



Chapter 3

Hydraulics

This chapter provides formulas and guidelines to aid in the hydraulic design of concrete pressure pipe. The three formulas typically used to determine pipeline capacity are presented and compared. Factors that contribute to the decrease with age of pipeline carrying capacity are also described. The methods used to determine minor and total head losses are presented, as well as the methodology for determining an economical pipe size. Throughout this chapter, discussions will be limited to combinations of pipe sizes and flows that provide a range of velocities and diameters most commonly used with concrete pressure pipe.

FLOW FORMULAS

The formulas commonly used to determine the capacity of pipelines are the Hazen–Williams formula, the Darcy–Weisbach formula, and the Manning formula. Many other flow formulas may be found in technical literature, but these three are most frequently used. It is impossible to say that any one of these formulas is superior to the others for all pipe under all circumstances, and the reader is cautioned not to expect identical answers to a problem from all three formulas. Judgment must be used when selecting a flow formula and the roughness coefficient for a particular hydraulics problem.

The Hazen–Williams Formula

The empirical Hazen–Williams formula is probably the most commonly used flow formula in the waterworks industry. The basic form of the equation is

$$V = 1.318 C_h R^{0.63} S^{0.54} \quad (\text{Eq 3-1})$$

Where:

- V = velocity, ft/sec
- C_h = Hazen–Williams roughness coefficient
- R = hydraulic radius, ft, which is the cross-sectional area of the pipe divided by the wetted perimeter, i.e., for circular pipe flowing full, the internal diameter in feet divided by 4
- S = slope of the hydraulic grade line, ft/ft calculated as h_L/L , where h_L equals head loss in feet occurring in a pipe of length L in ft

For circular conduits flowing full, the equation becomes

$$V = 0.550C_h d^{0.63} (h_L/L)^{0.54} \quad (\text{Eq 3-2})$$

Where:

- V = velocity, ft/sec
- C_h = Hazen–Williams roughness coefficient
- d = inside diameter of the pipe, ft
- h_L = head loss, ft
- L = pipe length, ft

Table 3-1 lists the various forms of the Hazen–Williams formula. Figures 3-1 and 3-2 show the hydraulic grade line, head losses, and static grade line for gravity and pumped flow systems.

A detailed investigation of the available flow test data for concrete pipe was performed by Swanson and Reed (1963), who concluded that the Hazen–Williams formula most closely matches the test results for the range of velocities normally encountered in water transmission. A statistical analysis of Swanson and Reed's test data led to the development of the following equation for determining C_h :

$$C_h = 139.3 + 2.028d \quad (\text{Eq 3-3})$$

Where:

- C_h = Hazen–Williams roughness coefficient
- d = inside diameter of pipe, ft

Table 3-2 compares theoretical Hazen–Williams C_h values using Equation 3-3 to tested values for 67 tests reported by Swanson and Reed.

Equation 3-3 can be used to calculate a C_h value for any size pipe; however, for design purposes the following conservative values are suggested:

Diameter, <i>in.</i>	C_h Value
16 to 48	140
54 to 108	145
114 and larger	150

Table 3-1 Various forms of the Hazen–Williams formula

In Terms of:	Velocity, <i>fps</i>	Flow Rate, <i>cfs</i>
General equation	$V = 0.550C_h d^{0.63} (h_L/L)^{0.54}$	$Q = 0.432C_h d^{2.63} (h_L/L)^{0.54}$
Head loss, <i>ft</i>	$h_L = 3.021(L/d^{1.167})(V/C_h)^{1.852}$	$h_L = 4.726(L/d^{4.870})(Q/C_h)^{1.852}$
Pipe diameter, <i>ft</i>	$d = 2.580(V/C_h)^{1.587}(L/h_L)^{0.857}$	$d = 1.376(Q/C_h)^{0.380}(L/h_L)^{0.205}$
Pipe length, <i>ft</i>	$L = 0.331h_L d^{1.167}(C_h/V)^{1.852}$	$L = 0.212h_L d^{4.870}(C_h/Q)^{1.852}$
Roughness coefficient	$C_h = 1.817(V/d^{0.63})(L/h_L)^{0.54}$	$C_h = 2.313(Q/d^{2.63})(L/h_L)^{0.54}$

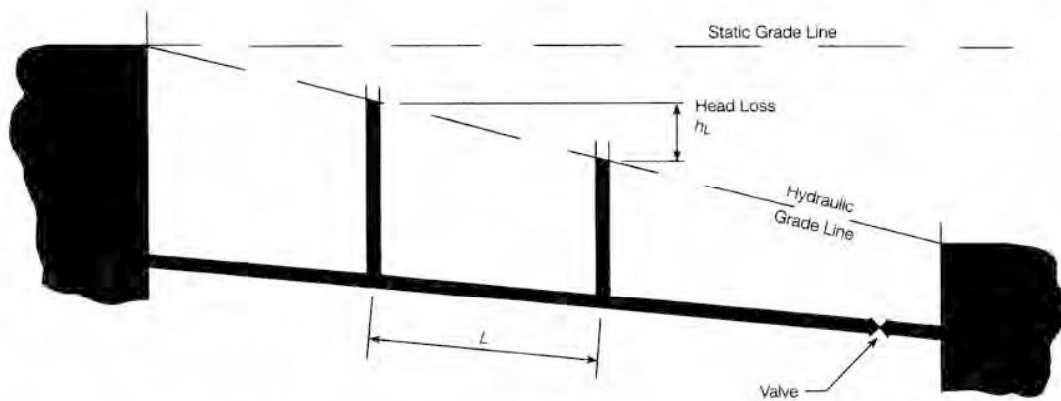


Figure 3-1 Hydraulic profile for a gravity flow system

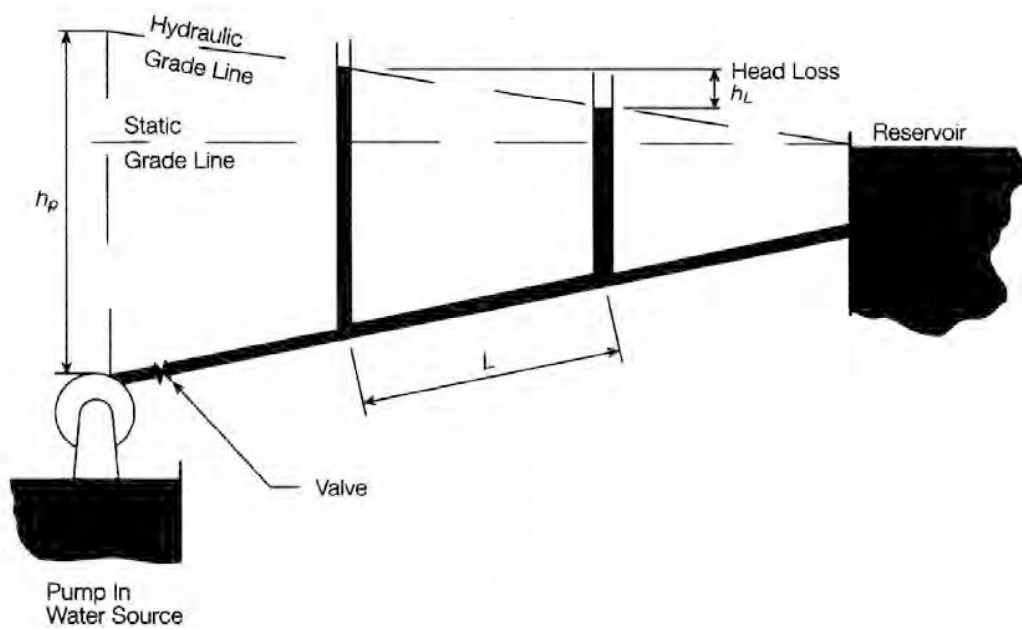


Figure 3-2 Hydraulic profile for a pumped flow system

Table 3-2 Comparison of theoretical Hazen–Williams C_h values to tested C_h values*

Sample Number	Diameter <i>in.</i>	C_h Measured	C_h Theoretical	Percent of Theoretical
1	24.0	145.0	143.4	101.1
2	30.0	147.0	144.4	101.8
3	30.0	145.0	144.4	100.4
4	31.5	147.0	144.6	101.6
5	31.8	152.0	144.7	105.1
6	36.0	143.0	145.4	98.4
7	36.0	150.0	145.4	103.2
8	36.0	141.5	145.4	97.3
9	36.0	142.5	145.4	98.0
10	36.0	150.0	145.4	103.2
11	39.0	147.0	145.9	100.8
12	42.0	147.5	146.4	100.8
13	42.0	142.0	146.4	97.0
14	42.0	149.0	146.4	101.8
15	42.0	149.0	146.4	101.8
16	42.0	142.0	146.4	97.0
17	42.0	147.0	146.4	100.4
18	46.0	144.0	147.1	97.9
19	48.0	146.0	147.4	99.0
20	48.0	150.5	147.4	102.1
21	48.0	144.0	147.4	97.7
22	51.0	142.0	147.9	96.0
23	54.0	152.0	148.4	102.4
24	54.0	141.0	148.4	95.0
25	54.0	140.5	148.4	94.7
26	60.0	145.5	149.4	97.4
27	60.0	152.5	149.4	102.0
28	46.0	148.5	147.1	101.0
29	54.0	150.0	148.4	101.1
30	36.0	138.0	145.4	94.9
31	36.0	137.0	145.4	94.2
32	36.0	142.0	145.4	97.7
33	48.0	149.0	147.4	101.1
34	54.0	151.5	148.4	102.1
35	54.0	145.0	148.4	97.7
36	54.0	146.0	148.4	98.4
37	54.0	147.5	148.4	99.4
38	60.0	147.0	149.4	98.4
39	60.0	154.0	149.4	103.1
40	60.0	156.0	149.4	104.4
41	60.0	143.5	149.4	96.0
42	66.0	142.0	150.5	94.4
43	48.0	156.5	147.4	106.2
44	48.0	152.0	147.4	103.1
45	48.0	149.5	147.4	101.4
46	48.0	153.5	147.4	104.1
47	54.0	154.0	148.4	103.8
48	54.0	155.5	148.4	104.8
49	54.0	151.0	148.4	101.7
50	54.0	152.0	148.4	102.4

Table continued next page.

Table 3-2 Comparison of theoretical Hazen–Williams C_h values to tested C_h values* (continued)

Sample Number	Diameter <i>in.</i>	C_h Measured	C_h Theoretical	Percent of Theoretical
51	54.0	143.0	148.4	96.3
52	72.0	154.0	151.5	101.7
53	12.0	145.0	141.3	102.6
54	18.0	147.0	142.3	103.3
55	20.0	134.0	142.7	93.9
56	24.0	143.5	143.4	100.1
57	30.0	141.0	144.4	97.7
58	30.0	143.0	144.4	99.1
59	33.0	147.0	144.9	101.5
60	33.0	146.0	144.9	100.8
61	33.0	146.0	144.9	100.8
62	36.0	134.0	145.4	92.2
63	31.4	138.0	144.6	95.4
64	36.0	147.0	145.4	101.1
65	36.0	150.0	145.4	103.2
66	41.9	150.0	146.4	102.5
67	42.0	150.0	146.4	102.5

* The theoretical values were calculated using Eq 3-3 and the test values are from Swanson and Reed (1963).

These values are applicable to concrete pipelines in which the fitting losses are a minor part of the total loss and to pipelines free from deposits or organic growths that can materially affect the pipe's carrying capacity.

The Darcy–Weisbach Formula

An alternative to the Hazen–Williams formula is the Darcy–Weisbach formula, which is

$$h_L = \frac{fLV^2}{d2g} \quad (\text{Eq 3-4})$$

Where:

- h_L = head loss, in feet of water
- f = Darcy friction factor
- L = pipeline length, ft
- V = velocity, ft/sec
- d = inside diameter of the pipe, ft
- g = gravitational constant, 32.2 ft/sec²

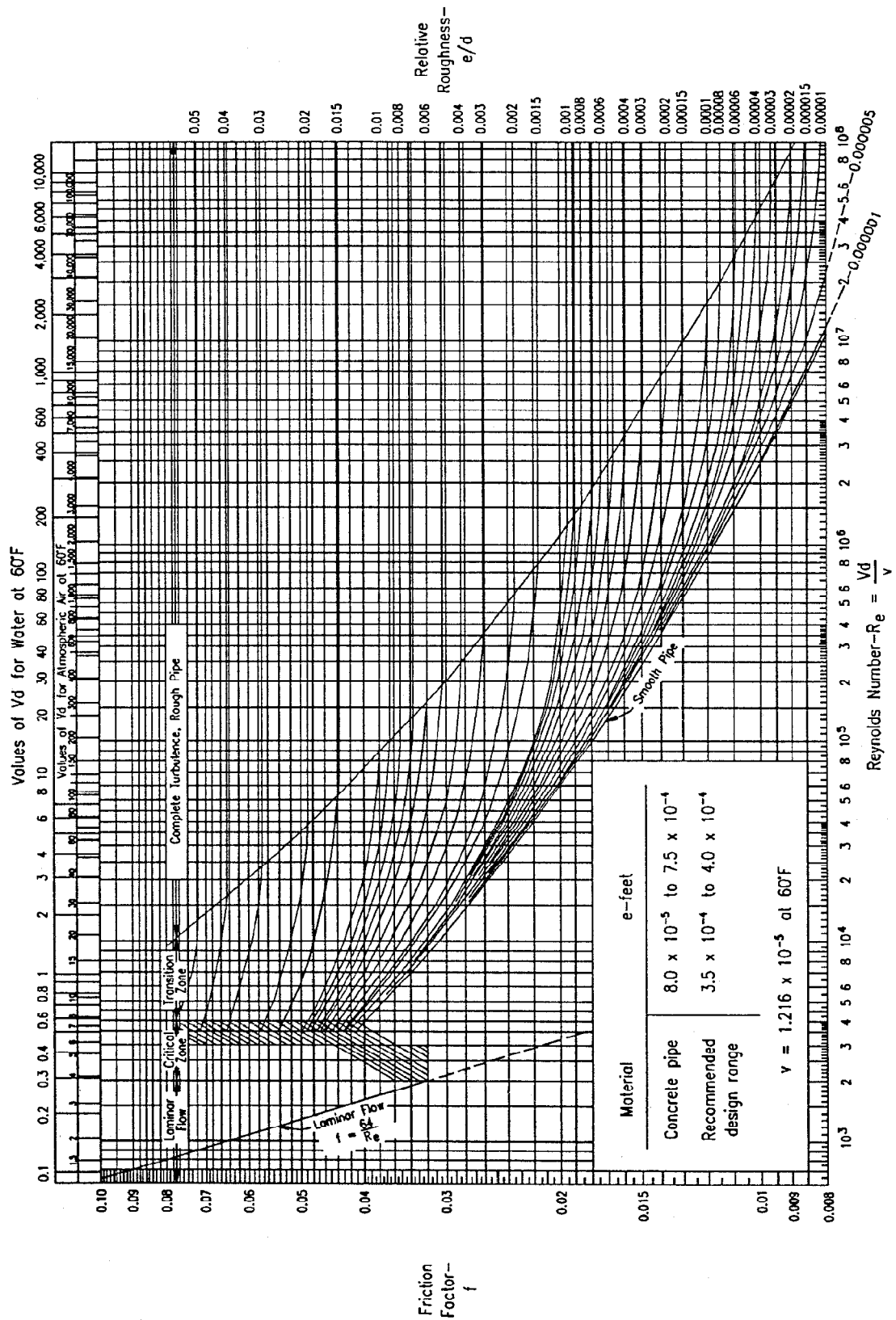


Figure 3-3 The Moody diagram for friction in pipe

The Darcy friction factor (f) can be determined from the Moody diagram, presented in Figure 3-3. To use this diagram, the Reynolds number (R_e) and the relative roughness (e/d) must first be calculated. Both terms are defined in the paragraphs that follow.

The Reynolds number is a function of the flow in the pipe and may be calculated as

$$R_e = \frac{dV}{\nu} \quad (\text{Eq 3-5})$$

Where:

- R_e = Reynolds number
- d = inside diameter of the pipe, ft
- V = velocity, ft/sec
- ν = kinematic viscosity of the fluid, ft²/sec

The kinematic viscosity of water at various temperatures from freezing to boiling is presented in Table 3-3.

The relative roughness (e/d) of a pipe is a function of the absolute roughness (e) of the interior surface of the pipe and the pipe diameter (d). Values of the absolute roughness (e) for concrete pipe range from 8.0×10^{-5} to 7.5×10^{-4} , and the recommended design range is 3.5×10^{-4} to 4.0×10^{-4} . Once the Reynolds number and the relative roughness are determined, the Moody diagram (Figure 3-3) can be used to determine values of the Darcy friction factor (f), which may then be used to solve the Darcy–Weisbach formula.

If an analytical solution for the Darcy friction factor (f) is preferred, it may be obtained by iteration from the Colebrook–White equation:

$$\frac{1}{\sqrt{f}} = -2 \log_{10} \left[\frac{e}{3.7d} + \frac{2.51}{R_e \sqrt{f}} \right] \quad (\text{Eq 3-6})$$

Where:

- f = Darcy friction factor
- e = absolute roughness, ft
- d = inside diameter of the pipe, ft
- R_e = Reynolds number

Table 3-3 Kinematic viscosity of water

Temperature, °F	Kinematic Viscosity (ν), ft ² /s
32	1.931 E-05
40	1.664 E-05
50	1.410 E-05
60	1.216 E-05
70	1.059 E-05
80	09.30 E-06
90	08.23 E-06
100	07.36 E-06
120	06.10 E-06
150	04.76 E-06
180	03.85 E-06
212	03.19 E-06

The Manning Formula

The Manning formula is more commonly used to determine the flow in partially filled gravity lines; however, it can be used for fully developed flow in conduits. The basic form of the equation is

$$V = 1.486R^{0.67}S^{0.5}/n_M \quad (\text{Eq 3-7})$$

Where:

- V = velocity, ft/sec
- R = hydraulic radius, in feet, which is the cross-sectional area of the pipe divided by the wetted perimeter (e.g., for circular pipe flowing full, the internal diameter divided by 4)
- S = slope of the hydraulic grade line, ft/ft
- n_M = Manning roughness coefficient

For pressure flow in round conduit, this equation becomes

$$V = (0.590d^{0.67}/n_M)(h_L/L)^{0.5} \quad (\text{Eq 3-8})$$

Where:

- V = velocity, ft/sec
- n_M = Manning roughness coefficient
- d = inside diameter of the pipe, ft
- h_L = head loss, in feet, occurring in a pipe of length L , ft

For concrete pressure pipe, the value for the Manning roughness coefficient n_M should be approximately 0.011 when the velocity is 3 ft/s and 0.010 when the velocity is 5 ft/s.

EFFECTS OF AGING ON CARRYING CAPACITY

The carrying capacity of concrete pressure pipe is usually not affected by age. However, some aggressive waters can roughen the lining surface, and there are several organic growths and some types of deposits that reduce the carrying capacity of pipe of any material, including concrete pressure pipe. These conditions are often associated with raw water transmission mains and can generally be prevented by chemical pretreatment. If left unchecked, however, excessive growths or deposits can develop. When this occurs, chemical treatment may not be adequate to remove the deposits if they have achieved a substantial buildup. In these cases, the growths or deposits can be removed by scraping or pigging the line. Industry has not witnessed deterioration of the mortar or concrete lining as a result of growths or deposits.

HEAD LOSSES

The total head loss in a pipeline is the sum of minor losses due to changes in flow geometry added to the head loss created due to the friction caused by flow through the pipe. The head loss caused by pipe friction may be determined from the formulas already presented. The calculation procedure for determining minor losses is presented in the following section. A method to easily include minor losses with frictional losses to simplify pipe selection calculations is also presented.

Minor Losses

Minor losses in pipelines are caused by turbulence resulting from changes in flow geometry. Minor losses, which are generally expressed as a function of the velocity head, will occur at entrances, outlets, contractions, enlargements, bends, and other fittings. In long pipelines, the minor losses are usually small compared to losses from pipe friction and may be neglected; however, in shorter lines or plant piping, these minor losses may become significant. The formula for calculating minor losses is

$$h_L = C_L V^2 / (2g) \quad (\text{Eq 3-9})$$

Where:

- h_L = head loss, ft
- C_L = a dimensionless coefficient
- V = velocity, ft/sec
- g = gravitational constant, 32.2 ft/sec²

Figure 3-4 presents values of C_L for common flow configurations.

Equivalent Length Method

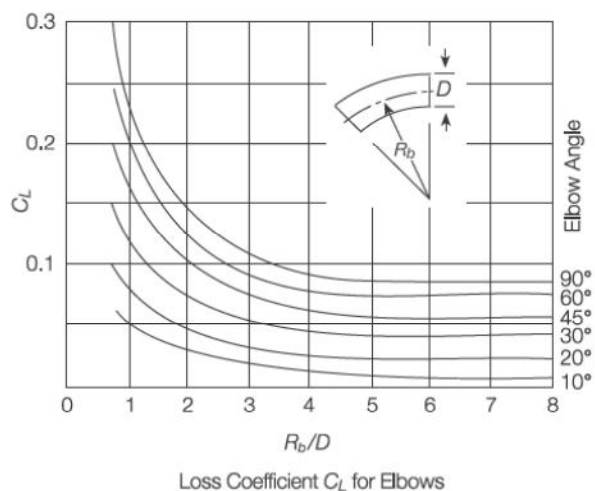
Each minor head loss in a piping system can be expressed in terms of an equivalent length of pipe L_e . This equivalent length of pipe is the number of feet of straight pipe that would have friction head loss equal to the minor losses created by the fitting. Table 3-4 lists the equivalent length formulas that correspond to the flow formulas presented earlier.

DETERMINING AN ECONOMICAL PIPE DIAMETER

To determine the most economical pipe diameter, the first cost (purchase cost plus installation cost) must be compared to the pumping cost incurred during the design life of the pipe. While a greater diameter pipe will result in a greater first cost, the reduced head loss will lead to a lower pumping cost and provide capacity for subsequent increased flow. These costs should be determined for a number of pipe sizes and the results compared in order to select the pipe size that provides the lowest total cost during the life of the line.

Experience has shown the most cost-effective water pipe diameter will usually yield design flow velocities in the range of 3 to 6 ft/s (0.9 to 1.8 m/s). Therefore, it is generally best to restrict pipe size evaluations to diameters that will deliver the required volume of water at a velocity between 2 and 7 ft/s (0.6 and 2.1 m/s), although in some rare circumstances involving short pipeline length, it may be cost effective to use velocities approaching 10 ft/s (3.0 m/s). The inside diameter of pipe for various flow velocities can be determined from

$$d = (1.27Q/V)^{0.5} \quad (\text{Eq 3-10})$$



Flow Configurations		C_L
Elbow		See figure above
Reentrant Entrance		0.80
Square-Edged Entrance		0.50
Slightly Rounded Entrance		0.23
Well-Rounded Entrance		0.04
Flow Into Reservoir		1.00
Reducer With $\alpha \leq 15^\circ$		0.04
Enlarger: $\beta = 10^\circ$ $\beta = 20^\circ$ $\beta = 30^\circ$		0.20 0.40 0.70
Tee, Through Run		0.60
Tee, Through Side Outlet		1.80
Tee Into Side Outlet		1.50
45° Wye		1.30
Gate Valve, Fully Open		0.20
Swing Check Valve, Fully Open		2.50
Butterfly Valve, Fully Open		0.25

Figure 3-4 Approximate loss coefficients for commonly encountered flow configurations

Table 3-4 Equivalent length formulas

Friction Formula	Equivalent Length Formula
Hazen-Williams	$L_e = 0.331 C_L \frac{V^{0.148} d^{1.167} C_h^{1.852}}{2g}$
Darcy-Weisbach	$L_e = C_L (d/f)$
Manning	$L_e = 0.348 C_L \frac{d^{1.333}}{2g(n_M)^2}$

NOTE: Definition of variables:

- L_e = length of pipe that would have a frictional head loss equal to the minor loss created by fitting, ft
 C_L = loss coefficient
 V = velocity, ft/sec
 d = inside diameter of the pipe, ft
 C_h = Hazen-Williams roughness coefficient
 g = gravitational constant, 32.2 ft/sec²
 f = Darcy friction factor
 n_M = Manning roughness coefficient

Where:

- d = inside diameter of the pipe, ft
 Q = flow rate, ft³/sec
 V = flow velocity, ft/sec

For each pipeline diameter to be evaluated, minor losses should be converted to equivalent lengths of pipe as outlined in the preceding section, and the friction head should be calculated from one of the flow equations presented earlier. As an example, the Hazen-Williams formula, in the following form, can be used to calculate total head loss for the sum of actual pipe lengths and equivalent lengths:

$$h_f = (4.726 L' / d^{4.870}) (Q / C_h)^{1.852} \quad (\text{Eq 3-11})$$

Where:

- h_f = total head loss, ft
 d = pipe diameter, ft
 L' = the actual length of the pipeline (L) plus the sum of the calculated equivalent lengths (L_e), ft
 Q = flow rate, ft³/sec
 C_h = Hazen-Williams roughness coefficient

The power required to overcome the total head loss may be determined from

$$P_p = (Qh_f\gamma)/550 \quad (\text{Eq 3-12})$$

Where:

- P_p = pump power requirements, hp
- Q = flow rate, ft³/sec
- h_f = total head loss, ft
- γ = unit weight of the fluid, lb/ft³; for water at 50°F, $\gamma = 62.41$ lb/ft³

The preceding equations for total head loss and power requirements may be combined to yield the following equation for the annual pumping power cost relative to frictional head:

$$\text{Cost/year} = \frac{(0.745 \times P_p \times \text{power cost} \times \text{hours/year})}{(\text{pump eff.} \times \text{motor eff.})} \quad (\text{Eq 3-13})$$

Where:

- Cost/year = pumping power cost per year, \$
- P_p = pump power requirements, hp
- power cost = power cost, \$/kW·h
- hours/year = number of hours per year of pump operation
- pump eff. = the pump efficiency, expressed as a decimal
- motor eff. = the motor efficiency, expressed as a decimal

It should be noted that the cost calculated using the preceding formulas is the cost of power required to overcome total head loss and does not include the cost of pumping required to overcome changes in elevation. Since the elevation change will be the same for all pipe diameters being evaluated, it can be ignored in the comparison of pipe diameters. If the designer desires to graphically present the hydraulic grade line for one or more of the pipe diameters to calculate total head or to calculate the total annual power cost, then the static head should be combined with the frictional head, and the sum of the two should be used in the preceding equations.

When selecting pipe diameters based on the preceding formulas, it is important that the designer consider both average annual pumping conditions and peak pumping conditions. The annual power cost equation, if used in conjunction with the peak pumping rate, will lead the designer to select a diameter that is larger than needed. In addition, the designer must consider that power factors or demand charges affect the rates paid for power and that power costs will increase with time. Therefore, it may be prudent to select an escalation rate so that a realistic comparison between the first cost of the pipe and the long-term pumping costs may be equitably made.

AIR ENTRAPMENT AND RELEASE

Air entrapment in poorly vented pipelines can reduce pipeline carrying capacity, cause unexpected pressure surges, and produce objectionable “white water.” The following three conditions must exist before air binding presents a serious problem:

- There must be a source of air in excess of that normally held in solution in the flow.
- The air must separate from the water and collect at high points or descending legs.
- The collected air must remain near the high points or in the descending legs.

It is desirable to vent high points and/or descending legs on transmission mains with automatic air-release valves to relieve this condition. Improperly sized air-release valves can contribute to surge pressure conditions as described in chapter 4.

BLOWOFF OUTLETS

Blowoffs may be located at low points in the profile of a pipeline. They are mainly used to drain lines for maintenance or inspection. Occasionally, blowoffs are used to remove silt or sediment from a line. If there is the possibility of backsiphoning contaminated water into water transmission mains, the outlet for blowoff discharge piping should be elevated. Blowoffs should generally be sized so the main pipeline velocity change is limited to 1 ft/s (0.3 m/s) or less during blowoff operation.

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