 1994). Outside of these conditions, use of the Hazen-Williams equation is strongly discouraged. To further support these quantitative limitations, Street and colleagues (1996) and Liou (1998) have shown that the Hazen-Williams coefficient has a strong Reynolds number dependence, and is mostly applicable where the pipe is relatively smooth and in the early part of its transition to rough flow. Furthermore, Jain and colleagues (1978) have shown that an error of up to 39% can be expected in the evaluation of the velocity by the Hazen-Williams formula over a wide range of diameters and slopes. In spite of these cautionary notes, the Hazen-Williams formula is frequently used in the United States for the design of large water-supply pipes without regard to its limited range of applicability, a practice that can have very detrimental effects on pipe design and could potentially lead to litigation (Bombardelli and García, 2003).

A second empirical formula that is sometimes used to describe flow in pipes is the Manning formula, which is given by

$$\boxed{V = \frac{1}{n} R^{\frac{2}{3}} S_f^{\frac{1}{2}}} \tag{2.85}$$

where V, R, and S_f have the same meaning and units as in the Hazen-Williams formula, and n is the Manning roughness coefficient. Values of n for a variety of commonly used pipe materials are given in Table 2.2 (Velon and Johnson, 1993). Solving Equations 2.85 and 2.83 yields the following expression for the frictional head loss

$$h_f = 6.35 \frac{n^2 L V^2}{D^{\frac{4}{3}}} \tag{2.86}$$

The Manning formula applies to fully turbulent flows, where the frictional head losses depend primarily on the relative roughness.

Example 2.8.

Water flows at a velocity of 1 m/s in a 150 mm new ductile iron pipe. Estimate the head loss over 500 m using: (a) the Hazen-Williams formula, (b) the Manning formula, and (c) the Darcy-Weisbach equation. Compare your results.

Solution.

(a) The Hazen-Williams roughness coefficient, C_H, can be taken as 130 (Table 2.2), L = 500 m, D = 0.150 m, V = 1 m/s, and therefore the head loss, h_f, is given by Equation 2.84 as

$$h_f = 6.82 \frac{L}{D^{1.17}} \left(\frac{V}{C_H}\right)^{1.85} = 6.82 \frac{500}{(0.15)^{1.17}} \left(\frac{1}{130}\right)^{1.85} = 3.85 \text{ m}$$

(b) The Manning roughness coefficient, n, can be taken as 0.013 (approximation from Table 2.2), and therefore the head loss, h_f, is given by Equation 2.86 as

$$h_f = 6.35 \frac{n^2 L V^2}{D^{\frac{4}{3}}} = 6.35 \frac{(0.013)^2 (500)(1)^2}{(0.15)^{\frac{4}{3}}} = 6.73 \text{ m}$$

(c) The equivalent sand roughness, k_s, can be taken as 0.26 mm (Table 2.1), and the Reynolds number, Re, is given by

$$\text{Re} = \frac{VD}{\nu} = \frac{(1)(0.15)}{1.00 \times 10^{-6}} = 1.5 \times 10^5$$

where $\nu = 1.00 \times 10^{-6}$ m^2/s at 20°C. Substituting k_s, D, and Re into the Colebrook equation yields the friction factor, f, where

$$\frac{1}{\sqrt{f}} = -2 \log \left[\frac{k_s}{3.7D} + \frac{2.51}{\text{Re}\sqrt{f}}\right] = -2 \log \left[\frac{0.26}{3.7(150)} + \frac{2.51}{1.5 \times 10^5 \sqrt{f}}\right]$$

Solving by trial and error leads to

$$f = 0.0238$$

The head loss, h_f, is therefore given by the Darcy-Weisbach equation as

$$h_f = f\frac{L}{D}\frac{V^2}{2g} = 0.0238\frac{500}{0.15}\frac{1^2}{2(9.81)} = 4.04 \text{ m}$$

It is reasonable to assume that the Darcy-Weisbach equation yields the most accurate estimate of the head loss. In this case, the Hazen-Williams formula gives a head loss that is 5% too low and the Manning formula yields a head loss that is 67% too high.

2.3 Pipe Networks

Pipe networks are commonly encountered in the context of water-distribution systems. The performance criteria of these systems are typically specified in terms of minimum flow rates and pressure heads that must be maintained at the specified points in the network. Analyses of pipe networks are usually within the context of (1) designing a new network, (2) designing a modification to an existing network, and/or (3) evaluating the reliability of a network. The procedure for analyzing a pipe network usually aims at finding the flow distribution within the network, with the pressure distribution being derived from the flow distribution using the energy equation. A typical pipe network is illustrated in Figure 2.9, where the boundary conditions consist of inflows, outflows, and constant-head boundaries such as storage reservoirs. Inflows are typically from water-treatment fa-

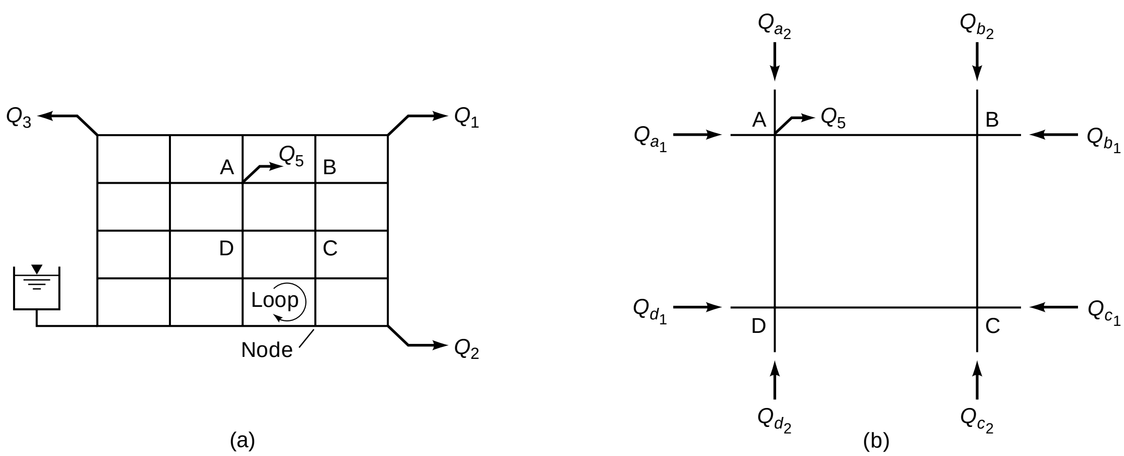


Figure 2.9: Typical Pipe Network

cilities, outflows from consumer withdrawals or fires. All outflows are assumed to occur at network junctions.

The basic equations to be satisfied in pipe networks are the continuity and energy equations. The continuity equation requires that, at each node in the network, the sum of the outflows is equal to the sum of the inflows. This requirement is expressed by the relation

$$\boxed{\sum_{i=1}^{NP(j)} Q_{ij} - F_j = 0, \qquad j = 1, NJ} \tag{2.87}$$

where Q_{ij} is the flowrate in pipe i at junction j (inflows positive); $NP(j)$ is the number of pipes meeting at junction j; F_j is the external flow rate (outflows positive) at junction j; and NJ is the total number of junctions in the network. The energy equation requires that the heads at each of the nodes in the pipe network be consistent with the head losses in the pipelines connecting the nodes. There are two principal methods of calculating the flows in pipe networks: the nodal method and the loop method. In the nodal method, the energy equation is expressed in terms of the heads at the network nodes, while in the loop method the energy equation is expressed in terms of the flows in closed loops within the pipe network.

2.3.1 Nodal Method

In the nodal method, the energy equation is written for each pipeline in the network as

$$\boxed{h_2 = h_1 - \left(\frac{fL}{D} + \sum k_m\right)\frac{Q|Q|}{2gA^2} + \frac{Q}{|Q|}h_p} \tag{2.88}$$

where h_2 and h_1 are the heads at the upstream and downstream ends of a pipe; the terms in parentheses measure the friction loss and minor losses, respectively; and h_p is the head added by pumps in the pipeline. The energy equation stated in Equation 2.88 has been modified to account for the fact that the flow direction is in many cases unknown, in which case a positive flow direction in each pipeline must be assumed, and a consistent set of energy equations stated for the entire network. Equation 2.88 assumes that the positive flow direction is from node 1 to node 2. Application of the nodal method in practice is usually limited to relatively simple networks.

Example 2.9.
The high-pressure ductile-iron pipeline shown in Figure 2.10 becomes divided at point B and rejoins at point C. The pipeline characteristics are given in the following tables.

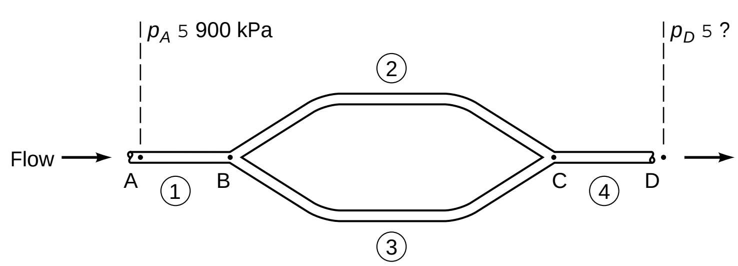


Figure 2.10: Pipe Network

Pipe	Diameter (mm)	Length (m)
1	750	500
2	400	600
3	500	650
4	700	400

Location	Elevation (m)
A	5.0
B	4.5
C	4.0
D	3.5

If the flowrate in Pipe 1 is 2 m^3/s and the pressure at point A is 900 kPa, calculate the pressure at point D. Assume that the flows are fully turbulent in all pipes.

Solution.
The equivalent sand roughness, k_s, of ductile-iron pipe is 0.26 mm, and the pipe and flow characteristics are as follows:

Pipe	Area (m^2)	Velocity (m/s)	k_s/D	f
1	0.442	4.53	0.000347	0.0154
2	0.126	—	0.000650	0.0177
3	0.196	—	0.000520	0.0168
4	0.385	5.20	0.000371	0.0156

where it has been assumed that the flows are fully turbulent. Taking $\gamma = 9.79$ kN/m^3, the head at location A, h_A, is given by

$$h_A = \frac{p_A}{\gamma} + \frac{V_1^2}{2g} + z_A = \frac{900}{9.79} + \frac{4.53^2}{(2)(9.81)} + 5 = 98.0 \text{ m}$$

and the energy equations for each pipe are as follows

$$\begin{aligned}
\text{Pipe 1:}\quad h_B &= h_A - \frac{f_1 L_1}{D_1}\frac{V_1^2}{2g} = 98.0 - \frac{(0.0154)(500)}{0.75}\frac{4.53^2}{(2)(9.81)} \\
&= 87.3 \text{ m} && (2.89)\\
\text{Pipe 2:}\quad h_C &= h_B - \frac{f_2 L_2}{D_2}\frac{Q_2^2}{2gA_2^2} = 87.3 - \frac{(0.0177)(600)}{0.40}\frac{Q_2^2}{(2)(9.81)(0.126)^2} \\
&= 87.3 - 85.2Q_2^2 && (2.90)\\
\text{Pipe 3:}\quad h_C &= h_B - \frac{f_3 L_3}{D_3}\frac{Q_3^2}{2gA_3^2} = 87.3 - \frac{(0.0168)(650)}{0.50}\frac{Q_3^2}{(2)(9.81)(0.196)^2} \\
&= 87.3 - 29.0Q_3^2 && (2.91)\\
\text{Pipe 4:}\quad h_D &= h_C - \frac{f_4 L_4}{D_4}\frac{Q_4^2}{2gA_4^2} = h_C - \frac{(0.0156)(400)}{0.70}\frac{Q_4^2}{(2)(9.81)(0.385)^2} \\
&= h_C - 3.07Q_4^2 && (2.92)
\end{aligned}$$

and the continuity equations at the two pipe junctions are

$$\begin{aligned}
\text{Junction B:}\quad Q_2 + Q_3 &= 2 \text{ m}^3\text{/s} && (2.93)\\
\text{Junction C:}\quad Q_2 + Q_3 &= Q_4 && (2.94)
\end{aligned}$$

Equations 2.90 to 2.94 are five equations in five unknowns: h_C, h_D, Q_2, Q_3, and Q_4. Equations 2.93 and 2.94 indicate that

$$Q_4 = 2 \text{ m}^3\text{/s}$$

Combining Equations 2.90 and 2.91 leads to

$$87.3 - 85.2Q_2^2 = 87.3 - 29.0Q_3^2$$

and therefore

$$Q_2 = 0.583Q_3 \tag{2.95}$$

Substituting Equation 2.95 into Equation 2.93 yields

$$2 = (0.583 + 1)Q_3$$

or

$$Q_3 = 1.26 \text{ m}^3/\text{s}$$

and from Equation 2.95

$$Q_2 = 0.74 \text{ m}^3/\text{s}$$

According to Equation 2.91

$$h_C = 87.3 - 29.0Q_3^2 = 87.3 - 29.0(1.26)^2 = 41.3 \text{ m}$$

and Equation 2.92 gives

$$h_D = h_C - 3.07Q_4^2 = 41.3 - 3.07(2)^2 = 29.0 \text{ m}$$

Therefore, since the total head at D, h_D, is equal to 29.0 m, then

$$29.0 = \frac{p_D}{\gamma} + \frac{V_4^2}{2g} + z_D = \frac{p_D}{9.79} + \frac{5.20^2}{(2)(9.81)} + 3.5$$

which yields

$$p_D = 236 \text{ kPa}$$

Therefore, the pressure at location D is 236 kPa.

This problem has been solved by assuming that the flows in all pipes are fully turbulent. This is generally not known for sure a priori, and therefore a complete solution would require repeating the calculations until the assumed friction factors are consistent with the calculated flowrates.

2.3.2 Loop Method

In the loop method, the energy equation is written for each loop of the network, in which case the algebraic sum of the head losses within each loop is equal to zero. This requirement is expressed by the relation

$$\boxed{\sum_{j=1}^{NP(i)} (h_{L,ij} - h_{p,ij}) = 0, \qquad i = 1, NL} \tag{2.96}$$

where $h_{L,ij}$ is the head loss in pipe j of loop i, and $h_{p,ij}$ is the head added by any pumps that may exist in line ij. Combining Equations 2.87 and 2.96 with an expression for calculating the head losses in pipes, such as the Darcy-Weisbach equation, and the pump characteristic curves, which relate the head added by the pump to the flowrate through the pump, yields a complete mathematical description of the flow problem. Solution of this system of flow equations is complicated by the fact that the equations are nonlinear, and numerical methods must be used to solve for the flow distribution in the pipe network.

Hardy Cross Method. The Hardy Cross method (Cross, 1936) is a simple technique for hand-solution of the loop system of equations governing flow in pipe networks. This iterative method was developed before the advent of computers, and much more efficient algorithms are used for numerical computations. In spite of this limitation, the Hardy Cross method is presented here to illustrate the iterative solution of the loop equations in pipe networks. The Hardy Cross method assumes that the head loss, h_L, in each pipe is proportional to the discharge, Q, raised to some power n, in which case

$$h_L = rQ^n \tag{2.97}$$

The proportionality constant, r, depends on which head loss equation is used and the types of losses in the pipe. Clearly, if all head losses are due to friction and the Darcy-Weisbach equation is used to calculate the head losses, then r is given by

$$r = \frac{fL}{2gA^2D} \tag{2.98}$$

If the flow in each pipe is approximated as $\hat{Q}$, and ΔQ is the error in this estimate, then the actual flowrate, Q, is related to $\hat{Q}$ and ΔQ by

$$Q = \hat{Q} + \Delta Q \tag{2.99}$$

and the head loss in each pipe is given by

$$\begin{aligned} h_L &= rQ^n \\ &= r(\hat{Q} + \Delta Q)^n \\ &= r\left[\hat{Q}^n + n\hat{Q}^{n-1}\Delta Q + \frac{n(n-1)}{2}\hat{Q}^{n-2}(\Delta Q)^2 + \cdots + (\Delta Q)^n\right] \end{aligned} \tag{2.100}$$

If the error in the flow estimate, ΔQ, is small, then the higher order terms in ΔQ can be neglected and the head loss in each pipe can be approximated by the relation

$$h_L \approx r\hat{Q}^n + rn\hat{Q}^{n-1}\Delta Q \tag{2.101}$$

This relation approximates the head loss in the flow direction. However, in working with pipe networks, it is required that the algebraic sum of the head losses in any loop of the network (see Figure 2.9) must be equal to zero. We must therefore define a positive flow direction (such as clockwise), and count head losses as positive in pipes when the flow is in the positive direction and negative when the flow is opposite to the selected positive direction. Under these circumstances, the sign of the head loss must be the same as the sign of the flow direction. Further, when the flow is in the positive direction, positive values of ΔQ require a positive correction to the head loss; when the flow is in the negative direction, positive values in ΔQ also require a positive correction to the calculated head loss. To preserve the algebraic relation among head loss, flow direction, and flow error (ΔQ), Equation 2.101 for each pipe can be written as

$$h_L = r\hat{Q}|\hat{Q}|^{n-1} + rn|\hat{Q}|^{n-1}\Delta Q \tag{2.102}$$

where the approximation has been replaced by an equal sign. On the basis of Equation 2.102, the requirement that the algebraic sum of the head losses around each loop be equal to zero can be written as

$$\sum_{j=1}^{NP(i)} r_{ij}Q_j|Q_j|^{n-1} + \Delta Q_i \sum_{j=1}^{NP(i)} r_{ij}n|Q_j|^{n-1} = 0, \qquad i = 1, NL \tag{2.103}$$

where $NP(i)$ is the number of pipes in loop i, r_{ij} is the head-loss coefficient in pipe j (in loop i), Q_j is the estimated flow in pipe j, ΔQ_i is the flow correction for the pipes in loop i, and NL is the number of loops in the entire network. The approximation given by Equation 2.103 assumes that there are no pumps in the loop, and that the flow correction, ΔQ_i, is the same for each pipe in each loop. Solving Equation 2.103 for ΔQ_i leads to

$$\boxed{\Delta Q_i = -\frac{\sum_{j=1}^{NP(i)} r_{ij} Q_j |Q_j|^{n-1}}{\sum_{j=1}^{NP(i)} n r_{ij} |Q_j|^{n-1}}} \tag{2.104}$$

This equation forms the basis of the Hardy Cross method.

The steps to be followed in using the Hardy Cross method to calculate the flow distribution in pipe networks are:

1. Assume a reasonable distribution of flows in the pipe network. This assumed flow distribution must satisfy continuity.

2. For each loop, i, in the network, calculate the quantities $r_{ij} Q_j |Q_j|^{n-1}$ and $n r_{ij} |Q_j|^{n-1}$ for each pipe in the loop. Calculate the flow correction, ΔQ_i, using Equation 2.104. Add the correction algebraically to the estimated flow in each pipe. [*Note*: Values of r_{ij} occur in both the numerator and denominator of Equation 2.104; therefore, values proportional to the actual r_{ij} may be used to calculate ΔQ_i.]

3. Proceed to another circuit and repeat step 2.

4. Repeat steps 2 and 3 until the corrections (ΔQ_i) are small.

The application of the Hardy Cross method is best demonstrated by an example.

Example 2.10.
Compute the distribution of flows in the pipe network shown in Figure 2.11(a), where the head loss in each pipe is given by

$$h_L = rQ^2$$

and the values of r are shown in Figure 2.11(a). The flows are taken as dimensionless for the sake of illustration.

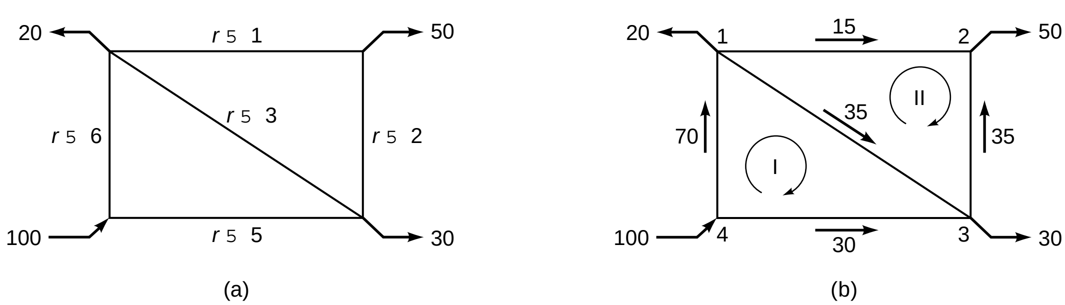


Figure 2.11: Flows in Pipe Network

Solution.
The first step is to assume a distribution of flows in the pipe network that satisfies continuity. The assumed distribution

of flows is shown in Figure 2.11(b), along with the positive-flow directions in each of the two loops. The flow correction for each loop is calculated using Equation 2.104. Since $n = 2$ in this case, the flow correction formula becomes

$$\Delta Q_i = -\frac{\sum_{j=1}^{NP(i)} r_{ij} Q_j |Q_j|}{\sum_{j=1}^{NP(i)} 2r_{ij} |Q_j|}$$

The calculation of the numerator and denominator of this flow correction formula for loop I is tabulated as follows

Loop	Pipe	Q	$rQ\|Q\|$	$2r\|Q\|$
I	4–1	70	29,400	840
	1–3	35	3,675	210
	3–4	−30	−4,500	300
			28,575	1,350

The flow correction for loop I, ΔQ_I, is therefore given by

$$\Delta Q_I = -\frac{28575}{1350} = -21.2$$

and the corrected flows are

Loop	Pipe	Q
I	4–1	48.8
	1–3	13.8
	3–4	−51.2

Moving to loop II, the calculation of the numerator and denominator of the flow correction formula for loop II is given by

Loop	Pipe	Q	$rQ\|Q\|$	$2r\|Q\|$
II	1–2	15	225	30
	2–3	−35	−2,450	140
	3–1	−13.8	−574	83
			−2,799	253

The flow correction for loop II, ΔQ_{II}, is therefore given by

$$\Delta Q_{II} = -\frac{-2799}{253} = 11.1$$

and the corrected flows are

Loop	Pipe	Q
II	1–2	26.1
	2–3	−23.9
	3–1	−2.7

This procedure is repeated in the following table until the calculated flow corrections do not affect the calculated flows, to the level of significant digits retained in the calculations.

Iteration	Loop	Pipe	Q	$rQ\|Q\|$	$2r\|Q\|$	ΔQ	Corrected Q
2	I	4–1	48.8	14,289	586		47.7
		1–3	2.7	22	16		1.6
		3–4	−51.2	−13,107	512		−52.3
				1,204	1,114	−1.1	
	II	1–2	26.1	681	52		29.1
		2–3	−23.9	−1,142	96		−20.9
		3–1	−1.6	−8	10		1.4
				−469	157	3.0	
3	I	4–1	47.7	13,663	573		47.7
		1–3	1.4	6	8		1.4
		3–4	−52.3	−13,666	523		−52.3
				3	1,104	0.0	
	II	1–2	29.1	847	58		29.2
		2–3	−20.9	−874	84		−20.8
		3–1	1.4	6	8		1.5
				−21	150	0.1	
4	I	4–1	47.7	13,662	573		47.7
		1–3	1.5	7	9		1.5
		3–4	−52.3	−13,668	523		−52.3
				1	1,104	0.0	
	II	1–2	29.2	853	58		29.2
		2–3	−20.8	−865	83		−20.8
		3–1	1.5	7	9		1.5
				−5	150	0.0	

The final flow distribution, after four iterations, is given by

Pipe	Q
1–2	29.2
2–3	−20.8
3–4	−52.3
4–1	47.7
1–3	−1.5

It is clear that the final results are fairly close to the flow estimates after only one iteration.

As the above example illustrates, complex pipe networks can generally be treated as a combination of simple circuits or loops, with each balanced in turn until compatible flow conditions exist in all loops. Typically, after the flows have been computed for all pipes in a network, the elevation of the hydraulic grade line and the pressure are computed for each junction node. These pressures are then assessed relative to acceptable operating pressures.

In practice, analyses of complex pipe networks are usually done using commercial computer programs that solve the system of continuity and energy equations that govern the flows in pipe networks. These computer programs, such as EPANET (Rossman, 2000), generally use algorithms that are computationally more efficient than the Hardy Cross method, such as the linear theory method, the Newton-Raphson method, and the gradient algorithm (Lansey and Mays, 1999).

2.4 Pumps

Pumps are hydraulic machines that convert mechanical energy to fluid energy. They can be classified into two main categories: (1) positive displacement pumps, and (2) rotodynamic or kinetic pumps. Positive displacement pumps deliver a fixed quantity of fluid with each revolution of the pump rotor, such as with a piston or cylinder, while rotodynamic pumps add energy to the fluid by accelerating it through the action of a rotating impeller. Rotodynamic pumps are far more common in engineering practice and will be the focus of this section.

Three types of rotodynamic pumps commonly encountered are centrifugal pumps, axial-flow pumps, and mixed-flow pumps. In centrifugal pumps, the flow enters the pump chamber along the axis of the impeller and is discharged radially by centrifugal action, as illustrated in Figure 2.12(a). In axial-flow pumps, the flow enters and leaves the pump chamber along the axis of the

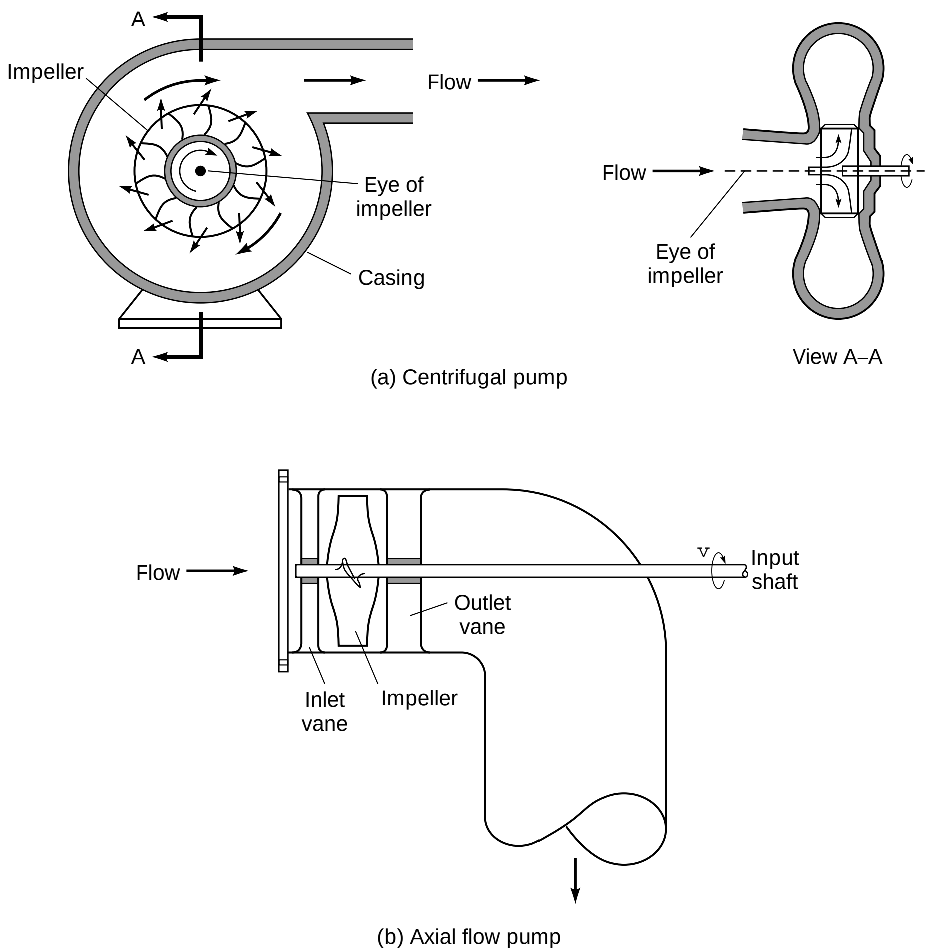


Figure 2.12: Types of Pumps

impeller, as shown in Figure 2.12(b). In mixed-flow pumps, outflows have both radial and axial components. Typical centrifugal and axial-flow pump installations are illustrated in Figure 2.13. Key components of the centrifugal pump are a foot valve installed in the suction pipe to prevent

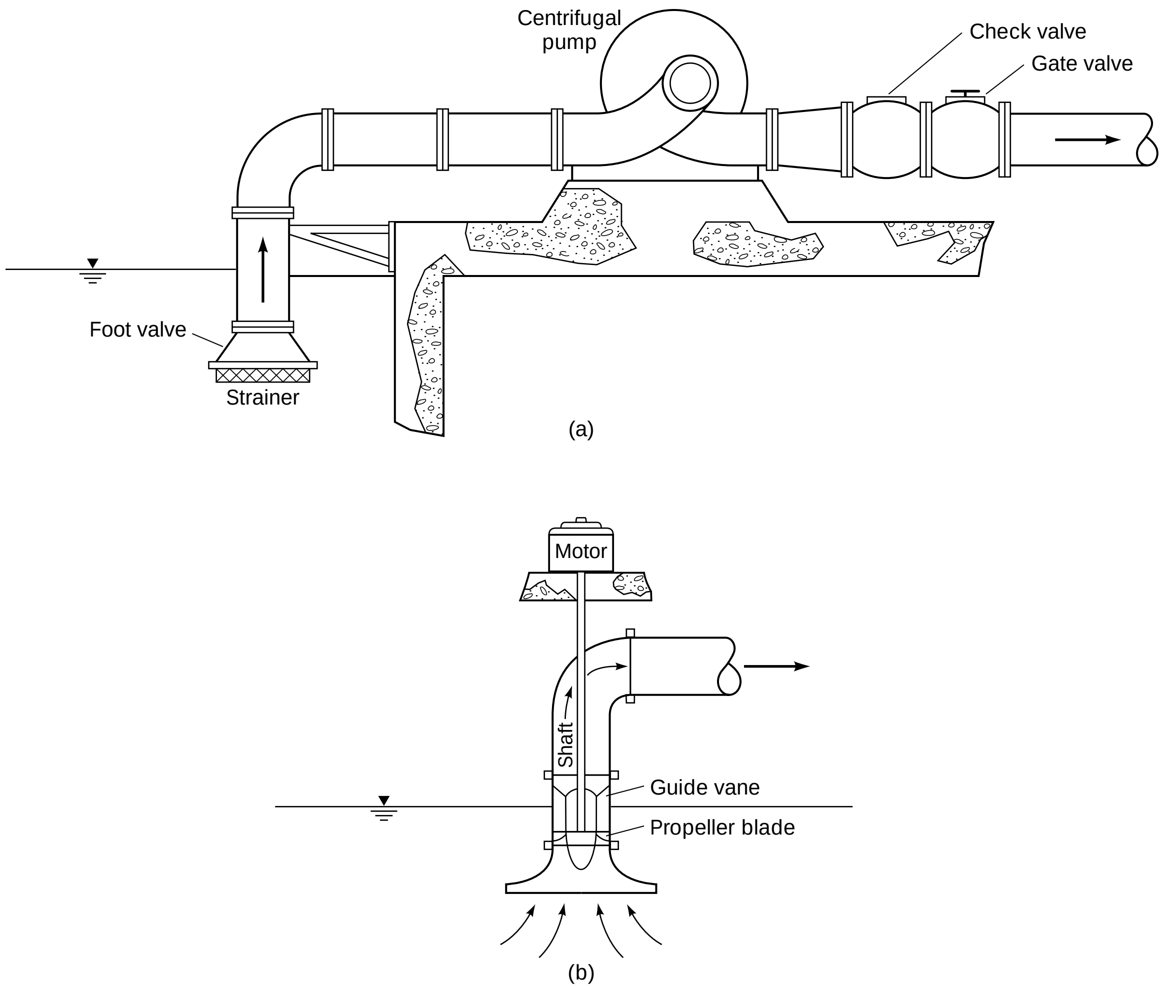


Figure 2.13: Centrifugal and Axial Flow Pump Installations
Source: Finnemore and Franzini (2002).

water from leaving the pump when it is stopped and a check valve in the discharge pipe to prevent backflow if there is a power failure. If the suction line is empty prior to starting the pump, then the suction line must be *primed* prior to startup. Unless the water is known to be very clean, a strainer should be installed at the inlet to the suction piping. The pipe size of the suction line should never be smaller than the inlet connection on the pump; if a reducer is required, it should be of the eccentric type since concentric reducers place part of the supply pipe above the pump inlet where an air pocket could form. The discharge line from the pump should contain a valve close to the pump to allow service or pump replacement.

The pumps illustrated in Figure 2.13 are both *single-stage* pumps, which means that they have only one impeller. In *multistage* pumps, two or more impellers are arranged in series in such a way that the discharge from one impeller enters the eye of the next impeller. If a pump has three impellers in series, it is called a three-stage pump. Multistage pumps are typically used when large pumping heads are required.

The performance of a pump is measured by the head added by the pump and the pump efficiency. The head added by the pump, h_p, is equal to the difference between the total head on the discharge side of the pump and the total head on the suction side of the pump, and is sometimes referred to as the *total dynamic head*. The efficiency of the pump, η, is defined by

$$\eta = \frac{\text{power delivered to the fluid}}{\text{power supplied to the shaft}} \tag{2.105}$$

Pumps are inefficient for a variety of reasons, such as frictional losses as the fluid moves over the solid surfaces, separation losses, leakage of fluid between the impeller and the casing, and mechanical losses in the bearings and sealing glands of the pump. The pump-performance parameters, h_p and η, can be expressed in terms of the fluid properties and the physical characteristics of the pump by the functional relation

$$gh_p \quad \text{or} \quad \eta = f_1(\rho, \mu, D, \omega, Q) \tag{2.106}$$

where the energy added per unit mass of fluid, gh_p, is used instead of h_p (to remove the effect of gravity), f_1 is an unknown function, ρ and μ are the density and dynamic viscosity of the fluid respectively, Q is the flowrate through the pump, D is a characteristic dimension of the pump (usually the inlet or outlet diameter), and ω is the speed of the pump impeller. Equation 2.106 is a functional relationship between six variables in three dimensions. According to the Buckingham pi theorem, this relationship can be expressed as a relation between three dimensionless groups as follows

$$\frac{gh_p}{\omega^2 D^2} \quad \text{or} \quad \eta = f_2\left(\frac{Q}{\omega D^3}, \frac{\rho \omega D^2}{\mu}\right) \tag{2.107}$$

where $gh_p/\omega^2 D^2$ is called the *head coefficient*, and $Q/\omega D^3$ is called the *flow coefficient* (Douglas et al., 1995). In most cases, the flow through the pump is fully turbulent and viscous forces are negligible relative to the inertial forces. Under these circumstances, the viscosity of the fluid is neglected and Equation 2.107 becomes

$$\frac{gh_p}{\omega^2 D^2} \quad \text{or} \quad \eta = f_3\left(\frac{Q}{\omega D^3}\right) \tag{2.108}$$

This relationship describes the performance of all pumps in which viscous effects are negligible, but the exact form of the function in Equation 2.108 depends on the geometry of the pump. A series of pumps having the same shape (but different sizes) are expected to have the same functional relationships between $gh_p/(\omega^2 D^2)$ and $Q/(\omega D^3)$ as well as η and $Q/(\omega D^3)$. A class of pumps that have the same shape is called a *homologous series*, and the performance characteristics of a homologous series of pumps are described by curves such as those in Figure 2.14. Pumps are selected to meet specific design conditions and, since the efficiency of a pump varies with the operating condition, it is usually desirable to select a pump that operates at or near the point of maximum efficiency, indicated by the point P in Figure 2.14.

The point of maximum efficiency of a pump is commonly called the *best efficiency point* (bep), and is sometimes called the *nameplate* or *design point*. Maintaining operation near the bep will allow a pump to function for years with little maintenance, and as the operating point moves away from the bep, pump thrust and radial loads increase which increases the wear on the pump

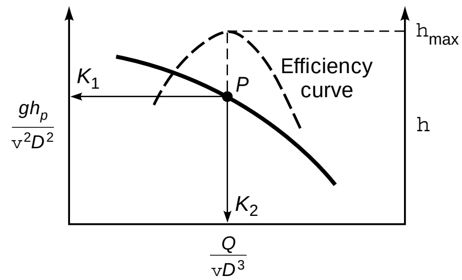


Figure 2.14: Performance Curves of a Homologous Series of Pumps

bearings and shaft. For these reasons, it is generally recommended that pump operation should be maintained between 70% and 130% of the bep flowrate (Lansey and El-Shorbagy, 2001). At the best efficiency point in Figure 2.14,

$$\frac{gh_p}{\omega^2 D^2} = K_1, \quad \text{and} \quad \frac{Q}{\omega D^3} = K_2 \tag{2.109}$$

Eliminating D from these equations yields

$$\frac{\omega Q^{\frac{1}{2}}}{(gh_p)^{\frac{3}{4}}} = \sqrt{\frac{K_2}{K_1^{\frac{3}{2}}}} \tag{2.110}$$

The term on the righthand side of this equation is a constant for a homologous series of pumps and is denoted by the *specific speed*, n_s, defined by

$$n_s = \frac{\omega Q^{\frac{1}{2}}}{(gh_p)^{\frac{3}{4}}} \tag{2.111}$$

where any consistent set of units can be used. The specific speed, n_s, is also called the *shape number* (Hwang and Houghtalen, 1996; Wurbs and James, 2002) or the *type number* (Douglas et al., 2001). In SI units, ω is in rad/s, Q in m^3/s, g in m/s^2, and h_p in meters. The most efficient operating point for a homologous series of pumps is therefore specified by the specific speed. This nomenclature is somewhat unfortunate since the specific speed is dimensionless and hence does not have units of speed. It is common practice in the United States to define the specific speed by N_s, as

$$\boxed{N_s = \frac{\omega Q^{\frac{1}{2}}}{h_p^{\frac{3}{4}}}} \tag{2.112}$$

where N_s is not dimensionless, ω is in revolutions per minute (rpm), Q is in gallons per minute (gpm), and h_p is in feet (ft). Although N_s has dimensions, the units are seldom stated in practice. The required pump operating point gives the flowrate, Q, and head, h_p, required from the pump; the rotational speed, ω, is determined by the synchronous speeds of available motors; and the specific speed calculated from the required operating point is the basis for selecting the appropriate pump. Since the specific speed is independent of the size of a pump, and all homologous pumps (of varying sizes) have the same specific speed, then the calculated specific speed at the desired operating point indicates the type of pump that must be selected to ensure optimal efficiency.

Table 2.3: Pump Selection Guidelines

Type of pump	Range of specific speeds, n_s*	Typical flowrates (L/s)	Typical efficiencies (%)
Centrifugal	0.15–1.5 (400–4,000)	< 60	70–94
Mixed flow	1.5–3.7 (4,000–10,000)	60–300	90–94
Axial flow	3.7–5.5 (10,000–15,000)	> 300	84–90

*The specific speeds in parentheses correspond to N_s given by Equation 2.112, with ω in rpm, Q in gpm, and h_p in ft.

The types of pump that give the maximum efficiency for given specific speeds, n_s, are listed in Table 2.3 along with typical flowrates delivered by the pumps. Table 2.3 indicates that centrifugal pumps have low specific speeds, $n_s < 1.5$; mixed-flow pumps have medium specific speeds, $1.5 < n_s < 3.7$; and axial-flow pumps have high specific speeds, $n_s > 3.7$. This indicates that centrifugal pumps are most efficient at delivering low flows at high heads, while axial flow pumps are most efficient at delivering high flows at low heads. The efficiencies of radial-flow (centrifugal) pumps increase with increasing specific speed, while the efficiencies of mixed-flow and axial-flow pumps decrease with increasing specific speed. Pumps with specific speed less than 0.3 tend to be inefficient (Finnemore and Franzini, 2002). Since axial-flow pumps are most efficient at delivering high flows at low heads, this type of pump is commonly used to move large volumes of water through major canals, and an example of this application is shown in Figure 2.15; where there are three axial-flow pumps operating in parallel, and these pumps are driven by motors housed in the pump station.

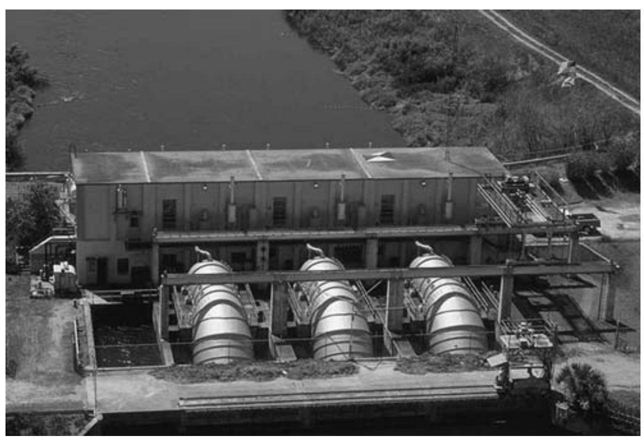


Figure 2.15: Axial Flow Pump Operating in a Canal

Most pumps are driven by standard electric motors. The standard speed of AC synchronous induction motors at 60 cycles and 220 to 440 volts is given by

$$\text{Synchronous speed (rpm)} = \frac{3600}{\text{no. of pairs of poles}} \tag{2.113}$$

A common problem is that, for the motor speed chosen, the calculated specific speed does not exactly equal the specific speed of available pumps. In these cases, it is recommended to choose a pump with a specific speed that is close to and greater than the required specific speed. In rare cases, a new pump may be designed to meet the design conditions exactly, however, this is usually very costly and only justified for very large pumps.

2.4.1 Affinity Laws

The performance curves for a homologous series of pumps is illustrated in Figure 2.14. Any two pumps in the homologous series are expected to operate at the same efficiency when

$$\boxed{\left(\frac{Q}{\omega D^3}\right)_1 = \left(\frac{Q}{\omega D^3}\right)_2, \text{and} \left(\frac{h_p}{\omega^2 D^2}\right)_1 = \left(\frac{h_p}{\omega^2 D^2}\right)_2} \tag{2.114}$$

These relationships are sometimes called the *affinity laws for homologous pumps*. An affinity law for the power delivered to the fluid, P, can be derived from the affinity relations given in Equation 2.114, since P is defined by

$$P = \gamma Q h_p \tag{2.115}$$

which leads to the following derived affinity relation

$$\boxed{\left(\frac{P}{\omega^3 D^5}\right)_1 = \left(\frac{P}{\omega^3 D^5}\right)_2} \tag{2.116}$$

In accordance with the dimensional analysis of pump performance, Equation 2.107, the affinity laws for scaling pump performance are valid as long as viscous effects are negligible. The effect of viscosity is measured by the Reynolds number, Re, defined by

$$\text{Re} = \frac{\rho\omega D^2}{\mu} \tag{2.117}$$

and scale effects are negligible when $\text{Re} > 3\times10^5$ (Gerhart et al., 1992). In lieu of stating a Reynolds number criterion for scale effects to be negligible, it is sometimes stated that larger pumps are more efficient than smaller pumps and that the scale effect on efficiency is given by (Moody and Zowski, 1969; Stepanoff, 1957)

$$\frac{1-\eta_2}{1-\eta_1} = \left(\frac{D_1}{D_2}\right)^{\frac{1}{4}} \tag{2.118}$$

where η_1 and η_2 are the efficiencies of homologous pumps of diameters D_1 and D_2, respectively. The effect of changes in flowrate on efficiency can be estimated using the relation

$$\frac{0.94-\eta_2}{0.94-\eta_1} = \left(\frac{Q_1}{Q_2}\right)^{0.32} \tag{2.119}$$

where Q_1 and Q_2 are corresponding homologous flowrates.

Example 2.11.
A pump with a 1,200 rpm motor has a performance curve of

$$h_p = 12 - 0.1Q^2$$

where h_p is in meters and Q is in cubic meters per minute. If the motor is changed to 2,400 rpm, estimate the new performance curve.

Solution.
The performance characteristics of a homologous series of pumps is given by

$$\frac{gh_p}{\omega^2 D^2} = f\left(\frac{Q}{\omega D^3}\right)$$

For a fixed pump size, D, for two different motor speeds, ω_1 and ω_2, the general performance curve can be written as

$$\frac{h_1}{\omega_1^2} = f\left(\frac{Q_1}{\omega_1}\right)$$

and

$$\frac{h_2}{\omega_2^2} = f\left(\frac{Q_2}{\omega_2}\right)$$

where h_1 and Q_1 are the head added by the pump and the flowrate, respectively, when the speed is ω_1; and h_2 and Q_2 are the head and flowrate when the speed is ω_2. Since the pumps are part of a homologous series, then, neglecting scale effects, the function f is fixed and therefore when

$$\frac{Q_1}{\omega_1} = \frac{Q_2}{\omega_2}$$

then

$$\frac{h_1}{\omega_1^2} = \frac{h_2}{\omega_2^2}$$

These relations are simply statements of the affinity laws. In the present case, $\omega_1 = 1{,}200$ rpm and $\omega_2 = 2{,}400$ rpm and the affinity laws give that

$$\begin{aligned} Q_1 &= \frac{\omega_1}{\omega_2} Q_2 = \frac{1200}{2400} Q_2 = 0.5 Q_2 \\ h_1 &= \frac{\omega_1^2}{\omega_2^2} h_2 = \frac{1200^2}{2400^2} h_2 = 0.25 h_2 \end{aligned}$$

Since the performance curve of the pump at speed ω_1 is given by

$$h_1 = 12 - 0.1 Q_1^2$$

then the performance curve at speed ω_2 is given by

$$0.25 h_2 = 12 - 0.1(0.5 Q_2)^2$$

which leads to

$$h_2 = 48 - 0.1 Q_2^2$$

The performance curve of the pump with a 2,400 rpm motor is therefore given by

$$h_p = 48 - 0.1 Q^2$$

2.4.2 Pump Selection

Selection of a pump generally requires specification of a manufacturer, model (homologous) series, impeller size, D, and rotational speed, ω. For each model series and rotational speed, pump manufacturers provide a *performance curve* or *characteristic curve* that shows the relationship between the head, h_p, added by the pump and the flowrate, Q, through the pump. A typical example of a set of pump characteristic curves (h_p versus Q) provided by a manufacturer for a

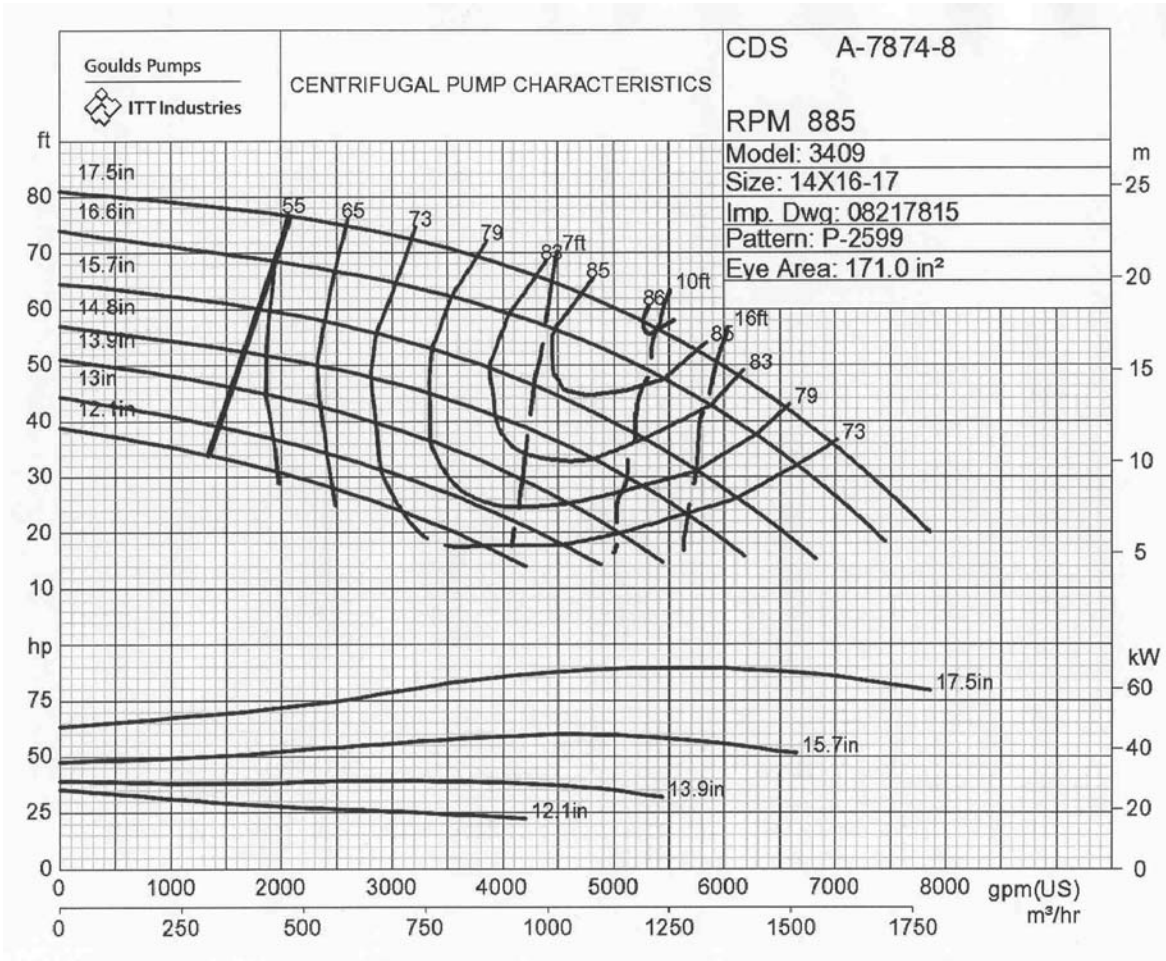


Figure 2.16: Pump Performance Curve
Source: Goulds Pumps (`www.gouldspumps.com`).

homologous series of pumps is shown in Figure 2.16. In this case, the homologous series of pumps (Model 3409) have impeller diameters ranging from 12.1 in. (307 mm) to 17.5 in. (445 mm) with a rotational speed of 885 revolutions per minute. Superimposed on the characteristic curves are constant-efficiency lines for efficiencies ranging from 55% to 86%, and (dashed) isolines of net positive suction head which is the allowable difference between the pressure head on the suction side of the pump and the pressure head at which water vaporizes (i.e. saturation vapor pressure). In Figure 2.16, the allowable net positive suction head ranges from 16 ft (4.9 m) for higher flowrates down to approximately zero, which is indicated by a bold line that meets the 55% efficiency contour. Also shown in Figure 2.16, below the characteristic curves, is the power delivered to the pump (in kW) for various flowrates and impeller diameters. This power input to the pump shaft is called the *brake horsepower*.

The goal in pump selection is to select a pump that operates at a point of maximum efficiency and with a net positive suction head that exceeds the minimum allowable value. Pumps are placed in pipeline systems such as that illustrated in Figure 2.17, in which case the energy equation for the pipeline system requires that the head, h_p, added by the pump is given by

$$\boxed{h_p = \Delta z + Q^2 \left[\sum \frac{fL}{2gA^2D} + \sum \frac{K_m}{2gA^2} \right]} \tag{2.120}$$

where Δz is the difference in elevation between the water surfaces of the inflow and outflow reservoirs, the first term in the square brackets is the sum of the head losses due to friction, and the second term is the sum of the minor head losses. Equation 2.120 gives a relationship between h_p

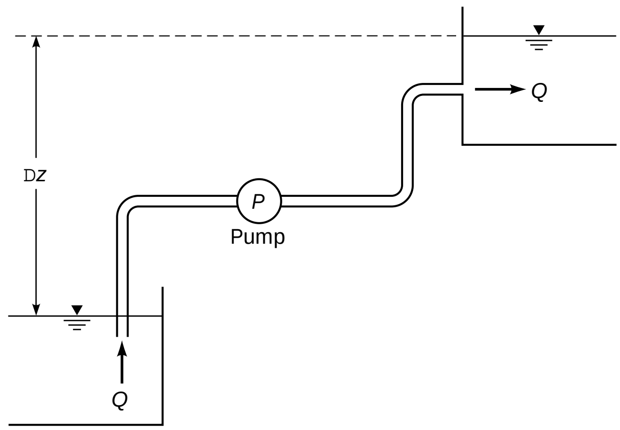


Figure 2.17: Pipeline System

and Q for the pipeline system, and this relationship is commonly called the *system curve*. Because the flowrate and head added by the pump must satisfy both the system curve and the pump characteristic curve, Q and h_p are determined by simultaneous solution of Equation 2.120 and the pump characteristic curve. The resulting values of Q and h_p identify the *operating point* of the pump. The location of the operating point on the performance curve is illustrated in Figure 2.18.

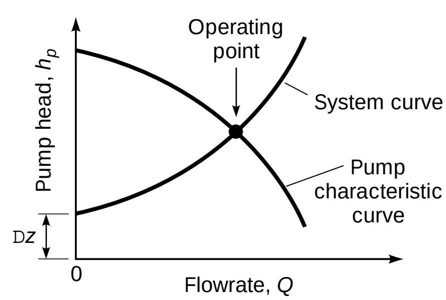


Figure 2.18: Operating Point in Pipeline System

Example 2.12.
Water is being pumped from a lower to an upper reservoir through a pipeline system similar to the one shown in Figure 2.17. The reservoirs differ in elevation by 15.2 m, and the length of the steel pipe ($k_s = 0.046$ mm) connecting the reservoirs is 21.3 m. The pipeline is 50-mm in diameter, and the performance curve of the 2400-rpm centrifugal pump being considered for selection is given by

$$h_p = 24.4 - 7.65Q^2 \tag{2.121}$$

where h_p is in meters and Q is in liters per second. Using this pump, what flow do you expect in the pipeline? If the calculated operating point of the pump coincides with the point of maximum efficiency, which is the goal of pump selection, calculate the specific speed of the pump in U.S. Customary units and verify that a centrifugal-type pump should be used.

Solution.
Neglecting minor losses, the energy equation for the pipeline system is given by

$$h_p = 15.2 + \frac{fL}{2gA^2D}Q^2 \tag{2.122}$$

where h_p is the head added by the pump, f is the friction factor, L is the pipe length, A is the cross-sectional area of the pipe, and D is the diameter of the pipe. The area, A, is given by

$$A = \frac{\pi}{4}D^2 = \frac{\pi}{4}(0.05)^2 = 0.00196 \text{ m}^2$$

In general, f is a function of both the Reynolds number and the relative roughness. However, if the flow is fully turbulent, then the friction factor depends only on the relative roughness according to Equation 2.34

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\epsilon}{3.7}\right) \tag{2.123}$$

where ϵ is the relative roughness of the pipe given by

$$\epsilon = \frac{k_s}{D}$$

and k_s is the equivalent sand roughness. In this case, $k_s = 0.046$ mm and $D = 50$ mm. Therefore

$$\epsilon = \frac{0.046}{50} = 0.000920$$

Substituting for ϵ into Equation 2.123 yields

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{0.000920}{3.7}\right) = 7.21$$

Therefore

$$f = 0.0192$$

Substituting this value of f into the system equation (Equation 2.122) yields

$$h_p = 15.2 + \frac{fL}{2gA^2D}Q^2 = 15.2 + \frac{(0.0192)(21.3)}{(2)(9.81)(0.00196)^2(0.05)}Q^2 = 15.2 + 108500Q^2$$

This equation is for Q in m^3/s, and the corresponding equation for Q in L/s is

$$h_p = 15.2 + 0.109Q^2 \tag{2.124}$$

Combining the system curve, Equation 2.124, with the pump characteristic curve, Equation 2.121, leads to

$$\begin{aligned} 15.2 + 0.109Q^2 &= 24.4 - 7.65Q^2 \\ Q &= 1.09 \text{ L/s} \end{aligned}$$

This flowrate was derived by assuming that the flow in the pipeline is fully turbulent. This assumption can now be verified by recalculating the friction factor with the Reynolds number corresponding to the calculated flowrate. The velocity in the pipeline, V, is given by

$$V = \frac{Q}{A} = \frac{1.09 \times 10^{-3}}{0.00196} = 0.556 \text{ m/s}$$

Since the kinematic viscosity, ν, at 20°C is 1.00×10^{-6} m^2/s, the Reynolds number, Re, is

$$\text{Re} = \frac{VD}{\nu} = \frac{(0.556)(0.05)}{1.00 \times 10^{-6}} = 2.78 \times 10^4$$

The friction factor can now be recalculated using the Jain equation (Equation 2.38), where

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{\epsilon}{3.7} + \frac{5.74}{\mathrm{Re}^{0.9}}\right] = -2\log\left[\frac{0.000920}{3.7} + \frac{5.74}{(2.78\times10^4)^{0.9}}\right] = 6.17$$

which leads to

$$f = 0.0263$$

Therefore, the original approximation of $f = 0.0192$ was in error and the estimation of the flow in the pipeline must be recalculated using $f = 0.0263$. After calculating the revised flow, the friction factor must again be calculated to see if it is equal to the assumed value. If not, the process is repeated until the assumed and calculated friction factors are equal. In this case, one more iteration shows that $f = 0.0263$ and

$$Q = 1.09 \text{ L/s}$$

According to the pump characteristic curve, the head added by the pump, h_p, is

$$h_p = 24.4 - 7.65Q^2 = 24.4 - 7.65(1.09)^2 = 15.3 \text{ m}$$

In U.S. Customary units, $Q = 1.09$ L/s = 17.3 gpm; $h_p = 15.3$ m = 50.2 ft; $\omega = 2{,}400$ rpm; and the specific speed, N_s, is given by Equation 2.112 as

$$N_s = \frac{\omega Q^{\frac{1}{2}}}{h_p^{\frac{3}{4}}} = \frac{(2400)(17.3)^{\frac{1}{2}}}{(50.2)^{\frac{3}{4}}} = 529$$

According to Table 2.3, with a pump operating at a specific speed of $N_s = 529$, the best type of pump is a centrifugal pump.

2.4.3 Limits on Pump Location

If the absolute pressure on the suction side of a pump falls below the saturation vapor pressure of the fluid, the water will begin to vaporize. This process of vaporization is called *cavitation*. Cavitation is usually a transient phenomenon that occurs as water enters the low-pressure suction side of a pump and experiences the even lower pressures adjacent to the rotating pump impeller. As the water containing vapor cavities moves toward the high-pressure environment of the discharge side of the pump, the vapor cavities are compressed and ultimately implode, creating small localized high-velocity jets that can cause considerable damage to the pump machinery. Collapsing vapor cavities have been associated with jet velocities on the order of 110 m/s, and pressures of up to 800 MPa when the jets strike a solid wall (Finnemore and Franzini, 2002; Knapp et al., 1970). The damage caused by collapsing vapor cavities usually manifests itself as *pitting* of the metal casing and impeller, reduced pump efficiency, and excessive vibration of the pump. The noise generated by imploding vapor cavities resembles the sound of gravel going through a centrifugal pump. Since the saturation vapor pressure increases with temperature, a system that operates satisfactorily without cavitation during the winter may have problems with cavitation during the summer.

The potential for cavitation is measured by the net positive suction head, NPSH, defined as the difference between the head on the suction side of the pump and the head when cavitation begins, hence

$$\boxed{\mathrm{NPSH} = \left(\frac{p_s}{\gamma} + \frac{V_s^2}{2g} + z_s\right) - \left(\frac{p_v}{\gamma} + z_s\right) = \frac{p_s}{\gamma} + \frac{V_s^2}{2g} - \frac{p_v}{\gamma}} \tag{2.125}$$

where p_s, V_s, and z_s are the pressure, velocity, and elevation of the fluid at the suction side of the pump, and p_v is the saturation vapor pressure of water at the temperature of the fluid. In cases where water is being pumped from a reservoir, the NPSH can be calculated by applying the energy equation between the reservoir and the suction side of the pump and, in this case the calculated NPSH is called the *available net positive suction head*, NPSH_A, and is given by

$$\boxed{\text{NPSH}_A = \frac{p_o}{\gamma} - \Delta z_s - h_L - \frac{p_v}{\gamma}} \tag{2.126}$$

where p_o is the pressure at the surface of the reservoir (usually atmospheric), Δz_s is the difference in elevation between the suction side of the pump and the fluid surface in the reservoir (called the *suction lift* or *static suction head* or *static head*), h_L is the head loss in the pipeline between the reservoir and suction side of the pump (including minor losses), and p_v is the saturation vapor pressure. In applying either Equation 2.125 or 2.126 to calculate NPSH, care must be taken to use a consistent measure of the pressures, using either gage pressures or absolute pressures. Absolute pressures are usually more convenient, since the vapor pressure is typically given as an absolute pressure. A pump requires a minimum NPSH to prevent the onset of cavitation within the pump, and this minimum NPSH is called the *required net positive suction head*, NPSH_R, which is generally supplied by the pump manufacturer. A typical illustration of this is shown in Figure 2.16, where isolines of NPSRR are overlaid on the pump performance curves. In the case shown, the allowable net positive suction head ranges from 16 ft (4.9 m) for higher flowrates down to approximately zero, which is indicated by a bold line that meets the 55% efficiency contour.

Example 2.13.
A pump with a performance curve of $h_p = 12 - 0.01Q^2$ (h_p in m, Q in L/s) is located 3 m above a water reservoir and pumps water from the reservoir at a rate of 24.5 L/s through a 102-mm diameter ductile iron pipe ($k_s = 0.26$ mm). The length of the pipeline between the reservoir and the suction side of the pump is 3.5 m, and the temperature of the water is 20°C. Calculate the available net positive suction head at the pump. If the pump manufacturer gives the required net positive suction head under the current operating conditions as 1.2 m, determine the maximum height above the surface of the reservoir that the pump can be located and still deliver the same flow.

Solution.
The available net positive suction head, NPSH_A is given by Equation 2.126 as

$$\text{NPSH}_A = \frac{p_o}{\gamma} - \Delta z_s - h_L - \frac{p_v}{\gamma} \tag{2.127}$$

Atmospheric pressure, p_o, can be taken as 101 kPa; the specific weight of water, γ, is 9.79 kN/m^3; the suction lift, Δz_s, is 3 m; and at 20°C, and the saturated vapor pressure of water, p_v, is 2.34 kPa. The head loss, h_L, must be estimated using the Darcy-Weisbach equation and minor loss coefficients. The flowrate, Q, is 24.5 L/s = 0.0245 m^3/s, and the average velocity of flow in the pipe, V, is given by

$$V = \frac{Q}{A} = \frac{0.0245}{\frac{\pi}{4}(0.102)^2} = 3.0 \text{ m/s}$$

At 20°C the kinematic viscosity, ν, is 1.00×10^{-6} m^2/s and the Reynolds number of the flow, Re, is given by

$$\text{Re} = \frac{VD}{\nu} = \frac{(3)(0.102)}{1.00 \times 10^{-6}} = 3.06 \times 10^5$$

Using Re $= 3.06 \times 10^5$, $k_s = 0.26$ mm and $D = 0.102$ m, the Colebrook equation gives $f = 0.0257$. Assuming an inlet head loss of $V^2/2g$ (for a projecting inlet), then the head loss, h_L, in the pipeline between the reservoir and the suction side of the pump is given by

$$h_L = \left(1 + \frac{fL}{D}\right)\frac{V^2}{2g} = \left[1 + \frac{(0.0257)(3.5)}{(0.102)}\right]\frac{(3.0)^2}{2(9.81)} = 0.863 \text{ m}$$

Now that all the parameters necessary to calculate the available net positive suction head, NPSH_A, have been determined, Equation 2.127 gives

$$\mathrm{NPSH}_A = \frac{p_o}{\gamma} - z_s - h_L - \frac{p_v}{\gamma} = \frac{101}{9.79} - 3 - 0.863 - \frac{2.34}{9.79} = 6.21 \text{ m}$$

It is comforting to know that the available net positive suction head ($\mathrm{NPSH}_A = 6.21$ m) significantly exceeds the required net positive suction head ($\mathrm{NPSH}_R = 1.2$ m).

The maximum allowable static lift occurs when $\mathrm{NPSH}_A = \mathrm{NPSH}_R = 1.2$ m, in which case Equation 2.127 gives

$$1.2 = \frac{p_o}{\gamma} - z_s - h_L - \frac{p_v}{\gamma} \tag{2.128}$$

where $p_o = 101$ kPa, and $p_v = 2.34$ kPa. The head loss, h_L, between the pump and the reservoir can be estimated by the relation

$$h_L = \left[1 + \frac{f(z_s + 0.5)}{D}\right]\frac{V^2}{2g}$$

where the length of the pipe from the reservoir to the pump is estimated as $z_s + 0.5$ m, $D = 0.102$ m, $f = 0.0257$, and $V = 3.0$ m/s. Therefore,

$$h_L = \left[1 + \frac{(0.0257)(z_s + 0.5)}{0.102}\right]\frac{3^2}{2(9.81)} = 0.517 + 0.116 z_s$$

Substituting this equation into Equation 2.128 gives

$$\begin{aligned} 1.2 &= \frac{p_o}{\gamma} - z_s - h_L - \frac{p_v}{\gamma} \\ &= \frac{101}{9.79} - z_s - (0.517 + 0.116 z_s) - \frac{2.34}{9.79} \\ &= 9.56 - 1.116 z_s \end{aligned}$$

Solving for z_s yields

$$z_s = 7.49 \text{ m}$$

Therefore, the pump should be no more than 7.49 m above the reservoir.

It is interesting to note that, although the practical limit for the suction lift is typically on the order of 7 m, difficulties in keeping air out of the suction pipe frequently limit the suction lift to around 3 m (Kay, 1998).

2.4.4 Multiple-Pump Systems

In cases where a single pump is inadequate to achieve a desired operating condition, multiple pumps can be used. Combinations of pumps are referred to as *pump systems*, and the pumps within these systems are typically arranged either in series or parallel. The characteristic curve of a pump system is determined by the arrangement of pumps within the system. Consider the case of two identical pumps in series, illustrated in Figure 2.19(a). The flow through each pump is equal to Q, and the head added by each pump is h_p. For the two-pump system, the flow through the system is equal to Q and the head added by the system is $2h_p$. Consequently, the characteristic curve of the two-pump (in series) system is related to the characteristic curve of each pump in that for any flow Q the head added by the system is twice the head added by a single pump, and the relationship between the single-pump characteristic curve and the two-pump characteristic curve is illustrated in Figure 2.19(b). This analysis can be extended to cases where the pump system contains n identical pumps in series, in which case the n-pump characteristic curve is derived from the single-pump

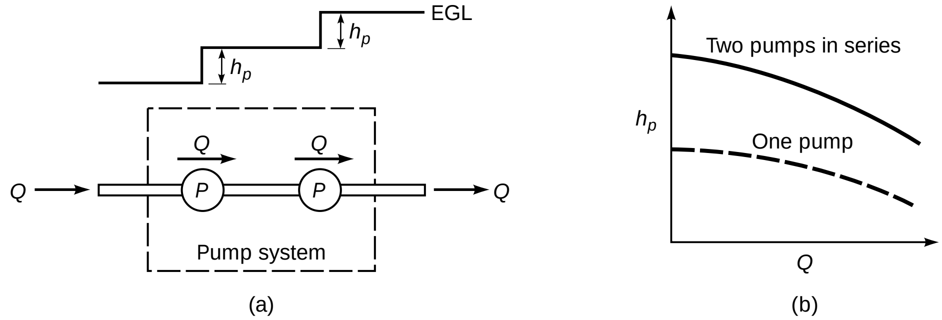


Figure 2.19: Pumps in Series

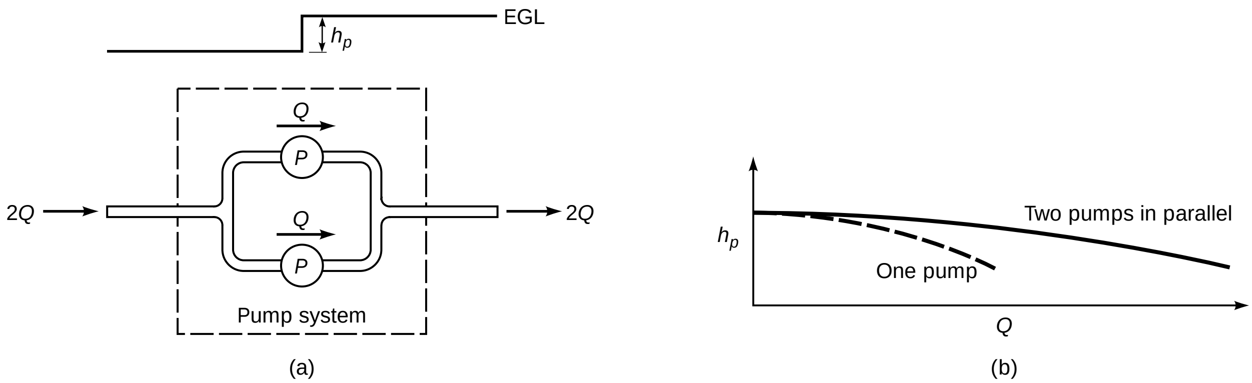


Figure 2.20: Pumps in Parallel

characteristic curve by multiplying the ordinate of the single-pump characteristic curve (h_p) by n. Pumps in series are used in applications involving unusually high heads.

The case of two identical pumps arranged in parallel is illustrated in Figure 2.20. In this case, the flow through each pump is Q and the head added is h_p; therefore, the flow through the two-pump system is equal to $2Q$, while the head added is h_p. Consequently, the characteristic curve of the two-pump system is derived from the characteristic curve of the individual pumps by multiplying the abscissa (Q) by two. This is illustrated in Figure 2.20(b). In a similar manner, the characteristic curves of systems containing n identical pumps in parallel can be derived from the single-pump characteristic curve by multiplying the abscissa (Q) by n. Pumps in parallel are used in cases where the desired flowrate is beyond the range of a single pump and also to provide flexibility in pump operations, since some pumps in the system can be shut down during low-demand conditions or for service. This arrangement is common in sewage pump stations and water-distribution systems, where flowrates vary significantly during the course of a day.

When pumps are placed either in series or parallel, it is usually desirable that these pumps be identical; otherwise, the pumps will be loaded unequally and the efficiency of the pump system will be less than optimal. In cases where nonidentical pumps are placed in series, the characteristic curve of the pump system is obtained by summing the heads added by the individual pumps for a given flowrate. In cases where nonidentical pumps are placed in parallel, the characteristic curve of the pump system is obtained by summing the flowrates through the individual pumps for a given head.

Example 2.14.
If a pump has a performance curve given by

$$h_p = 12 - 0.1Q^2$$

then what is the performance curve for: (a) a system having three of these pumps in series; and (b) a system having three of these pumps in parallel?

Solution.

(a) For a system with three pumps in series, the same flow, Q, goes through each pump, and each pump adds one-third of the head, H_p, added by the pump system. Therefore,

$$\frac{H_p}{3} = 12 - 0.1Q^2$$

and the characteristic curve of the pump system is

$$H_p = 36 - 0.3Q^2$$

(b) For a system consisting of three pumps in parallel, one-third of the total flow, Q, goes through each pump, and the head added by each pump is the same as the total head, H_p added by the pump system. Therefore

$$H_p = 12 - 0.1\left(\frac{Q}{3}\right)^2$$

and the characteristic curve of the pump system is

$$H_p = 12 - 0.011Q^2$$

2.5 Design of Water Distribution Systems

Water distribution systems move water from treatment plants to homes, offices, industries, and other consumers. The major components of a water distribution system are pipelines, pumps, storage facilities, valves, and meters. The primary requirements of distribution system are to supply each customer with a sufficient volume of water at adequate pressure, to deliver safe water that satisfies the quality expectations of customers, and to have sufficient capacity and reserve storage for fire protection. (AWWA, 2003c).

2.5.1 Water Demand

Major considerations in designing water-supply systems are the water demands of the population being served, the fire flows needed to protect life and property, and the proximity of the service area to sources of water. There are usually several categories of water demand within any populated area, and these sources of demand can be broadly grouped into residential, commercial, industrial, and public. Residential water use is associated with houses and apartments where people live; commercial water use is associated with retail businesses, offices, hotels, and restaurants; industrial water use is associated with manufacturing and processing operations; and public water use includes governmental facilities that use water. Large industrial requirements are typically satisfied by sources other than the public water supply.

A typical distribution of water use for an average city in the United States is given in Table 2.4. These rates vary from city to city as a result of differences in local conditions that are unrelated

Table 2.4: Typical Distribution of Water Demand

Category	Average use (liters/day)/person	Percent of total
Residential	380	56
Commercial	115	17
Industrial	85	12
Public	65	9
Loss	40	6
Total	685	100

Source: Solley, 1998

to the efficiency of water use. Water consumption is frequently stated in terms of the average amount of water delivered per day (to all categories of water use) divided by the population served, which is called the *average per capita demand*. The distribution of average per capita rates among 392 water-supply systems serving approximately 95 million people in the United States is shown in Table 2.5. The average per capita water usage in this sample was 660 L/d, with a standard

Table 2.5: Distribution of Per Capita Water Demand

Range (liters/day)/person	Number of systems	Percent of systems
190–370	30	7.7
380–560	132	33.7
570–750	133	33.9
760–940	51	13.0
950–1130	19	4.8
> 1140	27	6.9

Source: Reprinted from *1984 Water Utility Operating Data*, by permission. Copyright © 1986 American Water Works Association.

deviation of 270 L/d. Generally, high per capita rates are found in water-supply systems servicing large industrial or commercial sectors (Dziegielewski et al., 1996).

In the planning of municipal water-supply projects, the water demand at the end of the design life of the project is usually the basis for design. For existing water-supply systems, the American Water Works Association (AWWA, 1992) recommends that every 5 or 10 years, as a minimum, water-distribution systems be thoroughly reevaluated for requirements that would be placed on it by development and reconstruction over a 20-year period into the future. The estimation of the design flowrates for components of the water-supply system typically requires prediction of the population of the service area at the end of the design life, which is then multiplied by the per capita water demand to yield the design flowrate. Whereas the per capita water demand can usually be assumed to be fairly constant, the estimation of the future population typically involves a nonlinear extrapolation of past population trends.

A variety of methods are used in population forecasting. The simplest models treat the population as a whole, forecast future populations based on past trends, and fit empirical growth functions to historical population data. The most complex models disaggregate the population into various groups and forecast the growth of each group separately. A popular approach that segregates the population by age and gender is *cohort analysis* (Sykes, 1995). High levels of disaggregation have the advantage of making forecast assumptions very explicit, but these models tend to be complex and require more data than the empirical models that treat the population as a whole. Over relatively short time horizons, on the order of 10 years, detailed disaggregation models may not be any more accurate than using empirical extrapolation models of the population as a whole. Several conventional extrapolation models are illustrated in the following paragraphs.

Populated areas tend to grow in at varying rates, as illustrated in Figure 2.21. In the early

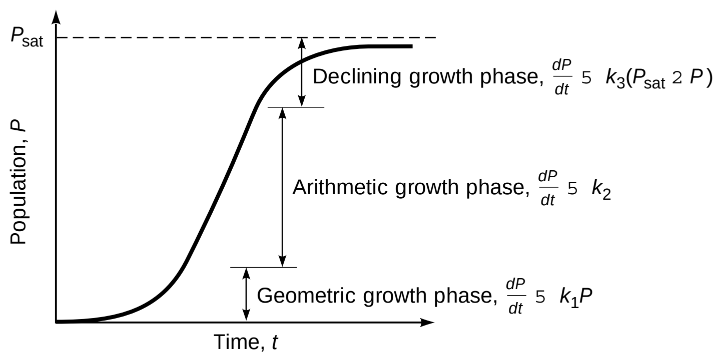


Figure 2.21: Growth Phases in Populated Areas

stages of growth, there are wide open spaces. Population, P, tends to grow geometrically according to the relation

$$\frac{dP}{dt} = k_1 P \tag{2.129}$$

where k_1 is a growth constant. Integrating Equation 2.129 gives the following expression for the population as a function of time

$$\boxed{P(t) = P_o e^{k_1 t}} \tag{2.130}$$

where P_o is the population at some initial time designated as $t = 0$. Beyond the initial geometric growth phase, the rate of growth begins to level off and the following arithmetic growth relation may be more appropriate

$$\frac{dP}{dt} = k_2 \tag{2.131}$$

where k_2 is an arithmetic growth constant. Integrating Equation 2.131 gives the following expression for the population as a function of time

$$\boxed{P(t) = P_o + k_2 t} \tag{2.132}$$

where P_o is the population at $t = 0$. Ultimately, the growth of population centers becomes limited by the resources available to support the population, and further growth is influenced by the saturation population of the area, P_{sat}, and the population growth is described by a relation such as

$$\frac{dP}{dt} = k_3(P_{\text{sat}} - P) \tag{2.133}$$

where k_3 is a constant. This phase of growth is called the *declining growth* phase. Almost all communities have zoning regulations that control the use of both developed and undeveloped areas within their jurisdiction (sometimes called a master plan), and a review of these regulations will yield an estimate of the saturation population of the undeveloped areas. Integrating Equation 2.133 gives the following expression for the population as a function of time

$$\boxed{P(t) = P_{\text{sat}} - (P_{\text{sat}} - P_o)e^{-k_3 t}} \tag{2.134}$$

where P_o is the population at $t = 0$.

The time scale associated with each growth phase is typically on the order of 10 years, although the actual duration of each phase can deviate significantly from this number. The duration of each phase is important in that population extrapolation using a single-phase equation can only be justified for the duration of that growth phase. Consequently, single-phase extrapolations are typically limited to 10 years or less, and these population predictions are termed *short-term projections* (Viessman and Welty, 1985). Extrapolation beyond 10 years, called *long-term projections*, involve fitting an S-shaped curve to the historical population trends and then extrapolating using the fitted equation. The most commonly fitted S-curve is the so-called *logistic curve*, which is described by the equation

$$\boxed{P(t) = \frac{P_{\text{sat}}}{1 + ae^{bt}}} \tag{2.135}$$

where a and b are constants. The conventional methodology to fit the population equations to historical data is to plot the historical data, observe the trend in the data, and fit the curve that best matches the population trend. Regardless of which method is used to forecast the population, errors less than 10% can be expected for planning periods shorter than 10 years, and errors greater than 50% can be expected for planning periods longer than 20 years (Sykes, 1995).

Example 2.15.
You are in the process of designing a water-supply system for a town, and the design life of your system is to end in the year 2020. The population in the town has been measured every 10 years since 1920 by the U.S. Census Bureau, and the reported populations are tabulated here. Estimate the population in the town using (a) graphical extension, (b) arithmetic growth projection, (c) geometric growth projection, (d) declining growth projection (assuming a saturation concentration of 600,000 people), and (e) logistic curve projection.

Year	Population
1920	125,000
1930	150,000
1940	150,000
1950	185,000
1960	185,000
1970	210,000
1980	280,000
1990	320,000

Solution.
The population trend is plotted in Figure 2.22, where a geometric growth rate approaching an arithmetic growth rate is indicated.

(a) A growth curve matching the trend in the measured populations is indicated in Figure 2.22. Graphical extension to the year 2020 leads to a population estimate of 530,000 people.

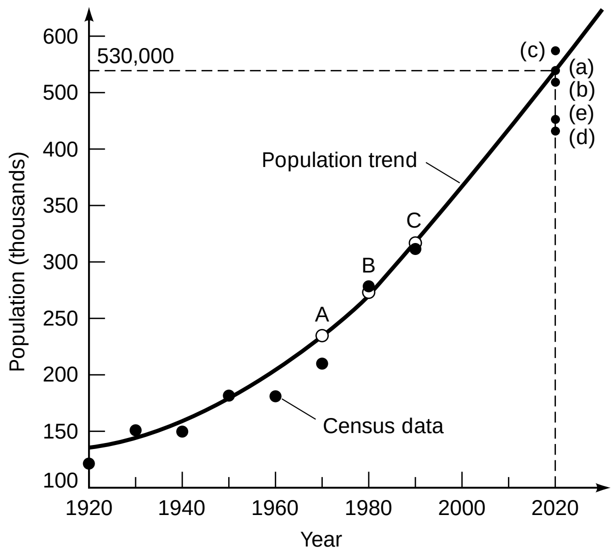


Figure 2.22: Population Trend

(b) Arithmetic growth is described by Equation 2.132 as

$$P(t) = P_o + k_2 t \tag{2.136}$$

where P_o and k_2 are constants. Consider the arithmetic projection of a line passing through points B and C on the approximate growth curve shown in Figure 2.22. At point B, $t = 0$ (year 1980) and $P = 270{,}000$; at point C, $t = 10$ (year 1990) and $P = 330{,}000$. Applying these conditions to Equation 2.136 yields

$$P = 270000 + 6000t \tag{2.137}$$

In the year 2020, $t = 40$ years and the population estimate given by Equation 2.137 is

$$P = 510{,}000 \text{ people}$$

(c) Geometric growth is described by Equation 2.130 as

$$P = P_o e^{k_1 t} \tag{2.138}$$

where k_1 and P_o are constants. Using points A and C in Figure 2.22 as a basis for projection, then at $t = 0$ (year 1970), $P = 225{,}000$, and at $t = 20$ (year 1990), $P = 330{,}000$. Applying these conditions to Equation 2.138 yields

$$P = 225000e^{0.0195t} \tag{2.139}$$

In the year 2020, $t = 50$ years and the population estimate given by Equation 2.139 is

$$P = 597{,}000 \text{ people}$$

(d) Declining growth is described by Equation 2.134 as

$$P(t) = P_{\text{sat}} - (P_{\text{sat}} - P_o)e^{-k_3 t} \tag{2.140}$$

where P_o and k_3 are constants. Using points A and C in Figure 2.22, then at $t = 0$ (year 1970), $P = 225{,}000$, at $t = 20$ (year 1990), $P = 330{,}000$, and $P_{\text{sat}} = 600{,}000$. Applying these conditions to Equation 2.140 yields

$$P = 600000 - 375000e^{-0.0164t} \tag{2.141}$$

In the year 2020, $t = 50$ years and the population given by Equation 2.141 is given by

$$P = 434{,}800 \text{ people}$$

(e) The logistic curve is described by Equation 2.135. Using points A and C in Figure 2.22 to evaluate the constants in Equation 2.135 ($t = 0$ in 1970) yields

$$P = \frac{600000}{1 + 1.67e^{-0.0357t}} \tag{2.142}$$

In 2020, $t = 50$ years and the population given by Equation 2.142 is

$$P = 469{,}000 \text{ people}$$

These results indicate that the population projection in 2020 is quite uncertain, with estimates ranging from 597,000 for geometric growth to 434,800 for declining growth. The projected results are compared graphically in Figure 2.22. Closer inspection of the predictions indicate that the declining and logistic growth models are limited by the specified saturation population of 600,000, while the geometric growth model is not limited by saturation conditions and produces the highest projected population.

The multiplication of the population projection by the per capita water demand is used to estimate the *average daily demand* for a municipal water-supply system. The (annual) average daily demand is equal to the average of the daily demands over one year and is typically given in m^3/s.

Variations in Demand. Water demand generally fluctuates between being below the average daily demand in the early morning hours and above the average daily demand during the midday hours. Typical daily cycles in water demand are shown in Figure 2.23. On a typical day in most

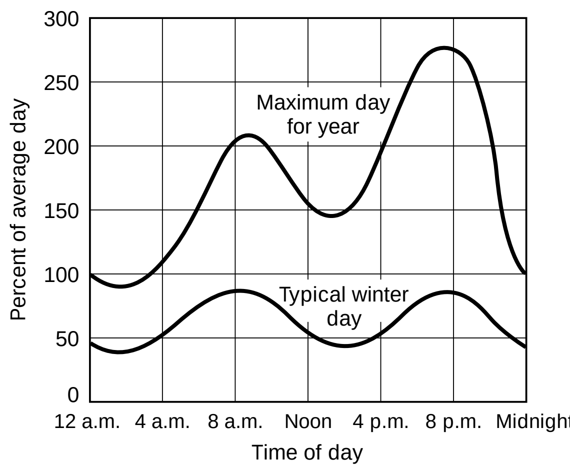


Figure 2.23: Typical Daily Cycles in Water Demand
Source: Linsley, Ray K. et al, *Water-Resources Engineering*. Copyright © 1992 by The McGraw-Hill Companies.

communities, water use is lowest at night (11 p.m. to 5 a.m.) when most people are asleep. Water use rises rapidly in the morning (5 a.m. to 11 a.m.) followed by moderate usage through midday (11 a.m. to 6 p.m.). Use then increases in the evening (6 p.m. to 10 p.m.) and drops rather quickly around 10 p.m. Overall, water-use patterns within a typical 24-hour period are characterized by demands that are 25% to 40% of the average daily demand during the hours between midnight and 6:00 a.m. and 150% to 175% of the average daily demand during the morning or evening peak periods (Velon and Johnson, 1993).

The range of demand conditions that are to be expected in water-distribution systems are specified by *demand factors* or *peaking factors* that express the ratio of the demand under certain conditions to the average daily demand. Typical demand factors for various conditions are given in Table 2.6, where the *maximum daily demand* is defined as the demand on the day of the year that uses the most volume of water, and the *maximum hourly demand* is defined as the demand during the hour that uses the most volume of water. The demand factors in Table 2.6 should serve only

Table 2.6: Typical Demand Factors

Condition	Range of demand factors	Typical value
Daily average in maximum month	1.10–1.50	1.20
Daily average in maximum week	1.20–1.60	1.40
Maximum daily demand	1.50–3.00	1.80
Maximum hourly demand	2.00–4.00	3.25
Minimum hourly demand	0.20–0.60	0.30

Source: Velon and Johnson (1993). Reprinted by permission of The McGraw-Hill Companies.

as guidelines, with the actual demand factors in any one distribution system being best estimated from local measurements. In small water systems, demand factors may be significantly higher than those shown in Table 2.6.

Fire Demand. Besides the fluctuations in demand that occur under normal operating conditions, water-distribution systems are usually designed to accommodate the large (short-term) water demands associated with fighting fires. Although there is no legal requirement that a governing body must size its water-distribution system to provide fire protection, the governing bodies of most communities provide water for fire protection for reasons that include protection of the tax base from destruction by fire, preservation of jobs, preservation of human life, and reduction of human suffering. Flowrates required to fight fires can significantly exceed the maximum flowrates in the absence of fires, particularly in small water systems. In fact, for communities with populations less than 50,000, the need for fire protection is typically the determining factor in sizing water mains, storage facilities, and pumping facilities (AWWA, 2003c). In contrast to urban water systems, many rural water systems are designed to serve only domestic water needs, and fire flow requirements are not considered in the design of these systems (AWWA, 2003c).

Numerous methods have been proposed for estimating fire flows, the most popular of which was proposed by the Insurance Services Office, Inc. (ISO, 1980), which is an organization representing the fire insurance underwriters. The required fire flow for individual buildings can be estimated using the formula (ISO, 1980)

$$\boxed{\mathrm{NFF}_i = C_i O_i (X+P)_i} \tag{2.143}$$

where NFF_i is the *needed fire flow* at location i, C_i is the *construction factor* based on the size of the building and its construction, O_i is the *occupancy factor* reflecting the kinds of materials stored in the building (values range from 0.75 to 1.25), and $(X+P)_i$ is the sum of the *exposure factor* (X_i) and *communication factor* (P_i) that reflect the proximity and exposure of other buildings (values range from 1.0 to 1.75). The construction factor, C_i, is the portion of the NFF attributed to the size of the building and its construction and is given by

$$C_i = 220F\sqrt{A_i} \tag{2.144}$$

where C_i is in L/min; A_i (m^2) is the effective floor area, typically equal to the area of the largest floor in the building plus 50% of the area of all other floors; and F is a coefficient based on the class of construction, given in Table 2.7.

Table 2.7: Construction Coefficient, F

Class of construction	Description	F
1	frame	1.5
2	joisted masonry	1.0
3	noncombustible	0.8
4	masonry, noncombustible	0.8
5	modified fire resistive	0.6
6	fire resistive	0.6

Source: AWWA (1992).

The maximum value of C_i calculated using Equation 2.144 is limited by the following: 30,000 L/min for construction classes 1 and 2; 23,000 L/min for construction classes 3, 4, 5, and 6; and 23,000 L/min for a one-story building of any class of construction. The minimum value of C_i is 2,000 L/min, and the calculated value of C_i should be rounded to the nearest 1,000 L/min. The occupancy factors, O_i, for various classes of buildings are given in Table 2.8. Detailed tables for

Table 2.8: Occupancy Factors, O_i

Combustibility class	Examples	O_i
C-1 Noncombustible	steel or concrete products storage	0.75
C-2 Limited combustible	apartments, churches, offices	0.85
C-3 Combustible	department stores, supermarkets	1.00
C-4 Free burning	auditoriums, warehouses	1.15
C-5 Rapid burning	paint shops, upholstering shops	1.25

Source: Reprinted from *Distribution System Requirements for Fire Protection* (M31), by permission.

estimating the exposure and communication factors, $(X+P)_i$, can be found in AWWA (1992), and values of $(X+P)_i$ are typically on the order of 1.4. The NFF calculated using Equation 2.143 should not exceed 45,000 L/min, nor be less than 2,000 L/min. According to AWWA (1992), 2,000 L/min is the minimum amount of water with which any fire can be controlled and suppressed safely and effectively. The NFF should be rounded to the nearest 1,000 L/min if less than 9,000 L/min, and to the nearest 2,000 L/min if greater than 9,000 L/min. For one- and two-family dwellings not exceeding two stories in height, the NFF listed in Table 2.9 should be used. For other habitable buildings not listed in Table 2.9, the NFF should not exceed 13,000 L/min maximum.

Usually the local water utility will have a policy on the upper limit of fire protection that it will provide to individual buildings. Those wanting higher fire flows need to either provide their own system or reduce fire-flow requirements by installing sprinkler systems, fire walls, or fire-retardant materials (Walski, 1996; AWWA, 1992). Estimates of the needed fire flow calculated using Equation 2.143 are used to determine the fire-flow requirements of the water-supply system, where the needed fire flow is calculated at several representative locations in the service area, and it is assumed that

Table 2.9: Needed Fire Flow for One- and Two-Family Dwellings

Distance between buildings (m)	Needed fire flow (L/min)
> 30	2,000
9.5–30	3,000
3.5–9.5	4,000
< 3.5	6,000

Source: Reprinted from *Distribution System Requirements for Fire Protection* (M31), by permission. Copyright © 1992 American Water Works Association.

only one building is on fire at any time (Sykes, 1995). The design duration of the fire should follow the guidelines in Table 2.10. If these durations cannot be maintained, insurance rates are typically

Table 2.10: Required Fire Flow Durations

Required fire flow (L/min)	Duration (h)
< 9000	2
11,000–13,000	3
15,000–17,000	4
19,000–21,000	5
23,000–26,000	6
26,000–30,000	7
30,000–34,000	8
34,000–38,000	9
38,000–45,000	10

Source: Reprinted from *Distribution System Requirements for Fire Protection* (M31), by permission. Copyright © 1992 American Water Works Association.

increased accordingly. A more detailed discussion of the requirements for fire protection has been published by the American Water Works Association (AWWA, 1992).

Example 2.16.
Estimate the flowrate and volume of water required to provide adequate fire protection to a 10-story noncombustible building with an effective floor area of 8,000 m^2.

Solution.
The NFF can be estimated by Equation 2.143 as

$$\text{NFF}_i = C_i O_i (X + P)_i$$

where the construction factor, C_i, is given by

$$C_i = 220F\sqrt{A_i}$$

For the 10-story building, $F = 0.8$ (Table 2.7, noncombustible, Class 3 construction), and $A_i = 8{,}000$ m^2, hence

$$C_i = 220(0.8)\sqrt{8000} = 16000 \text{ L/min}$$

where C_i has been rounded to the nearest 1,000 L/min. The occupancy factor, O_i, is given by Table 2.8 as 0.75 (C-1 Noncombustible); $(X + P)_i$ can be estimated by the median value of 1.4; and hence the needed fire flow, NFF, is given by

$$\text{NFF}_i = (16000)(0.75)(1.4) = 17000 \text{ L/min}$$

This flow must be maintained for a duration of four hours (Table 2.10). Hence the required volume, V, of water is given by

$$V = 17000 \times 4 \times 60 = 4.08 \times 10^6 \text{ L} = 4080 \text{ m}^3$$

Fire hydrants are placed throughout the service area to provide either direct hose connections for firefighting or connections to pumper trucks (also known as fire engines). A single-hose stream is generally taken as 950 L/min, and hydrants are typically located at street intersections or spaced 60–150 m apart (McGhee, 1991). In high-value districts, additional hydrants may be necessary in the middle of long blocks to supply the required fire flows. Fire hydrants may also be used to release air at high points in the water-distribution system and blow off sediments at low points in the system.

Design Flows. The design capacity of various components of the water-supply system are given in Table 2.11, where *low-lift pumps* refer to low-head, high-rate units that convey the raw-water supply to the treatment facility and *high-lift pumps* deliver finished water from the treatment facility into the distribution network at suitable pressures. The required capacities are based on per person use and population projections for full development of the service area. The required capacities shown in Table 2.11 consist of various combinations of the maximum daily demand, maximum hourly demand, and the fire demand. Typically, the delivery pipelines from the water source to the treatment plant, as well as the treatment plant itself, are designed with a capacity equal to the maximum daily demand. However, because of the high cost of providing treatment, some utilities have trended towards using peak flows averaged over a longer period than one day to design water treatment plants (such as 2 to 5 days), and relying on system storage to meet peak demands above treatment capacity. This approach has serious water quality implications and should be avoided if possible (AWWA, 2004). The flowrates and pressures in the distribution system are analyzed under both maximum daily plus fire demand and the maximum hourly demand, and the larger flowrate governs the design. Pumps are sized for a variety of conditions from maximum daily to maximum hourly demand, depending on their function in the distribution system. Additional reserve capacity is usually installed in water-supply systems to allow for redundancy and maintenance requirements.

Example 2.17.

A metropolitan area has a population of 130,000 people with an average daily demand of 600 L/d/person. If the needed fire flow is 20,000 L/min, estimate: (a) the design capacities for the wellfield and the water-treatment plant; (b) the duration that the fire flow must be sustained and the volume of water that must be kept in the service reservoir in case of a fire; and (c) the design capacity of the main supply pipeline to the distribution system.

Solution.

(a) The design capacity of the wellfield should be equal to the maximum daily demand (Table 2.11). With a demand factor of 1.8 (Table 2.6), the per capita demand on the maximum day is equal to $1.8 \times 600 = 1{,}080$

Table 2.11: Design Periods and Capacities in Water-Supply Systems

Component	Design period (years)	Design capacity
1. Source of supply:		
River	indefinite	maximum daily demand
Wellfield	10–25	maximum daily demand
Reservoir	25–50	average annual demand
2. Conveyance:		
Intake conduit	25–50	maximum daily demand
Conduit to treatment plant	25–50	maximum daily demand
3. Pumps:		
Low-lift	10	maximum daily demand, one reserve unit
High-lift	10	maximum hourly demand, one reserve unit
4. Treatment plant	10–15	maximum daily demand
5. Service reservoir	20–25	working storage plus fire demand plus emergency storage
6. Distribution system:		
Supply pipe or conduit	25–50	greater of (1) maximum daily demand plus fire demand, or (2) maximum hourly demand
Distribution grid	full development	same as for supply pipes

Source: Reprinted by permission of Waveland Press, Inc. from R. S. Gupta, *Hydrology and Hydraulic Systems*, Second Edition. (Prospect Heights, IL; Waveland Press, Inc., 2001).

L/day/person. Since the population served is 130,000 people, the design capacity of the wellfield, $Q_{\rm well}$, is given by

$$Q_{\rm well} = 1080 \times 130000 = 1.4 \times 10^8 \text{ L/d} = 1.62 \text{ m}^3/\text{s}$$

The design capacity of the water-treatment plant is also equal to the maximum daily demand, and therefore should also be taken as 1.62 m^3/s.

(b) The needed fire flow, $Q_{\rm fire}$, is 20000 L/min = 0.33 m^3/s. According to Table 2.10, the fire flow must be sustained for 5 hours. The volume, $V_{\rm fire}$, required for the fire flow will be stored in the service reservoir and is given by

$$V_{\rm fire} = 0.33 \times 5 \times 3600 = 5{,}940 \text{ m}^3$$

(c) The required flowrate in the main supply pipeline is equal to the maximum daily demand plus fire demand or the maximum hourly demand, whichever is greater.

$$\begin{aligned} \text{Maximum daily demand + fire demand} &= 1.62 + 0.33 = 1.95 \text{ m}^3/\text{s} \\ \text{Maximum hourly demand} &= \frac{3.25}{1.80} \times 1.62 = 2.92 \text{ m}^3/\text{s} \end{aligned}$$

where a demand factor of 3.25 has been assumed for the maximum hourly demand. The main supply pipe to the distribution system should therefore be designed with a capacity of 2.92 m^3/s. The water pressure within the distribution system must be above acceptable levels when the system demand is 2.92 m^3/s.

2.5.2 Pipelines

Water-distribution systems typically consist of connected pipe loops throughout the service area. Pipelines in water-distribution systems include *transmission lines*, *arterial mains*, and *distribution mains*. Transmission lines carry flow from the water-treatment plant to the service area, typically have diameters greater than 600 mm, and are usually on the order of 3 km apart. Arterial mains are connected to transmission mains and are laid out in interlocking loops with the pipelines not more than 1 km apart and diameters in the range of 400–500 mm. Smaller form a grid over the entire service area, with diameters in the range of 150–300 mm, and supply water to every user. Pipelines in distribution systems are collectively called *water mains*, and a pipe that carries water from a main to a building or property is called a *service line*. Water mains are normally installed within the rights-of-way of streets. Dead ends in water-distribution systems should be avoided whenever possible, since the lack of flow in such lines may contribute to water-quality problems.

Pipelines in water-distribution systems are typically designed with constraints relating to the minimum pipe size, maximum allowable velocity, and commercially available materials that will perform adequately under operating conditions.

Minimum Size. The size of a water main determines its carrying capacity. Main sizes must be selected to provide the capacity to meet peak domestic, commercial, and industrial demands in the area to be served, and must also provide for fire flow at the necessary pressure. For fire protection, insurance underwriters typically require a minimum main size of 150 mm for residential areas and 200 mm for high-value districts (such as sports stadiums, shopping centers, and libraries) if cross-connecting mains are not more than 180 m apart. On principal streets, and for all long lines not connected at frequent intervals, 300-mm and larger mains are required.

Service Lines. Service lines are pipes, including accessories, that carry water from the main to the point of service, which is normally a meter setting or curb stop located at the property line. Service lines can be any size, depending on how much water is required to serve a particular customer. Single-family residences are most commonly served with 20-mm (3/4 in.) diameter service lines, while larger residences and buildings located far from the main connection should have a 25-mm (1 in.) or larger service line. To properly size service lines it is essential to know the peak demands than any service tap will be called on to serve. A common method to estimate service flows is to sum the *fixture units* associated with the number and type of fixtures served by the service line and then use a curve called the *Hunter curve* to relate the peak flowrate to total fixture units. This relationship is included in most local plumbing codes and is contained in the Uniform Plumbing Code (UPC). Recent research has indicated that the peak flows estimated from fixture units and the Hunter curve provide conservative estimates of peak flows (AWWA, 2004). Irrigation demands that occur simultaneously with peak domestic demands must be added to the estimated peak domestic demands. Service lines are sized to provide an adequate service pressure downstream of the water meter when the service line is delivering the peak flow. This requires that the pressure and elevation at the tap, length of service pipe, head loss at the meter, elevation at the water meter, valve losses, and desired pressure downstream of the meter be known. Using the energy equation, the minimum service line diameter is calculated using this information. It is usually better to overdesign a service line than to underdesign a service line because of the cost of replacing a service line if service pressures turn out to be inadequate. To prevent water hammer, velocities greater than 3 m/s should be avoided. Materials used for service-line pipe and tubing are typically either copper (tubing) or plastic, which includes polyvinyl chloride (PVC) and polyethylene (PE). Type K copper is the most commonly used material for copper service lines. Older service lines used lead and galvanized iron, which are no longer recommended. The valve used to connect a small-diameter service line to a water main is called a *corporation stop*, which is sometimes loosely referred to as the corporation cock, corporation tap, corp stop, corporation, or simply corp or stop (AWWA, 2003c). Tapping a water main and inserting a corporation stop directly into the pipe wall requires a tapping machine, and taps are typically installed at the 10 or 2 o'clock position on the pipe. Guidelines for designing water service lines and meters are given in AWWA Manual M22 (AWWA, 2004). Good construction practices must be used when installing service lines to avoid costly repairs in the future. This must include burying the pipe below frost lines, maintaining proper ditch conditions, proper backfill, trench compaction, and protection from underground structures that may cause damage to the ppe.

Allowable Velocities. Maximum allowable velocities in pipeline systems are imposed to control friction losses and hydraulic transients. Maximum allowable velocities of 0.9 to 1.8 m/s are common in water-distribution pipes, and the American Water Works Association recommends a limit of about 1.5 m/s under normal operating conditions, but velocities may exceed this guideline under fire-flow conditions (AWWA, 2003c). The importance of controlling the maximum velocities in water distribution systems is supported by the fact that a change in velocity of 0.3 m/s in water transmission and distribution systems can increase the pressure in a pipe by approximately 345 kPa, while the standard design for DIP includes only a 690 kPa allowance (AWWA, 2003d).

Material. Pipeline materials should generally be selected based on a consideration of service conditions, availability, properties of the pipe, and economics. In selecting pipe materials the following considerations should be taken into account:

- Most water distribution mains in older cities in the United States are made of (gray) cast iron pipe (CIP), with many cities having CIP over 100 years old and still providing satisfactory service (Mays, 2000; AWWA, 2003d). CIP is no longer manufactured in the United States.
- For new distribution mains, ductile iron pipe (DIP) is most widely used for pipe diameters up to 760 mm (30 in.), and it has largely replaced CIP in new construction. DIP has all the good qualities of CIP plus additional strength and ductility. DIP is manufactured in diameters from 76 to 1625 mm (3–64 in.). For diameters from 100 to 500 mm (4–20 in.), standard commercial sizes are available in 50-mm (2 in.) increments, while for diameters from 600 to 1200 mm (24–48 in.), the size increments are 150 mm (6 in.). The standard lengths of DIP are 5.5 m (18 ft) and 6.1 m (20 ft). DIP is usually coated (outside and inside) with an bituminous coating to minimize corrosion. An internal cement-mortar lining 1.5 - 3 mm thick is common, and external polyethylene wraps are used to reduce corrosion in corrosive soil environments. DIP used in water systems in the United States are provided with a cement-mortar lining unless otherwise specified by the purchaser. The design of DIP and fittings are covered in AWWA Manual M41

(AWWA, 2003d) and guidance for DIP lining is covered in AWWA Standard C105 (latest edition). Tests conducted by the Ductile Iron Pipe Research Association (DIPRA) suggest a Hazen-Williams C-value of 140 is appropriate for the design of cement-mortar lined DIP. A variety of joints are available for use with DIP, which includes push-on (the most common), mechanical, flanged, ball-and-socket, and numerous joint designs. A stack of DIP is shown in Figure 2.24, and the the bell and spigot pipe ends that facilitate push-on connection are apparent. A rubber gasket, to ensure a tight fit, is contained in the bell side of the pipe.

Figure 2.24: Ductile Iron Pipe

- Steel pipe usually compares favorably with DIP for diameters larger than 400 mm (16 in.) As a consequence, steel pipe is primarily used for transmission lines in water distribution systems. Steel pipe available in diameters from 100 to 3600 mm (4–144 in.). The standard length of steel pipe is 12.2 m (40 ft).The interior of steel pipe is usually protected with either cement mortar or epoxy, and the exterior is protected by a variety of plastic coatings, bituminous materials, and polyethylene tapes depending on the degree of protection required. Guidance for the design of steel pipe are covered in AWWA Manual M11 (latest edition), and linings for steel pipe are covered under AWWA Standard C205 (latest edition) and AWWA Standard C210 (latest edition).
- Plastic materials used for fabricating water main pipe include polyvinyl chloride (PVC), polyethylene (PE), and polybutylene (PB). PVC pipe is by far the most widely used type of plastic pipe material for small-diameter water mains. The American Water Works Association standard (C900) for PVC pipe in sizes from 100 mm to 300 mm and laying lengths of 6.1 m (20 ft) is based on the same outside diameter as for DIP. In this way, standard DIP fittings can be used with PVC pipe. PVC pipe is commonly available in diameters from 100 to 914 mm. Extruded PE and PB pipe are primarily used for water service pipe in small sizes, however, the use of PB has decreased remarkably because of structural difficulties caused by premature pipe failures. Research has documented that pipe materials such as PVC, PE, and PB may be subject to permeation by lower molecular weight organic solvents or petroleum products (AWWA, 2002b). If a water

pipe must pass through an area subject to contamination, caution should be used is selecting PVC, PE, and PB pipes. In the hydraulic design of PVC pipes, a roughness height of 0.0015 mm or a Hazen-Williams C-value of 150 are appropriate for design (AWWA, 2002b). Details of large-diameter PE pipe are found in AWWA Standard C906 (latest edition) and information on PVC water main pipe is available in AWWA Manual M23 (2002b).

- Asbestos-cement (A-C) pipe has been widely installed in water distribution systems, especially in areas where metallic pipe is subject to corrosion, such as in coastal areas. It has also been installed in remote areas where its light weight makes it much easier to install than CIP. Common diameters are in the range of 100 to 890 mm. The U.S. Environmental Protection Agency banned most uses of asbestos in 1989 and, due to the manufacturing ban, new A-C pipe is no longer being installed in the United States.

- Fiberglass pipe is available for potable water used is sizes from 25 to 3600 mm. Advantages of fiberglass pipe include corrosion resistance, light weight, low installation cost, ease of repair, and hydraulic smoothness. Disadvantages include susceptibility to mechanical damage, low modulus of elasticity, and lack of standard joining system. Fiberglass pipe is covered in AWWA Standard C950 (latest edition).

- The use of concrete pressure pipe has grown rapidly since 1950. The pipe provides a combination of the high tensile strength of steel and the high compressive strength and corrosion resistance of concrete. The pipe is available in diameters ranging from 250 to 6400 mm and in standard lengths from 3.7 to 12.2 m. The design of concrete pressure pipe is covered in AWWA Manual M9 (latest edition). Concrete pipe is available with various types of liners and reinforcement, and the four types in common use in the United States and Canada are: prestressed concrete cylinder pipe, bar-wrapped concrete cylinder pipe, reinforced concrete cylinder pipe, and reinforced concrete noncylinder pipe. The manufacture of prestressed concrete cylinder pipe is covered under AWWA Standard C301 (latest edition) and AWWA Standard C304 (latest edition), bar-wrapped concrete cylinder pipe is covered under AWWA Standard C303 (latest edition), reinforced concrete cylinder pipe is covered under AWWA Standard C300 (latest edition), and reinforced concrete noncylinder pipe is covered under AWWA Standard C302 (latest edition).

Pipelines in water-distribution systems should be buried to a depth below the frost line in northern climates and at a depth sufficient to cushion the pipe against traffic loads in warmer climates (Clark, 1990). Generally, a cover of 1.2 m to 1.5 m is used for large mains and 0.75 m to 1.0 m for smaller mains. In areas where frost penetration is a significant factor, mains can have as much as 2.5 m of cover. Trenches for water mains should be as narrow as possible and still be wide enough to allow for proper joining and compaction around the pipe. The suggested trench width is the nominal pipe diameter plus 0.6 m; in deep trenches, sloping may be necessary to keep the bank from caving in. Trench bottoms should be undercut 15 to 25 cm, and sand, clean fill, or crushed stone installed to provide a cushion against the bottom of the excavation, which is usually rock (Clark, 1990). Standards for pipe construction, installation, and performance are published by the American Water Works Association in its C-series standards, which are continuously being updated.

2.5.3 Pumps

Service pressures are typically maintained by pumps, with head losses and increases in pipeline elevations acting to reduce pressures and decreases in pipeline elevations acting to increase pressures. When portions of the distribution system are separated by long distances or significant changes in elevation, *booster pumps* are sometimes used to maintain acceptable service pressures. In some cases, *fire-service pumps* are used to provide additional capacity for emergency fire protection. Pumps operate at the intersection of the pump performance curve and the system curve. Since the system curve is significantly affected by variations in water demand, there is a significant variation in pump operating conditions. In most cases, the range of operating conditions is too wide to be met by a single pump, and multiple-pump installations or variable speed pumps are required (Velon and Johnson, 1993).

2.5.4 Valves

Valves in water distribution systems are designed to perform several different functions, and the primary functions are to start and stop flow, isolate piping, regulate pressure and throttle flow, prevent backflow, and relieve pressure. *Shutoff valves* or *gate valves* are typically provided at 350-m intervals so that areas within the system can be isolated for repair or maintenance; *air-relief valves* or *air-and-vacuum relief valves* are required at high points to release trapped air; *blowoff valves* or *drain valves* may be required at low points; and *backflow prevention devices* are required by applicable regulations to prevent contamination from backflows of nonpotable water into the distribution system from system outlets. To maintain the performance of water-distribution systems it is recommended that each valve should be operated through a full cycle and then returned to its normal position on a regular schedule. The time interval between operations should be determined by the manufacturer's recommendations, size of the valve, severity of the operating conditions, and the importance of the installation (AWWA, 2003d).

2.5.5 Meters

The water meter s a changeable component of a customer's water system. Unlike the service line and water tap, whic when incorrectly sized will generally require expensive excavation and retapping, water meters can usually be changed less expensively. Selection of the type and size of a meter should be based primarily on the range of flow, and the pressure loss through the meter should also be a consideration. For many single-family residences, a 20-mm service line with a 15-mm meter is typical, while in areas where irrigation is prevalent, 20-mm or 25-mm meter may be more prevalent. Undersizing the meter can cause pressure-related problems, and oversizing the meter can result in reduced revenue and inaccurate meter recordings (since the flows do not register). Some customers such as hospitals, schools, and factories with processes requiring uninterrupted water service should have bypasses installed around the meter so that meter test and repair activities can be performed at scheduled intervals without inconvenience to either the customer or the utility. The bypass should be locked and valved appropriately.

2.5.6 Fire Hydrants

Fire hydrants are one of the few parts of a water distribution system that are visible to the public, so keeping them well maintained can help a water utility project a good public image. Fire hydrants are direct connections to the water mains and, in addition to providing an outlet for fire protection, fire hydrants are used for flushing water mains, flushing sewers, filling tank trucks for street washing, tree spraying, and providing a temporary water source for construction jobs. A typical fire hydrant is shown in Figure 2.25, along with the pipe connection between the water main and the fire hydrant. The vertical pipe connecting the water main to the fire hydrant is commonly called the *riser*. The

Figure 2.25: Fire Hydrant and Connection to Water Main

water utility is usually responsible for keeping hydrants in working order, although fire departments sometime assume this responsibility. Standard practice is to install hydrants only on mains 150 mm or larger, however, larger mains are often necessary to ensure that the residual pressure during fire flow remains greater than 140 kPa. Guidelines for the placement of hydrants are as follows (AWWA, 2003c):

- Not too close to buildings since fire fighters will not position their fire (pumper) trucks where a building wall could fall on them.
- Preferably located near street intersections, where the hose can be strung to fight a fire in any of several directions.
- Far enough from a roadway to minimize the danger of them being struck by vehicles.
- Close enough to the pavement to ensure a secure connection with the pumper and hydrant without the risk of the truck getting stuck in mud or snow.
- In areas of heavy snow, hydrants must be located where they are least liable to be covered by plowed snow or struck by snow-removal equipment.
- Hydrants should be high enough off the ground that valve caps can be removed with a standard wrench, without the wrench hitting the ground.

Fire hydrants should be inspected and operated through a full cycle on a regular schedule, and the hydrant should be flushed to prevent sediment buildup in the hydrant or connecting piping.

2.5.7 Water Storage Reservoirs

Water usually enters the system at a fairly constant rate from the treatment plant. To accommodate fluctuations in demand, a storage reservoir is typically located at the head of the system to store the excess water during periods of low demand and provide water from storage during periods of high demand. In addition to the operational storage required to accommodate diurnal (24-hour cycle) variations in water demand, storage facilities are also used to provide storage to fight fires, to provide storage for emergency conditions, and to equalize pressures in water-distribution systems.

Storage facilities are classified as either ground storage, elevated storage, or standpipes. The function and relative advantages of these types of systems are as follows:

Elevated-Storage Tanks. *Elevated storage tanks* are constructed above ground such that the height of the water in the elevated storage tank is sufficient to deliver water to the distribution system at the required pressure. The storage tank is generally supported by a steel or concrete tower, the tank is directly connected to the distribution system through a pipe called a *riser*, the water level in the storage tank is equal to the elevation of the hydraulic grade line in the distribution system (at the outlet of the storage tank), and the elevated storage tank is said to *float* on the system. Elevated storage is useful in the case of fires and emergency conditions since pumping of water from elevated tanks is not necessary, although the water must generally be pumped into elevated storage tanks. Occasionally, system pressure could become so high that the tank would overflow, and therefore altitude valves must be installed on the tank fill line to keep the tank from overflowing. Elevated tanks are usually made of steel.

Ground-Storage Reservoirs. *Ground-storage reservoirs* are constructed at or below ground level and usually discharge water to the distribution system through pumps. These systems, which are sometimes referred to simply as *distribution-system reservoirs* or *ground-level tanks*, are usually used where very large quantities of water must be stored or when an elevated tank is objectionable to the public. When a ground-level or buried reservoir is located at a low elevation on the distribution system, water is admitted through a remotely operated valve, and a pump station is provided to transfer water into the distribution system. Completely buried reservoirs are often used where an above-ground structure is objectionable, such as in a residential neighborhood. In some cases, the land over a buried reservoir can be used for recreational facilities such as a ball field or tennis court. Ground-storage reservoirs are typically constructed of steel or concrete.

Standpipes. A tank that rests on the ground with a height that is greater than its diameter is generally referred to as a *standpipe*. In most installations, only water in the upper portion of the tank will furnish usable system pressure, so most larger standpipes are equipped with an adjacent pumping system that can be used in an emergency to pump water to the system from the lower portion of the tank. Standpipes combine the advantages of elevated storage with the ability to store large quantities of water, and standpipes are usually constructed of steel. Standpipes taller than 15 m are usually uneconomical, since for taller standpipes it tends to be more economical to build an elevated tank than to accommodate the dead storage that must be pumped into the system.

Storage facilities in a distribution system are required to have sufficient volume to meet the following criteria (Velon and Johnson, 1993): (1) adequate volume to supply peak demands in excess of the maximum daily demand using no more than 50% of the available storage capacity; (2) adequate volume to supply the critical fire demand in addition to the volume required for meeting the maximum daily demand fluctuations; and (3) adequate volume to supply the average daily demand of the system for the estimated duration of a possible emergency. Conventional design practice is to rely on pumping to meet the daily operational demands up to the maximum daily demand; where detailed demand data are not available, the storage available to supply the peak demands should equal 20% to 25% of the maximum daily demand volume. Sizing the storage volume for fire protection is based on the product of the critical fire flow and duration for the service area. In extremely large systems where fire demands be only a small fraction of the maximum daily demand fire storage may not be necessary (Walski, 2000). Emergency storage is generally necessary to provide water during power outages, breaks in water mains, problems at treatment plants, unexpected shutdowns of water-supply facilities, and other sporadic events. Emergency volumes for most municipal water-supply systems vary from one to two days of supply capacity at the average daily demand. The recommended standards for water works developed by the Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers suggest a minimum emergency storage capacity equal to the average daily system demand.

In cases where elevated storage tanks are used, the minimum acceptable height of water in an elevated storage tank is determined by computing the minimum acceptable piezometric head in the service area and then adding to that figure an estimate of the head losses between the critical service location and the location of the elevated service tank, under the condition of average daily demand. The maximum height of water in the elevated tank is then determined by adding the minimum acceptable piezometric head to the head loss between the tank location and the critical service location under the condition of maximum hourly demand. The difference between the calculated minimum and maximum heights of water in the elevated storage tank is then specified as the normal operating range within the tank. The normal operating range for water in elevated tanks is usually limited to 4.5 to 6 meters, so that fluctuations in pressure are limited to 35 to 70 kPa. In most cases, the operating range is located in the upper half of the storage tank, with storage in the lower half of the tank reserved for firefighting and emergency storage. Any water stored in elevated tanks less than 14 m (46 ft) above ground is referred to as *ineffective storage* (Walski et al., 1990) since the pressure in connected distribution pipes will be less than the usual minimum acceptable pressure during emergency conditions of 140 kPa (20 psi). Operational storage in elevated tanks is normally at elevations of more than 25 m (81 ft) above the ground, since under these conditions the pressure in connected distribution pipes will exceed the usual minimum acceptable pressure during normal operations of 240 kPa (35 psi). A typical elevated storage tank is illustrated in Figure 2.26. These types of storage facilities generally have only a single pipe connection to the distribution system, and this single pipe handles both inflows and outflows from the storage tank. This piping arrangement is in contrast to ground storage reservoirs, which have separate inflow and outflow piping. The inflow piping delivers the outflow from the water-treatment facility to the reservoir, while the outflow piping delivers the water from the reservoir to the pumps that input water into the distribution system. Elevated tanks are generally made of steel (Walski, 1996), and the largest elevated storage tank in the United States (as of 2000) has a volume of 15,520 m^3 (ASCE, 2000). Elevated storage tanks are best placed on the downstream side of the largest demand from the source, with the advantages that: (1) if a pipe breaks near the source, the break will not result in disconnecting all the storage from the customers; and (2) if flow reaches the center of demand from more than one direction, the flow carried by any individual pipe will be lower and pipe sizes will generally be smaller, with associated cost savings (Walski, 2000). If there are multiple storage tanks in the distribution system, the tanks should be placed roughly the same distance from the source or sources, and all tanks should have approximately the same overflow elevation (otherwise, it may be impossible to fill the highest tank without overflowing or shutting off the lower tanks.

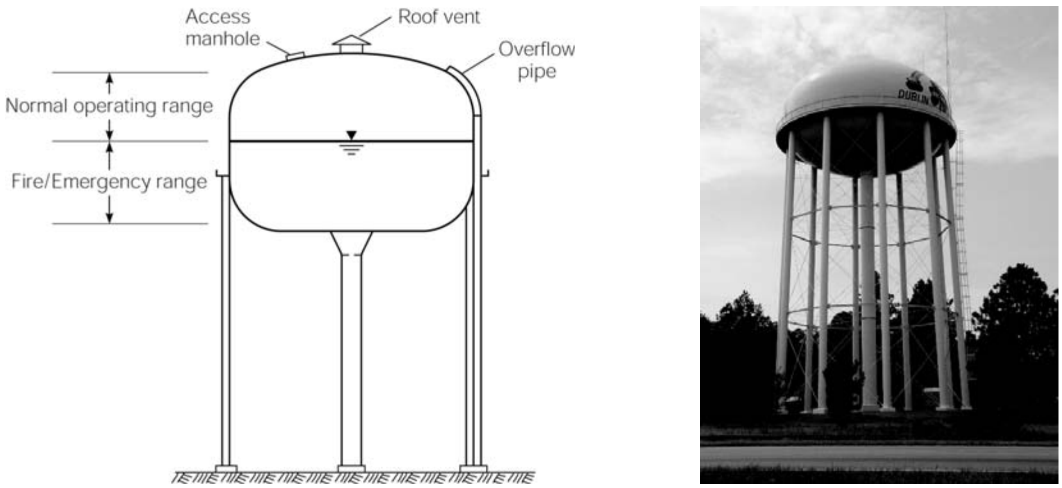


Figure 2.26: Elevated Storage Tank

Example 2.18.
A service reservoir is to be designed for a water-supply system serving 250,000 people with an average demand of 600 L/d/capita, and a needed fire flow of 37,000 L/min. Estimate the required volume of service storage.

Solution.
The required storage is the sum of three components: (1) volume to supply the demand in excess of the maximum daily demand, (2) fire storage, and (3) emergency storage.

The volume to supply the peak demand can be taken as 25% of the maximum daily demand volume. Taking the maximum daily demand factor as 1.8 (Table 2.6), then the maximum daily flowrate, Q_m, is given by

$$Q_m = (1.8)(600)(250000) = 2.7 \times 10^8 \text{ L/d} = 2.7 \times 10^5 \text{ m}^3/\text{d}$$

The storage volume to supply the peak demand, V_{peak}, is therefore given by

$$V_{\text{peak}} = (0.25)(2.7 \times 10^5) = 67500 \text{ m}^3$$

According to Table 2.10, the 37,000 L/min (= 0.62 m^3/s) fire flow must be maintained for at least 9 hours. The volume to supply the fire demand, V_{fire}, is therefore given by

$$V_{\text{fire}} = 0.62 \times 9 \times 3600 = 20100 \text{ m}^3$$

The emergency storage, V_{emer}, can be taken as the average daily demand, in which case

$$V_{\text{emer}} = 250000 \times 600 = 150 \times 10^6 \text{ L} = 150{,}000 \text{ m}^3$$

The required volume, V, of the service reservoir is therefore given by

$$\begin{aligned} V &= V_{\text{peak}} + V_{\text{fire}} + V_{\text{emer}} \\ &= 67500 + 20100 + 150000 \\ &= 237{,}600 \text{ m}^3 \end{aligned}$$

The service reservoir should be designed to store 238,000 m^3 of water. This large volume will require a ground storage tank (recall that the largest elevated-tank volume in the United States is 15,520 m^3), and it is interesting to note that most of the storage in the service reservoir is reserved for emergencies.

Example 2.19.
A water-supply system is to be designed in an area where the minimum allowable pressure in the distribution system

is 300 kPa. A hydraulic analysis of the distribution network under average daily demand conditions indicates that the head loss between the low-pressure service location, which has a pipeline elevation of 5.40 m, and the location of the elevated storage tank is 10 m. Under maximum hourly demand conditions, the head loss between the low-pressure service location and the elevated storage tank is 12 m. Determine the normal operating range for the water stored in the elevated tank.

Solution.
Under average demand conditions, the elevation z_o of the hydraulic grade line (HGL) at the reservoir location is given by

$$z_o = \frac{p_{\min}}{\gamma} + z_{\min} + h_L$$

where $p_{\min} = 300$ kPa, $\gamma = 9.79$ kN/m^3, $z_{\min} = 5.4$ m, and $h_L = 10$ m, which yields

$$z_o = \frac{300}{9.79} + 5.4 + 10 = 46.0 \text{ m}$$

Under maximum hourly demand conditions, the elevation z_1 of the HGL at the service reservoir is given by

$$z_1 = \frac{300}{9.79} + 5.4 + 12 = 48.0 \text{ m}$$

Therefore, the operating range in the storage tank should be between elevations 46.0 m and 48.0 m.

It is important to keep in mind that the best hydraulic location and most economical design are not always the deciding factors in the location of an elevated tank. In some cases, the only acceptable location will be in an industrial area or public park. In cases where public feeling is very strong, a water utility may have to construct ground-level storage, which is more aesthetically acceptable.

2.5.8 Performance Criteria for Water-Distribution Systems

The primary functions of water-distribution systems (Zipparro and Hasen, 1993) are to (1) meet the water demands of users while maintaining acceptable pressures in the system; (2) supply water for fire protection at specific locations within the system, while maintaining acceptable pressures for normal service throughout the remainder of the system; and (3) provide a sufficient level of redundancy to support a minimum level of reliable service during emergency conditions, such as an extended loss of power or a major water-main failure. Real-time operation of water distribution systems are typically based on remote measurements of pressures and storage-tank water levels within the distribution system. The pressure and water-level data are typically transmitted to a central control facility via telemetry, and adjustments to the distribution system are made from the central facility by remote control of pumps and valves within the distribution system. These electronic control systems are generally called *supervisory control and data acquisition* (SCADA) systems (Chase, 2000). Operating criteria for service pressures and storage facilities are described below.

The requirement that adequate pressures be maintained in the distribution system while supplying the service demands requires that the system be analyzed on the basis of allowable pressures. Minimum acceptable pressures are necessary to prevent contamination of the water supply from cross-connections. Criteria for minimum acceptable service pressures recommended by the Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers (GLUMB, 1987) and endorsed by the American Water Works Association (AWWA, 2003c) are typical of most water-distribution systems, and they are listed in Table 2.12. During main breaks,

Table 2.12: Minimum Acceptable Pressures in Distribution Systems

Demand condition	Minimum acceptable pressure (kPa)
Average daily demand	240–410
Maximum daily demand	240–410
Maximum hourly demand	240–410
Fire situation	> 140
Emergency conditions	> 140

Source: GLUMB (1987).

when the pressure in water-supply pipelines can drop below 140 kPa, it is not uncommon for a water utility to issue a “boil water” advisory because of the possibility of system contamination from cross connections (Chase, 2000). There are several considerations in assessing the adequacy of service pressures, including (1) the pressure required at street level for excellent flow to a 3-story building is about 290 kPa; (2) flow is adequate for residential areas if the pressure is not reduced below 240 kPa; (3) the pressure required for adequate flow to a 20-story building is about 830 kPa, which is not desirable because of the associated leakage and waste; (4) very tall buildings are usually served with their own pumping equipment; and (5) it is usually desirable to maintain normal pressures of 410–520 kPa since these pressures are adequate for the following purposes:

- To supply ordinary consumption for buildings up to 10 stories.
- To provide adequate sprinkler service in buildings of 4 to 5 stories.
- To provide direct hydrant service for quick response.
- To allow larger margin for fluctuations in pressure caused by clogged pipes and excessive length of service pipes.

Pressures higher than 650 kPa should be avoided if possible because of excessive leakage and water use, and the added burden of installing and maintaining pressure-reducing valves and other specialized equipment (Clark, 1990). Customers do not generally like high pressure because water comes out of a quickly-opened faucet with too much force (AWWA, 2003c). In addition, excessive pressures decrease the the life of water heaters and other plumbing fixtures.

2.5.9 Water Quality

The quality of water delivered to consumers can be significantly influenced by various components of a water distribution system. The principal factors affecting water quality in distribution systems are the quality of the treated water fed to the system; the material and condition of the pipes, valves, and storage facilities that make up the system; and the amount of time that the water is kept in the system (Grayman et al., 2000; AWWA, 2003c). Key processes that affect water quality within the distribution system usually include the loss of disinfection residual with resulting microbial regrowth, and the formation of disinfection byproducts such as trihalomethanes. Water-quality deterioration is often proportional to the time the water is resident in the distribution system. The longer the water is in contact with the pipe walls and is held in storage facilities, the greater the opportunity for water-quality changes. Generally, a hydraulic detention time of less than 7 days in the distribution system is recommended (AWWA, 2003c).

The velocity of flow in most mains is normally very low because mains are designed to handle fire flow, which may be several times larger than domestic flow. As a result, corrosion products and other solids tend to settle on the pipe bottom, and this problem is especially bad in dead-end mains or in areas of low water consumption. These deposits can can be a source of color, odor, and taste in the water when the deposits are stirred up by an increase in flow velocity or a reversal of flow in the distribution system. To prevent these sediments from accumulating and causing water-quality problems, pipe flushing is a typical maintenance routine. Flushing involves opening a hydrant located near the problem area, and the hydrant should be kept open as long as needed to flush out the sediment, which typically requires the removal of up to three pipe volumes (AWWA, 2003c). Only through experience will an operator be able to how often or how long certain areas should be flushed. Some systems find that dead-end mains must be flushed as often as weekly to avoid customer complaints of rusty water. The flow required for effective flushing is in the range of 0.75 - 1.1 m/s, with velocities limited to less than 3.1 - 3.7 m/s to avoid excessive scouring (AWWA, 2003c). If flushing proves to be inadequate for cleaning mains, air purging or cleaning devices such as swabs or pigs may need to be used.

In recognition of the influence of the water distribution system on water quality, water quality regulations in the United States requires that water to be sampled at at the entry point to the distribution system, at various points within the distribution system, and at consumers' taps (Kirmeyer et al., 1999). The computer program EPANET (Rossman, 2000) is widely used to simulate the water quality in distribution networks.

2.5.10 Network Analysis

Methodologies for analyzing pipe networks were discussed in Section 2.3, and these methods can be applied to any given pipe network to calculate the pressure and flow distribution under a variety of demand conditions. In complex pipe networks, the application of computer programs to implement these methodologies is standard practice (Haestad Methods Inc., 1997a; 1997b; 2002)). Computer programs allow engineers to easily calculate the hydraulic performance of complex networks and such parameters as the age of water delivered to consumers and also to trace the origin of the delivered water. Water age is measured from the time the water enters the system and gives an indication of the overall quality of the delivered water. Steady-state analyses are usually adequate for assessing the performance of various components of the distribution system, including the pipelines, storage tanks, and pumping systems, while time-dependent simulations are useful in assessing the response of the system over short time periods (days or less), evaluating the operation of pumping stations and variable-level storage tanks, performing energy consumption and cost studies, and water quality modeling (Velon and Johnson, 1993; Haestad Methods, Inc., 2002). Modelers frequently refer to time-dependent simulations as *extended-period simulations*, and several examples can be found in Larock et al. (2000).

An important part of analyzing large water-distribution systems is the *skeletonizing* of the system, which consists of representing the full water-distribution system by a subset of the system that includes only the most important elements. For example, consider the case of a water supply to the subdivision shown in Figure 2.27(a), where the system shown includes the service connections to the houses. A slight degree of skeletonization could be achieved by omitting the household service

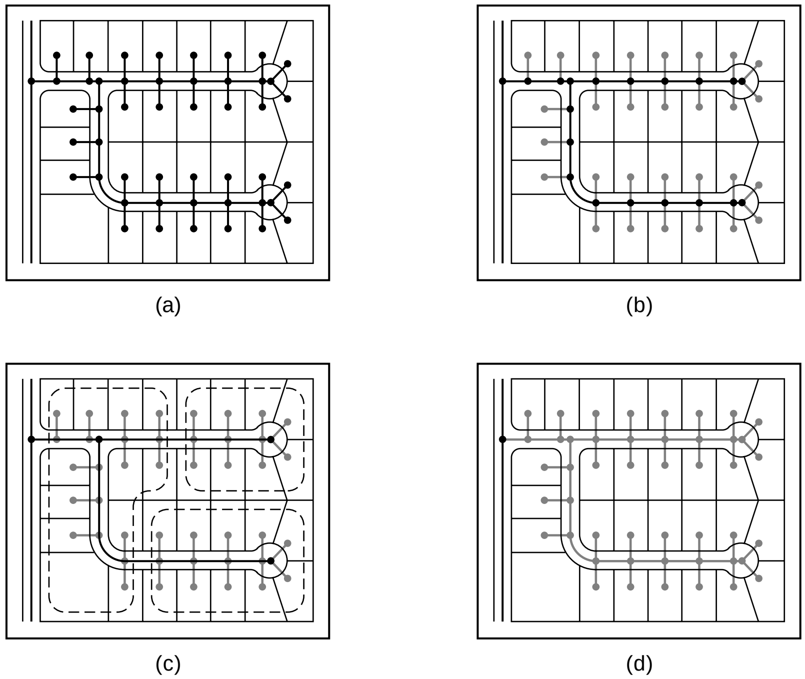


Figure 2.27: Skeletonizing a Water-Distribution System
Source: Haestad Methods, 1997 *Practical Guide: Hydraulics and Hydrology* p. 61–62. Copyright © 1997 by Haestad Methods, Inc. Reprinted by permission.

pipes (and their associated head losses) from consideration and accounting for the water demands at the tie-ins and as shown in Figure 2.27(b). This reduces the number of junctions from 48 to 19. Further skeletonization can be achieved by modeling just 4 junctions, consisting of the ends of the main piping and the major intersections shown in Figure 2.27(c). In this case, the water demands are associated with the nearest junctions to each of the service connections, and the dashed lines in Figure 2.27(c) indicate the service areas for each junction. A further level of skeletonization is shown in Figure 2.27(d), where the water supply to the entire subdivision is represented by a single node, at which the water demand of the subdivision is attributed.

Clearly, further levels of skeletonization could be possible in large water-distribution systems. As a general guideline, larger systems permit more degrees of skeletonization without introducing significant error in the flow conditions of main distribution pipes.

The results of a pipe-network analysis should generally include pressures and/or hydraulic grade line elevations at all nodes, flow, velocity, and head loss through all pipes as well as rates of flow into and out of all storage facilities. These results are used to assess the hydraulic performance and reliability of the network, and they are to be compared with the guidelines and specifications required for acceptable performance.

2.6 Computer Models

Several good computer models are available for simulating flow in closed conduits, with the majority of these models developed primarily for computing flows and pressure distributions in water-supply networks. In engineering practice, the use of computer models to apply the fundamental principles covered in this chapter is usually essential. In choosing a model for a particular application, there are usually a variety of models to choose from, however, in doing work that is to be reviewed by regulatory bodies, models developed and maintained by agencies of the United States government have the greatest credibility and, perhaps more importantly, are almost universally acceptable in supporting permit applications and defending design protocols. A secondary guideline in choosing a model is that the simplest model that will accomplish the design objectives should be given the most serious consideration.

EPANET. EPANET is a water-distribution-system modeling package developed by the U.S. Environmental Protection Agency's Water Supply and Water Resources Division. It performs extended-period simulation of hydraulic and water-quality behavior within pressurized pipe networks. A more detailed description of EPANET can be found in Rossman (2000), and the program can be downloaded from the World Wide Web at `www.epa.gov/ORD/NRMRL/wswrd/epanet.html`.

Summary

The hydraulics of flow in closed conduits is the basis for designing water-supply systems and other systems that involve the transport of water under pressure. The fundamental relationships governing flow in closed conduits are the conservation laws of mass, momentum, and energy; the forms of these equations that are most useful in engineering applications are derived from first principles. Of particular note is the momentum equation, the most useful form of which is the Darcy-Weisbach equation. Techniques for analyzing flows in both single and multiple pipelines, using the nodal and loop methods, are presented. Flows in closed conduits are usually driven by pumps, and the fundamentals of pump performance using dimensional analysis and similitude are presented. Considerations in selecting a pump include the specific speed under design conditions, the application of affinity laws in adjusting pump performance curves, the computation of operating points in pump-pipeline systems, practical limits on pump location based on the critical cavitation parameter, and the performance of pump systems containing multiple units.

Water-supply systems are designed to meet service-area demands during the design life of the system. Projection of water demand involves the estimation of per capita demands and population projections. Over short time scales, populations can be expected to follow either geometric, arithmetic, or declining growth models, while over longer time scales a logistic growth curve may be more appropriate. Components of water-supply systems must be designed to accommodate daily fluctuations in water demand plus potential fire flows. The design periods and capacities of various components of water-supply systems are listed in Table 2.11. Other key considerations in designing water distribution systems include required service pressures (Table 2.12), pipeline selection and installation, and provision of adequate storage capacity to meet fire demands and emergency conditions.

Problems

2.1. Water at 20°C is flowing in a 100-mm diameter pipe at an average velocity of 2 m/s. If the diameter of the pipe is suddenly expanded to 150 mm, what is the new velocity in the pipe? What are the volumetric and mass flowrates in the pipe?

2.2. A 200-mm diameter pipe divides into two smaller pipes each of diameter 100 mm. If the flow divides equally between the two smaller pipes and the velocity in the 200-mm pipe is 1 m/s, calculate the velocity and flowrate in each of the smaller pipes.

2.3. The velocity distribution in a pipe is given by the equation

$$v(r) = V_o \left[1 - \left(\frac{r}{R} \right)^2 \right] \tag{2.145}$$

where $v(r)$ is the velocity at a distance r from the centerline of the pipe, V_o is the centerline velocity, and R is the radius of the pipe. Calculate the average velocity and flowrate in the pipe in terms of V_o.

2.4. Calculate the momentum correction coefficient, β, for the velocity distribution given in Equation 2.145.

2.5. Water is flowing in a horizontal 200-mm diameter pipe at a rate of 0.06 m^3/s, and the pressures at sections 100 m apart are equal to 500 kPa at the upstream section and 400 kPa at the downstream section. Estimate the average shear stress on the pipe and the friction factor, f. [*Hint*: Use Equation 2.26 to calculate the shear stress and Equation 2.32 to calculate the friction factor.]

2.6. Water at 20°C flows at a velocity of 2 m/s in a 250-mm diameter horizontal ductile iron pipe. Estimate the friction factor in the pipe, and state whether the pipe is hydraulically smooth or rough. Compare the friction factors derived from the Moody diagram, the Colebrook equation, and the Jain equation. Estimate the change in pressure over 100 m of pipeline. How would the friction factor and pressure change be affected if the pipe is not horizontal but 1 m lower at the downstream section?

2.7. Show that the Colebrook equation can be written in the (slightly) more convenient form:

$$f = \frac{0.25}{\{\log[(k_s/D)/3.7 + 2.51/(\mathrm{Re}\sqrt{f})]\}^2}$$

Why is this equation termed "(slightly) more convenient"?

2.8. If you had your choice of estimating the friction factor either from the Moody diagram or from the Colebrook equation, which one would you pick? Explain your reasons.

2.9. Water leaves a treatment plant in a 500-mm diameter ductile iron pipeline at a pressure of 600 kPa and at a flowrate of 0.50 m^3/s. If the elevation of the pipeline at the treatment plant is 120 m, then estimate the pressure in the pipeline 1 km downstream where the elevation is 100 m. Assess whether the pressure in the pipeline would be sufficient to serve the top floor of a 10-story (approximately 30 m high) building.

2.10. A 25-mm diameter galvanized iron service pipe is connected to a water main in which the pressure is 400 kPa. If the length of the service pipe to a faucet is 20 m and the faucet is 2.0 m above the main, estimate the flowrate when the faucet is fully open.

2.11. A galvanized iron service pipe from a water main is required to deliver 300 L/s during a fire. If the length of the service pipe is 40 m and the head loss in the pipe is not to exceed 45 m, calculate the minimum pipe diameter that can be used. Use the Colebrook equation in your calculations.

2.12. Repeat Problem 2.11 using the Swamee-Jain equation.

2.13. Use the velocity distribution given in Problem 2.3 to estimate the kinetic energy correction factor, α, for turbulent pipe flow.

2.14. The velocity profile, $v(r)$, for turbulent flow in smooth pipes is sometimes estimated by the seventh-root law, originally proposed by Blasius (1911)

$$v(r) = V_o \left(1 - \frac{r}{R}\right)^{\frac{1}{7}}$$

where V_o is the maximum (centerline) velocity and R is the radius of the pipe. Estimate the energy and momentum correction factors corresponding to the seventh-root law.

2.15. Show that the kinetic energy correction factor, α, corresponding to the power-law velocity profile is given by Equation 2.75. Use this result to confirm your answer to Problem 2.14.

2.16. Water enters and leaves a pump in pipelines of the same diameter and approximately the same elevation. If the pressure on the inlet side of the pump is 30 kPa and a pressure of 500 kPa is desired for the water leaving the pump, what is the head that must be added by the pump, and what is the power delivered to the fluid?

2.17. Water leaves a reservoir at 0.06 m^3/s through a 200-mm riveted steel pipeline that protrudes into the reservoir and then immediately turns a 90° bend with a minor loss coefficient equal to 0.3. Estimate the length of pipeline required for the friction losses to account for 90% of the total losses, which includes both friction losses and so-called "minor losses". Would it be fair to say that for pipe lengths shorter than the length calculated in this problem that the word "minor" should not be used?

2.18. The top floor of an office building is 40 m above street level and is to be supplied with water from a municipal pipeline buried 1.5 m below street level. The water pressure in the municipal pipeline is 450 kPa, the sum of the local loss coefficients in the building pipes is 10.0, and the flow is to be delivered to the top floor at 20 L/s through a 150 mm diameter PVC pipe. The length of the pipeline in the building is 60 m, the water temperature is 20°C, and the water pressure on the top floor must be at least 150 kPa. Will a booster pump be required for the building? If so, what power must be supplied by the pump?

2.19. Water is pumped from a supply reservoir to a ductile iron water transmission line, as shown in Figure 2.28. The high point of the transmission line is at point A, 1 km downstream of the supply reservoir, and the low point of the transmission line is at point B, 1 km downstream of A. If the flowrate through the pipeline is 1 m^3/s, the diameter of the pipe is 750 mm, and the pressure at A is to be 350 kPa, then: (a) estimate the head that must be added by the pump; (b) estimate the power supplied by the pump; and (c) calculate the water pressure at B.

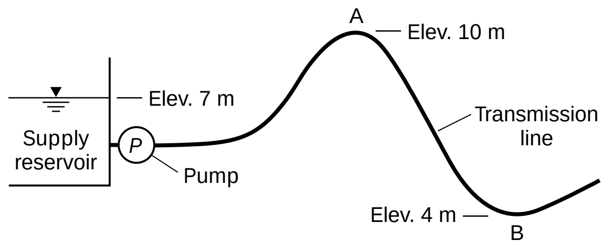


Figure 2.28: Problem 2.19

2.20. A pipeline is to be run from a water-treatment plant to a major suburban development 3 km away. The average daily demand for water at the development is 0.0175 m^3/s, and the peak demand is 0.578 m^3/s. Determine the required diameter of ductile iron pipe such that the flow velocity during peak demand is 2.5 m/s. Round the pipe diameter upward to the nearest 25 mm (i.e., 25 mm, 50 mm, 75 mm, ...). The water pressure at the development is to be at least 340 kPa during average demand conditions, and 140 kPa during peak demand. If the water at the treatment plant is stored in a ground-level reservoir where the level of the water is 10.00 m NGVD and the ground elevation at the suburban development is 8.80 m NGVD, determine the pump power (in kilowatts) that must be available to meet both the average daily and peak demands.

2.21. Water flows at 5 m^3/s in a 1 m $\times$ 2 m rectangular concrete pipe. Calculate the head loss over a length of 100 m.

2.22. Water flows at 10 m^3/s in a 2 m $\times$ 2 m square reinforced-concrete pipe. If the pipe is laid on a (downward) slope of 0.002, what is the change in pressure in the pipe over a distance of 500 m?

2.23. Derive the Hazen-Williams head-loss relation, Equation 2.84, starting from Equation 2.82.

2.24. Compare the Hazen-Williams formula for head loss (Equation 2.84) with the Darcy-Weisbach equation for head loss (Equation 2.33) to determine the expression for the friction factor that is assumed in the Hazen-Williams formula. Based on your result, identify the type of flow condition incorporated in the Hazen-Williams formula (rough, smooth, or transition).

2.25. Derive the Manning head-loss relation, Equation 2.86.

2.26. Compare the Manning formula for head loss (Equation 2.86) with the Darcy-Weisbach equation for head loss (Equation 2.33) to determine the expression for the friction factor that is assumed in the Manning formula. Based on your result, identify the type of flow condition incorporated in the Manning formula (rough, smooth, or transition).

2.27. Determine the relationship between the Hazen-Williams roughness coefficient and the Manning roughness coefficient.

2.28. Given a choice between using the Darcy-Weisbach, Hazen-Williams, or Manning equations to estimate the friction losses in a pipeline, which equation would you choose? Why?

2.29. Water flows at a velocity of 2 m/s in a 300-mm new ductile iron pipe. Estimate the head loss over 500 m using: (a) the Hazen-Williams formula; (b) the Manning formula; and (c) the Darcy-Weisbach equation. Compare your results. Calculate the Hazen-Williams roughness coefficient and the Manning coefficient that should be used to obtain the same head loss as the Darcy-Weisbach equation.

2.30. Reservoirs A, B, and C are connected as shown in Figure 2.29. The water elevations in reservoirs A, B, and C are 100 m, 80 m, and 60 m, respectively. The three pipes connecting the reservoirs meet at the junction J, with pipe AJ being 900 m long, BJ 800 m long, CJ 700 m long, and the diameter of all pipes equal to 850 mm. If all pipes are made of ductile iron and the water temperature is 20°C, find the flow into or out of each reservoir.

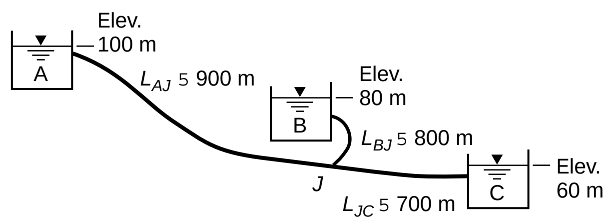


Figure 2.29: Problem 2.30

2.31. The water-supply network shown in Figure 2.30 has constant-head elevated storage tanks at A and B, with inflows and withdrawals at C and D. The network is on flat terrain, and the pipeline characteristics are as follows:

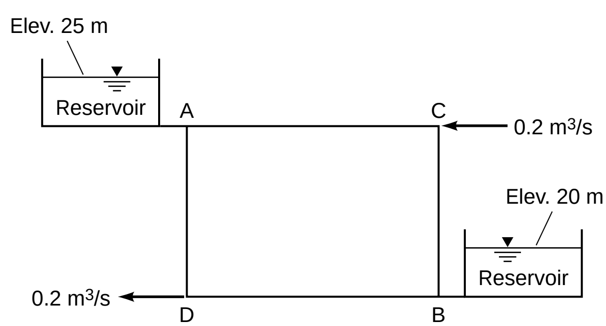


Figure 2.30: Problem 2.31

Pipe	L (km)	D (mm)
AD	1.0	400
BC	0.8	300
BD	1.2	350
AC	0.7	250

If all pipes are made of ductile iron, calculate the inflows/outflows from the storage tanks. Assume that the flows in all pipes are fully turbulent.

2.32. Consider the pipe network shown in Figure 2.31. The Hardy Cross method can be used to calculate the pressure distribution in the system, where the friction loss, h_f, is estimated using the equation

$$h_f = rQ^n$$

and all pipes are made of ductile iron. What value of r and n would you use for each pipe in the system? The pipeline characteristics are as follows:

Pipe	L (m)	D (mm)
AB	1,000	300
BC	750	325
CD	800	200
DE	700	250
EF	900	300
FA	900	250
BE	950	350

You can assume that the flow in each pipe is hydraulically rough.

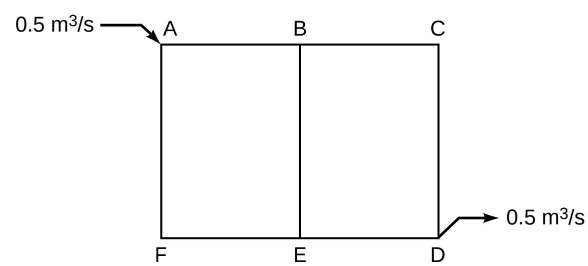


Figure 2.31: Problem 2.32

2.33. A portion of a municipal water distribution network is shown in Figure 2.32, where all pipes are made of ductile iron and have diameters of 300 mm. Use the Hardy Cross method to find the flowrate in each pipe. If the pressure at point P is 500 kPa and the distribution network is on flat terrain, determine the water pressures at each pipe intersection.

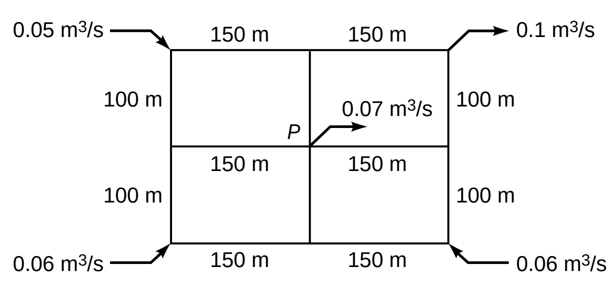


Figure 2.32: Problem 2.33

2.34. What is the constant that can be used to convert the specific speed in SI units (Equation 2.111) to the specific speed in U.S. Customary units (Equation 2.112)?

2.35. What is the highest synchronous speed for a motor driving a pump?

2.36. Derive the affinity relationship for the power delivered to a fluid by two homologous pumps. [*Note*: This affinity relation is given by Equation 2.116.]

2.37. A pump is required to deliver 150 L/s ($\pm$ 10%) through a 300-mm diameter PVC pipe from a well to a reservoir. The water level in the well is 1.5 m below the ground and the water surface in the reservoir is 2 m above the ground. The delivery pipe is 300 m long, and minor losses can be neglected. A pump manufacturer suggests using a pump with a performance curve given by

$$h_p = 6 - 6.67 \times 10^{-5} Q^2$$

where h_p is in meters and Q in L/s. Is this pump adequate?

2.38. A pump is to be selected to deliver water from a well to a treatment plant through a 300-m long pipeline. The temperature of the water is 20°C, the average elevation of the water surface in the well is 5 m below ground surface, the pump is 50 cm above ground surface, and the water surface in the receiving reservoir at the water-treatment plant is 4 m above ground surface. The delivery pipe is made of ductile iron (k_s = 0.26 mm) with a diameter of 800 mm. If the selected pump has a performance curve of $h_p = 12 - 0.1Q^2$, where Q is in m^3/s and h_p is in m, then what is the flowrate through the system? Calculate the specific speed of the required pump (in U.S. Customary units), and state what type of pump will be required when the speed of the pump motor is 1,200 rpm. Neglect minor losses.

2.39. A pump lifts water through a 100-mm diameter ductile iron pipe from a lower to an upper reservoir (Figure 2.33). If the difference in elevation between the reservoir surfaces is 10 m, and the performance curve of the 2400-rpm pump is given by

$$h_p = 15 - 0.1Q^2$$

where h_p is in meters and Q in L/s, then estimate the flowrate through the system. If the pump manufacturer gives the required net positive suction head under these operating conditions as 1.5 m, what is the maximum height above the lower reservoir that the pump can be placed and maintain the same operating conditions?

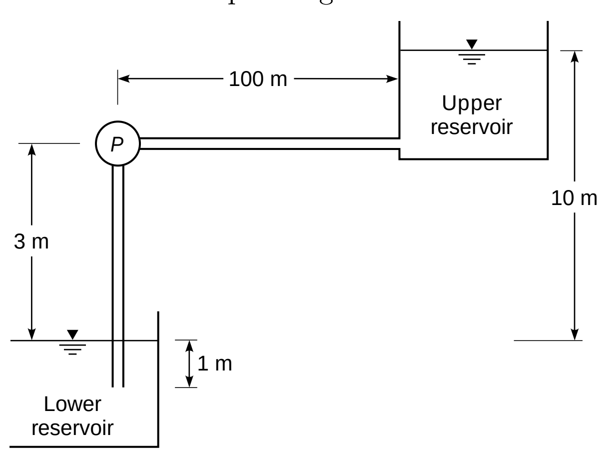


Figure 2.33: Problem 2.39

2.40. Water is being pumped from reservoir A to reservoir F through a 30-m long PVC pipe of diameter 150 mm (see Figure 2.34). There is an open gate valve located at C; 90° bends (threaded) located at B, D, and E; and the pump performance curve is given by

$$h_p = 20 - 4713Q^2$$

where h_p is the head added by the pump in meters and Q is the flowrate in m^3/s. The specific speed of the pump (in U.S. Customary units) is 3,000. Assuming that the flow is turbulent (in the smooth, rough, or transition range) and the temperature of the water is 20°C, then (a) write the energy equation between the upper and lower reservoirs, accounting for entrance, exit, and minor losses between A and F; (b) calculate the flowrate and velocity in the pipe; (c) if the required net positive suction head at the pump operating point is 3.0 m, assess the potential for cavitation in the pump (for this analysis you may assume that the head loss in the pipe is negligible between the intake and the pump); and (d) use the affinity laws to estimate the pump performance curve when the motor on the pump is changed from 800 rpm to 1,600 rpm.

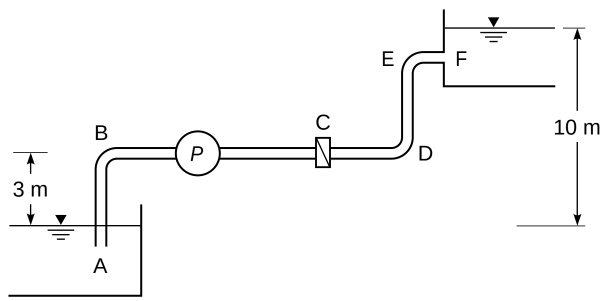


Figure 2.34: Problem 2.40

2.41. If the performance curve of a certain pump model is given by

$$h_p = 30 - 0.05Q^2$$

where h_p is in meters and Q is in L/s, what is the performance curve of a pump system containing n of these pumps in series? What is the performance curve of a pump system containing n of these pumps in parallel?

2.42. A pump is placed in a pipe system in which the energy equation (system curve) is given by

$$h_p = 15 + 0.03Q^2$$

where h_p is the head added by the pump in meters and Q is the flowrate through the system in L/s. The performance curve of the pump is

$$h_p = 20 - 0.08Q^2$$

What is the flowrate through the system? If the pump is replaced by two identical pumps in parallel, what would be the flowrate in the system? If the pump is replaced by two identical pumps in series, what would be the flowrate in the system?

2.43. Derive an expression for the population, P, versus time, t, where the growth rate is: (a) geometric, (b) arithmetic, and (c) declining.

2.44. The design life of a planned water-distribution system is to end in the year 2030, and the population in the town has been measured every 10 years since 1920 by the U.S. Census Bureau. The reported populations are tabulated below. Estimate the population in the town using: (a) graphical extension, (b) arithmetic growth projection, (c) geometric growth projection, (d) declining growth projection (assuming a saturation concentration of 100,000 people), and (e) logistic curve projection.

Year	Population
1920	25,521
1930	30,208
1940	30,721
1950	37,253
1960	38,302
1970	41,983
1980	56,451
1990	64,109

2.45. A city founded in 1950 had a population of 13,000 in 1960; 125,000 in 1975; and 300,000 in 1990. Assuming that the population growth follows a logistic curve, estimate the saturation population of the city.

2.46. The average demand of a population served by a water-distribution system is 580 L/d/capita, and the population at the end of the design life is estimated to be 100,000 people. Estimate the maximum daily demand and maximum hourly demand.

2.47. Estimate the flowrate and volume of water required to provide adequate fire protection to a five-story office building constructed of joisted masonry. The effective floor area of the building is 5,000 m^2.

2.48. What is the maximum fire flow and corresponding duration that can be estimated for any building?

2.49. A water-supply system is being designed to serve a population of 200,000 people, with an average per capita demand of 600 L/d/person and a needed fire flow of 28,000 L/min. If the water supply is to be drawn from a river, then what should be the design capacity of the supply pumps and water-treatment plant? For what duration must the fire flow be sustained, and what volume of water must be kept in the service reservoir to accommodate a fire? What should the design capacity of the distribution pipes be?

2.50. What is the minimum acceptable water pressure in a distribution system under average daily demand conditions?

2.51. Calculate the volume of storage required for the elevated storage reservoir in the water-supply system described in Problem 2.49.

Bibliography

[1] M. Abramowitz and I.A. Stegun. *Handbook of Mathematical Functions.* Dover, New York, New York, 1965.

[2] M. Abramowitz and I.A. Stegun. *Handbook of Mathematical Functions.* Dover, New York, New York, 1972.

[3] W. Abtew. Evapotranspiration Measurements and Modeling for Three Wetland Systems in South Florida. *Journal of the American Water Resources Association*, 32(3):465–473, 1996.

[4] W. Abtew, J.Obeysekera, M. Irizarry-Ortiz, D. Lyons, and A. Reardon. Evapotranspiration Estimation for South Florida. EMA 407, South Florida Water Management District, West Palm Beach, Florida, January 2003.

[5] W. Abtew and J. Obeysekera. Lysimeter study of evapotranspiration of cattails and comparison of three estimation methods. *Transactions of the American Society of Agricultural Engineers*, 38(1):121–129, 1995.

[6] M.M. Abu-Seida and A.A. Quraishi. A flow equation for submerged rectangular weirs. *Proceedings of the Institution of Civil Engineers*, 61(2):685–696, 1976.

[7] P. Ackers, W.R. White, J.A. Perkins, and A.J.M. Harrison. *Weirs and Flumes for Flow Measurement.* Wiley, Chichester, 1978.

[8] N. Ahmed and D.K. Sunada. Nonlinear flow in porous media. *Journal of the Hydraulics Division, ASCE*, 95(HY6):1847–1857, 1969.

[9] T.P. Ahrens. Well design criteria. *Water Well Journal, National Water Well Association*, September and November 1957.

[10] A.J. Aisenbrey, Jr., R.B. Hayes, H.J. Warren, D.L. Winsett, and R.B. Young. *Design of Small Canal Structures.* United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1974.

[11] A.O. Akan. *Urban Stormwater Hydrology.* Technomic Publishing Company, Inc., Lancaster, Pennsylvania, 1993.

[12] R. Aldridge and R.J. Jackson. Interception of rainfall by hard beech. *New Zealand Journal of Science*, 16:185–198, 1973.

[13] A. Alexandrou. *Principles of Fluid Mechanics*. Prentice-Hall, Upper Saddle River, New Jersey, 2001.

[14] Y. Alila. Regional rainfall depth-duration-frequency equations for Canada. *Water Resources Research*, 36(7):1767–1778, July 2000.

[15] R.G. Allen. A Penman for all seasons. *Journal of Irrigation and Drainage Engineering, ASCE*, 112(4):348–368, 1986.

[16] R.G. Allen. Self-calibrating method for estimating solar radiation from air temperature. *Journal of Irrigation and Drainage Engineering, ASCE*, 2(2):56–67, April 1997.

[17] R.G. Allen, M.E. Jensen, J.L. Wright, and R.D. Burman. Operational estimates of reference evapotranspiration. *Journal of Agronomy*, 81:650–662, 1989.

[18] R.G. Allen, L.S. Pereira, D. Raes, and M. Smith. Crop evapotranspiration. Guidelines for computer crop water requirements. Irrigation and Drainage Paper 56, Food and Agricultural Organization of the United Nations (FAO), Rome, Italy, 1998.

[19] R.G. Allen and W.O. Pruitt. Rational use of the FAO Blaney-Criddle formula. *Journal of Irrigation and Drainage Engineering, ASCE*, 112(IR2):139–155, 1986.

[20] R.G. Allen, M. Smith, L.S. Pereira, and A. Perrier. An update for the calculation of reference evapotranspiration. *ICID Bulletin*, 43(2):35–92, 1994b.

[21] R.G. Allen, M. Smith, A. Perrier, and L.S. Pereira. An update for the definition of reference evapotranspiration. *ICID Bulletin*, 43(2):1–34, 1994a.

[22] M. Amein. Stream flow routing on computer by characteristics. *Water Resources Research*, 2(1):123–130, 1966.

[23] M. Amein and C.S. Fang. Stream flow routing (with applications to North Carolina rivers). Report 17, Water Resources Research Institute of the University of North Carolina, Raleigh, North Carolina, 1969.

[24] American Concrete Pipe Association. *Concrete Pipe Handbook*. American Concrete Pipe Association, Vienna, Virginia, 1981.

[25] American Concrete Pipe Association. *Concrete Pipe Design Manual*. American Concrete Pipe Association, Vienna, Virginia, 1985.

[26] American Iron and Steel Institute. *Modern Sewer Design*. AISI, Washington, DC, third edition, 1995.

[27] American Society for Testing and Materials. Standard practice for design and installation of ground water monitoring wells in aquifers. Technical Report D 5092-1990, ASTM Committee D-18 on Soil and Rock, 1990.

[28] American Society for Testing and Materials. Standard guide for selection of aquifer test method in determining of hydraulic properties by well techniques. Technical Report D 4043-91, ASTM Committee D-18 on Soil and Rock, 1991a.

[29] American Society for Testing and Materials. Standard test method (field procedure) for withdrawal and injection well tests for determining hydraulic properties of aquifer systems. Technical Report D 4050-91, ASTM Committee D-18 on Soil and Rock, 1991b.

[30] American Society for Testing and Materials. Standard test method (analytical procedure) for determining transmissivity and storage coefficient of nonleaky confined aquifers by overdamped well response to instantaneous change in head(slug test). Technical Report D 4106-91, ASTM Committee D-18 on Soil and Rock, 1991c.

[31] American Society of Civil Engineers. Report of ASCE Task Force on Friction Factors in Open Channels. *Proceedings of the American Society of Civil Engineers*, 89(HY2):97, March 1963.

[32] American Society of Civil Engineers. *Gravity Sanitary Sewer Design and Construction*. American Society of Civil Engineers, New York, New York, 1982. Manual of Practice No.60.

[33] American Society of Civil Engineers. *Civil engineering guidelines for planning and designing hydroelectric developments*. ASCE, New York, New York, 1989.

[34] American Society of Civil Engineers. *Design and Construction of Urban Stormwater Management Systems*. American Society of Civil Engineers, New York, New York, 1992.

[35] American Society of Civil Engineers. *Hydrology Handbook*. American Society of Civil Engineers, New York, New York, second edition, 1996. Manual of Practice No.28.

[36] American Society of Civil Engineers. *Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers, No.19: Flood-Runoff Analysis*. ASCE Press, New York, New York, 1996.

[37] American Society of Civil Engineers. *Urban Runoff Quality Management*. American Society of Civil Engineers, New York, New York, 1998. Manual of Practice No.23.

[38] American Society of Heating, Refrigerating and Air Conditioning Engineers. *ASHRAE Handbook, 1981. Fundamentals*. ASHRAE, New York, New York, 1981.

[39] American Water Works Association. *1984 Water Utility Operating Data*. AWWA, Denver, Colorado, 1986.

[40] American Water Works Association. *Distribution System Requirements for Fire Protection, Manual of Water Supply Practices M31*. AWWA, Denver, Colorado, 1992.

[41] American Water Works Association. *Water Resources Planning, Manual of Water Supply Practices M50*. AWWA, Denver, Colorado, First edition, 2001.

[42] American Water Works Association. *PVC Pipe–Design and Installation, Manual of Water Supply Practices M23*. AWWA, Denver, Colorado, Second edition, 2002b.

[43] American Water Works Association. *Water Sources*. Principles and Practices of Water Supply Operations. AWWA, Denver, Colorado, Third edition, 2003a.

[44] American Water Works Association. *Groundwater, Manual of Water Supply Practices M21*. AWWA, Denver, Colorado, Third edition, 2003b.

[45] American Water Works Association. *Water Transmission and Distribution.* AWWA, Denver, Colorado, Third edition, 2003c.

[46] American Water Works Association. *Ductile-Iron Pipe and Fittings, Manual of Water Supply Practices M41.* AWWA, Denver, Colorado, Second edition, 2003d.

[47] American Water Works Association. *Sizing Water Service Lines and Meters, Manual of Water Supply Practices M22.* AWWA, Denver, Colorado, Second edition, 2004.

[48] American Water Works Association. *American National Standard for Polyethylene Encasement for Ductile-Iron Pipe Systems.* ANSI/AWWA C105/A21.5. AWWA, Denver, Colorado, latest edition.

[49] American Water Works Association. *AWWA Standard for Cement-Mortar Protective Lining and Coating for Steel Water Pipe–4 in. (100 mm) and Larger–Shop Applied.* ANSI/AWWA C205. AWWA, Denver, Colorado, latest edition.

[50] American Water Works Association. *AWWA Standard for Concrete Pressure Pipe, Bar-Wrapped, Steel Cylinder Type.* ANSI/AWWA C303. AWWA, Denver, Colorado, latest edition.

[51] American Water Works Association. *AWWA Standard for Design of Prestressed Concrete Cylinder Pipe.* ANSI/AWWA C304. AWWA, Denver, Colorado, latest edition.

[52] American Water Works Association. *AWWA Standard for Fiberglass Pressure Pipe.* ANSI/AWWA C950. AWWA, Denver, Colorado, latest edition.

[53] American Water Works Association. *AWWA Standard for Liquid-Epoxy Coating Systems for the Interior and Exterior of Steel Water Pipelines.* ANSI/AWWA C210. AWWA, Denver, Colorado, latest edition.

[54] American Water Works Association. *AWWA Standard for Polyethylene (PE) Pressure Pipe and Fittings, 4 in. (100 mm) through 63 in. (1575 mm) for Water Distribution and Transmission.* ANSI/AWWA C906. AWWA, Denver, Colorado, latest edition.

[55] American Water Works Association. *AWWA Standard for Prestressed Concrete Pressure Pipe, Steel Cylinder Type.* ANSI/AWWA C301. AWWA, Denver, Colorado, latest edition.

[56] American Water Works Association. *AWWA Standard for Reinforced Concrete Pressure Pipe, Steel-Cylinder Type.* ANSI/AWWA C300. AWWA, Denver, Colorado, latest edition.

[57] American Water Works Association. *Manual M11, Steel Pipe–A Guide for Design and Installation.* AWWA, Denver, Colorado, latest edition.

[58] American Water Works Association. *Manual M9, Concrete Pressure Pipe.* AWWA, Denver, Colorado, latest edition.

[59] W.F. Ames. *Numerical Methods for Partial Differential Equations.* Academic Press, New York, New York, second edition, 1977.

[60] A.G. Anderson, A.. Painta, and J.T. Davenport. Tentative design procedure for riprap lined channels. Technical report, Highway Research Board, National Academy of Sciences, Washington, DC, 1970. NCHRP Report No. 108.

[61] M.G. Anderson and T.P. Burt. The role of topography in controlling throughflow generation. *Earth Surface Processes and Landforms*, 3:27–36, 1978.

[62] M.P. Anderson and W.W. Woessner. *Applied Groundwater Modeling, Simulation of Flow and Transport*. Academic Press, New York, New York, 1992.

[63] ASCE. Elevated water tank rises to new levels. *Civil Engineering*, 70(1):19, January 2000.

[64] ASCE Task Committee on Ground Water Protection. *Ground Water, Protection Alternatives and Strategies in the U.S.A.* ASCE, New York, 1997.

[65] ASME B36.19M. *Stainless Steel Pipe*. American Society of Mechanical Engineers, New York, New York, 1985.

[66] Atlanta Regional Commission/Georgia Department of Natural Resources-Environmental Protection Division, Atlanta, Georgia. *Georgia Stormwater Management Manual*, 2003. www.georgiastormwater.com.

[67] G. Aussenac and C. Boulangeat. Interception des precipitation et evapotranspiration reelle dans des peuplements de feuillu (*fagus silvatica* L.) et de resineux (*pseudotsuga menziesii* (Mirb.) Franco). *Annales des Sciences Forestieres*, 37(2):91–107, 1980.

[68] H.E. Babbitt and E.R. Baumann. *Sewerage and Sewage Treatment*. John Wiley & Sons, New York, New York, 1958.

[69] W. Badon-Ghyben. Nota in Verband met de Voorgenomen Putboring Nabij Amsterdam (Notes on the probable results of well drilling near Amsterdam). Technical Report 1888/9, Tijdschr. Kon. Inst. Ing., The Hague, 1888.

[70] B.A. Bakhmeteff. *Hydraulics of Open Channel Flow*. McGraw-Hill, Inc., New York, New York, 1932.

[71] Barre de Saint-Venant. Theory of unsteady water flow, with application to river floods and to propagation of tides in river channels. *French Academy of Science*, 73:148–154, 237–240, 1871.

[72] V. Batu. *Aquifer Hydraulics*. John Wiley & Sons, New York, New York, 1998.

[73] J. Bear. *Dynamics of Fluids in Porous Media*. Dover Publications, Inc., New York, New York, 1972.

[74] J. Bear. *Hydraulics of Groundwater*. McGraw-Hill Book Company, New York, New York, 1979.

[75] J. Bear and G. Dagan. The transition zone between fresh and salt waters in a coastal aquifer, Progress Report 1:The steady interface between two immiscible fluids in a two-dimensional field of flow. Technical report, Hydraulic Lab, Technion, Haifa, Israel, 1962.

[76] J.S. Beard. Rainfall interception by grass. *South African Forestry Journal*, 42:12–25, 1962.

[77] P.B. Bedient and W.C. Huber. *Hydrology and Floodplain Analysis*. Prentice-Hall, Upper Saddle River, New Jersey, 1992.

[78] P.B. Bedient and W.C. Huber. *Hydrology and Floodplain Analysis*. Prentice-Hall, Upper Saddle River, New Jersey, third edition, 2002.

[79] P.B. Bedient, H.S. Rifai, and C.J. Newell. *Ground Water Contamination*. Prentice-Hall, Upper Saddle River, New Jersey, second edition, 1999.

[80] T.V. Belanger and M.T. Montgomery. Seepage meter errors. *Limnol. & Oceanogr.*, 37:1787–1795, 1992.

[81] J.R. Benjamin and C. Allin Cornell. *Probability, Statistics, and Decision for Civil Engineers*. McGraw-Hill, Inc., New York, New York, 1970.

[82] R.P. Betson. What is watershed runoff? *Journal of Geophysical Research*, 69(8):1541–1552, 1964.

[83] W.R. Bidlake, W.M. Woodham, and M.A. Lopez. Evapotranspiration from Areas of Native Vegetation in West-Central Florida. Water-Supply Paper 2430, U.S. Geological Survey, 1996.

[84] R.B. Billings and C.V. Jones. *Forecasting Urban Water Demand*. American Water Works Association, Denver, Colorado, 1996.

[85] J.H. Black and K.L. Kipp. Observation well response time and its effect upon aquifer test results. *Journal of Hydrology*, 34:297–306, 1977.

[86] P.E. Black. *Watershed Hydrology*. Ann Arbor Press, Inc., Chelsea, Michigan, second edition, 1996.

[87] H. Blasius. Das Ähnlichkeitsgesetz bei Reibungsvorgängen in Flüssigkeiten. *Forsch. Gebiete Ingenieurw.*, 131, 1913.

[88] R.D. Blevins. *Applied Fluid Dynamics Handbook*. Van Nostrand Reinhold, New York, New York, 1984.

[89] F.E. Blow. Quantity and hydrologic characteristics of litter upon upland oak forests in eastern Tennessee. *Journal of Forestry*, 53:190–195, 1955.

[90] B.B. Bobée and R. Robitaille. Correction of bias in estimation of the coefficient of skewness. *Water Resources Research*, 11(6):851–854, December 1975.

[91] G.L. Bodhaine. Measurement of Peak Discharge at Culverts by Indirect Methods. Techniques of Water Resources Investigations Chapter A3, Book 3, U.S. Geological Survey, Washington, DC, 1976.

[92] G.H. Bolt and P.H. Groenvelt. Coupling phenomena as a possible cause for non-darcian behavior of water in soil. *Bull. Intern. Assoc. Sci. Hydrol.*, 14:17–26, 1969.

[93] F.A. Bombardelli and M.H. García. Hydraulic design of Large-Diameter Pipes. *Journal of Hydraulic Engineering*, 129(11):839–846, November 2003.

[94] J. Boonstra. Well hydraulics and aquifer tests. In J.W. Delleur, editor, *The Handbook of Groundwater Engineering*, pages 8.1–8.34. CRC Press, Inc., Boca Raton, Florida, 1998a.

[95] L.E. Borgman. Risk criteria. *Journal of Waterways and Harbors Division, ASCE*, 89(WW3):1–35, August 1963.

[96] M.G. Bos. *Discharge Measurement Structures*. Wageningen, the Netherlands, third revised edition, 1988. ILRI Publication 20.

[97] Boulder County. *Drainage Criteria Manual*. Boulder County, Colorado, 1984.

[98] D. Bousmar and Y. Zech. Momentum transfer for practical flow computation in compound channels. *Journal of Water Resources Planning and Management*, 125(7):696–706, July 1999.

[99] H. Bouwer. *Groundwater Hydrology*. McGraw-Hill Book Company, New York, New York, 1978.

[100] H. Bouwer. Elements of soil science and groundwater hydrology. In G. Bitton and C.P. Gerba, editors, *Groundwater Pollution Microbiology*, pages 9–38. Krieger Publishing Company, Malabar, Florida, 1984.

[101] H. Bouwer. The Bouwer and Rice slug test: An update. *Ground Water*, 27:304–309, 1989.

[102] H. Bouwer. Discussion of Bouwer and Rice slug test review articles. *Ground Water*, 34:171, 1996.

[103] H. Bouwer, J.T. Back, and J.M. Oliver. Predicting infiltration and ground-water mounds for artificial recharge. *Journal of Hydrologic Engineering*, 4(4):350–357, 1999.

[104] H. Bouwer and R.C. Rice. A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells. *Water Resources Research*, 12:423–428, 1976.

[105] N.C. Brady. *The Nature and Properties of Soils*. Macmillan, New York, 8th edition, 1974.

[106] E.F. Brater, H.W. King, J.E. Lindell, and C.Y. Wei. *Handbook of Hydraulics*. McGraw-Hill, Inc., New York, New York, seventh edition, 1996.

[107] W.C. Bridges. Techniques for estimating magnitude and frequency of floods on natural-flow streams in Florida. Water-Resources Investigations Report 82-4012, United States Geological Survey, Reston, Virginia, 1982.

[108] J. Bromley, M. Robinson, and J.A. Barker. Scale-dependency of hydraulic conductivity: an example from Thorne Moor, a raised mire in South Yorkshire, UK. *Hydrological Processes*, 18:973–985, 2004.

[109] K.N. Brooks, P.F. Folliott, H.M. Gregersen, and J.L. Thames. *Hydrology and the Management of Watersheds*. Iowa State University Press, Ames, Iowa, 1991.

[110] D.S. Brookshire and D. Whittington. Water resources issues in developing countries. *Water Resources Research*, 29(7):1883–1888, July 1993.

[111] G.O. Brown. Henry Darcy and the making of a law. *Water Resources Research*, 38(7), doi:10.1029/2001WR000727, 2002.

[112] G.M. Brune. Trap Efficiency of Reservoirs. *Transactions of the American Geophysical Union*, 34(3):407–418, 1953.

[113] D. Brunt. Notes on radiation in the atmosphere. *Quarterly Journal of Royal Meteorological Society*, 58:389–418, 1932.

[114] W. Brutsaert. *Evaporation into the Atmosphere: Theory, History, and Applications.* R. Deidel, Dordrecht, The Netherlands, 1982.

[115] P.H. Burgi. Bicycle-Safe Grate Inlets Study: Volume 2–Hydraulic Characteristics of Three Selected Grate Inlets on Continuous Grades. Technical Report FHWA-RD-78-4, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1978.

[116] F.L. Burton. Wastewater-Collection Systems. In L.W. Mays, editor, *Water Resources Handbook*, pages 19.1–19.53. McGraw-Hill, Inc., New York, New York, 1996.

[117] K. Bury. *Statistical Distributions in Engineering.* Cambridge University Press, New York, New York, 1999.

[118] J.A. Businger. Some remarks on Penman's equations for the evapotranspiration. *Netherlands J. Agric. Sci.*, 4:77, 1956.

[119] D. Butler and J.W. Davies. *Urban Drainage.* E & FN Spon, London, England, 2000.

[120] J.J. Butler Jr., C.D. McElwee, and W. Liu. Improving the quality of parameter estimates obtained from slug tests. *Ground Water*, 34(3):480–490, 1996.

[121] T.R. Camp. Design of sewers to facilitate flow. *Sewer Works Journal*, 18:3, 1946.

[122] G.S. Campbell. *An Introduction to Environmental Physics.* Springer, New York, 1977.

[123] M.D. Campbell and J.H. Lehr. *Water Well Technology.* McGraw-Hill Book Company, Inc., New York, New York, 1972.

[124] P.C. Carman. Fluid flow through a granular bed. *Transaction of the Institutions of Chemical Engineers*, 15:150–156, 1937.

[125] P.C. Carman. *Flow of Gasses Through Porous Media.* Butterworths, London, Great Britain, 1956.

[126] D. Carroll. Rainwater as a chemical agent and geologic processes- A review. Water-Supply Paper 1535-G, United States Geological Survey, 1962.

[127] H.W. Carslaw and J.C. Jaeger. *Conduction of Heat in Solids.* Oxford University Press, New York, New York, second edition, 1959.

[128] T.V. Cech. *Principles of Water Resources*. John Wiley & Sons, New York, New York, 2002.

[129] H.R. Cedergren, K.H. O'Brien, and J.A. Arman. Guidelines for the design of subsurface drainage systems for highway structural sections. Technical Report FHWA-RD-72-30, Federal Highway Administration, June 1972.

[130] A. Chadwick and J. Morfett. *Hydraulics in Civil and Environmental Engineering*. E & FN Spon, London, England, second edition, 1993.

[131] R.A. Chandler and D.B. McWhorter. Upconing of the salt-water–fresh-water interface beneath a pumping well. *Ground Water*, 13:354–359, 1975.

[132] M. Chang. *Forest Hydrology: An Introduction to Water and Forests*. CRC Press, Inc., Boca Raton, Florida, 2002.

[133] R.J. Charbeneau. *Groundwater Hydraulics and Pollutant Transport*. Prentice-Hall, Upper Saddle River, New Jersey, 2000.

[134] D.V. Chase. Operation of water distribution systems. In L.W. Mays, editor, *Water Distribution Systems Handbook*, pages 15.1–15.16. McGraw-Hill Book Company, New York, New York, 2000.

[135] M.H. Chaudhry. *Open-Channel Flow*. Prentice-Hall, Upper Saddle River, New Jersey, 1993.

[136] C. Chen. Rainfall intensity-duration-frequency formulas. *Journal of Hydraulic Engineering*, 109(12):1603–1621, 1983.

[137] C.-N. Chen and T.S.W. Wong. Critical rainfall duration for maximum discharge from overland plane. *Journal of Hydraulic Engineering*, 119(9):1040–1045, September 1993.

[138] A.H.-D. Cheng. *Multilayered Aquifer Systems: Fundamentals and Applications*. Marcel Dekker, Inc., New York, New York, 2000.

[139] D.A. Chin. Leakage of clogged channels that partially penetrate surficial aquifers. *Journal of Hydraulic Engineering*, 117(4):467–488, April 1991.

[140] D.A. Chin. Analysis and Prediction of South-Florida Rainfall. Technical Report CEN-93-2, University of Miami, Coral Gables, Florida, 1993.

[141] D.A. Chin. A risk management strategy for wellhead protection. Technical Report CEN93-3, University of Miami, Department of Civil Engineering, Coral Gables, Florida, 1993.

[142] D.A. Chin and P.V.K. Chittaluru. Risk management in wellhead protection. *Journal of Water Resources Planning and Management*, 120(3):294–315, 1994.

[143] C.-L. Chiu and N.-C. Tung. Maximum Velocity and Regularities in Open-Channel Flow. *Journal of Hydraulic Engineering*, 128(4):390–398, April 2002.

[144] V.T. Chow. Frequency analysis of hydrologic data with special application to rainfall intensities. Bulletin 414, University of Illinois Engineering Experiment Station, 1953.

[145] V.T. Chow. The log-probability law and its emerging applications. *Proceedings of the American Society of Civil Engineers*, 80:536–1 to 536–25, 1954.

[146] V.T. Chow. *Open-Channel Hydraulics.* McGraw-Hill, Inc., New York, New York, 1959.

[147] V.T. Chow. *Handbook of Applied Hydrology.* McGraw-Hill, Inc., New York, New York, 1964.

[148] V.T. Chow, D.R. Maidment, and L.W. Mays. *Applied Hydrology.* McGraw-Hill, Inc., New York, New York, 1988.

[149] J.E. Christiansen. Pan evaporation and evapotranspiration from climatic data. *Journal of Irrigation and Drainage Engineering, ASCE*, 94:243–265, 1968.

[150] J.E. Christiansen and G.H. Hargreaves. Irrigation requirements from evaporation. *Trans. Int. Comm. on Irrig. and Drain.*, III:23.569–23.596, 1969.

[151] C.O. Clark. Storage and the Unit Hydrograph. *Transactions of the American Society of Civil Engineers*, 110:1419–1446, 1945.

[152] R.M. Clark. Water Supply. In R.A. Corbitt, editor, *Standard Handbook of Environmental Engineering*, pages 5.1–5.225. McGraw-Hill, Inc., New York, New York, 1990.

[153] C.F. Colebrook. Turbulent Flow in Pipes, with Particular Reference to the Transition Region Between the Smooth and Rough Pipe Laws. *Journal, Institution of Civil Engineers (London)*, 11:133–156, February 1939.

[154] H.H. Cooper, J.D. Bredehoeft, and I.S. Papadopulos. Response of a finite-diameter well to an instantaneous charge of water. *Water Resources Research*, 3(1):263–269, 1967.

[155] H.H. Cooper and C.E. Jacob. A generalized graphical method for evaluating formation constants and summarizing well-field history. *Transactions of the American Geophysical Union*, 27:526–534, 1946.

[156] R.A. Corbitt. Wastewater Disposal. In R.A. Corbitt, editor, *Standard Handbook of Environmental Engineering*, pages 6.1–6.274. McGraw-Hill, Inc., New York, New York, 1990.

[157] G.K. Cotton. Hydraulic Design of Flood Control Channels. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 16.1–16.42. McGraw-Hill, Inc., New York, New York, 2001.

[158] N.H. Crawford and R.K. Linsley. Digital Simulation in Hydrology, Stanford Watershed Model IV. Technical Report 39, Department of Civil Engineering, Stanford University, Stanford, California, 1966.

[159] H. Cross. Analysis of Flow in Networks of Conduits or Conductors. Bulletin 286, University of Illinois Engineering Experiment Station, 1936.

[160] Z. Şen. *Applied Hydrogeology for Scientists and Engineers.* Lewis Publishers, Boca Raton, Florida, 1995.

[161] J.A. Cunge. On the subject of a flood propagation computation method (Muskingum method). *Journal of Hydraulic Research, IAHR*, 7(2):205–230, 1969.

[162] C. Cunnane. Methods and merits of regional flood frequency analysis. *Journal of Hydrology*, 100:269–290, 1988.

[163] G. Dagan. *Flow and Transport in Porous Formations*. Springer-Verlag, New York, New York, 1989.

[164] M. Dagg. Evaporation Pans in East Africa, 1968. Proceedings Fourth Specialist Meeting on Applied Meteorology in East Africa.

[165] J.W. Daily and D.R.F. Harleman. *Fluid Dynamics*. Addison-Wesley Publishing Company, Inc., Reading, Massachusetts, 1966.

[166] H. Darcy. *Les Fontaines Publiques de la Ville de Dijon*. Victor Dalmont, Paris, France, 1856.

[167] T. Davie. *Fundamentals of Hydrology*. Routledge, London, England, 2002.

[168] G. de Marsily. *Quantitative Hydrogeology, Groundwater Hydrology for Engineers*. Academic Press, Inc., San Diego, California, 1986.

[169] T.N. Debo and A.J. Reese. *Municipal Storm Water Management*. Lewis Publishers, Inc., Boca Raton, Florida, 1995.

[170] G.J. DeGlee. *Over Grondwaterstromingen bij Wateronttrekking Door Middel van Putten* (in Dutch). J. Waltman, Delft, 1930.

[171] J. Delleur. The Evolution of Urban Hydrology: Past, Present, and Future. *Journal of Hydraulic Engineering*, 129(8):563–573, August 2003.

[172] J.L. Devore. *Probability and Statistics for Engineering and the Sciences*. Brooks/Cole, Pacific Grove, California, 2000.

[173] S.L. Dingman. *Fluvial Hydrology*. W.H. Freeman and Company, New York, New York, 1984.

[174] S.L. Dingman. *Physical Hydrology*. Prentice-Hall, Upper Saddle River, New Jersey, second edition, 2002.

[175] R.D. Dodson. *Storm Water Pollution Control*. McGraw-Hill, Inc., New York, New York, second edition, 1998.

[176] R.D. Dodson. Floodplain hydraulics. In L.W. Mays, editor, *Hydraulic Design Handbook*, pages 19.1–19.50. McGraw-Hill Book Company, New York, New York, 1999.

[177] R.D. Dodson and C.T. Maske. Regulation of Stormwater Collection Systems in the United States. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 2.1–2.25. McGraw-Hill, Inc., New York, New York, 2001.

[178] P.A. Domenico and M.D. Mifflin. Water from low-permeability sediments and land subsidence. *Water Resources Research*, 1:563–576, 1965.

[179] P.A. Domenico and F.W. Schwartz. *Physical and Chemical Hydrogeology.* John Wiley & Sons, New York, New York, second edition, 1998.

[180] J. Doorenbos and W.O. Pruitt. Guidelines for the prediction of crop water requirements. Irrigation and Drainage Paper 24, UN Food and Agriculture Organization, Rome, Italy, 1975.

[181] J. Doorenbos and W.O. Pruitt. Guidelines for the prediction of crop water requirements. Irrigation and Drainage Paper 24, UN Food and Agriculture Organization, Rome, Italy, 1977. Second Edition.

[182] J.F. Douglas, J.M. Gasiorek, and J.A. Swaffield. *Fluid Mechanics.* Prentice-Hall, Upper Saddle River, New Jersey, fourth edition, 2001.

[183] F.G. Driscoll. *Ground Water and Wells.* Johnson Division, St. Paul, Minnesota, second edition, 1986.

[184] N.E. Driver. Summary of nationwide analysis of storm-runoff quality and quantity in urban watersheds. In Y. Iwasa and T. Sueishi, editors, *Proceedings of the Fifth International Conference on Urban Storm Drainage, Osaka, Japan*, volume 1, pages 333–338, 1990.

[185] N.E. Driver and D.J. Lystrom. Estimation of urban storm runoff loads, urban runoff quality–Impact and quality enhancement technology. In B. Urbonas and L.A. Roesner, editors, *Proceedings of an Engineering Foundation Conference, New Hampshire*, pages 222–232, Reston, Virginia, 1986. American Society of Civil Engineers.

[186] N.E. Driver and G.D. Tasker. Techniques for estimation of storm-runoff loads, volumes, and selected constituent concentrations in urban watersheds in the United States. Open File Report 88-191, U.S. Geological Survey, Denver, Colorado, 1988.

[187] N.E. Driver and G.D. Tasker. Techniques for estimation of storm-runoff loads, volumes, and selected constituent concentrations in urban watersheds in the United States. Water-Supply Paper 2363, U.S. Geological Survey, Washington, DC, 1990.

[188] M. Duchene, E.A. McBean, and N.R. Thomson. Modeling of infiltration from trenches for storm-water control. *Journal of Water Resources Planning and Management*, 120(3):276–293, May/June 1994.

[189] J.A. Duffie and W.A. Beckman. *Solar Engineering of Thermal Processes.* John Wiley and Sons, Inc., New York, 1980.

[190] T. Dunne and R.D. Black. Partial area contributions to storm runoff in a small New England watershed. *Water Resources Research*, 6:1296–1311, 1970.

[191] T. Dunne and L.B. Leopold. *Water in Environmental Planning.* W.H. Freeman and Company, San Francisco, California, 1978.

[192] J. Dupuit. *Études Théoriques et Pratiques sur le Mouvement des Eaux dans les Canaux DéCauverts et á Travers les Terrains Perméables.* Dunod, Paris, France, second edition, 1863.

[193] H.B. Dwight. *Tables of Integrals and Other Mathematical Data*. Macmillan Publishing Company, New York, New York, fourth edition, 1961.

[194] B. Dziegielewski and E. Opitz. Water Demand Analysis. In L.W. Mays, editor, *Urban Water Supply Handbook*, pages 5.1–5.55. McGraw-Hill Book Company, New York, New York, 2002.

[195] B. Dziegielewski, E.M. Opitz, and D. Maidment. Water Demand Analysis. In L.W. Mays, editor, *Water Resources Handbook*, pages 23.1–23.62. McGraw-Hill, Inc., New York, New York, 1996.

[196] A.A. Dzurik. *Water Resources Planning*. Rowman and Littlefield Publishers, Inc., Lanham, Maryland, Third edition, 2003.

[197] P.S. Eagleson. Dynamics of flood frequency. *Water Resources Research*, 8:878–898, 1972.

[198] S.M. Easa. Geometric Design. In W.F. Chen, editor, *The Civil Engineering Handbook*, pages 2287–2319. CRC Press, Boca Raton, Florida, 1995.

[199] G. Echávez. Increase in losses coefficient with age for small diameter pipes. *Journal of Hydraulic Engineering*, 123(2):157–159, February 1997.

[200] J.W. Eheart and J.R. Lund. Water-Use Management: Permit and Water-Transfer Systems. In L.W. Mays, editor, *Urban Water Supply Handbook*, pages 4.1–4.41. McGraw-Hill Book Company, New York, New York, 2002.

[201] H.A. Einstein. Der hydraulische oder profil-radius. *Schweizerische Bauzeitung*, 103(8):89–91, February 1934.

[202] H.A. Einstein. The Bed Load Function for Sediment transport in Open Channel Flows. Technical Bulletin 1026, U.S. Department of Agriculture, Washington, DC, 1950.

[203] H.A. Einstein and R.B. Banks. Fluid resistance of composite roughness. *Transactions of the American Geophysical Union*, 31(4):603–610, 1951.

[204] M.Z. Ejeta and L.W. Mays. Water Pricing and Drought Management. In L.W. Mays, editor, *Urban Water Supply Handbook*, pages 6.1–6.43. McGraw-Hill Book Company, New York, New York, 2002.

[205] N. El-Jabi and S. Sarraf. Effect of maximum rainfall position on rainfall-runoff relationship. *Journal of Hydraulic Engineering*, 117(5):681–685, May 1991.

[206] W.J. Elliot. Precipitation. In A.D. Ward and W.J. Elliot, editors, *Environmental Hydrology*, pages 19–49. Lewis Publishers, Inc., Boca Raton, Florida, 1995.

[207] E.T. Engman. Roughness Coefficients for Routing Surface Runoff. *Journal of Irrigation and Drainage Engineering, ASCE*, 112(1):39–53, 1986.

[208] Environmental and Water Resources Institute, Standardization of Reference Evapotranspiration Task Committee. The ASCE Standardized Reference Evapotranspiration Equation. Draft Report July 9, 2002, American Society of Civil Engineers, 2002.

[209] Environmental Protection Agency. Results of the nationwide urban runoff program, final report. Technical Report Accession No. PB84-185552, NTIS, Washington, DC, December 1983.

[210] Environmental Protection Agency. Methodology for analysis of detention basins for control of urban runoff quality. Technical Report EPA 440/5-87-001, U.S. Environmental Protection Agency, Office of Water Regulations and Standards, Washington, DC, 1986.

[211] D.R. Erickson. A study of littoral groundwater seepage at Williams Lake, Minnesota using seepage meters and wells. Master's thesis, University of Minnesota, Minneapolis, 1981.

[212] W.H. Espey, Jr. and D.G. Altman. Nomographs for Ten-minute Unit Hydrographs for Small Urban Watersheds. Addendum 3 of Urban Runoff Control Planning EPA-600/9-78-035, U.S. Environmental Protection Agency, Washington, DC, 1978.

[213] W.H. Espey, Jr., D.G. Altman, and C.B. Graves. Nomographs for Ten-Minute Unit Hydrographs for Small Urban Watersheds. ASCE Urban Water Resources Research Program, Technical Memorandum 32 (NTIS PB-282158), ASCE, New York, New York, 1977.

[214] C.-Y. Fan, R. Field, W.C. Pisano, J. Barsanti, J.J. Joyce, and H. Sorenson. Sewer and tank flushing for sediment, corrosion, and pollution control. *Journal of Water Resources Planning and Management*, 127(3):194–201, May/June 2001.

[215] J.A. Fay. *Introduction to Fluid Mechanics*. MIT Press, Cambridge, Massachusetts, 1994.

[216] Federal Emergency Management Agency. Flood insurance study guidelines and specifications for study contractors. Report 37, FEMA, Washington, DC, 1993.

[217] Water Pollution Control Federation. *Gravity Sanitary Sewer Design and Construction, WPCF Manual of Practice No.FD-5*. Water Pollution Control Federation, Washington, DC, 1982.

[218] M.C. Feller. Water balances in *eucalyptus regnans*, *e.obliqua*, and *pinus radiata* forests in Victoria. *Australian Forestry*, 44(3):153–161, 1981.

[219] V. Ferro. Friction factor for gravel-bed channel with high boulder concentration. *Journal of Hydraulic Engineering*, 125(7):771–778, July 1999.

[220] C.W. Fetter. *Contaminant Hydrogeology*. Prentice-Hall, Upper Saddle River, New Jersey, second edition, 1999.

[221] C.W. Fetter. *Applied Hydrogeology*. Prentice-Hall, Upper Saddle River, New Jersey, fourth edition, 2001.

[222] L. Feyen, P.J. Ribeiro Jr., F. DeSmedt, and P.J. Diggle. Stochastic delineation of capture zones: classical versus Bayesian approach. *Journal of Hydrology*, 281:313–324, 2003.

[223] F.R. Fiedler. Simple, Practical Method for Determining Station Weights Using Thiessen Polygons and Isohyetal Maps. *Journal of Hydrologic Engineering*, 8(4):219–221, 2003.

[224] R. Field, J.P. Heaney, and R. Pitt. *Innovative Urban Wet-Weather Flow Management Systems.* Technomic Publishing Company, Inc., Lancaster, Pennsylvania, 2000.

[225] E.J. Finnemore and J.B. Franzini. *Fluid Mechanics with Engineering Applications.* McGraw-Hill, Inc., New York, New York, tenth edition, 2002.

[226] R.A. Fisher and L.H.C. Tippett. Limiting forms of the frequency distribution of the largest or smallest member of a sample. *Proc. Cambridge Phil. Soc.*, 24, part II:180–191, 1928.

[227] C.R. Fitts. *Groundwater Science.* Academic Press, New York, New York, 2002.

[228] Florida Geological Survey. *Florida's Ground Water Quality Monitoring Program, Hydrogeological Framework.* Tallahassee, Florida, 1991. Special Publication No. 32.

[229] P. Forchheimer. *Hydraulik.* Teubner Verlagsgesellschaft, Stuttgart, Germany, 1930.

[230] S. Fortier and F.C. Scobey. Permissible canal velocities. *Transactions of the American Society of Civil Engineers*, 89:940–984, 1926.

[231] H.A. Foster. Theoretical frequency curves and their application to engineering problems. *Transactions of the American Society of Civil Engineers*, 87:142–173, 1924.

[232] R.W. Fox and A.T. McDonald. *Introduction to Fluid Mechanics.* John Wiley & Sons, New York, New York, fourth edition, 1992.

[233] M.A. Franklin and G.T. Losey. Magnitude and frequency of floods from urban streams in Leon County, Florida. Water-Resources Investigations Report 84-4004, United States Geological Survey, Reston, Virginia, 1984.

[234] J.B. Franzini and E.J. Finnemore. *Fluid Mechanics with Engineering Applications.* McGraw-Hill, Inc., New York, New York, ninth edition, 1997.

[235] R.H. Frederick, V.A. Myers, and E.P. Auciello. Five- to 60-minute precipitation frequency for the eastern and central United States. NOAA Technical Memorandum NWS HYDRO-35, National Oceanic and Atmospheric Administration, National Weather Service, Silver Spring, Maryland, June 1977.

[236] R.A. Freeze and J.A. Cherry. *Groundwater.* Prentice-Hall, Upper Saddle River, New Jersey, 1979.

[237] R.H. French. *Open-Channel Hydraulics.* McGraw-Hill, Inc., New York, New York, 1985.

[238] R.H. French. Hydraulics of Open Channel Flow. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 3.1–3.35. McGraw-Hill, Inc., New York, New York, 2001.

[239] M. Frére and G.F. Popov. Agrometeorological crop monitoring and forecasting. FAO Plant Production and Protection Paper 17, FAO, Rome, Italy, 1979. 38-43.

[240] E. Ganguillet and W.R. Kutter. Versuch zur Aufstellung einer neuen allegemeinen Formel für die gleichförmige Bewegung des Wassers in Canälen und Flüssen (An investigation to establish a new general formula for uniform flow of water in canals and rivers). *Zeitschrift des Oesterreichischen Ingenieur- und Architekten Vereines*, 21(1,2-3):6–25,46–59, 1869. (Published as a book in Bern, Switzerland, 1877; translated into English by Rudolph Hering and John C. Trautwine, Jr., as *A general Formula for the Uniform Flow of Water in Rivers and Other Channels* John Wiley & Sons, Inc., New York, 1st ed., 1888; 2d ed., 1891 and 1901).

[241] W.F. Geiger and H.R. Dorsch. Quantity-Quality Simulation (QQS): A detailed continuous planning model for urban runoff control in *Model Description, Testing and Application* Volume I. Technical Report EPA-600/2-80-011, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1980.

[242] P.M. Gerhart, R.J. Gross, and J.I. Hochstein. *Fundamentals of Fluid Mechanics.* Addison-Wesley Publishing Company, Inc., Reading, Massachusetts, second edition, 1992.

[243] P.H. Gleick. An introduction to global fresh water issues. In P.H. Gleick, editor, *Water in Crisis*, pages 3–12. Oxford University Press, New York, New York, 1993.

[244] GLUMB. *Recommended Standards for Water Works, 1987.* Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers, Health Research, Inc., 1987.

[245] L. Gordon. Mississippi river discharge. San Diego, California, 1992. RD Instruments.

[246] S.M. Gorelick, R.A. Freeze, D. Donohue, and J.F. Keely. *Groundwater Contamination.* Lewis Publishers, Inc., Boca Raton, Florida, 1993.

[247] R.A. Granger. *Fluid Mechanics.* Holt, Rinehart and Winston, New York, New York, 1985.

[248] D.M. Gray, editor. *Handbook on the Principles of Hydrology.* National Research Council, Port Washington, Canada, 1973. Water Information Center, Inc.

[249] Great Lakes-Upper Mississippi River Board of Sanitary Engineers. *Recommended Standards for Sewage Works.* Health Education Service, Albany, New York, 1978.

[250] W.H. Green and G.A. Ampt. Studies of soil physics, Part I: The flow of air and water through soils. *J. Agric. Sci.*, 4(1):1–24, 1911.

[251] J.E. Gribbin. *Hydraulics and Hydrology for Stormwater Management.* Delmar Publishers, Albany, New York, 1997.

[252] I.I. Gringorten. A plotting rule for extreme probability paper. *Journal of Geophysical Research*, 68(3):813–814, 1963.

[253] T.L. Grizzard, C.W. Randall, B.L. Weand, and K.L. Ellis. Effectiveness of extended detention ponds. In *Urban Runoff Quality.* American Society of Civil Engineers, 1986.

[254] E.J. Gumbel. Statistical theory of extreme values and some practical applications. Applied Mathematics Series 33, U.S. National Bureau of Standards, Washington, DC, 1954.

[255] E.J. Gumbel. *Statistics of Extremes*. Columbia University Press, New York, New York, 1958.

[256] J.C.Y. Guo. Street Storm Water Conveyance Capacity. *Journal of Irrigation and Drainage Engineering, ASCE*, 126(2):119–123, March/April 2000.

[257] J.C.Y. Guo. Design of Infiltration Basins for Stormwater. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 9.1–9.35. McGraw-Hill, Inc., New York, New York, 2001a.

[258] R.S. Gupta. *Hydrology and Hydraulic Systems*. Waveland Press, Inc., Prospect Heights, Illinois, second edition, 2001.

[259] C.T. Haan. *Statistical Methods in Hydrology*. The Iowa State University Press, Ames, Iowa, 1977.

[260] Haestad Methods, Inc. *1997 Practical Guide to Hydraulics and Hydrology*. Haestad Press, Waterbury, Connecticut, 1997.

[261] Haestad Methods, Inc. *Computer Applications in Hydraulic Engineering*. Haestad Press, Waterbury, Connecticut, 1997.

[262] Haestad Methods, Inc. *Computer Applications in Hydraulic Engineering*. Haestad Press, Waterbury, Connecticut, fifth edition, 2002.

[263] W.H. Hager. *Energy Dissipators and Hydraulic Jump*. Kluwer Academic Publishers, Dordrecht, Netherlands, 1991.

[264] W.H. Hager. *Wastewater Hydraulics*. Springer-Verlag, Berlin, Germany, 1999.

[265] A. Haldar and S. Mahadevan. *Probability, Reliability and Statistical Methods in Engineering Design*. John Wiley & Sons, New York, New York, 2000.

[266] R.L. Hall. Interception loss as a function of rainfall and forest types: stochastic modelling for tropical canopies revisited. *Journal of Hydrology*, 280:1–12, 2003.

[267] E.M. Hansen. Probable maximum precipitation for design floods in the United States. *Journal of Hydrology*, 96:267–278, 1987.

[268] E.M. Hansen, L.C. Schreiner, and J.F. Miller. Application of probable maximum precipitation estimates — United States east of the 105th meridian. NOAA Hydrometeorological Report 52, National Weather Service, Washington, DC, August 1982.

[269] M.S. Hantush. Analysis of data from pumping tests in leaky aquifers. *Transactions of the American Geophysical Union*, 37(6):702–714, 1956.

[270] M.S. Hantush. Modification of the theory of leaky aquifers. *Journal of Geophysical Research*, 65:3713–3725, 1960.

[271] M.S. Hantush. Hydraulics of wells. In V.T. Chow, editor, *Advances in Hydroscience*, volume 1, pages 281–432. Academic Press, New York, New York, 1964.

[272] M.S. Hantush. Growth and decay of groundwater-mounds in response to uniform percolation. *Water Resources Research*, 3(1):227–234, 1967.

[273] M.S. Hantush and C.E. Jacob. Plane potential flow of ground-water with linear leakage. *Transactions of the American Geophysical Union*, 35:917–936, 1954.

[274] M.S. Hantush and C.E. Jacob. Non-steady radial flow in an infinite leaky aquifer. *Transactions of the American Geophysical Union*, 36(1):95–100, 1955.

[275] A.W. Harbaugh and M.G. McDonald. User's documentation for MODFLOW-96, an update to the U.S. Geological Survey modular finite-difference ground-water flow model. Open File Report 96-485, United States Geological Survey, 1996.

[276] G.H. Hargreaves and Z.A. Samani. Estimating potential evapotranspiration. *Journal of Irrigation and Drainage Engineering, ASCE*, 108(3):225–230, 1982.

[277] G.L. Hargreaves, G.H. Hargreaves, and J.P. Riley. Agric. benefits for Senegal River Basin. *Journal of Irrigation and Drainage Engineering, ASCE*, 111(2):113–124, 1985.

[278] G.L. Hargreaves and Z.A. Samani. Reference-crop evapotranspiration from temperature. *Applied Engrg. in Agric.*, 1(2):96–99, 1985.

[279] D.R.F. Harleman, P.F. Melhorn, and R.R. Rumer. Dispersion-permeability correlation in porous media. *Journal of the Hydraulics Division, ASCE*, 89:67–85, 1963.

[280] W.G. Harmon. Forecasting Sewage at Toledo under Dry Weather Conditions. *Engineering News Record*, 80, 1918.

[281] B.W. Harrington. Design and construction of infiltration trenches. In L.A. Roesner, B. Urbonas, and M.B. Sonnen, editors, *Design of Urban Runoff Quality Controls*, pages 291–306. ASCE, New York, New York, 1989.

[282] L.P. Harrison. Fundamental concepts and definitions relating to humidity. In A. Wexler, editor, *Humidity and Moisture*. Reinhold Publishing Company, New York, New York, 1963.

[283] J.P. Hartigan. Basis for design of wet detention basin BMP's. In L.A. Roesner, B. Urbonas, and M.B. Sonnen, editors, *Design of Urban Runoff Quality Controls*, pages 122–144. ASCE, New York, New York, 1989.

[284] G.A. Hathaway. Design of Drainage Facilities. *Transactions of the American Society of Civil Engineers*, 110, 1945.

[285] R.H. Hawkins, A.T. Hjelmfelt, and A.W. Zevenbergen. Runoff probability, storm depth, and curve numbers. *Journal of the Irrigation and Drainage Division, ASCE*, 111(4):330–340, December 1985.

[286] A. Hazen. Discussion of "Dams on Sand Foundations, by A.C. Koenig". *Transactions of the American Society of Civil Engineers*, 73:199, 1911.

[287] J.P. Heany, W.C. Huber, and S.J. Nix. Storm water management model: Level I–Preliminary screening procedures. Report EPA-600/2-76-275, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1976.

[288] J.P. Heany, W.C. Huber, H. Sheikh, M.A. Medina, J.R. Doyle, W.A. Peltz, and J.E. Darling. Nationwide evaluation of combined sewer overflows and urban stormwater discharges, Volume 2, Cost assessment and impacts. Report EPA-600/2-77-064, U.S. Environmental Protection Agency, Washington, DC, 1977.

[289] H.G. Heinemann. A New Sediment Trap Efficiency Curve for Small Reservoirs. *Water Resources Bulletin*, 17:825–830, 1981.

[290] J.D. Helvey. Rainfall interception by hardwood forest litter in the Southern Appalachians. Research Paper SE-8, U.S. Department of Agriculture, Forest Service, Southeastern Forest Experiment Station, Asheville, NC, 1964. 9 p.

[291] J.D. Helvey. Interception by eastern white pine. *Water Resources Research*, 3(3):723–729, 1967.

[292] J.D. Helvey. Biological Effects in the Hydrological Cycle. In E.J. Monke, editor, *Proceedings of the Third International Seminar for Hydrology Professors*, pages 103–113, West Lafayette, IN, 1971. Purdue University.

[293] J.D. Helvey and J.J. Patric. Canopy and litter interception of rainfall by hardwoods of eastern United States. *Water Resources Research*, 1:193–206, 1965.

[294] J.C. Hemain. Statistically based modelling of urban runoff quality: State of the art. In H.C. Torno et al., editor, *Urban Runoff Pollution*, volume 10 of *NATO ASI Ser., Ser.G*, pages 277–304. Springer-Verlag, New York, New York, 1986.

[295] H.F. Hemond and E.J. Fechner. *Chemical Fate and Transport in the Environment*. Academic Press, New York, New York, 1993.

[296] F.M. Henderson. *Open Channel Flow*. Macmillan Publishing Co., Inc., New York, New York, 1966.

[297] H.R. Henry. Discussion of 'Diffusion of submerged jets,' by M.L. Albertson, Y.B. Dai, R.A. Jensen, and H. Rouse. *Transactions of the American Society of Civil Engineers*, 115:687–694, 1960.

[298] J.A. Henry, K.M. Portier, and J. Coyne. *The Climate and Weather of Florida*. Pineapple Press, Inc., Sarasota, Florida, 1994.

[299] J.F. Hermance. *A Mathematical Primer on Groundwater Flow*. Prentice-Hall, Upper Saddle River, New Jersey, 1999.

[300] D.M. Hershfield. Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years. Technical Paper 40, U.S. Department of Commerce, Weather Bureau, Washington, DC, May 1961.

[301] D.M. Hershfield. Method for estimating probable maximum precipitation. *Journal of the American Water Works Association*, 57:965–972, 1965.

[302] A. Herzberg. Die Wasserversorgung einiger Nordseebaden (The water supply on parts of the North Sea coast in Germany). *Z. Gasbeleucht. Wasserversorg.*, 44:815–819, 824–844, 1901.

[303] B.L. Herzog. Slug tests for determining hydraulic conductivity of natural geologic deposits. In D.E. Daniel and S.J. Trautwein, editors, *Hydraulic Conductivity and Waste Contaminant Transport in Soil*, pages 95–110. ASTM, Philadelphia, Pennsylvania, 1994.

[304] J.D. Hewlett and A.R. Hibbert. Factors affecting the response of small watersheds to precipitation in humid areas. In W.E. Sopper and H.W. Lull, editors, *Forest Hydrology*, pages 275–290. Pergamon, New York, New York, 1967.

[305] W.I. Hicks. A method of computing urban runoff. *Transactions of the American Society of Civil Engineers*, 109:1217, 1944.

[306] F. Hildebrand. *Advanced Calculus for Applications*. Prentice-Hall, Inc., Englewood Cliffs, New Jersey, second edition, 1976.

[307] A.T. Hjelmfelt, Jr. Negative Outflows from Muskingum Flood Routing. *Journal of Hydraulic Engineering*, 111(6), June 1985.

[308] A.T. Hjelmfelt, Jr. Investigation of curve number procedure. *Journal of Hydraulic Engineering*, 117(6):725–737, 1991.

[309] R.A. Horton. Separate roughness coefficients for channel bottom and sides. *Engineering News Record*, 111(22):652–653, November 1933a.

[310] R.E. Horton. Rainfall interception. *Monthly Weather Review*, 47:603–623, 1919.

[311] R.E. Horton. The role of infiltration in the hydrologic cycle. *Transactions of the American Geophysical Union*, 14:446–460, 1933b.

[312] R.E. Horton. Analysis of runoff-plat experiments with varying infiltration-capacity. *Trans. American Geophysical Union*, 20:693–711, 1939.

[313] R.E. Horton. An approach towards physical interpretation of infiltration capacity. *Soil Science Society of America Proceedings*, 5:399–417, 1940.

[314] R.E. Horton. Erosional development of streams and their drainage basins: Hydrophysical approach to quantitative morphology. *Geological Society of America Bulletin*, 56:275–370, 1945.

[315] J.R.M. Hosking and J.R. Wallis. *Regional Frequency Analysis*. Cambridge University Press, New York, New York, 1997.

[316] C.H. Howe and F.P. Linaweaver. The Impact of Price on Residential Water Demand and Its Relation to System Design and Price Structure. *Water Resources Research*, 13(1), February 1967.

[317] T.A. Howell, S.R. Evett, A.D. Schneider, D.A. Dusek, and K.S. Copeland. Irrigated fescue grass ET compared with calculated reference grass ET. In *Proceedings, Fourth National Irrigation Symposium*, pages 228–242, St. Joseph, Michigan, 2000. American Society of Agricultural Engineers.

[318] W.C. Huber and R.E. Dickinson. Storm water management model, Version 4, User's manual, with addendums. Technical Report EPA-600/3-88/001a, U.S. Environmental Protection Agency, Athens, Georgia, 1988.

[319] F.A. Huff. Characteristics and contributing causes of an abnormal frequency of flood-producing rainstorms at Chicago. *Water Resources Bulletin*, 31:703–714, 1995.

[320] L. Huisman. *Groundwater Recovery.* Macmillan, London, 1972.

[321] B. Hunt. Dispersion model for mountain streams. *Journal of Hydraulic Engineering*, 125(2):99–105, February 1999.

[322] M.J. Hvorslev. Time lag and soil permeability in ground-water observation. Bulletin 36, Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksburg, Mississippi, 1951.

[323] Hydrologic Engineering Center. Probable Maximum Storm (Eastern U.S.) HMR 52, User's Manual. Technical report, U.S. Army Corps of Engineers, Davis, California, March 1984.

[324] Hydrologic Engineering Center. HEC-FFA Flood Frequency Analysis, Users Manual. Report CPD-13, U.S. Army Corps of Engineers, Davis, California, 1992.

[325] Hydrologic Engineering Center. HEC-1 Flood Hydrograph Package Users Manual. Technical report, U.S. Army Corps of Engineers, Davis, California, June 1998.

[326] Hydrologic Engineering Center. HMS Hydrologic Modeling System Technical Reference Manual. Technical report, U.S. Army Corps of Engineers, Davis, California, March 2000.

[327] Hydrologic Engineering Center. HEC-HMS Hydrologic Modeling System Users Manual. Technical report, U.S. Army Corps of Engineers, Davis, California, January 2001.

[328] Insurance Services Office. Fire Suppression Rating Schedule. New York, 1980.

[329] Interagency Advisory Committee on Water Data. Guidelines for determining flood flow frequency. Bulletin 17B of the Hydrology Committee, U.S. Geological Survey, Reston, Virginia, 1982.

[330] S. Irmak, R.G. Allen, and E.B. Whitty. Daily grass and alfalfa-reference evapotranspiration estimates and alfalfa-to-grass evapotranspiration ratios in Florida. *Journal of Irrigation and Drainage Engineering, ASCE*, 129(5):360–370, 2003c.

[331] S. Irmak, A. Irmak, R.G. Allen, and J.W. Jones. Solar and Net Radiation-Based Equations to Estimate Reference Evapotranspiration in Humid Climates. *Journal of Irrigation and Drainage Engineering, ASCE*, 129(5):336–347, September/October 2003b.

[332] S. Irmak, A. Irmak, J.W. Jones, T.A. Howell, J.M. Jacobs, R.G. Allen, and G. Hoogenboom. Predicting Daily Net Radiation Using Minimum Climatological Data. *Journal of Irrigation and Drainage Engineering, ASCE*, 129(4):256–269, July/August 2003a.

[333] S.A. Isiorho and J.H. Meyer. The effects of bag type and meter size on seepage meter measurements. *Ground Water*, 37:411–413, 1999.

[334] O.W. Israelsen and R.C. Reeve. Canal lining experiments in the Delta Area. Utah Technical Bulletin 313, Utah Agricultural Experiment Station, Logan, Utah, 1944.

[335] D. Itenfisu, R.L. Elliott, R.G. Allen, and I.A. Walter. Comparison of Reference Evapotranspiration Calculations as Part of the ASCE Standardization Effort. *Journal of Irrigation and Drainage Engineering, ASCE*, 129(6):440–448, November/December 2003.

[336] C.F. Izzard. The surface-profile of overland flow. *Transactions of the American Geophysical Union*, 25:959–969, 1944.

[337] C.F. Izzard. Hydraulics of Runoff from Developed Surfaces. *Proceedings of the Highway Research Board*, 26:129–146, 1946.

[338] C.R. Jackson, S.J. Burges, X. Liang, K.M. Leytham, K.R. Whiting, D.M. Hartley, C.W. Crawford, B.N. Johnson, and R.R. Horner. Development and Application of Simplified Continuous Hydrologic Modeling for Drainage Design and Analysis. In M.S. Wigmosta and S.J. Burges, editors, *Land Use and Watersheds*, pages 39–58. American Geophysical Union, Washington, DC, 2001.

[339] I.J. Jackson. Problems of throughfall and interception assessment under tropical forest. *Journal of Hydrology*, 12:234–254, 1971.

[340] Jackson and others. Contaminant hydrogeology of toxic organic chemicals at a disposal site, Gloucester, Ontario. 1. Chemical concepts and site assessment. Iwd scientific series 141, Environment Canada, 1985. 114 p.

[341] C.E. Jacob. Drawdown test to determine effective radius of artesian well. *Transactions of the American Society of Civil Engineers*, 112:1047–1064, 1947. Paper 2321.

[342] A.K. Jain. Accurate explicit equation for friction factor. *ASCE, Journal of the Hydraulics Division*, 102(HY5):674–677, May 1976.

[343] A.K. Jain, D.M. Mohan, and P. Khanna. Modified Hazen-Williams formula. *Journal of the Environmental Engineering Division, ASCE*, 104(EE1):137–146, February 1978.

[344] S.C. Jain. *Open-Channel Flow*. John Wiley & Sons, New York, New York, 2001.

[345] S.V. Jakovlev, J.A.A. Karelm, A.I. Zukov, and S. Kolobanov. *Kanalizacja*. Stroizdat, Moscow, 1975.

[346] L.D. James and R.R. Lee. *Economics of Water Resources Planning*. McGraw-Hill, Inc., New York, New York, 1971.

[347] W.S. Janna. *Introduction to Fluid Mechanics*. PWS-KENT Publishing Company, Boston, Massachusetts, third edition, 1993.

[348] A.F. Jenkinson. The frequency distribution of the annual maximum (or minimum) value of meteorological elements. *Quarterly Journal of Royal Meteorological Society*, 81:158–171, 1955.

[349] M.E. Jennings, W.O. Thomas, and H.C. Riggs. Nationwide Summary of U.S. Geological Survey Regional Regresson Equations for Estimating Magnitude and Frequency of Floods for Ungaged Sites. Water-Resources Investigations Report 94-4002, United States Geological Survey, Reston, Virginia, 1994.

[350] S.W. Jens. Design of urban highway drainage- The state of the art. Report FHWA-TS-79-225, U.S. Department of Transportation, Federal Highway Administration, Washington, DC, 1979.

[351] M.E. Jensen. Empirical methods of estimating or predicting evapotranspiration using radiation. In *Proceedings of Conference on Evapotranspiration*, pages 57–61, 64, Chicago, Illinois, December 1966. American Society of Agricultural Engineers.

[352] M.E. Jensen and H.R. Haise. Estimating evapotranspiration from solar radiation. *Journal of Irrigation and Drainage Engineering, ASCE*, 89:15–41, 1963.

[353] M.E. Jensen, R.D.Burman, and R.G. Allen, editors. *Evapotranspiration and Irrigation Water Requirements*. American Society of Civil Engineers, New York, New York, 1990. Manual of Practice No.70.

[354] M.E. Jensen, J.L. Wright, and B.J. Pratt. Estimating soil moisture depletion from climate, crop, and soil data. *Transactions of the American Society of Agricultural Engineers*, 14:954–959, 1971.

[355] T.K. Jewell and D.D. Adrian. Development of improved stormwater quality models. *Journal of the Environmental Engineering Division, ASCE*, 107:957–974, 1981.

[356] T.K. Jewell and D.D. Adrian. Statistical analysis to derive improved stormwater quality models. *Journal of the Water Pollution Control Federation*, 54:489–499, 1982.

[357] R.C. Johanson, J.C. Imhoff, J.L. Kittle, and S. Donigan. Hydrologicaal Simulation Program-Fortran (HSPF): User manual for release 8.0. Technical Report EPA/600/3-84/066, U.S. Environmental Protection Agency, Athens, Georgia, 1984.

[358] D. Johnson. Down the Drain. *Los Angeles Times*, V1, February 2, 1988.

[359] D. Johnson, M. Smith, V. Koren, and B. Finnerty. Comparing mean areal precipitation estimates from NEXRAD and rain gage networks. *Journal of Hydrologic Engineering*, 4(2):117–124, April 1999.

[360] F.L. Johnson and F.M. Chang. Drainage of highway pavements. Hydraulic Engineering Circular 12, Federal Highway Administration, McLean, VA, 1984.

[361] J.L. Johnson. Local wellhead protection in Florida. Technical report, City of Tallahassee, Water Quality Division, Aquifer Protection Section, Tallahassee, Florida, 1997.

[362] P.A. Johnson. Uncertainty in hydraulic parameters. *Journal of Hydraulic Engineering*, 22(2):112–114, February 1996.

[363] C. Jothityangkoon and M. Sivapalan. Towards estimation of extreme floods: examination of the roles of runoff process changes and floodplain flows. *Journal of Hydrology*, 281:206–229, 2003.

[364] A.G. Journel. *Fundamentals of Geostatistics in Five Lessons*, volume 8 of *Short Course in Geology*. American Geophysical Union, Washington, DC, 1989.

[365] Z.J. Kabala. Well response in a leaky aquifer and computational interpretation of pumping tests. In W.S. Hsieh, S.T. Su, and F. Wen, editors, *Hydraulic Engineering, Proceedings of ASCE National Conference on Hydraulic Engineering and International Symposium on Engineering Hydrology*, page 21, New York, New York, 1993. ASCE.

[366] A.I. Kashef. *Groundwater Engineering*. McGraw-Hill Book Company, Inc., New York, New York, 1986.

[367] M. Kay. *Practical Hydraulics*. E & FN Spon, London, England, 1998.

[368] M.W. Kemblowki and G.E. Urroz. Subsurface flow and transport. In L.W. Mays, editor, *Hydraulic Design Handbook*, pages 4.1–4.26. McGraw-Hill Book Company, New York, New York, 1999.

[369] W.S. Kerby. Time of Concentration for Overland Flow. *Civil Engineering*, 29(3):174, March 1959.

[370] G.H. Keulegan. Laws of turbulent flow in open channels. *Journal of Research of N.B.S.*, 21:707–741, 1938.

[371] D.F. Kibler. Desk-top runoff methods. In D.F. Kibler, editor, *Urban Stormwater Hydrology*, pages 87–135. American Geophysical Union, Water Resources Monograph Series, Washington, DC, 1982.

[372] G.J. Kirmeyer, L. Kirby, B.M. Murphy, P.F. Noran, K. Martel, T.W. Lund, J.L. Anderson, and R. Medhurst. *Maintaining Water Quality in Finished Water Storage Facilities*. AWWA Research Foundation and American Water Works Association, Denver, CO, 1999.

[373] P.Z. Kirpich. Time of Concentration of Small Agricultural Watersheds. *Civil Engineering*, 10(6):362, June 1940.

[374] G.W. Kite. *Frequency and Risk Analysis in Hydrology*. Water Resources Publications, Fort Collins, Colorado, 1977.

[375] R.T. Knapp, J.W. Daily, and F.G. Hammitt. *Cavitation*. McGraw-Hill, Inc., New York, New York, 1970.

[376] S.F. Korom, K.F. Bekker, and O.J. Helweg. Influence of Pump Intake Location on Well Efficiency. *Journal of Hydrologic Engineering*, 8(4):197–203, July/August 2003.

[377] N.T. Kottegoda and R. Rosso. *Statistics, Probability, and Reliability for Civil and Environmental Engineers*. McGraw-Hill, Inc., New York, New York, 1997.

[378] D. Koutsoyiannis. A probabilistic view of Hershfield's method for estimating probable maximum precipitation. *Water Resources Research*, 35(4):1313–1322, April 1999.

[379] N. Kouwen, R.M. Li, and D.B. Simons. Velocity measurements in a channel lined with flexible plastic roughness elements. Technical report, Department of Civil Engineering, Colorado State University, Fort Collins, Colorado, 1980. Technical Report No. CER79-80-RML-DBS-11.

[380] J. Kozeny. Über Kapillare Leitung des Wassers im Boden. *Sitzungsber. Akad. Wiss.*, 36:271–306, 1927. (in German).

[381] J. Kozeny. Theorie und Berechnung der Brunnen. *Wasserkraft und Wassenwirtschaft*, 28:88–92, 101–105, 113–116, 1933. (in German).

[382] M. Krishnamurthy and B.A. Christensen. Equavalent roughness for shallow channels. *Journal of the Hydraulics Division, ASCE*, 98(12):2257–2263, 1972.

[383] W.C. Krumbein and G.D. Monk. Permeability as a function of the size parameters of unconsolidated sand. *Am. Inst. Min. & Met. Eng. Tech. Pub.*, 150(11):1492, 1942.

[384] W.C. Krumbein and G.D. Monk. Permeability as a function of the size parameters of unconsolidated sand. *Am. Inst. Min. & Met. Eng. Tech. Pub.*, 151:153–163, 1943.

[385] E. Kuichling. The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts. *Transactions of the American Society of Civil Engineers*, 20:1–56, 1889.

[386] A.L. Lagvankar and J.P. Velon. Wastewater and stormwater piping systems. In M.L. Nayyar, editor, *Piping Handbook*, pages C.619–C.665. McGraw-Hill Book Company, New York, New York, seventh edition, 2000.

[387] E.W. Lane. Design of stable channels. *Transactions of the American Society of Civil Engineers*, 120:1234–1279, 1955.

[388] K. Lansey and W. El-Shorbagy. Design of Pumps and Pump Facilities. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 12.1–12.41. McGraw-Hill, Inc., New York, New York, 2001.

[389] K. Lansey and L.M. Mays. Hydraulics of Water Distribution Systems. In L.W. Mays, editor, *Hydraulic Design Handbook*, pages 9.1–9.38. McGraw-Hill Book Company, New York, New York, 1999.

[390] B.E. Larock, R.W. Jeppson, and G.Z. Watters. *Hydraulics of Pipeline Systems*. CRC Press, Inc., Boca Raton, Florida, 2000.

[391] G. Leclerc and J.C. Schaake. Derivation of hydrologic frequency curves. Report 142, R.M. Parsons Laboratory of Hydrodynamics and Water Resources, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1972.

[392] D.R. Lee. A device for measuring seepage flux in lakes and estuaries. *Limnol. & Oceanogr.*, 22:140–147, 1977.

[393] J.G. Lee and J.P. Heany. Estimation of Urban Imperviousness and its Impacts on Storm Water Systems. *Journal of Water Resources Planning and Management*, 129(5):419–426, September/October 2003.

[394] T.-C. Lee. *Applied Mathematics in Hydrogeology.* Lewis Publishers, Inc., Boca Raton, Florida, 1999.

[395] J. Lehr, S. Hurlburt, B. Gallagher, and J. Voytek. *Design and Construction of Water Wells, A Guide for Engineers.* Van Nostrand Reinhold, New York, New York, 1988.

[396] D.H. Lennox. Analysis of step-drawdown test. *Journal of the Hydraulics Division, ASCE*, 92(HY6):25–48, 1966.

[397] B. Levy and R. McCuen. Assessment of storm duration for hydrologic design. *Journal of Hydrologic Engineering*, 4(3):209–213, July 1999.

[398] L. Leyton, E.R.C. Reynolds, and F.B. Thompson. Rainfall interception in forest and moorland. In W.E. Sopper and H.W. Lull, editors, *International symposium on forest hydrology; 1965 August 29 - September 10; Pennsylvania State University, University Park, PA*, pages 163–178. Oxford: Pergamon Press, 1967.

[399] J.A. Liggett. *Fluid Mechanics.* McGraw-Hill, Inc., New York, New York, 1994.

[400] J.T. Limerinos. Determination of the Manning Coefficient from Measured Bed Roughness in Natural Channels. Water-Supply Paper 1898-B, U.S. Geological Survey, Washington, DC, 1970.

[401] C.H. Lin, J.F. Yean, and C.T. Tsai. Influence of Sluice Gate Contraction Coefficient on Distinguishing Condition. *Journal of Irrigation and Drainage Engineering, ASCE*, 128(4):249–252, July/August 2002.

[402] S.S.T. Lin and W.A. Perkins. Review of pre-development runoff analysis methods, Volume I. Report DRE 270, South Florida Water Management District, West Palm Beach, Florida, 1989.

[403] R.K. Linsley, J.B. Franzini, D.L. Freyberg, and G. Tchobanoglous. *Water-Resources Engineering.* McGraw-Hill, Inc., New York, New York, fourth edition, 1992.

[404] R.K. Linsley, M.A. Kohler, and J.L.H. Paulhus. *Hydrology for Engineers.* McGraw-Hill, Inc., New York, New York, third edition, 1982.

[405] C.P. Liou. Limitations and proper use of the Hazen-Williams equation. *Journal of Hydraulic Engineering*, 124(9):951–954, September 1998.

[406] D.E. Lloyd-Davies. The elimination of storm water from sewerage systems. *Proc. Inst. Civ. Eng.*, 164:41–67, 1906.

[407] G.V. Loganathan, D.F. Kibler, and T.J. Grizzard. Urban Stormwater Management. In L.W. Mays, editor, *Water Resources Handbook*, pages 26.1–26.35. McGraw-Hill, Inc., New York, New York, 1996.

[408] S.W. Lohman. Ground-water hydraulics. Professional Paper 708, U.S. Geological Survey, 1972.

[409] M.A. Lopez and W.M. Woodham. Magnitude and frequency of flooding on small urban watersheds in the Tampa Bay area, west-central Florida. Water-Resources Investigations Report 82-42, United States Geological Survey, Reston, Virginia, 1982.

[410] G.K. Lotter. Considerations of hydraulic design of channels with different roughness of walls. *Trans. All Union Scientific Research, Institute of Hydraulic Engineering*, 9:238–241, 1933.

[411] D.P. Loucks. Surface-Water Quality Management Models. In A.K. Biswas, editor, *Systems Approach to Water Management*. McGraw-Hill Book Company, New York, New York, 1976.

[412] W. Maddaus, G. Gleason, and J. Darmody. Integrating Conservation into Water Supply Planning: How can water suppliers achieve an appropriate balance between capacity expansion and conservation? *Journal of the American Water Works Association*, November 1996.

[413] R. Manning. Flow of water in open channels and pipes. *Trans. Inst. Civil Engrs. (Ireland)*, 20, 1890.

[414] A. Mantoglou. Pumping management of coastal aquifers using analytic models of saltwater intrusion. *Water Resources Research*, 39(12), 1335, doi:10.1029/2002WR001891, 2003.

[415] J. Marsalek. Calibration of the tipping bucket raingage. *Journal of Hydrology*, 53:343–354, 1981.

[416] D.L. Martin and J. Gilley. Irrigation Water Requirements. In *National Engineering Handbook*, chapter 2, part 623. U.S. Department of Agriculture, Natural Resources Conservation Service, 1993.

[417] E.S. Martins and J.R. Stedinger. Generalized maximum-liklihood generalized extreme-value quantile estimators for hydrologic data. *Water Resources Research*, 36(3):737–744, March 2000.

[418] L.W. Mays. Water Resources: An Introduction. In L.W. Mays, editor, *Water Resources Handbook*, pages 1.3–1.35. McGraw-Hill, Inc., New York, New York, 1996.

[419] L.W. Mays. Introduction. In L.W. Mays, editor, *Water Distribution Systems Handbook*, pages 1.1–1.30. McGraw-Hill Book Company, New York, New York, 2000.

[420] L.W. Mays. *Water Resources Engineering*. John Wiley & Sons, New York, New York, 2001.

[421] G.T. McCarthy. The unit-hydrograph and flood routing. Unpublished article at *U.S. Army Corps of Engineers North Atlantic Division Conference*, 1938.

[422] R.H. McCuen. *Hydrologic Analysis and Design*. Prentice-Hall, Upper Saddle River, New Jersey, 1989.

[423] R.H. McCuen. *Hydrologic Analysis and Design*. Prentice-Hall, Upper Saddle River, New Jersey, second edition, 1998.

[424] R.H. McCuen. Do water resources engineers need factors of safety for design? *Journal of Water Resources Planning and Management*, 127(1):4–5, 2001.

[425] R.H. McCuen. *Modeling Hydrologic Change: Statistical Methods*. Lewis Publishers, Inc., Boca Raton, Florida, 2002a.

[426] R.H. McCuen. *Hydrologic Analysis and Design*. Prentice-Hall, Upper Saddle River, New Jersey, third edition, 2005.

[427] R.H. McCuen and R.E. Beighley. Seasonal flow frequency analysis. *Journal of Hydrology*, 279:43–56, 2003.

[428] R.H. McCuen, P.A. Johnson, and R.M. Ragan. Hydrology. Technical Report HDS-2, U.S. Federal Highway Administration, Washington, DC, September 1996.

[429] R.H. McCuen and O. Okunola. Extension of TR-55 for Microwatersheds. *Journal of Hydrologic Engineering*, 7(4):319–325, July/August 2002.

[430] R.H. McCuen, S.L. Wong, and W.J. Rawls. Estimating urban time of concentration. *Journal of Hydraulic Engineering*, 110(7):887–904, 1984.

[431] M.G. McDonald and A.W. Harbaugh. A modular three-dimensional finite-difference groundwater flow model. Techniques of Water Resources Investigations of the United States Geological Survey, Book 6, Chapter A1, U.S. Geological Survey, Reston, Virginia, 1988.

[432] B.M. McEnroe. Characteristics of intense storms in Kansas. In *World Water Issues in Evolution*, pages 933–940. ASCE, New York, New York, 1986.

[433] T.J. McGhee. *Water Supply and Sewerage*. McGraw-Hill, Inc., New York, New York, sixth edition, 1991.

[434] R.G. Mein and C.L. Larson. Modeling infiltration during a steady rain. *Water Resources Research*, 9(2):384–394, 1973.

[435] R.A. Meriam. A note on the interception loss equation. *Journal of Geophysical Research*, 65:3850–3851, 1960.

[436] Metcalf & Eddy, Inc. *Wastewater Engineering: Collection and Pumping of Wastewater*. McGraw-Hill, Inc., New York, New York, 1981.

[437] P.E. Meyer-Peter and R. Muller. Formulas for Bed Load Transport. In *Proceedings of the Third International Association for Hydraulic Research, Stockholm, Sweden*, pages 39–64, 1948.

[438] Miami-Dade Public Works Department. *Public Works Manual, Part 2: Design and Construction*, 2001.

[439] D.W. Miller, J.J. Geraghty, and R.S. Collins. *Water Atlas of the United States*. Water Information Center, Inc., Port Washington, New York, 1962.

[440] J.F. Miller. Probable maximum precipitation and rainfall frequency data for Alaska. Technical report, U.S. Department of Commerce, Weather Bureau, Washington, DC, 1963.

[441] J.F. Miller, R.H. Frederick, and R.J. Tracey. Precipitation frequency atlas of the western United States. NOAA Atlas 2, U.S. National Weather Service, Silver Springs, MD, 1973. 11 Volumes.

[442] N. Miller and R. Cronshey. Runoff curve numbers – the next step. In B.C. Yen, editor, *Channel Flow and Catchment Runoff*, pages 910–916. Department of Civil Engineering, University of Virginia, Charlottseville, Virginia, 1989.

[443] S.K. Mishra, J.V. Tyagi, and V.P. Singh. Comparison of infiltration models. *Hydrological Processes*, 17:2629–2652, August 2003.

[444] J.L. Monteith. Evaporation and the environment. *Symp. Soc. Expl. Biol.*, 19:205–234, 1965.

[445] J.L. Monteith. *Principles of Environmental Physics*. Edward Arnold, London, 1973.

[446] J.L. Monteith. Evaporation and surface temperature. *Quarterly Journal of Royal Meteorological Society*, 107:1–27, 1981.

[447] S. Montes. *Hydraulics of Open Channel Flow*. ASCE Press, Reston, Virginia, 1998.

[448] J.H. Montgomery. *Groundwater Chemicals Desk Reference*. Lewis Publishers, Inc., Boca Raton, Florida, 3 edition, 2000.

[449] L.F. Moody. Friction Factors for Pipe Flow. *Trans. ASME*, 66(8), 1944.

[450] L.F. Moody. Some Pipe Characteristics of Engineering Interest. *Houille Blanche*, May-June 1950.

[451] L.F. Moody and T. Zowski. Hydraulic Machinery. In C.V. Davis and K.E. Sorensen, editors, *Handbook of Applied Hydraulics*. McGraw-Hill, Inc., New York, New York, third edition, 1989.

[452] H.J. Morel-Seytoux and J.P. Verdin. Extension of Soil Conservation Service rainfall runoff methodology for ungaged watersheds. Report FHWA/RD-81/060, Federal Highway Administration, Washington, DC, 1981.

[453] R. Morgan and D.W. Hulinghorst. Unit Hydrographs for Gauged and Ungauged Watersheds. Technical report, U.S. Engineers Office, Binghampton, New York, July 1939.

[454] D.A. Morris and A.I. Johnson. Summary of hydrologic and physical properties of rock and soil materials, as analysed by the Hydrologic Laboratory of the U.S. Geologic Survey, 1948-1960. Water-Supply Paper 1839-D, U.S. Geological Survey, 1967.

[455] A.K. Motayed and M. Krishnamurthy. Composite roughness of natural channels. *Journal of the Hydraulics Division, ASCE*, 106(6):1111–1116, 1980.

[456] R.L. Mott. *Applied Fluid Mechanics.* Merrill, New York, New York, 1994.

[457] L. Muhlhofer. Rauhigkeitsuntersuchungen in einem stollen mit betonierter sohle und unverkleideten wanden. *Wasserkraft und Wasserwirtschaft*, 28(8):85–88, 1933.

[458] T.J. Mulvaney. On the use of self-registering rain and flood gauges in making observations on the relations of rainfall and flood discharges in a given catchment. *Trans. Inst. Civ. Eng. Ireland*, 4(2):18–31, 1850.

[459] B.R. Munson, D.F. Young, and T.H. Okiishi. *Fundamentals of Fluid Mechanics.* John Wiley & Sons, New York, New York, second edition, 1994.

[460] L.C. Murdoch and S.E. Kelly. Factors affecting the performance of conventional seepage meters. *Water Resources Research*, 39(6), 1163, doi:10.1029/2002WR001347, 2003.

[461] T.E. Murphy. Spillway Crest Design. Misc. Paper H-73-5, Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksburg, MS, 1973.

[462] M. Muskat. Potential distribution in large cylindrical discs (homogeneous sands) with partially penetrating electrodes (partially penetrating wells). *Physics*, 2(5):329–384, 1932.

[463] M. Muskat. *The Flow of Homogeneous Fluids through Porous Media.* McGraw-Hill, Inc., New York, New York, 1937.

[464] W.R.C. Myers. Influence of geometry on discharge capacity of open channels. *Journal of Hydraulic Engineering*, 117(5):676–680, May 1991.

[465] S. Naoum and I.K. Tsanis. Temporal and spatial variation of annual rainfall on the island of Crete, Greece. *Hydrological Processes*, 17:1899–1922, 2003.

[466] National Research Council. *Safety of dams, flood and earthquake criteria.* National Academy Press, Washington, DC, 1985.

[467] National Research Council. *Opportunities in the Hydrologic Sciences.* National Academy Press, Washington, DC, 1991.

[468] National Weather Service (NWS). *NWSRFS User's Manual Documentation, Release Number 64*, June 2002. ⟨http://www.nws.noaa.gov/oh/hrl/nwsrfs/users_manual/htm/xrfsdochtm.htm⟩.

[469] L.C. Neale and R.E. Price. Flow characteristics of PVC sewer pipe. *Journal of the Sanitary Engineering Division, ACSE*, 90(SA3):109, 1964.

[470] S.P. Neuman. Theory of flow in unconfined aquifers considering delayed response of the water table. *Water Resources Research*, 8(4):1031–1045, August 1972.

[471] S.P. Neuman. Supplementary comments on 'Theory of flow in unconfined aquifers considering delayed response of the water table'. *Water Resources Research*, 9:1102–1103, 1973.

[472] S.P. Neuman. Analysis of pumping test data from anisotropic unconfined aquifers considering delayed gravity response. *Water Resources Research*, 11:329–342, 1975.

[473] S.P. Neuman and P.A. Witherspoon. Theory of flow in aquicludes adjacent to slightly leaky aquifers. *Water Resources Research*, 4(1):103–112, 1968.

[474] S.P. Neuman and P.A. Witherspoon. Theory of flow in a confined two aquifers system. *Water Resources Research*, 5(2):803–816, 1969a.

[475] S.P. Neuman and P.A. Witherspoon. Applicability of current theories of flow in leaky aquifers. *Water Resources Research*, 5(4):817–829, 1969b.

[476] S.P. Neuman and P.A. Witherspoon. Field determination of the hydraulic properties of leaky multiple aquifer systems. *Water Resources Research*, 8(5):1284–1298, 1972.

[477] B.D. Newport. Salt water intrusion in the United States. Technical Report 600/8-77-011, U.S. Environmental Protection Agency, Washington, DC, 1977.

[478] V. Nguyen and G.F. Pinder. Direct calculation of aquifer parameters in slug test analysis. In J. Rosenshein and G.D. Bennett, editors, *Groundwater Hydraulics*, Water Resources Monograph 9, pages 222–239. American Geophysical Union, 1984.

[479] J.W. Nicklow. Design of Stormwater Inlets. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 5.1–5.42. McGraw-Hill, Inc., New York, New York, 2001.

[480] J. Niemczynowicz. The dynamic calibration of tipping-bucket raingauges. *Nord. Hydrol.*, 17:203–214, 1986.

[481] J. Nikuradse. Gesetzmässigkeiten der turbulenten strömung in glatten röhren. *VDI-Forschungsh*, 356, 1932.

[482] J. Nikuradse. Strömungsgesetze in rauhen Röhren. *VDI-Forschungsh*, 362, 1933.

[483] N.S. Norman, E.J. Nelson, and A.K. Zundel. Improved Process for Foodplain Delineation from Digital Terrain Models. *Journal of Water Resources Planning and Management*, 129(5):427–436, September/October 2003.

[484] M.A. Nouh and R.D. Townsend. Shear stress distribution in stable channel bends. *Journal of the Hydraulics Division, ASCE*, 105(HY10):1233–1245, October 1979.

[485] P. Novak. Improvement of water quality in rivers by aeration at hydraulic structures. In M. Hino, editor, *Water Quality and its Control*, pages 147–168. A.A. Balkema Publishers, Brookfield, Vermont, 1994.

[486] V. Novotny, K.R. Imhoff, M. Olthof, and P.A. Krenkel. *Karl Imhoff's Handbook of Urban Drainage and Wastewater Disposal*. John Wiley & Sons, New York, New York, 1989.

[487] E. Nyer, G. Boettcher, and B. Morello. Using the properties of organic compounds to help design a treatment system. *Ground Water Monitoring Review*, 11(4):115–120, 1991.

[488] J.E. O'Connor and J.E. Costa. Spatial distribution of the largest rainfall-runoff floods from basins between 2.6 and 26,000 km^2 in the United States and Puerto Rico. *Water Resources Research*, 40, W01107, doi:10.1029/2003WR002247, 2004.

[489] American Society of Civil Engineers. *Quality of Ground Water, Guidelines for Selection and Application of Frequently Used Models.* American Society of Civil Engineers, New York, New York, 1996. Manual of Practice No.85.

[490] National Academy of Sciences. *Safety of Existing Dams: Evaluation and Improvement.* National Academy Press, Washington, DC, 1983.

[491] I.C. Olmsted. Stomatal resistance and water stress in melaleuca. Final Report, Contract with USDA, Forest Service, August 1978.

[492] R.M. Olson and S.J. Wright. *Essentials of Engineering Fluid Mechanics.* Harper & Row, Publishers, Inc., New York, New York, fifth edition, 1990.

[493] P.S. Osbourne. Suggested operating procedures for aquifer pumping tests. EPA Ground Water Issue EPA/540/S-93/503, U.S. Environmental Protection Agency, February 1993.

[494] D.E. Overton and M.E. Meadows. *Storm Water Modeling.* Academic Press, New York, New York, 1976.

[495] J.N. Paine and A.O. Akan. Design of Detention Systems. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 7.1–7.66. McGraw-Hill, Inc., New York, New York, 2001.

[496] W.C. Palmer. Meteorological Drought. Technical Report Research Paper No. 45, U.S. Department of Commerce, Weather Bureau, 1965.

[497] J.F. Pankow and J.A. Cherry. *Dense Chlorinated Solvents and other DNAPLs in Grounwater.* Waterloo Press, Portland Oregon, 1996.

[498] I.S. Papadopulos and H.H. Cooper, Jr. Drawdown in a well of large diameter. *Water Resources Research*, 3(1):241–244, 1967.

[499] N.N. Pavlovskii. K Voporosu o Raschetnoi dlia Ravnomernogo Dvizheniia v Vodotokahk s Neodnorodnymi Stenkami (On a design formula for uniform flow in channels with nonhomogeneous walls). *Izvestiia Vsesoiuznogo Nauchno-Issledovatel'skogo Instituta Gidrotekhniki (Trans. All-Union Sci. Res. Inst. Hydraulic Eng.)*, 3:157–164, 1931.

[500] K. Pearson. *Tables for Statisticians and Biometricians.* Columbia University Press, New York, New York, third edition, 1930.

[501] R. Pecher. The runoff coefficient and its dependence on rain duration. *Berichte aus dem Institut fur Wasserwirtschaft und Gesundheitsingenieurwesen, No.2*, 1969. TU Munich.

[502] H. Penman. Natural evaporation from open water, bare soil, and grass. *Proc. R. Soc. A.*, A193:120–146, 1948.

[503] H.L. Penman. Evaporation: An introductory survey. *Netherlands Journal of Agricultural Sciences, Series A*, 4:9–29, 1956.

[504] H.L. Penman. Vegetation and hydrology. Technical Communication 53, Commonwealth Bureau of Soils, Harpenden, England, 1963. 125pp.

[505] L.S. Pereira, A. Perrier, R.G. Allen, and I. Alves. Evapotranspiration: Concepts and future trends. *Journal of Irrigation and Drainage Engineering, ASCE*, 125(2):45–51, 1999.

[506] J.R. Philip. An infiltration equation with physical significance. *Soil Science*, 77:153–157, 1954.

[507] J.R. Philip. Variable-head ponded infiltration under constant or variable rainfall. *Water Resources Research*, 29(7):2155–2165, July 1993.

[508] N.N. Pillai. Effect of shape on uniform flow through smooth rectangular open channels. *Journal of Hydraulic Engineering*, 123(7):656–658, July 1997.

[509] R.D. Pomeroy and J.D. Parkhurst. The forecasting of sulfide buildup rates in sewers. *Progress in Water Technology*, 9, 1977.

[510] V.M. Ponce. *Engineering Hydrology, Principles and Practices*. Prentice-Hall, Upper Saddle River, New Jersey, 1989.

[511] V.M. Ponce, A.K. Lohani, and P.T. Huston. Surface albedo and water resources: Hydroclimatological impact of human activities. *Journal of Hydrologic Engineering*, 2(4):197–203, October 1997.

[512] V.M. Ponce, R.P. Pandey, and Sezar Ercan. Characterization of drought across climatic spectrum. *Journal of Hydrologic Engineering*, 5(2):222–224, April 2000.

[513] M.C. Potter and D.C. Wiggert. *Mechanics of Fluids*. Prentice-Hall, Upper Saddle River, New Jersey, 1991.

[514] M.C. Potter and D.C. Wiggert. *Mechanics of Fluids*. Brooks/Cole, Pacific Grove, California, third edition, 2001.

[515] A. Prakash. *Water Resources Engineering*. ASCE Press, New York, New York, 2004.

[516] C.H.B. Priestly and R.J. Taylor. On the assessment of surface heat flux and evaporation using large-scale parameters. *Mon. Weath. Rev.*, 100:81–92, 1972.

[517] L.G. Puls. Construction of flood flow routing curves, 1928. US 70th Congress, First Session, House Document 185, pp.46-52, and US 71st Congress, Second Session, House Document 328, pp.190-191.

[518] V.L. Quisenberry and R.E. Phillips. Percolation of surface-applied water in the field. *Soil Sci. Soc. Am. J.*, 40:484–489, 1976.

[519] M. Radojkovic and C. Maksimovic. On standardization of computational models for overland flow. In B.C. Yen, editor, *Proceedings of the Fourth International Conference on Urban Storm Drainage*, pages 100–105, Lausanne, Switzerland, 1987. International Association for Hydraulic Research.

[520] C.D. Rail. *Groundwater Contamination.* Technomic Publishing Company, Inc., Lancaster, Pennsylvania, 1989.

[521] N. Rajaratnam and K. Subramanya. Flow equation for the sluice gate. *Journal of Irrigation and Drainage Engineering, ASCE*, 93(3):167–186, 1967.

[522] C.E. Ramser. Runoff from Small Agricultural Areas. *Journal of Agricultural Research*, 34(9), 1927.

[523] R.E. Rathbun. Transport, behavior, and fate of volatile organic compounds in streams. Professional Paper 1589, U.S. Geological Survey, Washington, DC, 1998.

[524] A.J. Raudkivi and R.A. Callander. *Analysis of Groundwater Flow.* John Wiley & Sons, Inc., New York, New York, 1976.

[525] W. Rawls, P. Yates, and L. Asmussen. Calibration of selected infiltration equations for the Georgia Coastal Plain. Technical Report ARS-S-113, U.S. Department of Agriculture, Agricultural Research Service, Washington, DC, 1976.

[526] W.J. Rawls and D.L. Brakensiek. Estimating soil water retention from soil properties. *Journal of Irrigation and Drainage Engineering, ASCE*, 108(2):166–171, 1982.

[527] W.J. Rawls, D.L. Brakensiek, and N. Miller. Green-Ampt infiltration parameters from soils data. *Journal of Hydraulic Engineering*, 109(1):1316–1320, 1983.

[528] W.J. Rawls, D.L. Brakensiek, and K.E. Saxton. Estimation of Soil Water Properties. *Transactions of the American Society of Agricultural Engineers*, 25(5):1316–1320, 1328, 1982.

[529] A.J. Reese and S.T. Maynord. Design of Spillway Crest. *Journal of Hydraulic Engineering*, 113(4):476–490, 1987.

[530] O. Reynolds. An experimental investigation of the circumstances which determine whether the motion of water shall be direct or sinuous and of the law of resistance in parallel channel. *Philosophical Transactions, Royal Society of London*, 174:935–982, 1883.

[531] W. Rippl. The Capacity of Storage Reservoirs for Water Supply. *Proceedings of the Institution of Civil Engineers, London*, 71:270, 1883.

[532] J.A. Roberson and C.T. Crowe. *Engineering Fluid Mechanics.* John Wiley & Sons, New York, New York, sixth edition, 1997.

[533] F. Rodriguez, H. Andrieu, and J.-D. Creutin. Surface runoff in urban catchments: morphological identification of unit hydrographs from urban databanks. *Journal of Hydrology*, 283:146–168, 2003.

[534] M.I. Rorabaugh. Graphical and theoretical analysis of step-drawdown test of artsian well. *Proceedings of the American Society of Civil Enginers*, 79(362):23 pp., 1953.

[535] Roscoe Moss Company. *Handbook of Groundwater Development*. John Wiley & Sons, New York, New York, 1990.

[536] J.R. Rossing. *Identification of critical runoff generating areas using a variable source area model*. PhD thesis, Cornell University, Ithaca, New York, 1996.

[537] L.A. Rossman. Computer models/epanet. In L.W. Mays, editor, *Water Distribution Systems Handbook*, pages 12.1–12.23. McGraw-Hill Book Company, New York, New York, 2000.

[538] W.M. Grayman L.A. Rossman and E.E. Geldreich. Water quality. In L.W. Mays, editor, *Water Distribution Systems Handbook*, pages 9.1–9.24. McGraw-Hill Book Company, New York, New York, 2000.

[539] R.L. Rossmiller. The rational formula revisited. In *Proceedings of the International Symposium on Urban Storm Runoff, July 28-31*, Lexington, Kentucky, 1980. University of Kentucky.

[540] J. Rothacher. Net precipitation under a Douglas-fir forest. *Forest Science*, 9(4):423–429, 1963.

[541] H. Rouse. *Elementary Fluid Mechanics*. John Wiley & Sons, New York, New York, 1946.

[542] H. Rouse and S. Ince. *History of Hydraulics*. Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, 1957.

[543] L.K. Rowe. Rainfall interception by a beech-podocarp-hardwood forest near Reefton, North Westland, New Zealand. *Journal of Hydrology (N.Z.)*, 18(2):63–72, 1979.

[544] H. Rubin and J. Atkinson. *Environmental Fluid Mechanics*. Marcel Dekker, Inc., New York, New York, 2001.

[545] M.L. Sage and G.W. Sage. Vapor Pressure. In R.S. Boethling and D. Mackay, editors, *Handbook of Property Estimation Methods for Chemicals*, pages 53–65. Lewis Publishers, Inc., Boca Raton, Florida, 2000.

[546] J.D. Salas. Analysis and modeling of hydrologic time series. In D.R. Maidment, editor, *Handbook of Hydrology*, pages 19.1–19.72. McGraw-Hill Book Company, New York, New York, 1993.

[547] G.D. Salvucci and D. Entekhabi. Explicit expressions for Green-Ampt (delta function diffusivity) infiltration rate and cumulative storage. *Water Resources Research*, 30(9):2661–2663, September 1994.

[548] Z. Samani. Estimating solar radiation and evapotranspiration using minimum climatological data. *Journal of Irrigation and Drainage Engineering, ASCE*, 126(4):265–267, 2000.

[549] X. Sánchez-Vila, P.M. Meier, and J. Carrera. Pumping tests in heterogeneous aquifers: An analytical study of what can be obtained from their interpretation using Jacob's method. *Water Resources Research*, 35(4):943–952, April 1999.

[550] R. Schildgen. Unnatural disasters. *Sierra*, 84:48–57, 1999.

[551] H. Schlichting. *Boundary Layer Theory.* McGraw-Hill, Inc., New York, New York, seventh edition, 1979.

[552] S. Schmorak and A. Mercado. Upconing of fresh water-sea water interface below pumping wells, field study. *Water Resources Research*, 5:1290–1311, 1969.

[553] J.L. Schnoor. *Environmental Modeling: Fate and Transport of Pollutants in Water, Air, and Soil.* John Wiley & Sons, New York, New York, 1996.

[554] T.R. Schueler. *Controlling urban runoff: A practical manual for planning and designing urban BMPs.* Metropolitan Washington Council of Governments, Washington, DC, 1987.

[555] T.R. Schueler, P.A. Crumble, and M.A. Heraty. *Current Assessment of Urban Best Management Practices: Techniques for Reducing Non-point Source Pollution in the Coastal Zones.* Metropolitan Washington Council of Governments, Washington, DC, 1991.

[556] C.K. Sehgal. Design guidelines for spillway gates. *Journal of Hydraulic Engineering*, 122(3):155–165, March 1996.

[557] Soil Conservation Service. *SCS National Engineering Handbook, Section 4: Hydrology.* U.S. Department of Agriculture, Washinton, DC, 1993.

[558] R.M. Shane and W.R. Lynn. Mathematical model for flood risk evaluation. *Journal of the Hydraulics Division, ASCE*, 90(HY6):1–20, November 1964.

[559] L.K. Sherman. Stream-flow from rainfall by the unit graph method. *Engineering News Record*, 108:501–505, 1932.

[560] J.-T. Shiau and H.W. Shen. Recurrence analysis of hydrologic droughts of differing severity. *Journal of Water Resources Planning and Management*, 127(1):30–40, January/February 2001.

[561] V. Sichard. *Das Fassungsvermögen von Bohrbrunnen und Eine Bedeutung für die Grundwasserersenkung inbesondere für grossere Absentiefen.* PhD thesis, Tech. Hochschule, Berlin, 1927.

[562] J.R. Simanton and H.B. Osborn. Reciprocal-distance estimate of point rainfall. *Journal of the Hydraulic Engineering Division, ASCE*, 106(HY7), July 1980.

[563] D.B. Simons and E.V. Richardson. Resistance to Flow in Alluvial Channels. Professional Paper 422-J, U.S. Geological Survey, Washington, DC, 1966.

[564] V.P. Singh. *Hydrologic Systems, Rainfall-Runoff Modeling, Volume I.* Prentice-Hall, Englewood Cliffs, New Jersey, 1989.

[565] V.P. Singh. *Elementary Hydrology.* Prentice-Hall, Upper Saddle River, New Jersey, 1992.

[566] V.P. Singh and W.G. Strupczewski. On the status of flood frequency analysis. *Hydrological Processes*, 16:3737–3740, 2002b.

[567] M.K. Smith. Throughfall, stemflow, and interception in pine and eucalypt forests. *Australian Forestry*, 36:190–197, 1974.

[568] M.R. Smith, R.G. Allen, J.L. Monteith, L.S. Pereira, and A. Segeren. Report on the Expert Consultation on Procedures for Revision of FAO Guidelines for Predicting Crop Water Requirements. Technical report, Land and Water Development Division, Food and Agricultural Organization of the United Nations, Rome, 1991.

[569] R.E. Smith. Discussion of 'Runoff curve number: Has it reached maturity?' by V.M. Ponce and R.H. Hawkins. *Journal of Hydrologic Engineering*, 2(3):145–147, 1997.

[570] F.F Snyder. Synthetic Unit Hydrographs. *Transactions of the American Geophysical Union*, 19:447, 1938.

[571] Soil Conservation Service. Computer program for project formulation hydrology (draft). Technical Release 20, Soil Conservation Service, U.S. Department of Agriculture, Washington, DC, 1983.

[572] Soil Conservation Service. Urban hydrology for small watersheds. Technical Release 55, Soil Conservation Service, U.S. Department of Agriculture, Washington, DC, 1986.

[573] W.B. Solley. Estimated Use of Water in the United States in 1995. Circular 1200, U.S. Geological Survey, Washington, DC, 1998.

[574] South Carolina Land Resources Conservation Commission. South Carolina stormwater management and sediment reduction regulations. Technical report, Division of Engineering, Columbia, South Carolina, 1992.

[575] South Florida Water Management District. *Management and Storage of Surface Waters, Permit Information Manual, Volume IV.* West Palm Beach, Florida, May 1994.

[576] South Florida Water Management District. *Best Management Practices for South Florida Urban Stormwater Management Systems.* West Palm Beach, Florida, April 2002.

[577] M.S. Sperry and J.J. Pierce. A model for estimating the hydraulic conductivity of granular material based on grain shape, grain size, and porosity. *Ground Water*, 33(6):892–898, November-December 1995.

[578] P. Stahre and B. Urbonas. Swedish approach to infiltration and percolation design. In L.A. Roesner, B. Urbonas, and M.B. Sonnen, editors, *Design of Urban Runoff Quality Controls*, pages 307–323. ASCE, New York, New York, 1989.

[579] R.W. Stallman. Aquifer-test design, observation and data analysis. Techniques of Water Resources Investigations, Book 3, Chapter B1, U.S. Geological Survey, 1971.

[580] G. Stanhill. The control of field irrigation practice from measurements of evaporation. *Israel J. Agric. Res.*, 12:51–62, 1962.

[581] T.E. Stanton and J.R. Pannell. Similarity of motion in relation to surface friction of fluids. *Philosophical Transactions, Royal Society of London*, 214A:199–224, 1914.

[582] A.J. Stepanoff. *Centrifugal and Axial Flow Pumps.* John Wiley & Sons, New York, New York, second edition, 1957.

[583] W.J. Stone. *Hydrogeology in Practice.* Prentice-Hall, Upper Saddle River, New Jersey, 1999.

[584] R.L. Street, G.Z. Watters, and J.K. Vennard. *Elementary Fluid Mechanics.* John Wiley & Sons, New York, New York, seventh edition, 1996.

[585] V.L. Streeter and E.B. Wylie. *Fluid Mechanics.* McGraw-Hill, Inc., New York, New York, eighth edition, 1985.

[586] V.L. Streeter, E.B. Wylie, and K.W. Bedford. *Fluid Mechanics.* McGraw-Hill, Inc., New York, New York, ninth edition, 1998.

[587] A. Strickler. Contributions to the question of a velocity formula and roughness data for streams, channels and closed pipelines. Technical report, W.M. Keck Laboratory, California Institute of Technology, Pasadena, California, 1923. Translation by T. Roesgen and W.R. Brownlie (1981).

[588] W.G. Strupczewski, V.P. Singh, and S. Weglarczyk. Asymptotic bias of estimation methods caused by the assumption of false probability distribution. *Journal of Hydrology*, 258:122–148, 2002.

[589] J.M. Stubchaer. The Santa Barbara Urban Hydrograph Method. Proceedings, National Symposium of Hydrology and Sediment Control, University of Kentucky, Lexington, Kentucky, 1980.

[590] T. Sturm. *Open Channel Hydraulics.* McGraw-Hill, Inc., New York, New York, 2001.

[591] P.K. Swamee. Generalized rectangular weir equations. *Journal of Hydraulic Engineering*, 114(8):945–949, 1988.

[592] P.K. Swamee and A.K. Jain. Explicit Equations for Pipe-Flow Problems. *ASCE, Journal of the Hydraulics Division*, 102(HY5):657–664, May 1976.

[593] W.T. Swank, N.B. Goebel, and J.D. Helvey. Interception loss in loblolly pine stands in the piedmont of South Carolina. *Journal of Soil and Water Conservation*, 27(4):160–164, 1972.

[594] R.M. Sykes. Water and Wastewater Planning. In W.F. Chen, editor, *The Civil Engineering Handbook*, pages 169–223. CRC Press, Inc., Boca Raton, Florida, 1995.

[595] T. Szirtes. *Applied Dimensional Analysis and Modeling.* McGraw-Hill, Inc., New York, New York, 1998.

[596] L.M. Tallaksen. A Review of Baseflow Recession Analysis. *Journal of Hydrology*, 165, 1995.

[597] G.D. Tasker and J.R. Stedinger. Regional skew with weighted LS regression. *Journal of Water Resources Planning and Management*, 112(2):225–237, April 1986.

[598] B.N. Taylor. Guide for the use of the international system of units (si). Technical report, National Institute of Standards and Technology, 1995. NIST Special Publication 811, 1995 Edition.

[599] D.W. Taylor. *Fundamentals of Soil Mechanics*. Wiley, New York, New York, 1948.

[600] C.V. Theis. The relation between lowering of the piezometric surface and the rate and duration of discharge of a well using ground water storage. In *Transactions of the American Geophysical Union, 16th Annual Meeting, Part 2*, pages 519–524, 1935.

[601] H.J. Thiébaux. *Statistical Data Analysis for Ocean and Atmospheric Sciences*. Academic Press, Inc., San Diego, California, 1994.

[602] G. Thiem. *Hydrologische*. Gebhardt, Leipzig, Germany, 1906.

[603] A.H. Thiessen. Precipitation for large areas. *Monthly Weather Review*, 39:1082–1084, July 1911.

[604] A.L. Tholin and C.J. Keifer. The hydrology of urban runoff. *Transactions of the American Society of Civil Engineers*, 125:1308, 1960.

[605] A.S. Thom and H.R. Oliver. On Penman's equation for estimating regional evapotranspiration. *Quarterly Journal of Royal Meteorological Society*, 103:345–357, 1977.

[606] G.W. Thomas and R.E. Phillips. Consequences of water movement in macropores. *J. Environ. Qual.*, 8(2):149–152, 1979.

[607] L. Thomas. What Causes Wells to Lose Capacity? *Opflow*, 28(1):8–15, January 2002. American Water Works Association.

[608] C.W. Thornthwaite. An approach toward a rational classification of climate. *Geograph. Rev*, 38:55, 1948.

[609] C.W. Thornthwaite and J.R. Mather. The water balance. *Climatology*, 8(1), 1955. Lab. of Climat., Centeron, N.J.

[610] J.A. Tindall and J.R. Kunkel. *Unsaturated Zone Hydrology for Scientists and Engineers*. Prentice-Hall, Upper Saddle River, New Jersey, 1999.

[611] D.K. Todd. *Groundwater Hydrology*. John Wiley & Sons, New York, New York, second edition, 1980.

[612] M. Todorovic. Single-layer evapotranspiration model with variable canopy resistance. *Journal of Irrigation and Drainage Engineering, ASCE*, 125(5):235–245, 1999.

[613] P.A. Troch, F.P. De Troch, and W. Brutsaert. Effective water table depth to describe initial conditions prior to storm rainfall in humid regions. *Water Resources Research*, 29(2):427–434, 1993.

[614] I.K. Tuncock and L.W. Mays. Hydraulic design of culverts and highway structures. In L.W. Mays, editor, *Hydraulic Design Handbook*, pages 15.1–15.71. McGraw-Hill Book Company, New York, New York, 1999.

[615] L. Turc. Evaluation des besoins en eau d'irrigation, evapotranspiration potentielle, formule climatique simplifice et mise a jour (in French). (English title: Estimation of irrigation water requirements, potential evapotranspiration: A simple climatic formula evolved up to date). *Ann. Agron.*, 12:13–49, 1961.

[616] G.J.G. Upton and A.R. Rahimi. On-line detection of errors in tipping-bucket raingages. *Journal of Hydrology*, 278:197–212, 2003.

[617] Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual*. Denver Regional Council of Governments, Denver, Colorado, 1984.

[618] B. Urbonas and P. Stahre. *Stormwater: Best Management Practices and Detention for Water Quality, Drainage, and CSO Management*. Prentice-Hall, Upper Saddle River, New Jersey, 1993.

[619] B.R. Urbonas and L.A. Roesner. Hydrologic Design for Urban Drainage and Flood Control. In D.R. Maidment, editor, *Handbook of Hydrology*, page Chapter 28. McGraw-Hill, Inc., New York, New York, 1993.

[620] U.S. Army Corps of Engineers. Standard Project Flood Determination. Report EM 1110-2-1411, U.S. Army Corps of Engineers, Washington, DC, March 1965.

[621] U.S. Army Corps of Engineers. *Feasibility Studies for Small Scale Hydropower Additions, Volume 3*. Hydrologic Engineering Center, Davis, California, 1979.

[622] U.S. Army Corps of Engineers. *Reservoir system analysis for conservation, HEC-3*. Hydrologic Engineering Center, Davis, California, 1981.

[623] U.S. Army Corps of Engineers. *Simulation of flood control and conservation systems, HEC-5*. Hydrologic Engineering Center, Davis, California, 1982.

[624] U.S. Army Corps of Engineers. Hydropower engineering manual. Report EM-1110-2-1701, Engineering Design, U.S. Army Corps of Engineers, Washington, DC, 1985.

[625] U.S. Army Corps of Engineers. *Accuracy of Computed Water Water Surface Profiles*. Hydrologic Engineering Center, Davis, California, 1986.

[626] U.S. Army Corps of Engineers. Digest of water resources policies and authorities. Report EP 1165-2-1, U.S. Army Corps of Engineers, Washington, DC, 1989.

[627] U.S. Army Corps of Engineers. *Interior Flood Hydrology package, HEC-IFH*. Hydrologic Engineering Center, Davis, California, 1992.

[628] U.S. Army Corps of Engineers. Hydrologic Frequency Analysis. Report EM 1110-2-1415, U.S. Army Corps of Engineers, Washington, DC, 1993.

[629] U.S. Army Corps of Engineers. Hydraulic design of flood control channels. Report EM 1110-2-1601, U.S. Army Corps of Engineers, Washington, DC, 1994.

[630] U.S. Army Corps of Engineers. *Hydraulic Design of Flood Control Channels*. American Society of Civil Engineers, New York, New York, 1995.

[631] U.S. Army Corps of Engineers. HEC-RAS Hydraulic Reference Manual, Version 2.2. Technical Reference Manual, Hydrologic Engineering Center, Davis, California, 1998.

[632] U.S. Army Corps of Engineers. Hydrologic Modeling System HEC–HMS. Technical Reference Manual CPD-74B, Hydrologic Engineering Center, Davis, California, March 2000.

[633] U.S. Army Corps of Engineers. HEC-RAS River Analysis System, Version 3.1. Hydraulic Reference Manual CPD-69, Hydrologic Engineering Center, Davis, California, November 2002.

[634] U.S. Bureau of Reclamation. Chapter 6.10: Flood Routing. Technical Report Water Studies, Part 6: Flood Hydrology, Volume IV, USBR, Washington, DC, 1949.

[635] U.S. Bureau of Reclamation. Research Studies on Stilling Basins, Energy Dissipators, and Associated Appurtenances. Technical Report Hydraulics Laboratory Report, Hyd-399, USBR, Washington, DC, 1955.

[636] U.S. Bureau of Reclamation. *Design of Small Dams*. U.S. Department of Interior, Washington, DC, 1977.

[637] U.S. Bureau of Reclamation. *Design of Small Canal Structures*. U.S. Department of Interior, Denver, Colorado, 1978.

[638] U.S. Bureau of Reclamation. *Design of Small Dams*. U.S. Government Printing Office, Washington, DC, third edition, 1987.

[639] U.S. Department of Agriculture. Irrigation water requirements. Technical Release 21 (rev), Soil Conservation Service, 1970. 92pp.

[640] U.S. Department of Agriculture. Design of open channels. Technical Release No.25, Soil Conservation Service, Washington, DC, October 1977.

[641] U.S. Department of Commerce. Generalized estimates of probable maximum precipitation and rainfall frequency data for Puerto Rico and Virgin Islands. Technical Paper 42, Weather Bureau, Washington, DC, 1961.

[642] U.S. Department of Commerce. Rainfall-frequency atlas of the Hawaiian Islands. Technical Paper 43, Weather Bureau, Washington, DC, 1962.

[643] U.S. Department of the Army. Hydraulic Design of Spillways. Technical Report Engineering Manual 1110-2-1603, Office of the Chief of Engineers, USDOA, Washington, DC, 1986.

[644] U.S. Department of Transportation, Federal Highway Administration. *HEC No.12*. U.S. Department of Transportation, McLean, VA, 1984a.

[645] U.S. Department of Transportation, Federal Highway Administration. Hydraulic design of highway culverts. Hydraulic Design Series 5 (HDS-5) FHWA-IP-85-15, FHWA, Washington, DC, 1985. Reprinted 1998.

[646] U.S. Department of Transportation, Federal Highway Administration. HEC-15, Design of Roadside Channels with Flexible Linings. Technical Report FHWA-IP-87-7, FHWA, Washington, DC, April 1988.

[647] U.S. Department of Transportation, Federal Highway Administration. HEC-19, Hydrology. Technical Report FHWA-IP-95, FHWA, Washington, DC, 1995.

[648] U.S. Department of Transportation, Federal Highway Administration. HEC-22, Urban Drainage Design Manual. Technical Report FHWA-SA-96-078, FHWA, Washington, DC, 1996.

[649] U.S. Department of Transportation, Federal Highway Administration. Hydraulic Design Series 2 (HDS-2) *Hydrology*. Technical report, U.S. Federal Highway Administration, Washington, DC, September 1996.

[650] U.S. Environmental Protection Agency. Guidelines for delineation of wellhead protection areas. Technical Report EPA 440/6-87-010, Office of Ground-Water Protection, Washington, DC, June 1987.

[651] U.S. Environmental Protection Agency. Guide to ground-water supply contingency planning for local and state governments. Technical Assistance Document EPA 440/6-90-003, Office of Water, Washington, DC, 1990.

[652] U.S. Environmental Protection Agency. Superfund chemical data matrix (scdm), 1996. Downloaded from USEPA web site www.epa.gov, cited in Johnson (1999).

[653] U.S. Environmental Protection Agency. Oxygenates in Water: Critical Information and Research Needs. Technical Report EPA/600/R-98/048, USEPA, Washington, DC, 1998. Office of Research and Development.

[654] U.S. Geological Survey. A modular three-dimensional finite-difference ground-water flow model: MODFLOW. Technical report, USGS, Reston, Virginia, 2000.

[655] U.S. National Weather Service. Probable Maximum Precipitation Estimates–United States East of the 105th Meridian. Report 51, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Washington, DC, 1978. (reprinted 1986).

[656] U.S. National Weather Service. Application of Probable Maximum Precipitation Estimates–United States East of the 105th Meridian. Report 52, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Washington, DC, 1982.

[657] U.S. Weather Bureau. Two-to-ten-day precipitation for a return period of 2 to 100 years in the contiguous United States. Technical Paper 49, U.S. Department of Commerce, Washington, DC, 1964.

[658] U.S.S.R. National Committee for the International Hydrological Decade. World water balance and water resources of the earth. In *Studies and Reports in Hydrology*, volume 25. UNESCO, Paris, 1978. English translation.

[659] C.H.M. van Bavel. Potential evaporation: The combination concept and its experimental verification. *Water Resources Research*, 2(3):455–467, 1966.

[660] G. Van der Kamp. Determining aquifer transmissivity by means of well response tests: The underdamped case. *Water Resources Research*, 12(1):71–77, 1976.

[661] J.A. Van Mullem. Runoff and peak discharges using Green-Ampt infiltration model. *Journal of Hydraulic Engineering*, 117(3):354–370, March 1991.

[662] V.A. Vanoni. Velocity Distribution in Open Channels. *Civil Engineering*, 11:356–357, 1941.

[663] J. Vaze and H.S. Chiew. Comparative evaluation of urban storm water quality models. *Water Resources Research*, 39(10), 1280, doi:10.1029/2002WR001788, 2003.

[664] J.P. Velon and T.J. Johnson. Water Distribution and Treatment. In V.J. Zipparro and H. Hasen, editors, *Davis' Handbook of Applied Hydraulics*, pages 27.1–27.50. McGraw-Hill, Inc., New York, New York, fourth edition, 1993.

[665] P. Venkataraman and P.R.M. Rao. Darcian, transitional, and turbulent flow through porous media. *Journal of Hydraulic Engineering*, 124(8):840–846, August 1998.

[666] W. Viessman, Jr. and M.J. Hammer. *Water Supply and Pollution Control*. Prentice-Hall, Upper Saddle River, New Jersey, seventh edition, 2005.

[667] W. Viessman, Jr., J. Knapp, and G.L. Lewis. *Introduction to Hydrology*. Harper & Row, Publishers, Inc., New York, New York, second edition, 1977.

[668] W. Viessman, Jr. and G.L. Lewis. *Introduction to Hydrology*. HarperCollins College Publishers, New York, New York, fourth edition, 1996.

[669] W. Viessman, Jr. and G.L. Lewis. *Introduction to Hydrology*. Prentice-Hall, Upper Saddle River, New Jersey, fifth edition, 2003.

[670] W. Viessman, Jr. and C. Welty. *Water Management Technology and Institutions*. Harper & Row, Publishers, Inc., New York, New York, 1985.

[671] J.R. Villemonte. Submerged weir discharge studies. *Engineering News Record*, Dec 25 1947.

[672] F.N. Visher and J.F. Mink. Ground-Water Resources in Southern Oahu, Hawaii. Water-Supply Paper 1778, United States Geological Survey, 1964.

[673] G.K. Voigt. Distribution of rainfall under forest stands. *Forest Science*, 6:2–10, 1960.

[674] F. Von Hofe and O.J. Helweg. Modeling well hydrodynamics. *Journal of Hydraulic Engineering*, 124(12):197–203, July/August 1998.

[675] J.R. Wallis, N.C. Matalas, and J.R. Slack. Just a Moment. *Water Resources Research*, 10(2):211–219, 1974.

[676] T.M. Walski. Water Distribution. In L.W. Mays, editor, *Water Resources Handbook*, pages 18.1–18.45. McGraw-Hill, Inc., New York, New York, 1996.

[677] T.M. Walski. Hydraulic design of water distribution storage tanks. In L.W. Mays, editor, *Water Distribution Systems Handbook*, pages 10.1–10.20. McGraw-Hill Book Company, New York, New York, 2000.

[678] T.M. Walski, J. Gessler, and J.A.W. Sjostrom. *Water Distribution Systems: Simulation and Sizing*. Lewis Publishers, Inc., Chelsea, Michigan, 1990.

[679] I.A. Walter, R.G. Allen, R. Elliott, M.E. Jensen, D. Itenfisu, B. Mecham, T.A. Howell, R. Snyder, P. Brown, S. Eching, T. Spofford, M. Hattendorf, R.H. Cuenca, J.L. Wright, and D. Martin. ASCE's standardized reference evapotranspiration equation. In *Proceedings, Fourth National Irrigation Symposium*, pages 209–215, St. Joseph, Michigan, 2000. American Society of Agricultural Engineers.

[680] M.T. Walter, V.K. Metha, A.M. Marrone, J. Boll, P. Gérard-Marchant, T.S. Steenhuis, and M.F. Walter. Simple Estimation of Prevalence of Hortonian Flow in New York City Watersheds. *Journal of Hydrologic Engineering*, 8(4):214–218, July/August 2003.

[681] W.C. Walton. Selected analytical methods for well and aquifer evaluation. Bulletin 49, Illinois State Water Survey, Urbana, Illinois, 1962.

[682] W.C. Walton. *Principles of Groundwater Engineering*. CRC Press, Inc., Boca Raton, Florida, 1991.

[683] C. Wandmacher and A.I. Johnson. *Metric Units in Engineering, Going SI*. ASCE Press, New York, New York, 1995.

[684] N.H.C. Wang and R.J. Houghtalen. *Fundamentals of Hydraulic Engineering Systems*. Prentice-Hall, Upper Saddle River, New Jersey, third edition, 1996.

[685] M. Wanielista, R. Kersten, and R. Eaglin. *Hydrology: Water Quantity and Quality Control*. John Wiley & Sons, New York, New York, second edition, 1997.

[686] M.P. Wanielista. *Stormwater Management: Quantity and Quality*. Ann Arbor Science, Ann Arbor, Michigan, 1978.

[687] M.P. Wanielista and Y.A. Yousef. *Stormwater Management*. John Wiley & Sons, New York, New York, 1993.

[688] A.D. Ward and J. Dorsey. Infiltration and soil water procsses. In A.D. Ward and W.J. Elliot, editors, *Environmental Hydrology*, pages 51–90. Lewis Publishers, Inc., Boca Raton, Florida, 1995.

[689] A.D. Ward and S.W. Trimble. *Environmental Hydrology*. CRC Press, Inc., Boca Raton, Florida, second edition, 2004.

[690] R.C. Ward and M. Robinson. *Principles of Hydrology*. McGraw-Hill, Inc., New York, New York, fourth edition, 1970.

[691] D.L. Warner and J.H. Lehr. *Subsurface Wastewater Injection*. Premier Press, Berkeley, California, 1981.

[692] M.J. Waterloo, L.A. Bruijnzeel, H.F. Vugts, and T.T. Rawaqa. Evaporation from *Pinus caribaea* plantations on former grassland soils under maritime tropical conditions. *Water Resources Research*, 35(7):2133–2144, July 1999.

[693] T.C. Wei and J.L. McGuinness. Reciprocal distance squared method, a computer technique for estimating area precipitation. Technical Report ARS-NC-8, U.S. Agricultural Research Service, North Central Region, Coshocton, Ohio, 1973.

[694] W. Weibull. A statistical theory of the strength of materials. *Ing. Vetenskapsakad. Handl. (Stockh.)*, 151:15, 1939.

[695] H.G. Wenzel. The effect of raindrop impact and surface roughness on sheet flow. WRC Research Report 34, Water Resources Centre, University of Illinois, Urbana, Illinois, 1970.

[696] H.G. Wenzel, Jr. Rainfall for urban design. In D.F. Kibler, editor, *Urban Stormwater Hydrology*, pages 35–67. American Geophysical Union, Water Resources Monograph Series, Washington, DC, 1982.

[697] J.A. Westphal. Hydrology for Drainage System Design and Analysis. In L.W. Mays, editor, *Stormwater Collection Systems Design Handbook*, pages 4.1–4.44. McGraw-Hill, Inc., New York, New York, 2001.

[698] P.J. Whalen and M.G. Cullum. An assessment of urban land use/stormwater runoff quality relationships and treatment efficiencies of selected stormwater management systems. Technical Publication 88-9, South Florida Water Management District, West Palm Beach, Florida, July 1988.

[699] W. Whipple and C.W. Randall. *Detention and Flow Retardation Devices*. Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1983. Stormwater Management in Urbanizing Areas.

[700] F.M. White. *Fluid Mechanics*. McGraw-Hill, Inc., New York, New York, third edition, 1994.

[701] E.R. Wilcox. A comparative test of the flow of water in 8-inch concrete and vitrified clay sewer pipe. Experiment Station Series Bulletin 27, University of Washington, 1924.

[702] J.O. Wilkes. *Fluid Mechanics for Chemical Engineers*. Prentice-Hall, Upper Saddle River, New Jersey, 1999.

[703] Willeke. Discussion of 'Runoff curve number: Has it reached maturity?' by V.M. Ponce and R.H. Hawkins. *Journal of Hydrologic Engineering*, 2(3):147, 1997.

[704] G.S. Williams and A.H. Hazen. *Hydraulic Tables*. John Wiley & Sons, New York, New York, 1920.

[705] J. Williamson. The laws of motion in rough pipes. *La Houille Blanche*, 6(5):738, September-October 1951.

[706] M.D. Winsberg. *Florida Weather*. University of Central Florida Press, Orlando, Florida, 1990.

[707] E. Wohl. *Mountain Rivers*. American Geophysical Union, Washington, DC, 2000.

[708] T.S.W. Wong and C.-N. Chen. Time of concentration formula for sheet flow of varying flow regime. *Journal of Hydrologic Engineering*, 2(3):136–139, July 1997.

[709] E.F. Wood. Global Scale Hydrology: Advances in Land Surface Modeling, U.S. National Report to International Union of Geodesy and Geophysics 1987-1990. Technical report, American Geophysical Union, Washington, DC, 1991.

[710] S.L. Woodall. Rainfall interception losses from melaleuca forest in Florida. Research Note SE-323, United States Department of Agriculture, Southeastern Forest Experiment Station, Forest Resources Laboratory, Lehigh Acres, FL 33936, February 1984.

[711] D.A. Woolhiser, R.E. Smith, and J.-V. Giraldez. Effects of spatial variability of saturated hydraulic conductivity on Hortonian overland flow. *Water Resources Research*, 32(3):671–678, March 1996.

[712] World Meteorological Organization. Guide to Hydrological Practices, Volume II, Analysis, forecasting and other applications. Technical Report No.168, 4th ed., WMO, Geneva, Switzerland, 1983.

[713] World Meteorological Organization. Manual for estimation of probable maximum precipitation. Oper. Hydrol. No.1, 2nd ed., Publ.332, WMO, Geneva, Switzerland, 1986.

[714] J.L. Wright. Crop coefficients for estimates of daily crop evaporanspiration. In *Irrigation Scheduling for Water and Energy Conservation in the 80s*. ASAE, 1981.

[715] J.L. Wright. New evapotranspiration crop coefficients. *Journal of Irrigation and Drainage Engineering, ASCE*, 108(IR2):57–74, 1982.

[716] J.L. Wright, R.G. Allen, and T.A. Howell. Conversion between evapotranspiration references and methods. In *Proceedings, Fourth National Irrigation Symposium*, pages 251–259, St. Joseph, Michigan, 2000. American Society of Agricultural Engineers.

[717] J.L. Wright and M.E. Jensen. Peak water requirements of crops in southern Idaho. *Journal of Irrigation and Drainage Engineering, ASCE*, 96(IR1):193–201, 1972.

[718] S. Wu and N. Rajaratnam. Transition from hydraulic jump to open channel flow. *Journal of Hydraulic Engineering*, 122(9):526–528, September 1996.

[719] R.A. Wurbs and W.P. James. *Water Resources Engineering*. Prentice-Hall, Upper Saddle River, New Jersey, 2002.

[720] C.T. Yang. *Sediment Transport: Theory and Practice*. McGraw-Hill, Inc., New York, New York, 1996.

[721] D.L. Yarnell and S.M. Woodward. The flow of water in drain tile. Bulletin 854, Department of Agriculture, Washington, DC, 1924.

[722] M.V. Yates. Septic tank siting to minimize the contamination of ground water by microorganisms. Technical Report EPA 440/6-87-007, U.S. Environmental Protection Agency, Office of Ground-Water Protection, Washington, DC, 1987.

[723] B.C. Yen. Hydraulic resistance in open channels. In B.C. Yen, editor, *Channel flow resistance: Centennial of Manning's formula*, pages 1–135. Water Resource Publications, Highlands Ranch, Colorado, 1991.

[724] B.C. Yen. Open channel flow resistance. *Journal of Hydraulic Engineering*, 128(1):20–39, January 2002.

[725] B.C. Yen and A.O. Akan. Hydraulic design of urban drainage systems. In L.W. Mays, editor, *Hydraulic Design Handbook*, pages 14.1–14.114. McGraw-Hill Book Company, New York, New York, 1999.

[726] B.C. Yen and V.T. Chow. Design hyetographs for small drainage structures. *Journal of Hydraulic Engineering*, 106(HY6), 1980.

[727] V. Yevjevich. *Probability and Statistics in Hydrology.* Water Resources Publications, Littleton, Colorado, 1972.

[728] G. K. Young, Jr. and S.M. Stein. Hydraulic design of drainage for highways. In L.W. Mays, editor, *Hydraulic Design Handbook*, pages 13.1–13.44. McGraw-Hill Book Company, New York, New York, 1999.

[729] Y.A. Yousef and M.P. Wanielista. Efficiency optimization of wet-detention ponds for urban stormwater management. Florida Department of Environmental Regulation, Tallahassee, Florida, 1989.

[730] S.L. Yu, R.J. Kaighn Jr., and S. Liao. Testing of best management practices for controlling highway runoff. Phase II final report. Technical Report FHWA/VA-94-R21, Virginia Transportation Research Council, Richmond, VA, 1994.

[731] S.L. Yu, J.-T. Kuo, E.A. Fassman, and H. Pan. Field test of grassed-swale performance in removing runoff pollution. *Journal of Water Resources Planning and Management*, 127(3):168–171, May/June 2001.

[732] L. Zhang, K. Hickel, W.R. Dawes, F.H.S. Chiew, A.W. Western, and P.R. Briggs. A rational function approach for estimating mean annual evapotranspiration. *Water Resources Research*, 40, W02502, doi:10.1029/2003WR002710, 2004.

[733] V.J. Zipparro and H. Hasen. *Davis' Handbook of Applied Hydraulics.* McGraw-Hill, Inc., New York, New York, fourth edition, 1993.