United States Department of Agriculture

Natural Resources Conservation Service

Part 634 Hydraulics
National Engineering Handbook

Chapter 4 Pipe Flow

(210-634-H, 1st Ed., Feb 2022)

Issued: Feb 2022_

Table of Contents

634.0400 General 1

634.0401 Friction Loss Methods 2

634.0402 Manning's Equation for Pipelines 2

634.0403 Darcy-Weisbach Equation and Friction Factor 6

634.0404 Pipe Flow Solutions Using Hazen-Williams Formula 10

634.0405 Local Losses in Pipelines 14

634.0406 Pumps in Pipelines 20

634.0407 Pipelines and Networks 25

634.0407 EPANET 33

634.0408 WaterCAD 37

634.0409 Appurtenances in Pipelines and Networks 38

634.0410 Hydraulic Transients (Water Hammer) 51

634.0411 Cavitation 54

634.0412 Culverts 57

634.0413 Sprinkler Irrigation 68

634.0414 Microirrigation 69

634.0415 References 71

Table of Figures

Figure 4-1: Energy Heads in Pipe Flow 1

Figure 4-2: Pipe Flow calculations with the USDA-NRCS Hydraulics Formula 4

Figure 4-3: Values of Manning's resistance coefficient for pipe 5

Figure 4-4: Determining Velocity in a Pipeline Draining a Reservoir 6

Figure 4-5: Absolute roughness values for pipe materials 7

Figure 4-6: Iterative Solution for Q 9

Figure 4-7: Determining Flow in a Pipeline Draining a Reservoir using the Darcy-Weisbach Equation 10

Figure 4-8: Values of the Hazen-Williams coefficient. 12

Figure 4-9: Determining Discharge and Velocity in a Pipeline Draining a Reservoir using the Hazen-Williams Formula 13

Figure 4-10. Discharge from a pipe into a reservoir. 14

Figure 4-11. Entrance loss coefficients for typical pipe entrance shapes....................................15

Figure 4-12. Pipe entrance loss coefficients..16

Figure 4-13: Solution for pipe flow with entrance loss using USDA-NRCS Hydraulics Formula.16

Figure 4-14: Local loss coefficients for selected pipe fittings. 17

Figure 4-15: Schematic of pipe flow between two reservoirs..17

Figure 4-16: Solution for pipe flow including all local losses using USDA-NRCS Hydraulics Formula...20

Figure 4-17: Discharge versus Head from Pump Tests...21

Figure 4-18: Pump discharge-head graph 21

Figure 4-19: Manufacturer-provided Pump Curves and Pump Sizes..22

Figure 4-20: Pump-pipeline System for Analysis. 22

Figure 4-21: System Curve (Tabular) 23

Figure 4-22: Pump System Curve (Graphical)..23

Figure 4-23: System Schematic showing Motor *M*, the Electric Supply *E*, and Pump *P* 24

Figure 4-24: Three Pipelines in Series Connecting Two Reservoirs 25

Figure 4-25 Local Head Loss Coefficients for a Sudden Pipe Contraction 26

Figure 4-26: Effect of neglecting local losses on an example pipeline system 29

Figure 4-27: Three pipelines in parallel connecting two reservoirs. 30

Figure 4-28: Three pipelines converging to a single delivery point..31

Figure 4-29 Hydraulic example for EPANET 34

Figure 4-30: Pump Curve editor screen shot. Only one point is entered for this example and the curve is calculated from the program. 35

Figure 4-31: Screen shot of EPANET output...36

Figure 4-32: Output table of Junction information for EPANET example 36

Figure 4-33: Output table of links (Pipelines) for EPANET example. 36

Figure 4-34: Continuous acting air valve 39

Figure 4-35: Location and dimensions of air vent chamber near a water intake. 45

Figure 4-36: Spring operated pressure relief valve 46

Figure 4-37: Schematic of pump-pipeline system protected with a surge chamber....................49

Figure 4-38: Check valve installed on a pumping plant..51

Figure 4-39 Siphon conduit connecting two reservoirs. 54

Figure 4-40: Flow regimes for submerged inlet flow in culverts..57

Figure 4-41: Culvert discharge calculation 60

Figure 4-42: Culvert Discharge Convergence of Values 61

Figure 4-43: Culvert discharge calculation using USDA-NRCS Hydraulics Formula Tool 61

Figure 4-44: Nomogram for headwater depth for concrete pipe culverts with inlet control. 63

Figure 4-45: Nomogram for headwater depth for corrugated metal (CM) pipe culverts with inlet control. 65

Figure 4-46: Nomogram for concrete pipe culverts flowing full with outlet control, Manning's *n = 0.012*. 66

Figure 4-47: Nomogram for head for corrugated metal (CM) pipe culverts flowing full with outlet control, *n = 0.024*....67

Figure 4-48: Central pivot irrigation system near Grace, Idaho....68

Figure 4- 49: Schematic of a sprinkler system layout....69

Figure 4-50: A manifold with drip tape on a corn field....70

Part 634 – Hydraulics

Chapter 4 – Pipe Flow

634.0400 General

A. Pipe flow exists when a closed conduit of any form is flowing f'u1l of water. In pipe flow the cross-sectional area of flow is fixed by the cross section of the conduit and the water surface is not exposed to the atmosphere. The internal pressure in a pipe may be equal to, greater than, or less than the local atmospheric pressure.

B. The principles of pipe flow apply to the hydraulics of such structures as some culverts, drop inlets, siphons, and various types of pipelines.

C. Consider figure 4-1 which shows the hydraulic grade line, energy line and the energy heads at two points of the flow. Most pipe flow occurs through constant-diameter pipe, thus, making the velocity heads at sections (1) and (2) the same (*V1 = V2 = V*). This in turn means that the energy line (E.L.) and hydraulic grade line (H.G.L.) are parallel. The energy equation for constant-diameter pipe is written as:

$$z_1 + \frac{p_1}{\omega} = z_2 + \frac{p_2}{\omega} + h_f \qquad \text{(eq. 4-1)}$$

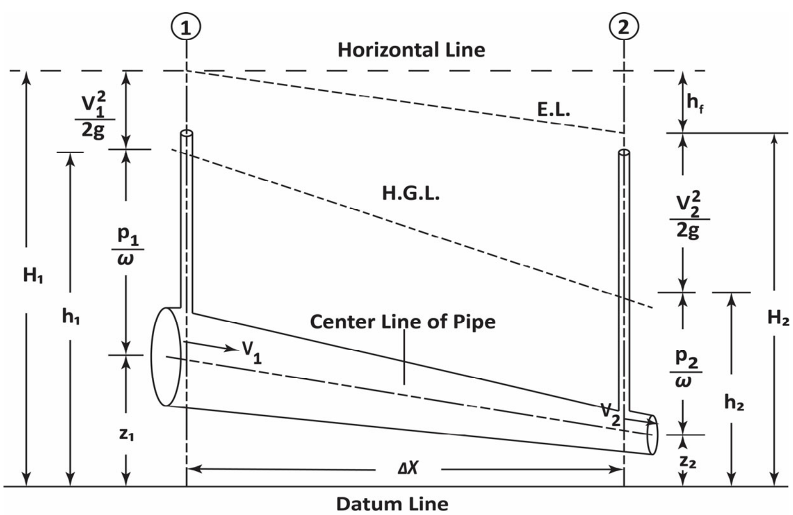
Where h_f represents the energy head losses between sections (1) and (2) in figure 4-1. Because the energy head losses are due to the friction between the moving fluid mass and the walls of the pipe, h_f is also referred to as the friction losses. Friction losses may be calculated by measuring the piezometric heads,

$$h_1 = z_1 + \frac{p_1}{\omega} \quad \text{and} \quad h_2 = z_2 + \frac{p_2}{\omega}$$

and taking the difference

$$h_f = h_1 - h_2$$

Figure 4-1: Energy Heads in Pipe Flow



D. The energy slope, or slope of the energy line, is the ratio of the friction losses, h_f, to the length of the pipe, L (Δx in figure 4-1), and may be expressed as:

$$S_f = \frac{h_f}{L} = \frac{h_1 - h_2}{L} = \frac{1}{L}\left[\left(z_1 + \frac{p_1}{\omega}\right) - \left(z_2 + \frac{p_2}{\omega}\right)\right] \qquad \text{(eq. 4-2)}$$

E. Flow in pipelines occurs in two different regimes, laminar and turbulent. Laminar flow occurs at relatively low velocities and is characterized by fluid particles moving in stable flow layers and with a parabolic velocity distribution. Increasing the flow velocity sufficiently causes instabilities of the flow layers which, in turn, cause the flow to become turbulent. In turbulent flow the fluid particles no longer move in stable layers, but rather clump together to form different-sized eddies. Turbulent flow is characterized by a more uniform velocity distribution across the pipe typically described mathematically by logarithmic or power-law functions.

634.0401 Friction Loss Methods

Calculation of friction losses is an important step Solving the energy equation, in analyzing flow in pipes, and in the selection of pipe sizes for specific applications. The easiest method of calculating friction losses in pipe can be performed using empirical equation. Several of the more common equation are Manning's equation, the Darcy-Weisbach equation, or the Hazen-Williams formula.

634.0402 Manning's Equation for Pipelines

A. Manning's equation for open-channel flow was presented as 634.03 equation 3-42 in terms of the flow velocity V, the hydraulic radius R, the channel bed slope S_o , the Manning's resistance coefficient n, and a constant C_u that depends on the system of units used ($Cu = 1.0$ for the International System, and $C_u = 1.486$ for the English System). When applied to pipelines, the bed slope So is replaced by the energy slope $S_f = h_f / L$, while the hydraulic radius is calculated in terms of the area, A (634.03 equation 3-2 repeated here), and wetted perimeter, P, of a full circular cross-section of diameter D, using:

$$A = \frac{\pi D^2}{4}$$

$$P = \pi D \qquad \text{(eq. 4-3)}$$

which results in:

$$R = \frac{A}{P} = \frac{D}{4} \qquad \text{(eq. 4-4)}$$

B. With these substitutions, the Manning's equation for a pipeline results in:

$$V = \frac{C_u}{n}\left(\frac{D}{4}\right)^{2/3}\sqrt{\frac{h_f}{L}} \qquad \text{(eq. 4-5)}$$

C. Typically, the equation is rewritten by isolating h_f:

$$h_f = \frac{4^{2/3}}{C_u^2}\left(\frac{n^2 L}{D^{4/3}}\right)V^2 \qquad \text{(eq. 4-6)}$$

D. Equations based on the above equation (velocity) for both the International System (SI) and English System (ES) follow:

(1) International System (SI): *h_f (m), Cu = 1.0, L(m), D(m), V(m/s)*

$$h_f = 6.3496\left(\frac{n^2L}{D^{4/3}}\right)V^2 \qquad \text{(eq. 4-7)}$$

(2) English System (ES): h_f (ft), *Cu = 1.486, L(ft), D(ft), V (ft/s, fps)*

$$h_f = 2.8755\left(\frac{n^2L}{D^{4/3}}\right)V^2 \qquad \text{(eq. 4-8)}$$

E. Sometimes it is preferred to work with the discharge *Q*, instead of the flow velocity *V*. For circular pipelines of diameter *D*, the equation of continuity is written as either 634.03 equation 3-3 or equation 3-4, which are reproduced below:

$$Q = VA = V\frac{\pi D^2}{4}$$

$$V = \frac{4Q}{\pi D^2}$$

F. Substituting equation 3-4 into equation 4-5, and solving for *Q*:

$$Q = \frac{\pi C_u D^{8/3}}{4^{5/3}(n)}\sqrt{\frac{h_f}{L}} \qquad \text{(eq. 4-9)}$$

or, solving for h_f.

$$h_f = \left(\frac{4^{10/3}}{\pi^2 C_u^2}\right)\frac{n^2L}{D^{16/3}}Q^2 \qquad \text{(eq. 4-10)}$$

G. Equations based on the above equation (discharge) for both the International System (SI) and English System (ES) follow:

(1) International System (SI): *h_f(m), C_u = 1.0, L(m), D(m), Q(m^3/s)*

$$h_f = 10.2936\frac{n^2L}{D^{16/3}}Q^2 \qquad \text{(eq. 4-11)}$$

(2) English System (ES): *hf (ft), C_u = 1.486, L (ft), D (ft), Q (ft^3/s, cfs)*

$$h_f = 4.6615\frac{n^2L}{D^{16/3}}Q^2 \qquad \text{(eq. 4-12)}$$

H. A demonstration of how to calculate a pipeline's discharge using Manning's equation can be found in Example 4-1

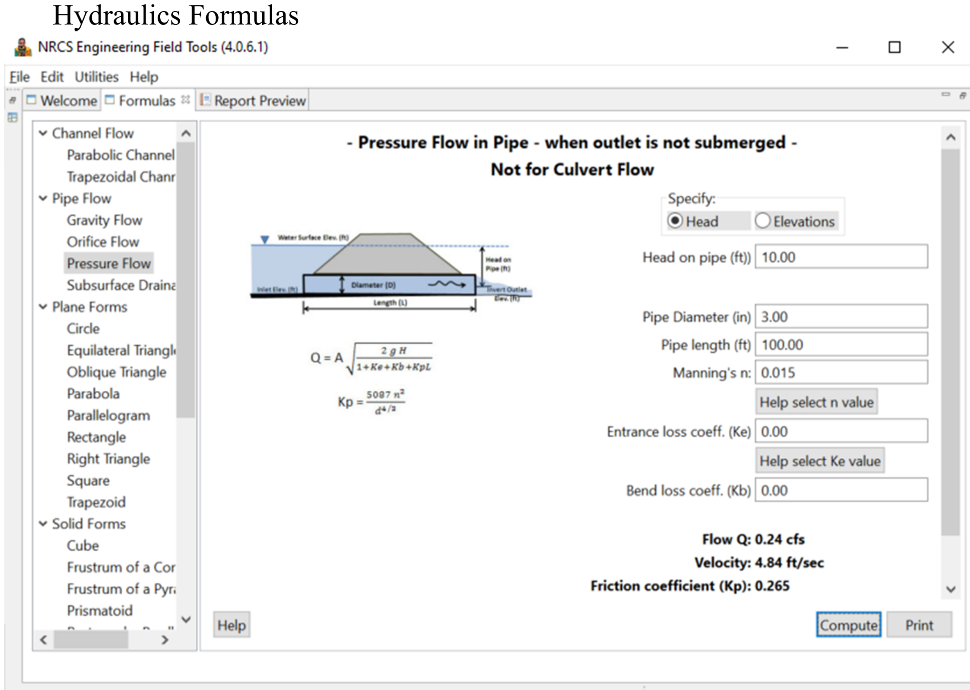
(1) Example 4-1 – To determine the discharge *Q* that can be conveyed by a *3.0-in*-diameter corrugated-plastic pipeline if a head *h_f = 10 ft*, is to be dissipated in a length *L = 100 ft* use equation 4-9 to calculate the discharge. The data to use are the following: *D = 3.0 in = 3.0/12 = 0.25 ft, hf = 10 ft, L = 100 ft, Cu = 1.486, n = 0.015* (from fig 4-2). The resulting discharge is:

$$Q = \frac{\pi C_u D^{8/3}}{4^{5/3}(n)}\sqrt{\frac{h_f}{L}} = \frac{(3.1416)(1.486)(0.25)^{8/3}}{4^{5/3}(0.015)}\sqrt{\frac{10}{100}} = 0.24\,cfs$$

I. The software USDA-NRCS Hydraulics Formula provides for the calculation of pipe flow using Manning's equation. To activate this solution, select the Pipe Flow tab, which produces the entry form shown in figure 4-2. The formula shown in the entry form is equivalent to equation 4-09, but with the diameter *D* in inches and with local loss coefficients equal to zero (i.e., $K_e = 0$, $K_b = 0$ reference 634.0405).

(1) Example 4-2 – Pipeline discharge calculation using the USDA-NRCS Hydraulics Formula software Using the Pipe Flow tab in the USDA-NRCS Hydraulics Formula software, with $Ke = 0$, $Kb = 0$, determine the discharge Q conveyed by a 3-in-diameter corrugated-plastic pipeline if a head $hf = 10\ ft$, is to be dissipated in a length $L = 100\ ft$. These data are the same as for the previous example 4-1. The resulting discharge is *0.24 cfs.* The equation for pipe flow shown in the figure 4-2 will be derived after introducing the concept of local losses in pipe (see section 634.0405).

Figure 4-2: Pipe Flow calculations with the USDA-NRCS Engineer Field Tools Hydraulics Formulas



J. One of the more important steps of using the Manning's equation is selecting the correct *n* value. Figure 4-3 gives a range of values for various materials. Manning's *n* values, shown in figure 4-3, are assembled from many hydraulic references, including Brater and King (1996), FHWA (2001), and USACE (2008).

Figure 4-3: Values of Manning's resistance coefficient for pipe

Pipe material	Manning's n		
	Minimum	Design	Maximum
Ductile iron, unlined	0.012	0.012 - 0.015	0.016
Ductile iron, polyurethane coated	0.010	0.012 - 0.014	0.014
Ductile iron, cement coated	0.011	0.012 - 0.014	0.015
Cast-iron, coated	0.010	0.012 - 0.014	0.014
Cast-iron, uncoated	0.011	0.013 - 0.015	0.015
Wrought iron, galvanized	0.013	0.015 - 0.017	0.017
Wrought iron, black	0.012	0.013 – 0.014	0.015
Steel, riveted and spiral	0.013	0.015 - 0.017	0.017
Annular corrugated metal [(1)]	0.021	0.021 - 0.025	0.025
Helical corrugated metal [(1)]	0.013	0.015 - 0.020	0.021
Wood stave	0.010	0.012 - 0.013	0.014
Neat cement surface	0.010		0.013
Concrete	0.010	0.012 - 0.017	0.017
Vitrified sewer pipe	0.010	0.013 - 0.015	0.017
Clay, common drainage tile	0.011	0.012 - 0.014	0.017
Corrugated plastic	0.014	0.015 - 0.016	0.017
PVC		0.009 -0.011	
Smooth Interior PE		0.009 - 0.015	0.02
Aluminum		0.01	
Gated Aluminum Pipe		0.013	

[(1)] N-values for corrugated metal pipe vary with pipe diameter. See FHWA (2001) or USACE (2008) to select a refined n-value.

K. Other examples of using the Manning's equation to solve for different variables are shown below.

(1) Example 4-3 – Using equation 4-5, determine flow velocity in a helical corrugated metal pipe. Use n = 0.016 (from fig. 4-3), for a 5-in diameter pipe that dissipates a head of $h_f = 6.5\ ft$ in a length $L = 300\ ft$. Using $D = 5\ in = 5/12\ ft = 0.4167\ ft$, and $Cu = 1.486$, equation 4-5 produces the following result:

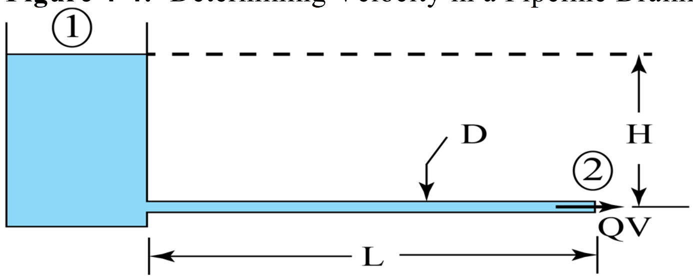
$$V = \frac{C_u}{n}\left(\frac{D}{4}\right)^{2/3}\sqrt{\frac{h_f}{L}} = \frac{1.486}{0.016}\left(\frac{0.4167}{4}\right)^{2/3}\sqrt{\frac{6.5}{300}} = 3.0\,ft/s$$

(2) Example 4-4 – Using equation 4-12, determine the head loss in *1500 ft* of a riveted and spiral steel pipe (use $n = 0.015$) with a *24-in (2-ft) diameter* that carries a discharge of *10 cfs*. Using equation 4-12:

$$h_f = 4.6615\frac{n^2 L}{D^{16/3}}Q^2 = 4.6615\frac{0.015^2(1500)}{2^{16/3}}10^2 = 3.9ft$$

(3) Example 4-5 – Determine the velocity in a pipeline draining a reservoir using Manning's equation. Consider a reservoir whose free surface is located at an elevation $z_1 = 60\ ft$, draining through a *0.5-ft-diameter*, *100-ft*-long, concrete pipe ($n = 0.012$) open to the atmosphere whose outlet is located at an elevation $z_2 = 55\ ft$. The system is depicted in figure 4-4. Entrance losses will be ignored.

Figure 4-4: Determining Velocity in a Pipeline Draining a Reservoir



Point 1 in the energy equation is at the reservoir free surface where $p1 = 0$ and $V1 = 0$. Point 2 is at the pipe outlet where $p2 = 0$ and $V_2 = V$, the pipe velocity. The energy head, $H = z_1 - z_2 = 60\ ft - 55\ ft = 5\ ft$, for this case. Applying the energy equation (634.03 equation 3-19) between points 1 and 2:

$$z_1 + \frac{p_1}{\omega} + \frac{V_1^2}{2g} = z_2 + \frac{p_2}{\omega} + \frac{V_2^2}{2g} + h_f$$

Using Manning's equation (eq. 4-8) to represent friction losses:

$$H + \frac{0}{\omega} + \frac{0^2}{2g} = \frac{0}{\omega} + \frac{V^2}{2g} + 2.8755\frac{n^2L}{D^{4/3}}V^2$$

This simplifies to:

$$H = V^2\left(\frac{1}{2g} + \frac{2.8755n^2L}{D^{4/3}}\right)$$

Solving for the velocity, V, and using $H = 5\ ft, L = 100\ ft, n = 0.012, D = 0.5\ ft, g = 32.2$

$$V = \sqrt{\frac{H}{\frac{1}{2g} + \frac{2.8755n^2L}{D^{4/3}}}} = \sqrt{\frac{5}{\frac{1}{2(32.2)} + \frac{2.8755(0.012)^2 100}{(0.5)^{4/3}}}} = 6.46\ ft/s$$

634.0403 Darcy-Weisbach Equation and Friction Factor

A. A second method for calculating friction losses in pipes is the Darcy-Weisbach equation written, in terms of the flow velocity V as:

$$h_f = f\frac{L}{D}\frac{V^2}{2g} \qquad \text{(eq. 4-13)}$$

B. The friction factor, f, is a function of a relative roughness, e/D, where "e" is known as the absolute roughness or equivalent sand roughness, and the Reynolds number of the flow.

C. The Reynolds number, defined in 634.0104 equation 1-10, is repeated here:

$$R_e = \frac{\rho VD}{\mu} = \frac{VD}{\nu}$$

where ρ is the density (mass per unit volume) of water, μ is the absolute or dynamic viscosity of water, and ν is the kinematic viscosity of water, defined by 634.01 equation 1-9, $\nu = \mu/\rho$.

Values of the density and viscosity of water related to temperature are available in 634.0107 Exhibit D, figures 1-9 (English Units) and 1-10 (SI Units).

D. The absolute roughness of a pipe material is the average height of the irregularities of the inner wall of the pipe. The first experiments on head losses in pipes were conducted in the early 20th century by coating glass pipes with uniform-size sand grains. The diameter of the sand grains was used to represent the absolute roughness of the pipe, *e*. Typical values of the absolute roughness of various pipe materials are presented in Figure 4-5. Absolute roughness values may be found in Streeter and Wylie (1998) and other similar fluid mechanics texts.

Figure 4-5: Absolute roughness values for pipe materials

Pipe material	e (mm)	e (ft)
Centrifugally spun cement	0.0015	0.000005
Concrete	0.3-3	0.001-0.01
Bituminous lining	0.0015	0.000005
Asphalt-dipped cast iron	0.12	0.0004
Cast iron	0.25	0.00085
Ductile iron	0.061	0.0002
Galvanized iron	0.15	0.0005
Wrought iron	0.046	0.00015
Commercial steel	0.046	0.00015
Riveted steel	0.9-9	0.003-0.03
Welded-steel pipe	0.046	0.00015
PVC, PE, HDPE pipe	0.002	0.0000066
Aluminum, with couplers	0.13	0.00043
Smooth surface (glass, plastic)	0.0015	0.000005
Drawn tubing, brass, lead, copper	0.0015	0.000005
Wood stave	0.18-0.9	0.0006-0.003

E. The Darcy-Weisbach friction factor has different expressions for laminar or turbulent pipe flow. Water flowing at a very low velocity, or in a very small-diameter pipe, typically flows in a laminar flow regime. In laminar flow the flow takes place in layers (Latin, laminae) that remain very stable and are easily identifiable by dye injected into the flow. As the velocity increases, conditions are reached in which the flow becomes turbulent. In turbulent flow, layers of flow are no longer identifiable, and the flow tends to break down into eddies that facilitate mixing. Laminar flow is rare, occurring at low flow velocity or in small pipe diameter. Most pipe flows of interest are turbulent.

F. Experiments to determine the laminar or turbulent nature of flow were carried out by Osborne Reynolds in the 19th century. Reynolds identified a dimensionless parameter, now known as the Reynolds number (634.0104 equation 1-10), to determine if a flow is laminar or turbulent. There is also a transitional flow regime in which the flow is neither laminar nor turbulent but shifts between conditions. The typical values for classifying flows according to laminar, transitional, or turbulent regimes are shown next:

(1) Laminar flow: $Re < 2000$
(2) Transitional flow: $2000 < Re < 4000$
(3) Turbulent flow: $Re > 4000$

The range $Re < 4000$ that encompasses laminar and transitional flow is a small range considering that Re can take values to 10^9 or larger. Thus, turbulent flow is more likely to occur in pipelines.

G. Example 4-6 How Reynolds number is used to classify pipe flow. Water is flowing in a 2-in diameter pipe at a rate of 0.08 cfs. The input data given is *D = 2 in = 2in/12in/ ft = 0.167 ft, Q = 0.08 cfs*, and from Title 210, National Engineering Handbook, Part 634, Chapter 1, (210-634-1), Section 634.0107 Exhibit D (634.01.7D), figures 1-8 (English Units) and 1-9 (SI Units) at a temperature of *70ºF* the kinematic viscosity of water is $\nu = .00001059\ ft^2/s = 1.059x10^{-5}\ ft^2/s$.

(1) If the water temperature is 70ºF, determine the Reynolds number of the flow and classify it as laminar, transitional, or turbulent.

(i) The flow velocity is calculated as:

$$V = \frac{Q}{A} = \frac{4Q}{\pi D^2} = \frac{(4)\left(0.08\ {ft^3}/{s}\right)}{3.1416(0.167)^2} = 3.7\ {ft}/{s}$$

(ii) And the Reynolds number is:

$$R_e = \frac{VD}{\nu} = \frac{\left(3.7\ {ft}/{s}\right)(0.167ft)}{1.059x10^{-5}\ {ft^2}/{s}} = 58347 \approx 5.83x10^4$$

(iv) Since *Re > 4000*, the flow is turbulent.

(2) In the laminar regime, the Darcy-Weisbach friction factor, *f*, is a function of the Reynolds number, given as:

$$f = \frac{64}{R_e} \qquad \text{(eq. 4-14)}$$

(3) In the turbulent regime, an equation relating the friction factor, *f*, the Reynolds number *Re*, and the relative roughness, *e/D*, is the Colebrook-White equation:

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{e}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right) \qquad \text{(eq. 4-15)}$$

(4) The difficulty in using this equation is that it is not explicit in *f* and any solution involving this equation requires a numerical approach. To facilitate solving explicitly for *f*, a close approximation (2 – 5% error) to the Colebrook-White equation is provided by the Swamee-Jain equation:

$$f = \frac{0.25}{log^2\left(\frac{e}{3.7D} + \frac{5.74}{Re^{0.9}}\right)} \qquad \text{(eq. 4-16)}$$

H. Pipe Flow Solutions Using the Darcy-Weisbach Equation –

(1) Solutions of turbulent pipe flow using the Darcy-Weisbach equation are often convenient to write in terms of discharge since the discharge, *Q*, is often known or can be calculated (equation 4-17).

$$h_f = \frac{8fLQ^2}{\pi^2 g D^5} \qquad \text{(eq. 4-17)}$$

(2) The Reynolds number *Re* can also be written in terms of the discharge *Q* as follows:

$$R_e = \frac{4\rho Q}{\pi\mu D} = \frac{4Q}{\pi\nu D} \qquad \text{(eq. 4-18)}$$

(3) With this definition of the Reynolds number, the Swamee-Jain equation becomes:

$$f = \frac{0.25}{log^2\left(0.27\left(\frac{e}{D}\right)+4.62\left(\frac{vD}{Q}\right)^{0.9}\right)} \qquad \text{(eq. 4-19)}$$

(4) Combining the Darcy-Weisbach equation (equation 4-17) with the Swamee-Jain equation (equation 4-19), and solving for the discharge, Q, produces the following equation:

$$Q = 2.22\left(\sqrt{\frac{gD^5h_f}{L}}\right)log^2\left(0.27\left(\frac{e}{D}\right)+4.62\left(\frac{vD}{Q}\right)^{0.9}\right) \qquad \text{(eq. 4-20)}$$

(5) This equation's variables are best solved by using a numerical spreadsheet application. Typically, there are three types of problems involving pipe friction losses, namely:

- Head loss problem: calculate h_f given D, Q or V, and g, L, e, v.
- Discharge problem: calculate Q or V, given D, h_f and g, L, e, v.
- Sizing problem: calculate D, given Q, h_f and g, L, e, v.

Where h_f is the friction loss, D is the pipe diameter, Q is the discharge, V is the velocity, g is the acceleration of gravity, L is the pipe length, e is the absolute roughness, and v is the kinematic viscosity.

I. Examples 4-7 through 4-9 demonstrate solving for pipe flow solutions with the Darcy-Weisbach equation

(1) Example 4-7 - Given $D = 0.3\ ft$, $Q = 0.20\ cfs$, $g = 32.2\ ft/s^2$, $L = 1000\ ft$, $e = 0.002\ in = 0.000166\ ft$, and $v = 1.13x10^{-5}\ ft^2/s$, find h_f.

(i) A numerical spreadsheet application of equation 4-20 gives $h_f = 8.86\ ft$.

(ii) Alternately, equation 4-20 can be re-arranged and solved directly for h_f, with a hand calculator.

(2) Example 4-8 - Given $D = 0.7\ ft$, $h_f = 15\ ft$, $g = 32.2\ ft/s2$, $L = 750\ ft$, $e = 0.005\ in = 0.000416\ ft$, and $v = 1.2x10^{-5}\ ft^2/s$, find Q.

(i) A numerical spreadsheet application of equation 4-20 gives $Q = 2.68\ cfs$.

(ii) Alternately, equation 4-20 can be solved iteratively for Q, with a hand calculator, as shown in figure 4-6:

Figure 4-6: Iterative Solution for Q

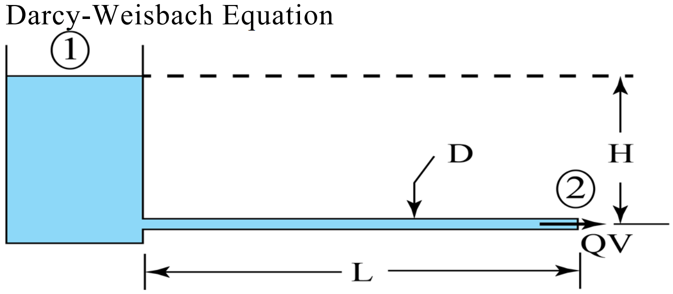
Trial Q	Calculated Q
1.5	2.63
2.1	2.66
2.6	2.681
2.7	2.684
2.68	2.683

(3) Example 4-9 - Given $Q = 3\ cfs$, $h_f = 10\ ft$, $g = 32.2\ ft/s^2$, $L = 1500\ ft$, $e = 0.01\ in = 0.000833\ ft$, and $v = 1.5x10\text{-}5\ ft^2/s$, find D.

A numerical spreadsheet application of equation 4-20 gives $D = 0.8591\ ft\ x\ 12\ in/ft = 10.29\ in$. The most likely value of commercial pipeline that can be used is 10 or 12 inch.

(4) Example 4-10 - Flow in pipeline draining a reservoir using the Darcy-Weisbach equation. Consider again (see example 4-5) a reservoir whose free surface is located at an elevation $z_1 = 60\ ft$, draining through a *0.5-ft-diameter*, *100-ft-long*, concrete pipe ($e = 0.003\ ft$) open to the atmosphere whose outlet is located at an elevation $z_2 = 55\ ft$. The system, which carries water at a temperature of $55°F$, is depicted in figure 4-7. Minor losses at the entrance from the reservoir into the pipe are ignored. Determine the discharge and velocity in the pipeline.

Figure 4-7: Determining Flow in a Pipeline Draining a Reservoir using the Darcy-Weisbach Equation



(i) As in example 4-5, the energy equation (634.03 equation 3-20) is applied, but the friction losses are estimated with the Darcy-Weisbach equation instead of Manning's equation. The energy equation, in terms of discharge, Q, simplifies to:

$$H = \frac{8Q^2}{\pi^2 g D^4}\left(1 + f\frac{L}{D}\right)$$

(ii) Re-arrange the equation to solve for Q and approximate the friction factor, f, by the Swamee-Jain equation (equation 4-19). Then using $H = 5\ ft$, $L = 100\ ft$, $e = 0.003\ ft$, $\nu = 1.3135 \times 10^{-5}\ ft^2/s$, $D = 0.5\ ft$, $g = 32.2\ ft/s^2$, and applying a numerical spreadsheet solution gives $Q = 1.285\ cfs$.

(iii) And the flow velocity is:

$$V = \frac{Q}{A} = \frac{4Q}{\pi D^2} = \frac{(4)\left(1.285\ {ft^3}/{s}\right)}{3.1416(0.5\ ft)^2} = 6.54\ \mathrm{ft/s}$$

634.0404 Pipe Flow Solutions Using Hazen-Williams Formula

A. A third method for calculating friction head losses in pipes is the Hazen-Williams formula. The Hazen-Williams formula was developed from empirical data of water flow in pipes. In terms of flow velocity, V, the Hazen-William formula is expressed as follows:

(1) International System (SI): *V(m/s)*

$$V = 0.849 C_{HW} R^{0.63} S_f^{0.54} \qquad \text{(eq. 4-21)}$$

(2) English System (ES): *V (ft/s, fps)*

$$V = 1.318 C_{HW} R^{0.63} S_f^{0.54} \qquad \text{(eq. 4-22)}$$

where V is velocity, C_{HW} is the Hazen-Williams coefficient, R is the hydraulic radius ($R=D/4$), and S_f is the energy slope ($S_f=h_f/L$).

B. Using the definitions of *R* and Sf, the Hazen-Williams formula may be written as:

(1) International System (SI): *h_f(m), L(m), D(m), V(m/s)*

$$V = 0.354 C_{HW} D^{0.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(eq. 4-23)}$$

(2) English System (ES): h_f (ft), L (ft), D (ft), V (ft/s, fps)

$$V = 0.550 C_{HW} D^{0.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(eq. 4-24)}$$

B. In terms of the discharge, the Hazen-Williams formula is written as:

(1) International System (SI): *h_f(m), L(m), D(m), Q(m^3/s)*

$$Q = 0.278 C_{HW} D^{2.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(eq. 4-25)}$$

(2) English System (ES): h_f(ft), L(ft), D(ft), Q(cfs)

$$Q = 0.432 C_{HW} D^{2.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(eq. 4-26)}$$

C. Solving for the head loss, h_f, in the Q-based equations:

(1) International System (SI): *h_f(m), L(m), D(m), Q(m^3/s)*

$$h_f = 10.704 \left(\frac{Q}{C_{HW}}\right)^{1.85} \frac{L}{D^{4.87}} \quad \text{(eq. 4-27)}$$

(2) English System (ES): h_f(ft), L(ft), D(ft), Q(cfs)

$$h_f = 4.732 \left(\frac{Q}{C_{HW}}\right)^{1.85} \frac{L}{D^{4.87}} \quad \text{(eq. 4-28)}$$

D. Solving for the diameter, D, in the Q-based equations:

(1) International System (SI): *h_f(m), L(m), D(m), Q(m^3/s)*

$$D = 1.627 \left(\frac{Q}{C_{HW}}\right)^{0.38} \left(\frac{L}{h_f}\right)^{0.205} \quad \text{(eq. 4-29)}$$

(2) English System (ES): h_f(ft), L(ft), D(ft), Q(cfs)

$$D = 1.376 \left(\frac{Q}{C_{HW}}\right)^{0.38} \left(\frac{L}{h_f}\right)^{0.205} \quad \text{(eq. 4-30)}$$

E. Figure 4-8, shows typical values for the Hazen-Williams coefficient. Hazen-Williams coefficients may be found in Lamont (1981), Mays (1991) and other similar publications.

Figure 4-8: Values of the Hazen-Williams coefficient.

Pipe description	Condition	C_{HW}
Very smooth	Straight alignment	140
	Slight curvature	130
Cast iron, uncoated or steel pipe or steel pipe	New	130
	5 years old	120
	10 years old	110
	15 years old	100
	20 years old	90
	30 years old	80
Cast iron, coated	All ages	130
Wrought iron or standard galvanized steel	Diameter 12 in. and up	110
	Diameter 4 to 12 in.	100
	Diameter 4 in or less	80
Brass or lead	New	140
Concrete	Very smooth, excellent joints	140
	Smooth, good joints	120
	Rough	110
Vitrified clays		110
Smooth wooden; wood stave		120
Asbestos cement		140
Corrugated metal		60
Pipes of small diameter	old, rough inside surface, as low as	40
PVC, PE, HDPE		150
Aluminum		120
Aluminum gated pipe		110

F. Solutions to pipe-flow problems with the Hazen-Williams formula require the use of equations (equation 4-21) through (equation 4-30), depending on the data given and which variable is being solved for. This is demonstrated in examples 4-11 through 4-15.

(1) Example 4-11 – Pipe velocity calculation using the Hazen-Williams formula. A pipeline with a length $L = 1200\ ft$, diameter $D = 0.75\ ft$, suffers a head loss $h_f = 12\ ft$. If the pipe is made of 5-year-old steel pipe ($C_{HW} = 120$), determine the pipe velocity.

Using equation 4-24 as follows:

$$V = 0.550 C_{HW} D^{0.63} \left(\frac{h_f}{L}\right)^{0.54} = 0.550(120)(0.75)^{.63} \left(\frac{12}{1200}\right)^{0.54} = 4.58\ ft/s$$

(2) Example 4-12 – Pipe discharge calculation using the Hazen-Williams formula. Determine the discharge Q for a rough concrete pipeline $(C_{HW} = 110)$ with a diameter $D = 1.5\ ft$ that produces a head loss $h_f = 8.5\ ft$ on a length of $L = 650\ ft$.

Using equation 4-26 as follows:

$$Q = 0.432 C_{HW} D^{2.63} \left(\frac{h_f}{L}\right)^{0.54} = 0.432(110)(1.5)^{2.63}\left(\frac{8.5}{650}\right)^{0.54} = 13.27\ cfs$$

(3) Example 4-13 – Head loss calculation using the Hazen-Williams formula. Determine the head loss h_f in a new PVC pipeline $(C_{HW} = 150)$ of length $L = 2000\ ft$ and diameter $D = 3.0\ ft$, that carries a discharge $Q = 20\ cfs$.

Using equation 4-28 as follows:

$$h_f = 4.732\left(\frac{Q}{C_{HW}}\right)^{1.85}\frac{L}{D^{4.87}} = 4.732\left(\frac{20}{150}\right)^{1.85}\frac{2000}{3^{4.87}} = 1.08 ft$$

(4) Example 4-14 – Diameter D calculation using the Hazen-Williams formula. Determine the diameter of coated cast iron ($C_{HW} = 130$) pipe that produces a friction head loss $h_f = 20\ ft$ in a length $L = 500\ ft$ while carrying a discharge $Q = 25\ cfs$.

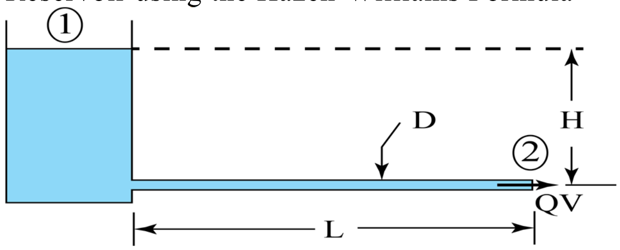
(i) Using equation 4-30 as follows:

$$D = 1.376\left(\frac{Q}{C_{HW}}\right)^{0.38}\left(\frac{L}{h_f}\right)^{0.205} = 1.376\left(\frac{25}{130}\right)^{0.38}\left(\frac{500}{20}\right)^{0.205} = 1.42 ft$$

(ii) The most likely value of commercial pipeline that can be used is 1.5ft.

(5) Example 4-15 – Flow in pipeline draining a reservoir using Hazen-Williams formula. Consider once more (see examples 4-5 and 4-10) the case of a reservoir whose free surface is located at an elevation $z_1 = 60\ ft$, draining through a *0.5-ft-diameter*, *100-ft-long*, rough concrete pipe $(C_{HW} = 110)$ open to the atmosphere whose outlet is located at an elevation $z_2 = 55\ ft$. The system is depicted in figure 4-9. Local losses at the entrance from the reservoir into the pipe are ignored. Determine the discharge and velocity in the pipeline.

Figure 4-9: Determining Discharge and Velocity in a Pipeline Draining a Reservoir using the Hazen-Williams Formula



(i) As in examples 4-5 and 4-10, the energy equation (634.0302 equation 3-13) is applied, but the friction losses are estimated with the Hazen-Williams formula. The energy equation, in terms of discharge, Q, simplifies to:

$$H = \frac{8Q^2}{\pi^2 g D^4} + 4.732\left(\frac{Q}{C_{HW}}\right)^{1.85}\frac{L}{D^{4.87}}$$

(ii) Using $H = 5\ ft,\ L = 100\ ft,\ C_{HW} = 110,\ D = 0.5\ ft,\ g = 32.2\ ft/s^2$, and applying a numerical spreadsheet solution gives $Q = 1.385\ cfs$, and the flow velocity is:

$$V = \frac{4Q}{\pi D^2} = \frac{4\left(1.385\ {ft^2}/{s}\right)}{3.1416(0.5ft)^2} = 7.05\ {ft}/{s}$$

(6) The same concrete pipe material, diameter, length, and elevations were used in examples 4-5, 4-10, and 4-15. These examples illustrate the use of Manning's equation, the Darcy- Weisbach equation (using Swamee-Jain equation for the friction factor), and the Hazen- Williams formula in estimating friction loss in pipe. The values of the flow velocity found using these three methods are listed below.
(i) Manning's equation: $V = 6.46\ fps$
(ii) Darcy-Weisbach (with Swamee-Jain): $V = 6.54\ fps$
(iii) Hazen-Williams formula: $V = 7.05\ fps$

634.0405 Local Losses in Pipelines

A. Local losses are energy losses due to the presence of appurtenances or changes in the pipeline such as valves, elbows, curves, reductions, or expansions. The term local losses indicate that these energy losses are concentrated at a location (rather than distributed along a pipeline as are friction losses). Local losses in pipelines are calculated using an equation of the form:

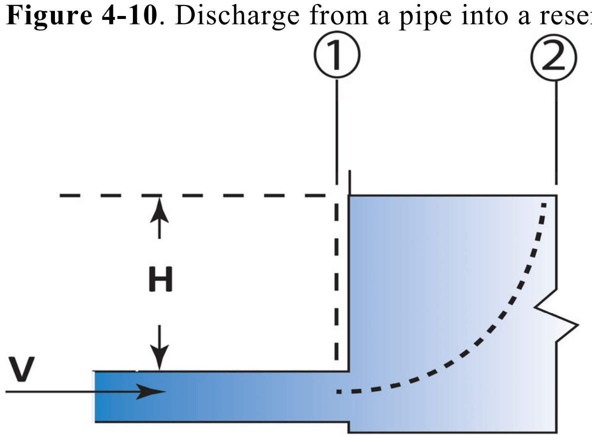
$$h_l = K\frac{V^2}{2g} = K\frac{8Q^2}{\pi g D^4} \qquad \text{(eq. 4-31)}$$

Where the local loss coefficient K, depends on the nature of the device or pipeline change producing the loss. Note that local losses are sometimes referred to as minor losses.

B. In addition to losses at appurtenances, local losses occur at pipeline expansions and contractions. Local losses also occur at entrances and discharge ends of pipe.

C. Discharge loss - Figure 4-10, below, shows the conditions of flow at the discharge end of a pipeline into a reservoir.

Figure 4-10. Discharge from a pipe into a reservoir.



(1) At section (1), right before the entrance, the pressure head is $p_1/\omega = H$, the elevation of the pipe centerline can be taken as $z_1 = 0$, and the velocity is $V_1 = V$. The curved line in the reservoir depicts a streamline of flow connecting the pipe outlet to the free

surface of the reservoir at section (2). In this section the pressure head is $p_2/\omega = 0$, the elevation is $z_2 = H$, and the local velocity is $V_2 = 0$. Energy head loss between sections (1) and (2) is a local head loss referred to as a discharge loss, and given by equation 4-31:

$$(h_l)_d = K_d \frac{V^2}{2g}$$

(2) Writing the energy equation (equation 3-20) between points (1) and (2), gives:

$$z_1 + \frac{p_1}{\omega} + \frac{V_1^2}{2g} = z_2 + \frac{p_2}{\omega} + \frac{V_2^2}{2g} + h_f$$

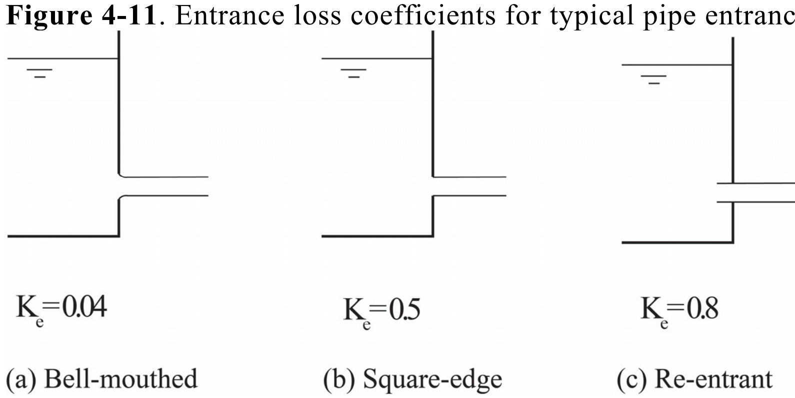
(3) Replacing the data for sections (1) and (2) as described above, the equation reduces to:

$$0 + H + \frac{V^2}{2g} = H + 0 + 0 + K_d \frac{V^2}{2g}$$

(4) From which it follows that the discharge loss coefficient, $K_d = 1.0$ (standard value used in pipe flow analyses).

D. Entrance loss - Three different conditions of entrance from a reservoir into a pipe, and their local loss coefficients, are depicted in figure 4-11.

Figure 4-11. Entrance loss coefficients for typical pipe entrance shapes



(1) The Pipe Flow tab in the USDA-NRCS Hydraulics Formula software provides additional values for the entrance coefficient K_e for other specific entrance conditions. These values are summarized in figure 4-12. Entrance loss coefficients may also be found in FHWA (2001) and USACE (2008), related to culvert analysis.

(2) Entrance losses are calculated using the expression (equation 4-31):

$$(h_l)_e = K_e \frac{V^2}{2g} = K_e \frac{8Q^2}{\pi g D^4}$$

Figure 4-12. Pipe entrance loss coefficients.

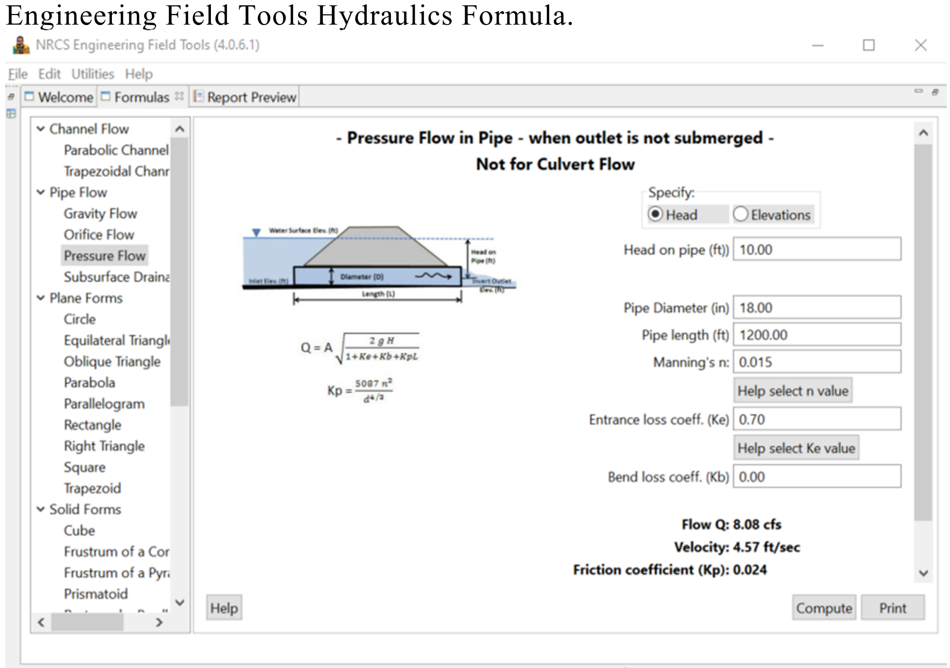
Type of Structure and Entrance Design	Ke
Pipe, Concrete	
Projecting from fill, socket end [groove-end]	0.2
Projecting from fill, square cut end	0.5
Headwall or Headwall and wingwalls with	
Socket end of pipe [groove-end]	0.2
Square end	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Pipe or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or Headwall and wingwalls square edge	0.5
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5

(3) The USDA-NRCS Hydraulics Formula software can be used to determine pipe flow calculation including local losses as shown in Example 4-16.

(i) Example 4-16 -Using the *Pipe Flow* tab in the *Hydraulics Formula* software, calculate the discharge Q and flow velocity V for a *18-in*-diameter, *1200-ft*-long, corrugated plastic pipe (*Manning's* $n = 0.015$) whose entrance is mitered to conform to a fill slope ($K_e = 0.7$). Head on the pipe is *10 ft*.

(ii) The solution, illustrated in the figure 4-13 indicates that $Q = 8.1$ *cfs*, and $V = 4.6$ *ft/s*.

Figure 4-13: Solution for pipe flow with entrance loss using USDA-NRCS Engineering Field Tools Hydraulics Formula.



E. Losses due to pipe fittings

(1) Pipe fittings such as valves, elbows, and bends produce local losses according to (equation 4-31). The values of selected pipe fittings are shown in figure 4-14.

Figure 4-14: Local loss coefficients for selected pipe fittings.

Pipe fitting	K
Globe valve, wide open	10
Alfalfa or stub valves	2.80
Close-return bend	2.20
Gate valve, half open	2.06
Portable hydrants	1.00
Valve opening elbows	1.00
Tee, through side outlet	1.80
90° short-radius elbow	0.90
90° medium-radius elbow	0.75
90° long-radius elbow	0.60
45° elbow	0.42
Gate valve, wide open	0.19
Ball valve, wide open	0.06

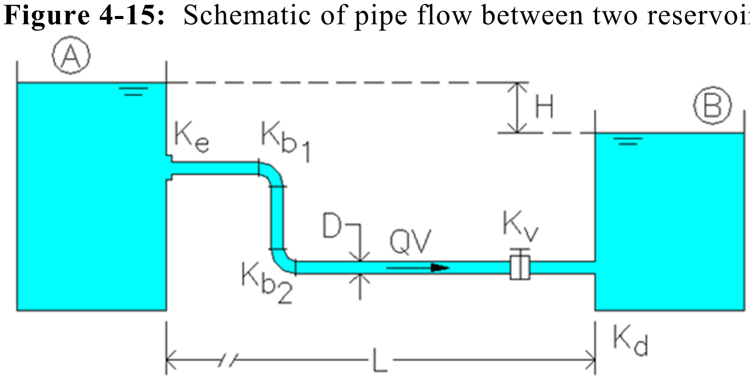
For a more complete set of values for local losses, refer to Brater and King (1996) or Idelchik (1999).

(2) Calculating Pipe flow between two reservoirs including local losses using Manning's equation is found in Example 4-17

(3) Example 4-17

(i) The figure 4-15 shows the steady flow between two reservoirs whose free surfaces have an elevation difference, H. The pipe has length L, diameter D, and resistance coefficient; the pipe carries a discharge Q and flows with a velocity, V.

Figure 4-15: Schematic of pipe flow between two reservoirs.



(ii) The velocity at the free surfaces (A) and (B) is practically zero, i.e., $V_A = V_B = 0$, and with the reservoirs open to the atmosphere, the gage pressure at those points are also zero, $p_A = p_B = 0$. Finally, the elevations of the free surfaces at points (A) and (B) can be taken as $z_A = H$ and $z_B = 0$ (i.e., the reference level for elevation is free surface (B)). The energy equation between points (A) and (B), including friction and local losses in the pipe, is written as (see 634.0302 equation 3-19):

$$z_A + \frac{p_B}{\omega} + \frac{V_A^2}{2g} = z_B + \frac{p_B}{\omega} + \frac{V_B^2}{2g} + \sum h_L$$

(iii) Introducing Manning's equation (equation 4-8) to estimate pipe friction losses, the local loss equation (equation 4-31), and the data presented above, the energy equation is written as:

$$H + \frac{0}{\omega} + \frac{0}{2g} = \frac{0}{\omega} + \frac{0}{2g} + 2.8755\left(\frac{n^2 L}{D^{4/3}}\right)V^2 + \sum K\frac{V^2}{2g}$$

Results in:

$$H = V^2\left(2.8755\left(\frac{n^2 L}{D^{4/3}}\right) + \frac{\sum K}{2g}\right)$$

In terms of the flow discharge, this equation is written as:

$$H = \frac{16Q^2}{\pi^2 D^4}\left(2.8755\left(\frac{n^2 L}{D^{4/3}}\right) + \frac{\sum K}{2g}\right)$$

(iv) The term $\sum K$ includes the sum of all local loss coefficients in the pipeline. Note that the Darcy-Weisbach equation or the Hazen-Williams formula is always an option to use in estimating pipe friction loss.

(v) Using the values *L = 550 ft, D = 2 ft, n = 0.012, Q = 35 cfs*, with the following local loss coefficients:

- Entrance: K_e = 0.04 (bell-mouth entrance)
- Elbow 1: K_{b1} = 0.90 (short-radius elbow)
- Elbow 2: K_{b2} = 0.75 (medium-radius elbow)
- Valve: K_v = 0.19 (gate valve, wide open)
- Discharge: K_d = 1.00 (standard value for discharge coefficient)

(vi) Thus, for this case

$$\sum K = K_e + K_{b1} + K_{b2} + K_v + K_d = 0.04 + 0.90 + 0.75 + 0.19 + 1.0$$
$$= 2.88$$

(vii) Find the difference in elevations between the reservoirs, *H*. Using the equation developed above:

$$H = \frac{(16)35^2}{\pi^2 2^4}\left(2.8755\left(\frac{0.012^2(550)}{2^{4/3}}\right) + \frac{2.88}{(2)32.2}\right) = 16.77ft$$

(viii) If the elevation difference between reservoirs is known, the resulting equation above may be solved for V to determine the velocity in the pipe. Then the discharge may be determined with the continuity equation, *Q = VA*.

F. Development of the Pipe Flow equation

(1) The equation from the Pipe Flow tab in the USDA-NRCS Engineering Field Tools Hydraulics Formula software is shown figure 4-13 and repeated here:

$$Q = A\sqrt{\frac{2gH}{1+K_e+K_b+K_pL}}\ ;\ K_p = \frac{5087n^2}{d^{4/3}};\ \text{d(in)} \qquad \text{(eq. 4-32)}$$

(2) To develop this equation:

$$\sum K = K_e + K_b + 1$$

with K_e being the entrance loss coefficient, K_b accounting for devices such as elbows and valves (referred to as Bend Coefficient in figure 4-13), and $K_d = 1$ being the discharge coefficient. Thus, the energy equation presented in example 4-17 (Manning's equation is used to estimate pipe friction losses) is written as:

$$H = V^2\left(2.8755\left(\frac{n^2L}{D^{4/3}}\right) + \frac{\sum K}{2g}\right) = \frac{V^2}{2g}\left((2g)2.8755\left(\frac{n^2L}{D^{4/3}}\right) + K_e + K_b + 1\right)$$

(3) Replacing the diameter D (in feet) with d (in inches) and defining:

$$K_p = \frac{5087n^2}{d^{4/3}}$$

(4) The energy equation becomes:

$$H = \frac{V^2}{2g}\left((K_p)(L) + K_e + K_b + 1\right) = \frac{V^2}{2g}\left(1 + K_e + K_b + (K_p)(L)\right)$$

(5) Solving for V, gives the following equation:

$$V = \sqrt{\frac{2gH}{(1 + K_e + K_b + (K_p)(L))}}$$

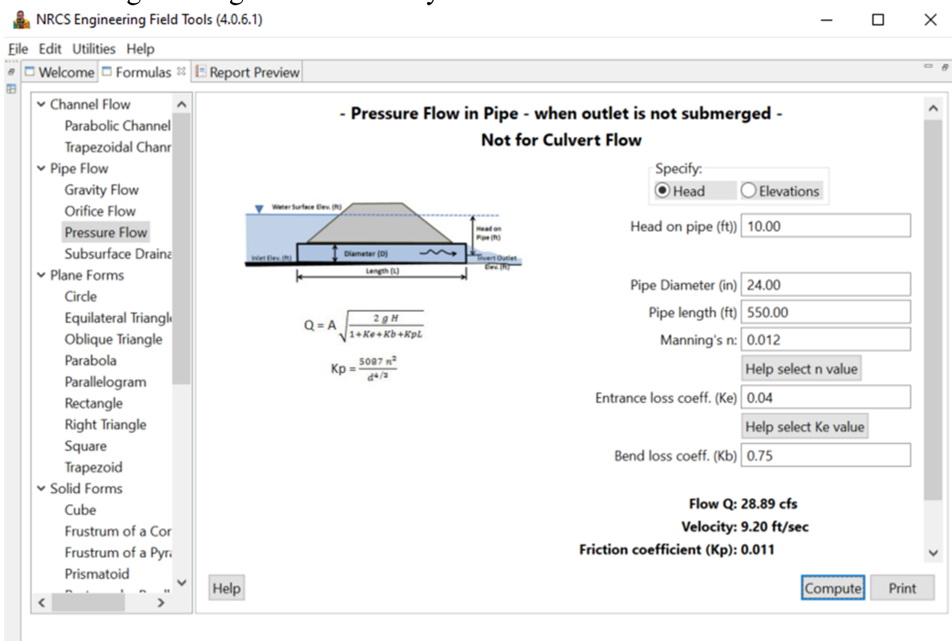
(6) Multiplying this result by the area of the cross-section and substituting Q for VA gives equation 4-32.

G. Pipe flow calculation including local losses using USDA-NRCS Engineering Field Tools Hydraulics Formula are shown in Example 4-18

(1) Example 4-18 - Use the Pipe Flow tab in the USDA-NRCS Engineering Field Tools Hydraulics Formula software to determine the discharge in a pipe with $L = 550$ ft, $D = 2$ ft, $n = 0.012$, $H = 10$ ft, with the following local loss coefficients, $K_e = 0.04$ (bell-mouth entrance), $Kb_2 = 0.75$ (medium-radius elbow), and $K_d = 1.00$ (standard value for discharge coefficient).

(2) The solution is shown in the following figure 4-16. The results shown are $Q = 28.9$ *cfs* and $V = 9.2$ *ft/s*. Also, the friction coefficient $Kp = 0.0106$.

Figure 4-16: Solution for pipe flow including all local losses using USDA-NRCS Engineering Field Tools Hydraulics Formula



634.0406 Pumps in Pipelines

A. Pumps are mechanical devices used to introduce energy into a pipeline system. Pumps can be used, for example, to lift water from a lower elevation to a higher one, or to overcome friction losses between reservoirs. The most common types of pumps used in pipelines are centrifugal pumps. In this section the analysis of pipeline systems with centrifugal pumps is presented.

B. A pump introduces an energy head hp into a pipeline system. The energy equation including a pump head is written as:

$$z_1 + \frac{p_1}{\omega} + \frac{V_1^2}{2g} + h_p = z_2 + \frac{p_2}{\omega} + \frac{V_2^2}{2g} + h_f + \sum h_L \qquad \text{(eq. 4-33)}$$

The normal convention is that energy added to the flow, such as the pump head hp, is placed on the left-hand side (upstream side) of the equation, while, energy extracted from or lost by the flow, such as the friction head hf and the sum of local losses $\sum h_L$, is placed on the right-hand side (downstream side) of the equation.

C. Pump Operational Characteristics

(1) In pipeline system hydraulics, an important operational characteristic of a pump is the variation of the pump head h_p with the discharge Q passing through the pump. For centrifugal pumps the relationship between *hp* and Q is given by a quadratic equation (also called a polynomial equation of the second degree) of the form:

$$h_p = aQ^2 + bQ + c \qquad \text{(eq. 4-34)}$$

(2) The coefficients *a, b,* and *c*, in this equation, can be determined by fitting data from tests performed on a given pump. The equation describing a given pump, or at least a graph of the relationship between h_p and Q, should be available from the pump manufacturer.

(3) Example 4-19 – Pump head and discharge analysis

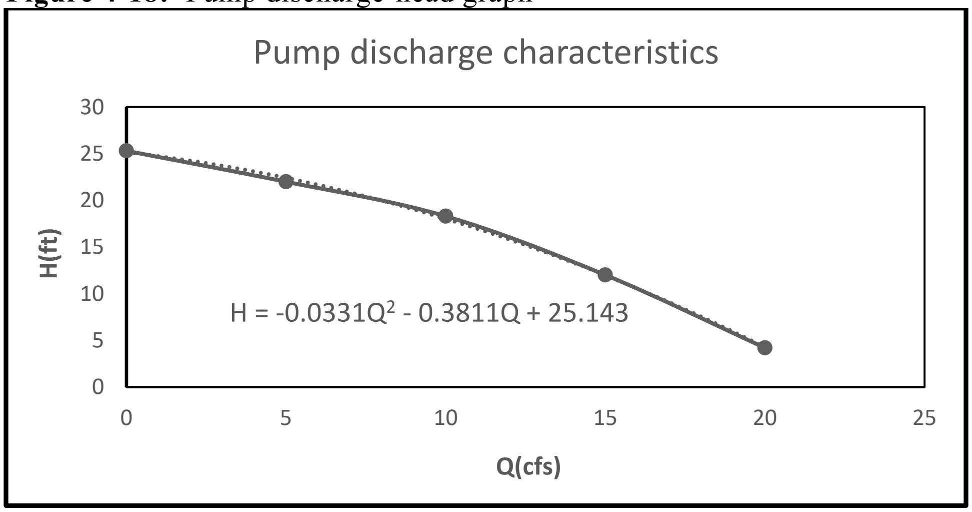
(i) Tests on a pump produce the following discharge-head data shown in figure 4-17:

Figure 4-17: Discharge versus Head from Pump Tests

Q(cfs)	H_p(ft)
0.0	25.3
5.0	22.0
10.0	18.3
15.0	12.0
20.0	4.2

(ii) The pump discharge graph and equation were produced with curve-fitting software and are shown in figure 4-18.

Figure 4-18: Pump discharge-head graph



(iii) Notice from the graph that the energy head for zero discharge is $H = 25.143\ ft$. This is the "shut-off head" or maximum head for this pump. If valves are closed and no flow occurs, the pump is capable of building *25.143 ft* of head.

(iv) The maximum pump capacity at free discharge is calculated from the pump curve equation by setting $H = 0$ and solving for Q. This gives

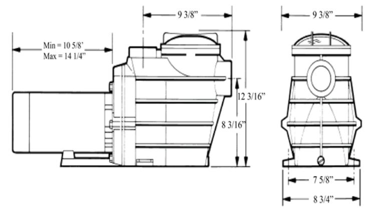
$$0 = -0.0331Q^2 - 0.3811Q + 25.143$$

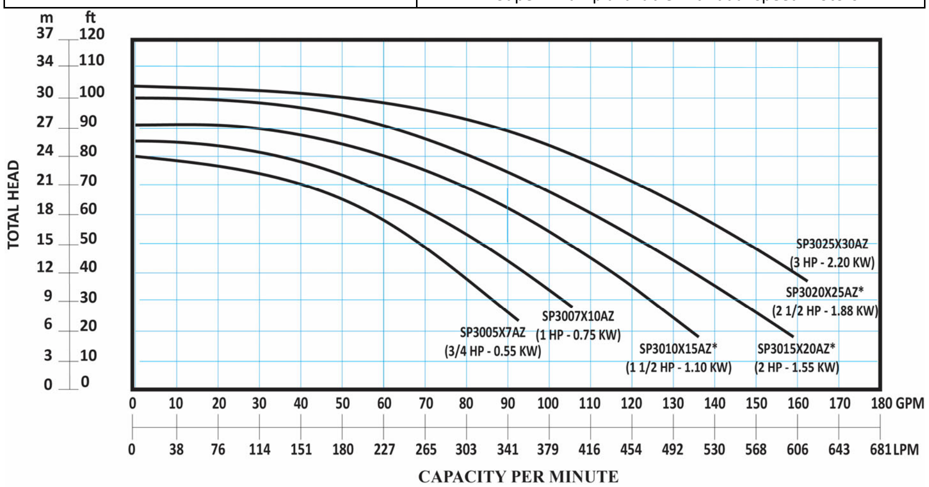
The positive solution to this quadratic equation produces the free-discharge, $Q = 22.4\ cfs$, which the pump delivers with $H = 0$ (no discharge pressure). This discharge is what the pump would deliver if it were disconnected from any pipeline and allowed to discharge freely to the atmosphere.

(v) Example 4-20 illustrates the use of manufacturer-provided pump curves in pump selection.

- The figure 4-19 shows the pump curves provided by a manufacturer for various pumps whose sizes are specified in the figure.

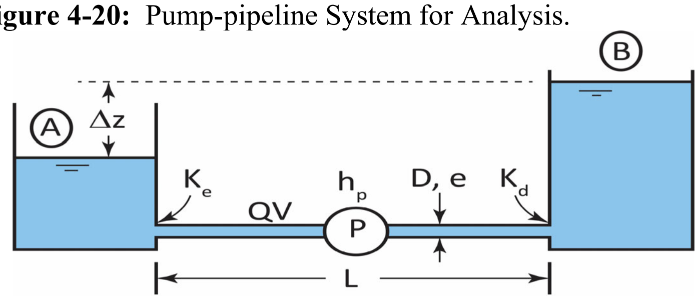
Figure 4-19: Manufacturer-provided Pump Curves and Pump Sizes

OVERALL DIMENSIONS	MODEL	Motor power		Pipe size	Dimension "A"	
		HP	KW	inches	inches	mm
	SP3005X7AZ	¾	0.55	1 ½	$10^{5/8}$	270
	SP3007X10AZ	1	0.75	1 ½	11	280
	SP3010X15AZ*	1½	1.10	2	$12^{1/8}$	308
	SP3015X20AZ*	2	1.55	2	$13^{1/16}$	332
	SP3020X25AZ*	2½	1.88	2	$13^{1/16}$	332
	SP3025X30AZ	3	2.20	2	$14^{13/64}$	361
	* Super II Pump available with dual-speed motors					



- Suppose that the pump is to be installed in a system as that shown in figure 4-20. The parameters of the system are the following: $L = 1000\ ft$, $D = 2\ in$, $e = 0.000005\ ft$, $g = 32.2\ ft/s^2$, $\nu = 1.20\times10^{-5} ft/s$, $K_e = 0.5$, $K_d = 1.0$, (i.e., $\Sigma K = 1.5$), and $\Delta z = 20\ ft$.

Figure 4-20: Pump-pipeline System for Analysis.



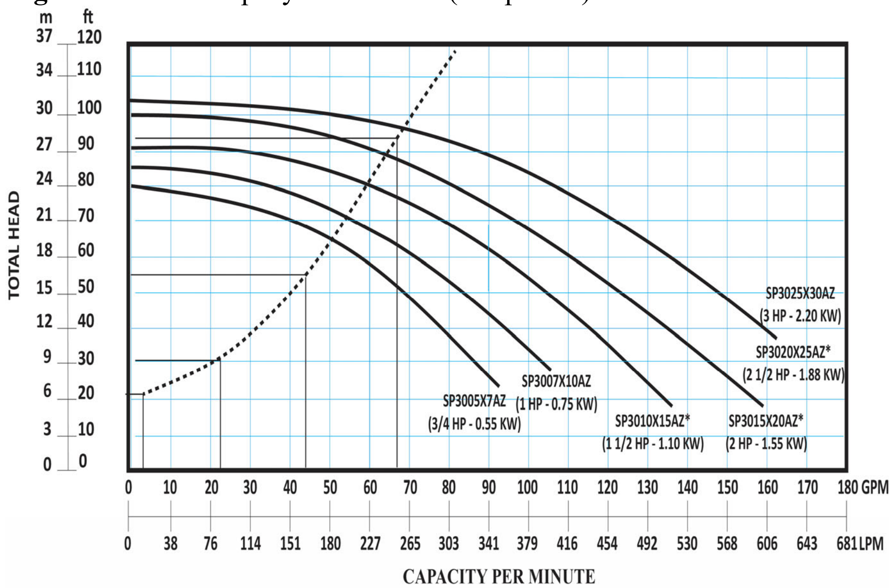
- A numerical spreadsheet application was used to develop the system curve, which is the capacity and head needed for various operating conditions. The tabular data for the system curve are shown in figure 4-21:

Figure 4-21: System Curve (Tabular)

System h_p (ft)	Q (gpm)
21	4
30	22
55	45
93	67
142	90
275	135
451	180

- The next step is to plot the system curve values of h_p (*ft*) and *Q(gpm)* over the manufacturer's pump curves, as shown in the figure 4-22:

Figure 4-22: Pump System Curve (Graphical)



- The points of intersection of the system curve with the pump curves are the operating points. The three system operating points for the pumps using 2-inch diameter pipe are:
  - Pump SP3010X15AZ, $Q = 59\ gpm$, $h_P = 80\ ft$
  - Pump SP3015X20AZ, $Q = 64\ gpm$, $h_P = 88\ ft$
  - Pumps SP3020X25AZ and SP3025X30AZ, $Q = 69\ gpm$, $h_P = 99\ ft$

  Generally, a pump should be selected with an operating point at 80 to 85% of the shut-off head (maximum head).

D. Pump Power and Efficiency

(1) The hydraulic power developed by a pump represents the amount of energy per unit time introduced by the pump into the flow. The hydraulic power, P_h, is calculated as:

$$P_h = \gamma_w Q h_p \qquad \text{(eq. 4-35)}$$

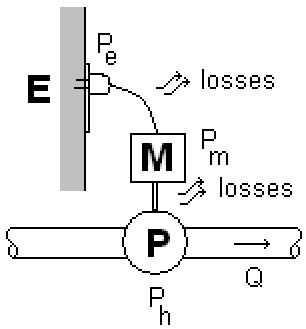
where γ_w is the specific weight of water. Using the units of the English system, namely, γ_w *(lb/ft^3), Q (cfs)*, and h_p*(ft)*, the power is given in units of lb·ft/s. A more commonly used unit of power in the English System is the *horsepower* (*hp*) defined as *1 hp = 550 lb·ft/s*. Thus, in terms of horsepower (P_h) the equation for the hydraulic power of a pump is:

$$P_h = \frac{\gamma_w Q h_p}{550} \qquad \text{(eq. 4-36)}$$

(2) In units of the International System (SI), namely, γ_w *(N/m³), Q (m³/s), and* h_p*(m),* the pump power P_h is given in Watts (*W*). For the SI use equation 4-35 to calculate the power.

(3) To provide hydraulic power, P_h, the pump *P* is activated by a motor *M* as illustrated in Figure 4-23. The motor can be powered by electricity at connection *E*.

Figure 4-23: System Schematic showing Motor *M*, the Electric Supply *E*, and Pump *P*



(4) The motor provides the pump with input power P_m, however, due to energy losses by friction in the pump shaft (which is dissipated as heat in the environment), and other losses, not all the motor power P_m is utilized by the pump to produce hydraulic power. Thus, in general, $P_h < P_m$, and the ratio between the hydraulic power and the motor power is a quantity smaller than one ($P_h/P_m<1$) known as the efficiency of the pump, η_p:

$$\eta = \frac{P_h}{P_m} < 1 \qquad \text{(eq. 4-37)}$$

(5) The motor itself is provided with a certain amount of electric power P_e; however, due to losses in the transmission line as well as in the motor mechanism the motor power P_m is smaller than the electric power, i.e., $P_m<P_e$. Thus, the *efficiency of the motor*, η_m, is a quantity smaller than one ($P_m/P_e<1$), defined as:

$$\eta = \frac{P_h}{P_e} < 1 \qquad \text{(eq. 4-38)}$$

(6) The combined motor-pump system, in turn, has an efficiency $\eta_{m\text{-}p}$ defined as

$$\eta_{m-p} = \frac{P_h}{P_e} = \left(\frac{P_h}{P_m}\right)\left(\frac{P_m}{P_e}\right) = \eta_p \eta_m < 1 \qquad \text{(eq. 4-39)}$$

(7) The various efficiencies, η_i, can be expressed as a percent by multiplying by 100.

(8) Example 4-21 - Calculating Pump power and efficiency.

(i) A pump produces a head $h_p = 41.67\ ft$ with a discharge $Q = 1.24\ cfs$. The pump efficiency is 80% ($\eta_p = 0.80$) and the motor efficiency is 90% ($\eta_m = 0.90$).

(ii) Determine: (a) the hydraulic power provided by the pump to the flow, *Ph*; (b) the power that the motor needs to provide to the pump, P_m; (c) the electric power needed to be provided to the motor, P_e; and (d) the efficiency of the motor-pump system, $\eta_{m\text{-}p}$.

- The hydraulic power (in hp) is calculated according to equation 4-36:

$$P_h = \frac{\omega Q h_p}{550} = \frac{(62.4)(1.24)(41.67)}{550} = 5.86hp$$

- The power provided by the motor follows from the definition of the pump efficiency (equation 4-37):

$$P_m = \frac{P_h}{\eta_p} = \frac{5.86}{0.80} = 7.33hp$$

- The electric power provided to the motor follows from the definition of the motor efficiency (equation 4-38):

$$P_e = \frac{P_m}{\eta_m} = \frac{7.33}{0.90} = 8.14hp$$

- The efficiency of the motor-pump system can be calculated with equation 4-39:

$$\eta_{m-p} = \eta_p \times \eta_m = 0.80 \times 0.90 = 0.72 = 72\%$$

or

$$\eta_{m-p} = \frac{P_h}{P_e} = \frac{5.86}{8.14} = 0.72 = 72\%$$

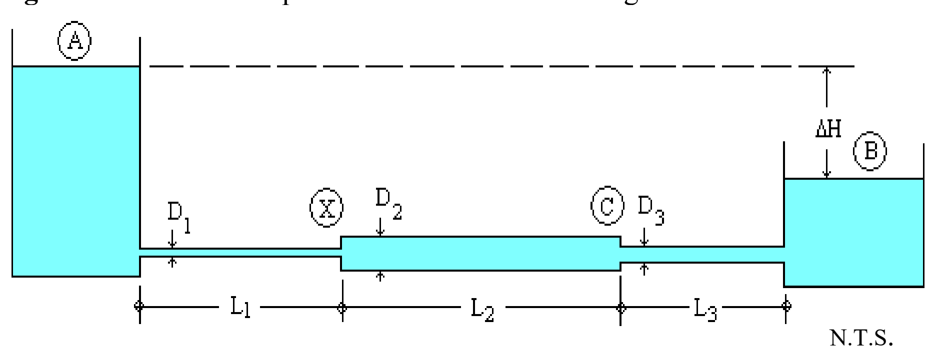
634.0407 Pipelines and Networks

A. Pipelines of different diameters can be combined into different configurations including pipes in series, pipes in parallel, pipes converging to a single point, and more complex pipe networks. These configurations are discussed in the following sections.

B. Pipelines in Series

(1) Pipelines in series consist of segments of pipeline connected one after the other so that the flow discharge follows a single path. An arrangement of pipelines in series is presented in figure 4-24.

Figure 4-24: Three Pipelines in Series Connecting Two Reservoirs



(2) Three pipelines of different diameters and lengths are shown carrying water between reservoirs (A) and (B). The pipelines have lengths L_1, L_2, and L_3, with corresponding diameters D_1, D_2, and D_3. In addition, the pipelines have different roughness coefficients. The elevation difference between reservoirs (A) and (B) is given by ΔH.

C. Local head losses due to an expansion or a contraction

(1) The system illustrated in figure 4-24 shows an expansion in diameter between pipelines 1 and 2 (point X), and a contraction in diameter between pipelines 2 and 3 (point C). Expansions and contraction in pipelines produce local head losses.

(2) For a sudden expansion, the local head losses are calculated as:

$$(hl)_m = \frac{(V_u - V_d)^2}{2g} = \left[1 - \left(\frac{D_u}{D_d}\right)^2\right]^2 \frac{V_u^2}{2g} = \left[\left(\frac{D_u}{D_d}\right)^2 - 1\right]^2 \frac{V_d^2}{2g} \quad \text{(eq. 4-40)}$$

(3) The subscripts, u and d refer to the upstream and downstream pipelines. Notice that the head loss in a sudden expansion depends only on the diameters upstream and downstream of the expansion (D_u, D_d). Defining expansion loss coefficients as:

$$K_{xu} = \left[1 - \left(\frac{D_u}{D_d}\right)^2\right]^2 \quad \text{(eq. 4-41)}$$

or

$$K_{xd} = \left[\left(\frac{D_u}{D_d}\right)^2 - 1\right]^2 \quad \text{(eq. 4-42)}$$

(4) The general form of the local losses equation is written as:

$$(h_L) = K_{xu}\left(\frac{V_u^2}{2g}\right) = K_{xd}\left(\frac{V_d^2}{2g}\right) \quad \text{(eq. 4-43)}$$

(5) For a sudden contraction, as the one between pipes 2 and 3 in figure 4-24, the local head losses are given by the equation:

$$(h_L)_c = K_c\left(\frac{V_d^2}{2g}\right) \quad \text{(eq. 4-44)}$$

where K_c is the contraction loss coefficient given in figure 4-25, below, and V_d is the flow velocity in the downstream pipeline, i.e., the one with the smallest diameter. In figure 4-24, D_d and D_u are the diameters of the pipe downstream and upstream of the contraction. See Brater and King (1996) or Idelchik (1999) for more information on pipeline contraction losses.

Figure 4-25 Local Head Loss Coefficients for a Sudden Pipe Contraction

D_d/D_u	Kc	D_d/D_u	Kc
0.1	0.45	0.6	0.28
0.2	0.42	0.7	0.22
0.3	0.39	0.8	0.15
0.4	0.36	0.9	0.06
0.5	0.33	1.0	0.00

D. Energy equation for the three-pipeline system

(1) In writing the energy equation between reservoirs (A) and (B) in figure 4-24 friction head losses in each pipeline will be included, as well as entrance losses from reservoir (A) into pipeline 1, expansion losses at point (X), contraction losses at point (C), and discharge losses from pipeline 3 into reservoir (B). For the points (A) and (B) in the surface of the reservoirs, $V_A = V_B = 0$, $p_A = p_B = 0$, $z_A = z_B + \Delta H$. The expressions for the local losses are the following:

(i) Entrance loss from reservoir (A) to pipe 1:

$(h_L)_e = K_e \frac{V_1^2}{2g}$; with K_e=0.5. (See figure 4-11.)

(ii) Expansion from pipe 1 to pipe 2:

$(h_L)_x = K_{x1} \frac{V_1^2}{2g}$; with $K_{x1} = \left[1 - \left(\frac{D_u}{D_d}\right)^2\right]^2$

(iii) Contraction from pipe 2 to pipe 3:

$(h_L)_c = K_c \frac{V_3^2}{2g}$; with K_c from figure 4-25 as a function of the ratio of D_3/D_2.

(iv) Discharge loss from pipe 3 into reservoir (B):

$(h_L)_d = K_d \frac{V_3^2}{2g}$, with K_d=1.0 always.

(2) The friction losses in each pipe may be calculated using either the Darcy-Weisbach equation, the Manning's equation, or the Hazen-Williams formula. The total friction loss is the sum of each pipe's friction loss.

(3) The energy equation for the system of figure 4-24 is written as:

$$z_A + \frac{p_A}{\omega} + \frac{V_A^2}{2g} = z_B + \frac{p_B}{\omega} + \frac{V_B^2}{2g} + \sum h_f + \sum h_L$$

(4) From which it follows that:

$$z_B + \Delta H + \frac{0}{\omega} + \frac{0}{2g} = z_B + \frac{0}{\omega} + \frac{0}{2g} + \sum h_f + K_e \frac{V_1^2}{2g} + K_{x1} \frac{V_1^2}{2g} + K_c \frac{V_3^2}{2g} + K_d \frac{V_3^2}{2g}$$

or,

$$\Delta H = \sum h_f + \sum h_L = \sum h_f + K_e \frac{V_1^2}{2g} + K_{x1} \frac{V_1^2}{2g} + K_c \frac{V_3^2}{2g} + K_d \frac{V_3^2}{2g} \qquad \text{(eq. 4-45)}$$

(5) In most analyses of pipelines in series the diameters of the pipelines are known, and the problem consists in determining the discharge Q for a given available head ΔH, or vice versa. The three examples of pipes in series presented below include the effect of local losses.

(i) Example 4-22 – Pipes in series using the Darcy-Weisbach equation. The system of Figure 4-24 has pipelines lengths, diameters, and roughness values: $L_1 = 200$ *ft*, $L_2 = 400$ *ft*, $L_3 = 150$ *ft*, $D_1 = 1.00$ *ft*, $D_2 = 1.50$ *ft*, $D_3 = 1.00$ *ft*, $e_1 = 0.0001$*ft (concrete)*, $e_2 = 0.00004$ *ft (PVC)*, and $e_3 = 0.00025$ *ft (welded steel)*. The kinematic viscosity is $\nu = 1\text{x}10^{-5}$ ft^2/s. If the discharge $Q = 5$ *cfs*, determine the needed head $\triangle H$. If the available head between the reservoirs is $\triangle H = 30$ *ft*, determine the discharge through the pipes.

A spreadsheet application gives the results. If the discharge is 5 cfs, the needed head is coincidentally *5 ft*.

Determination of head or discharge

Three pipelines in series - Friction losses using the Darcy-Weisbach equation with function fSJ for the friction factor

Q(cfs):	Pipe No.	L(ft)	D(ft)	A(ft^2)	V(fps)	$V^2/2g$	Re	e(ft)	Local loss	Coeff, K	h_L (ft)	f	h_f (ft)
5.000	1	200	1.00	0.7854	6.37	0.629	6.37E+05	0.0001	Entrance	0.500	0.3147	0.0141	1.77
ΔH (ft):	2	400	1.50	1.7671	2.83	0.124	4.24E+05	0.00004	Expansion	0.309	0.1942	0.0139	0.46
4.991	3	150	1.00	0.7854	6.37	0.629	6.37E+05	0.00025	Contraction	0.220	0.1385	0.0157	1.48
ν(ft^2/s)	D_2/D_1:	1.50	Expansion						Discharge	1.000	0.6293		
1.00E-05	D_3/D_2:	0.67	Contraction		Equation error:	0.0000	Solve Q			Σh_L:	1.2767	Σh_f:	3.71
							Solve H		Total head loss,	$\Sigma h_L+\Sigma h_f$:	4.991		

If the available head is *30 ft*, the discharge, $Q = 12.58$ *cfs*.

Determination of head or discharge

Three pipelines in series - Friction losses using the Darcy-Weisbach equation with function fSJ for the friction factor

Q(cfs):	Pipe No.	L(ft)	D(ft)	A(ft^2)	V(fps)	$V^2/2g$	Re	e(ft)	Local loss	Coeff, K	h_L (ft)	f	h_f (ft)
12.584	1	200	1.00	0.7854	16.02	3.986	1.60E+06	0.0001	Entrance	0.500	1.9931	0.0130	10.40
ΔH (ft):	2	400	1.50	1.7671	7.12	0.787	1.07E+06	0.00004	Expansion	0.309	1.2303	0.0121	2.55
30.000	3	150	1.00	0.7854	16.02	3.986	1.60E+06	0.00025	Contraction	0.220	0.8770	0.0150	8.96
ν(ft^2/s)	D_2/D_1:	1.50	Expansion						Discharge	1.000	3.9862		
1.00E-05	D_3/D_2:	0.67	Contraction		Equation error:	0.0000	Solve Q			Σh_L:	8.0866	Σh_f:	21.91
							Solve H		Total head loss,	$\Sigma h_L+\Sigma h_f$:	30.000		

(ii) Example 4-23 - Pipes in series using the Manning's equation. Using the same data as example 4-22, the pipe lengths, diameters, and Manning's n coefficients are given by $L_1 = 200$ *ft*, $L_2 = 400$ *ft*, $L_3 = 150$ *ft*, $D_1 = 1.00$ *ft*, $D_2 = 1.50$ *ft*, $D_3 = 1.00$ *ft*, $n_1 = 0.012$*(concrete)*, $n_2 = 0.01$*(PVC)*, *and* $n_3 = 0.013$*(welded steel)*. If the discharge $Q = 5$ *cfs*, determine the needed head ΔH. If the available head between the reservoirs is $\Delta H = 30$ ft, determine the discharge through the pipes.

A spreadsheet application gives the results. If the discharge is *5 cfs*, the needed head is *8.12 ft*.

Determination of head or discharge

Three pipelines in series - Friction losses using Manning's equation for friction losses

Q(cfs):	Pipe No.	L(ft)	D(ft)	A(ft^2)	V(fps)	$V^2/2g$	n	Local loss	Coeff, K	h_L (ft)		h_f (ft)
5.000	1	200	1.00	0.7854	6.37	0.629	0.012	Entrance	0.500	0.3147	h_{f1} (ft):	3.36
ΔH (ft):	2	400	1.50	1.7671	2.83	0.124	0.010	Expansion	0.309	0.1942	h_{f2} (ft):	0.54
8.123	3	150	1.00	0.7854	6.37	0.629	0.013	Contraction	0.220	0.1385	h_{f3} (ft):	2.95
	D_2/D_1:	1.50	Expansion					Discharge	1.000	0.6293		
	D_3/D_2:	0.67	Contraction		Equation:	0.0000	Solve Q		Σh_L:	1.2767	Σh_f:	6.85
							Solve H	Total head loss,	$\Sigma h_L+\Sigma h_f$:	8.123		

If the available head is *30 ft*, the discharge, $Q = 9.61$ *cfs*.

Determination of head or discharge

Three pipelines in series - Friction losses using Manning's equation for friction losses

Q(cfs):	Pipe No.	L(ft)	D(ft)	A(ft^2)	V(fps)	$V^2/2g$	n	Local loss	Coeff, K	h_L (ft)		h_f (ft)
9.608	1	200	1.00	0.7854	12.23	2.324	0.012	Entrance	0.500	1.1620	h_{f1} (ft):	12.39
ΔH (ft):	2	400	1.50	1.7671	5.44	0.459	0.010	Expansion	0.309	0.7173	h_{f2} (ft):	1.98
30.000	3	150	1.00	0.7854	12.23	2.324	0.013	Contraction	0.220	0.5113	h_{f3} (ft):	10.91
	D_2/D_1:	1.50	Expansion					Discharge	1.000	2.3240		
	D_3/D_2:	0.67	Contraction		Equation:	0.0009	Solve Q		Σh_L:	4.7146	Σh_f:	25.28
							Solve H	Total head loss,	$\Sigma h_L+\Sigma h_f$:	29.999		

(iii) Example 4-24 - Pipes in series using the Hazen-Williams formula. Using the same data as examples 4-22 and 4-23, the pipe lengths, diameters, and Hazen-Williams coefficients are given by $L_1 = 200$ *ft*, $L_2 = 400$ *ft*, $L_3 = 150$ *ft*, $D_1 = 1.00$ *ft*, $D_2 = 1.50$ *ft*, $D_3 = 1.00$ *ft*, $CHW_1 = 120$*(concrete)*, $CHW_2 = 150$*(PVC)*, *and* $CHW_3 = 120$*(welded steel)*. If the discharge $Q = 5$ *cfs*, determine the needed head ΔH. If the available head between the reservoirs is $\Delta H = 30$ *ft*, determine the discharge through the pipes.

A spreadsheet application gives the results. If the discharge is *5 cfs*, the needed head is *6.37 ft*.

Determination of head or discharge
Three pipelines in series - Friction losses using Hazen-William's formula for friction losses

Q(cfs):	Pipe No.	L(ft)	D(ft)	A(ft^2)	V(fps)	$V^2/2g$	C_{HW}	Local loss	Coeff, K	h_L (ft)		h_f (ft)
5	1	200	1.00	0.7854	6.37	0.629	120	Entrance	0.500	0.3147	h_{f1} (ft):	2.63
ΔH (ft):	2	400	1.50	1.7671	2.83	0.124	150	Expansion	0.309	0.1942	h_{f2} (ft):	0.48
6.365	3	150	1.00	0.7854	6.37	0.629	120	Contraction	0.220	0.1385	h_{f3} (ft):	1.97
	D_2/D_1:	1.50	Expansion					Discharge	1.000	0.6293		
	D_3/D_2:	0.67	Contraction		Equation:	0.0000	Solve Q		Σh_L:	1.2767	Σh_f:	5.09
							Solve H	Total head loss,	$\Sigma h_L+\Sigma h_f$:	6.365		

If the available head is *30 ft*, the discharge, *Q = 11.39 cfs.*

Determination of head or discharge
Three pipelines in series - Friction losses using Hazen-William's formula for friction losses

Q(cfs):	Pipe No.	L(ft)	D(ft)	A(ft^2)	V(fps)	$V^2/2g$	C_{HW}	Local loss	Coeff, K	h_L (ft)		h_f (ft)
11.39078	1	200	1.00	0.7854	14.50	3.266	120	Entrance	0.500	1.6331	h_{f1} (ft):	12.09
ΔH (ft):	2	400	1.50	1.7671	6.45	0.645	150	Expansion	0.309	1.0081	h_{f2} (ft):	2.22
30.000	3	150	1.00	0.7854	14.50	3.266	120	Contraction	0.220	0.7186	h_{f3} (ft):	9.07
	D_2/D_1:	1.50	Expansion					Discharge	1.000	3.2662		
	D_3/D_2:	0.67	Contraction		Equation:	-0.0001	Solve Q		Σh_L:	6.6259	Σh_f:	23.37
							Solve H	Total head loss,	$\Sigma h_L+\Sigma h_f$:	30.000		

(6) Calculations were also made ignoring local losses in examples 4-22, 4-23, and 4-24. Figure 4-26 shows the effect of neglecting local losses on discharge.

Figure 4-26: Effect of neglecting local losses on an example pipeline system

Friction loss method	Q (cfs) including local losses	Q(cfs) neglecting local losses	Percentage difference
Darcy-Weisbach	12.58	14.79	17.56%
Manning's	9.61	10.47	8.95%
Hazen-Williams	11.39	13.03	14.40%

(7) The percentage differences in example 4-23 are too large to neglect local loss. **A generally accepted criterion is that local losses should be included in a pipe design analysis if the local losses exceed 5% of the total head loss.**

E. Pipelines in Parallel

(1) Figure 4-27 illustrates reservoirs connected by three different parallel pipelines conducting water between the reservoirs. An energy equation can be written separately for each of the pipelines. Including entrance (h_e), discharge (exit) (h_d), and friction (h_f) losses for each pipeline, the corresponding energy equations can be written as:

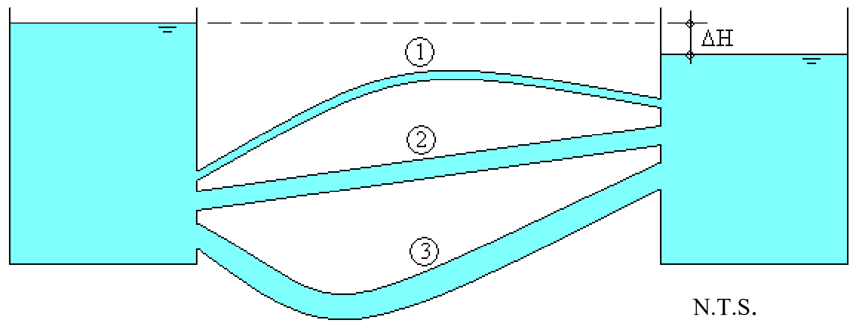
$$\Delta H = (h_e)_1 + (h_d)_1 + (h_f)_1$$

$$\Delta H = (h_e)_2 + (h_d)_2 + (h_f)_2$$

$$\Delta H = (h_e)_3 + (h_d)_3 + (h_f)_3 \qquad \text{(eq. 4-46)}$$

Where the subscripts (1), (2), and (3) refer to each of the pipelines as illustrated in figure 4-27.

Figure 4-27: Three pipelines in parallel connecting two reservoirs.



(2) A simplified analysis in which the local losses are neglected simplifies equation 4-46 to:

$$\Delta H = (h_f)_1 = (h_f)_2 = (h_f)_3 \qquad \text{(eq. 4-47)}$$

(3) A typical problem of parallel pipes consists in determining the total discharge Q delivered from the upstream to the downstream reservoir given the available head ΔH between the reservoirs. Given ΔH, the available head, the friction loss, h_f, is known. Then using either the Darcy-Weisbach equation (requires spreadsheet application for efficient calculation), the Manning's equation, or the Hazen-Williams formula, the individual discharges *Q1, Q2*, and *Q3* can be obtained. The individual discharges are summed to obtain the total discharge. The examples using Manning's equation and the Hazen-Williams formula presented below neglect local losses.

(i) Example 4-25 - Pipes in parallel using the Manning's equation. The system of figure 4-27 has pipelines lengths, diameters, and Manning's n coefficients: $L_1 = 200\ ft$, $L_2 = 400\ ft$, $L_3 = 150\ ft$, $D_1 = 1.00\ ft$, $D_2 = 1.50\ ft$, $D_3 = 1.00\ ft$, $n_1 = 0.012$, $n_2 = 0.018$, *and* $n_3 = 0.010$. If the available head is $\Delta H = 30\ ft$, determine the total discharge between the reservoirs.

- The head loss through each pipeline is 30 ft. Manning's equation (equation 4-9) is used to calculate the individual discharges:

$$Q_1 = 0.4632\frac{D_1^{8/3}}{n_1}\sqrt{\frac{(h_f)_1}{L_1}} = 0.4632\frac{1.0^{8/3}}{0.012}\sqrt{\frac{30}{200}} = 14.95cfs$$

$$Q_2 = 0.4632\frac{D_2^{8/3}}{n_2}\sqrt{\frac{(h_f)_2}{L_2}} = 0.4632\frac{1.5^{8/3}}{0.018}\sqrt{\frac{30}{400}} = 20.78cfs$$

$$Q_3 = 0.4632\frac{D_3^{8/3}}{n_3}\sqrt{\frac{(h_f)_3}{L_3}} = 0.4632\frac{1.0^{8/3}}{0.010}\sqrt{\frac{30}{150}} = 20.71cfs$$

- The total discharge is Q = 14.95 cfs + 20.78 cfs + 20.71 cfs = 56.44 cfs.
- Discharge calculations may also be made for individual pipes with Pipe Flow, NRCS Hydraulics Formula program, which uses Manning's equation for friction losses. The discharges calculated will be slightly less than those shown above because Pipe Flow, USDA-NRCS Hydraulics Formula automatically accounts for a local loss at the pipe exit ($K_d = 1.0$).

(ii) Example 4-26 - Pipes in parallel using the Hazen-Williams formula. The system of figure 4-27 has pipelines lengths, diameters, and Hazen-Williams coefficients:

$L_1 = 200$ ft, $L_2 = 400$ ft, $L_3 = 150$ ft, $D_1 = 1.00$ ft, $D_2 = 1.50$ ft, $D_3 = 1.00$ ft, $C_{HW1} = 100$, $C_{HW2} = 80$, and *$C_{HW3} = 120$*. If the available head is $\Delta H = 30$ *ft*, determine the total discharge between the reservoirs.

- The head loss through each pipeline is 30ft. Using (equation 4-26), the individual pipe discharges are calculated as follows:

$$Q_1 = 0.432C_{HW1}D_1^{2.63}\left(\frac{(h_f)_1}{L_1}\right)^{0.54} = 0.432(100)(1.00)^{2.63}\left(\frac{30}{200}\right)^{0.54}$$
$$= 15.50cfs$$

$$Q_2 = 0.432C_{HW2}D_2^{2.63}\left(\frac{(h_f)_2}{L_2}\right)^{0.54} = 0.432(80)(1.50)^{2.63}\left(\frac{30}{400}\right)^{0.54}$$
$$= 24.79cfs$$

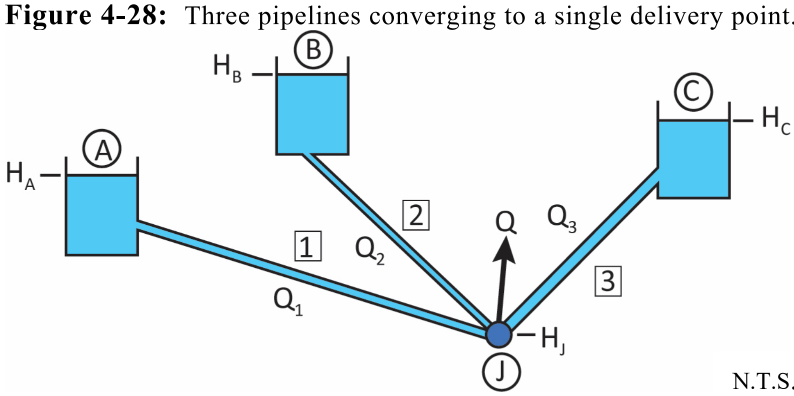
$$Q_3 = 0.432C_{HW3}D_3^{2.63}\left(\frac{(h_f)_3}{L_3}\right)^{0.54} = 0.432(120)(1.00)^{2.63}\left(\frac{30}{150}\right)^{0.54}$$
$$= 21.74cfs$$

- The total discharge is Q = 15.50 cfs + 24.79 cfs + 21.74 cfs = 62.03 cfs.

F. Pipelines Converging at a Single Point

(1) Figure 4-28 shows a pipeline system consisting of three pipelines connecting reservoirs A, B, and C to a single delivery point J (for Junction), where a total discharge Q is to be delivered. Shown in the figure are also the elevations of the free surface of the reservoirs, namely, H_A, H_B, and H_C, as well as the piezometric head elevation of the junction point, H_J.

Figure 4-28: Three pipelines converging to a single delivery point.



(2) Neglecting local or minor losses, the energy equations for the three pipelines shown above are written as:

$$H_A - H_J = (h_f)_1$$
$$H_B - H_J = (h_f)_2$$
$$H_C - H_J = (h_f)_3 \qquad \text{(eqs. 4-48)}$$

Where $(h_f)_1$, $(h_f)_2$, and $(h_f)_3$ are the friction losses in pipelines [1], [2], and [3], respectively.

(3) A typical problem for the system illustrated in figure 4-28 consists in determining the discharge Q given the elevations of the free surfaces in reservoirs A, B, and C (H_A, H_B, H_C) and that of point J (H_J). The individual pipe discharges are summed to obtain the total. The examples using Manning's equation and the Hazen-Williams formula presented below neglect local losses.

(i) Example 4-27, Converging pipelines using Manning's equation. The system of figure 4-28 has pipelines lengths, diameters, and Manning's n coefficients: $L_1 = 200\ ft$, $L_2 = 400\ ft$, $L_3 = 150\ ft$, $D_1 = 1.00\ ft$, $D_2 = 1.50\ ft$, $D_3 = 1.00\ ft$, $n_1 = 0.012$, $n_2 = 0.018$, and $n_3 = 0.010$. The elevations of interest are $H_A = 280\ ft$, $H_B = 290\ ft$, $H_C = 310\ ft$, and $H_J = 250\ ft$.

- Determine the total discharge delivered to junction J. The energy losses in each pipeline are calculated as follows:

$(h_f)_1 = H_A - H_J = 280\ ft - 250\ ft = 30\ ft,$

$(h_f)_2 = H_B - H_J = 290\ ft - 250\ ft = 40\ ft,$

$(h_f)_3 = H_C - H_J = 310\ ft - 250\ ft = 60\ ft.$

- As in example 4-25, Manning's equation is used to calculate the individual discharges as follows:

$$Q_1 = 0.4632\frac{D_1^{8/3}}{n_1}\sqrt{\frac{(h_f)_1}{L_1}} = 0.4632\frac{1.0^{8/3}}{0.012}\sqrt{\frac{30}{200}} = 14.95cfs$$

$$Q_2 = 0.4632\frac{D_2^{8/3}}{n_2}\sqrt{\frac{(h_f)_2}{L_2}} = 0.4632\frac{1.5^{8/3}}{0.018}\sqrt{\frac{40}{400}} = 23.99cfs$$

$$Q_3 = 0.4632\frac{D_3^{8/3}}{n_3}\sqrt{\frac{(h_f)_3}{L_3}} = 0.4632\frac{1.0^{8/3}}{0.010}\sqrt{\frac{60}{150}} = 29.3cfs$$

- The total discharge is Q = 14.95 cfs + 23.99 cfs + 29.3 cfs = 68.24 cfs
- As in the previous parallel pipes example 4-25, using Manning's formula for friction losses, discharge calculations may also be made for individual pipes with Pipe Flow, USDA-NRCS Hydraulics Formula program. The discharges calculated will be slightly less than those shown above because Pipe Flow, USDA-NRCS Hydraulics Formula automatically accounts for a local loss at the pipe exit ($K_d = 1.0$).

(ii) Example 4-28, Converging pipelines using Hazen-William's formula. The system of figure 4-28 has pipelines lengths, diameters, and Hazen-William's coefficients: $L_1 = 200\ ft$, $L_2 = 400\ ft$, $L_3 = 150\ ft$, $D_1 = 1.00\ ft$, $D_2 = 1.50\ ft$, $D_3 = 1.00\ ft$, $CHW_1 = 100$, $CHW_2 = 80$, and $CHW_3 = 120$. The elevations of interest are $H_A = 280\ ft$, $H_B = 290\ ft$, $H_C = 310\ ft$, and $H_J = 250\ ft$. Determine the total discharge delivered to junction J.

- The energy losses in each pipeline are calculated as follows:

$(h_f)_1 = H_A - H_J = 280\ ft - 250\ ft = 30\ ft,$

$(h_f)_2 = H_B - H_J = 290\ ft - 250\ ft = 40\ ft,$

$(h_f)_3 = H_C - H_J = 310\ ft - 250\ ft = 60\ ft.$

- As in example 4-25, the Hazen-Williams formula is used to calculate the individual pipe discharges as follows:

$$Q_1 = 0.432C_{HW1}D_1^{2.63}\left(\frac{(h_f)_1}{L_1}\right)^{0.54} = 0.432(100)(1.00)^{2.63}\left(\frac{30}{200}\right)^{0.54}$$
$$= 15.50cfs$$
$$Q_2 = 0.432C_{HW2}D_2^{2.63}\left(\frac{(h_f)_2}{L_2}\right)^{0.54} = 0.432(80)(1.50)^{2.63}\left(\frac{40}{400}\right)^{0.54}$$
$$= 28.95cfs$$
$$Q_3 = 0.432C_{HW3}D_3^{2.63}\left(\frac{(h_f)_3}{L_3}\right)^{0.54} = 0.432(120)(1.00)^{2.63}\left(\frac{60}{150}\right)^{0.54}$$
$$= 31.60cfs$$

- The total discharge is Q = 15.50 cfs + 28.95 cfs + 31.60 cfs = 76.05 cfs.

G. Pipeline Networks

(1) Pipe networks are utilized to supply water for urban and rural domestic and industrial uses. Networks are also used by irrigation districts and occasionally in livestock watering systems. A pipe network is simply a collection of pipes connected in a given geometric pattern and having at least one supply point and one delivery or consumption point. The points where individual pipelines join each other are known as junctions or nodes. Pipe networks are schematized as geometric constructs of lines representing the component pipes.

(2) The main purpose of analyzing a given pipe network is to determine the discharge in each pipe and the piezometric head at each node. These results can be used then to verify that certain design guidelines, such as minimum flow velocities and minimum junction pressures, are satisfied. The analysis of pipe networks requires the simultaneous solution of several mathematical equations that represent equations of continuity at the nodes, and equations of energy around closed loops and/or pseudo-loops.

(3) Analyses of simple pipe networks may be performed with a spreadsheet application, but spreadsheet use would become very tedious as the number of pipelines and loops increase. There are several publicly available and commercial software that can be used for network solution in a more efficient manner, such as the U.S. Environmental Protection Agency's EPANET software.

634.0407 EPANET

A. A software program available for performing pipeline hydraulic calculations is called EPANET. Developed and supported by the US Environmental Protection Agency, this program can be used for full pipe design. It is available online and an approved CCE version can be loaded on NRCS computers (at this time it must be installed by an IT specialist). It is free to use, and training can be self-based through tutorials or the web site.

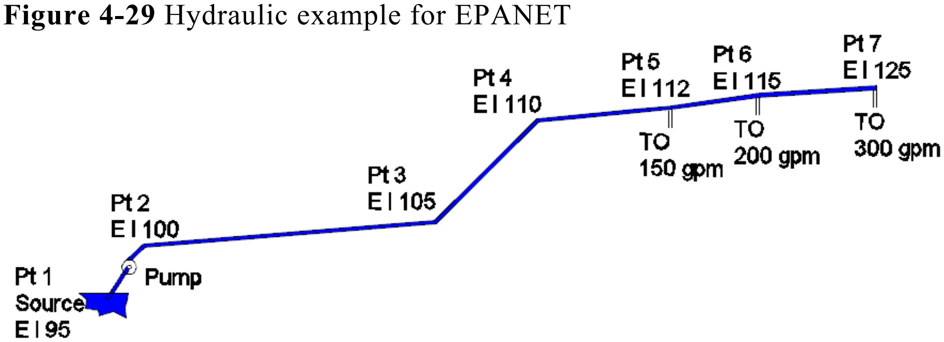
B. This program uses pipes, junctions, pumps, valves, storage tanks and reservoirs as the basis for design. It has the capability of carry out simulations over defined time frames computing flow, velocity, source, chemical tracking and water age. The pipe system can be simply a single pipeline or more complex using multiple pumps, water sources, networks and loops. Units can be English or metric as well as the results. Maps (as either *.bmp, *.emf, or

*.wmf files) can be used as backdrops for lay out, which can be used to scale the system or enter points and define the layout as the system is developed. There is no limit to size of the network being analyzed and calculations can be made using either Hazen-Williams, Darcy-Weisbach or Chezy-Mannings to compute friction.

C. Valves, pumps and junctions (or turnouts) can be set to turn on or off at different times of the day. Pumps can be entered with one or more flow/head points on the curve or the pump characteristics can be estimated using a one-point curve built by EPANET. Modeling of the flow, demands, quality or age can be estimated and colorized and seen as flow changes. There is a tutorial available that has step by step instructions to build a few different types of scenarios and explanations on how to use them. This program is very easy to use and quick to enter data to get hydraulic flow information.

(1) Example 4-29 Use of EPANET for a single pipeline. For a pumped pipeline from a water source at point 1 (reservoir) to 3 turnouts across a field using EPANET in the figure 4-29.

Figure 4-29 Hydraulic example for EPANET



(i) After opening the software, it is easy to set up the initial defaults. From the menu choose the View, Options and select the notation, flow arrows, symbols to use. Under Notation checking the Display Node Values, Display Link ID's and Display Link Values turns on the values to the screen for the user to see. For more complex systems with loops or multiple pumps or inlets, the flow arrows are good to see how the system is operating. All these settings can be selected at any time during the program operation.

(ii) Next, from the Project menu, choose Defaults, and under the Properties and the Hydraulics tabs to choose the hydraulic calculation method, the typical pipe size, length, and friction loss value. It is possible to have the software scale the dimensions based on an inserted background photo or you can adjust the pipe length at any time under the properties list for each pipe. You may also edit the view of the layout by changing the nodes under the view. This makes it easier to see water source, direction, which valves are open or closed and where possible design concerns are located.

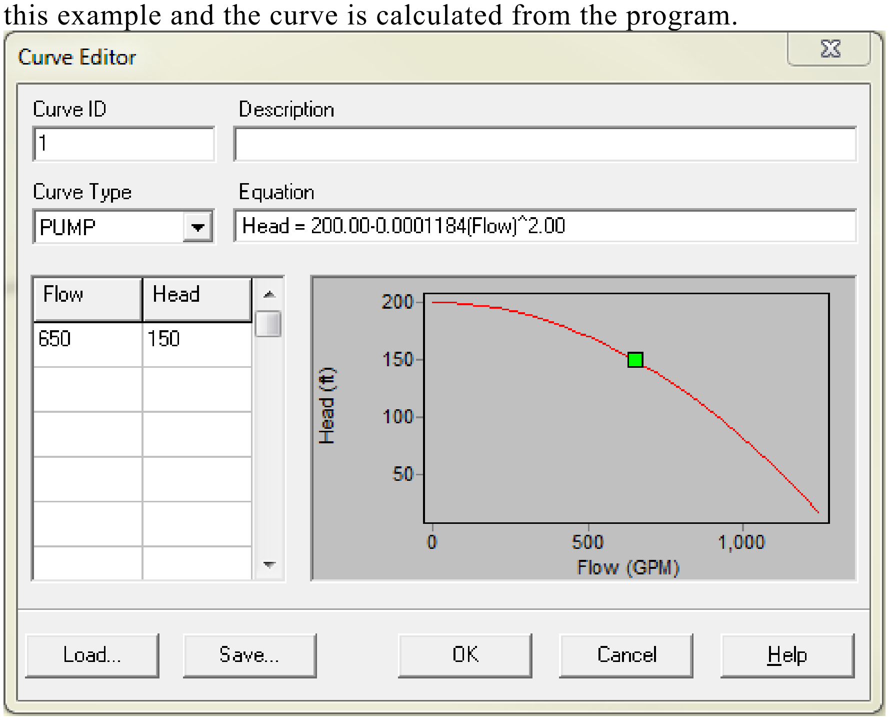
(iii) For this example, enter a reservoir as a water source (Pt 1) and nodes for each of the above points (Pt 2-7) into EPANET. The screen shot of what this could look like is shown above with flows in gpm at each of the turnout nodes (Pts. 5, 6 and 7). After entering the reservoir and nodes for the system, select the pump and connect it to the reservoir and to point 2. Then connect the pipes from node to node. Edit all the information in the properties table for each node elevation and pipe diameter, friction and length. All elevations are as shown. Use 500 ft. of 10" diameter pipe from the pump outlet at point 2 to the first turnout at point 5. From point 5 to the next turnout at point 6 use 200 ft. of 8" diameter pipe and from the second turnout (Pt 6) use 300 ft. of 6” diameter pipe the end (Pt 7), calculate the pump head needed in feet to get 45 psi of pressure at the third turnout (Pt 7).

(iv) With the pump located between the reservoir and point 2 it is connected between these two points and just ties into the line and does not take up length in the system. To get the system to run, at least a one-point pump curve (Q in gpm and H in ft) must be entered. This is done in two parts. First a pump curve needs to be entered under curves on the right under Data on the Browser menu. Next enter the pump curve number into the pump properties table. For this example, we want to produce a pressure of (45 psi) at the end of the line. To get this pressure, we can do trial and error for the pump. For the first estimate try a flow of 650 gpm and head 150 ft entered. The figure 4-30 is of the pump 1 Curve editor.

(v) Edits can be made by either selecting an item or by selecting from the Browser, Data menu on the right. After entering the correct friction coefficient, friction loss equation to use, pipe size and lengths, select the run button (lightning bolt) to see if it works or if there is an error. Errors usually come from pipes not connected to nodes or junctions, pump curves not entered correctly or a possible hydraulic issue. Support and troubleshooting can be found by using the support contained in the software.

(vi) The pump flow rate may show up as negative since it is losing that amount of water, as will the reservoir.

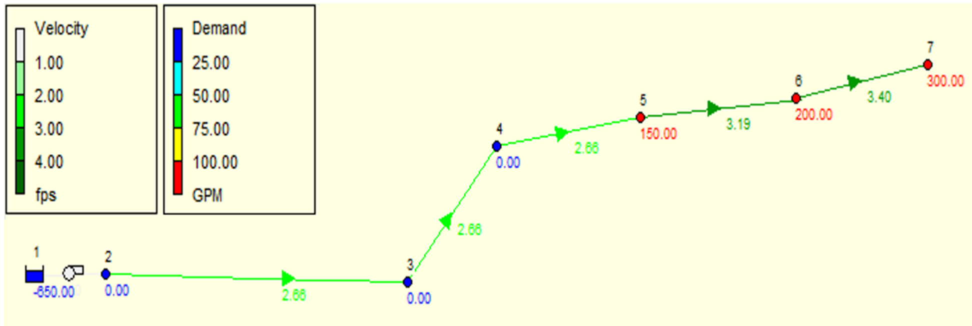
Figure 4-30: Pump Curve editor screen shot. Only one point is entered for this example and the curve is calculated from the program.



(vii) Velocity and demand graphics are included for the example below, which can be turned on from the Browser, Map menu. This color codes the EPANET results for each section of pipe and locations of turnouts with flow amounts.

(viii) Figure 4-31 shows an EPANET screen shot with flow direction and velocity shown next to the direction arrows, turnout flows in red in gpm, plus reservoir location and pipes. The pump is located between points 1 and 2.

Figure 4-31: Screen shot of EPANET output



(ix) To have 45 psi (44.9 psi in circle below) at the end of the line (Point 7) the pump would need to have an outlet head of 150 ft. (62.83 psi) as shown at Junction 2 in figure 4-28.

(2) The main disadvantage of EPANET is the lack of reports that are user friendly to NRCS. Reviewing the file in EPANET is the best way to see what is going on. Reports do not show all the details that use common NRCS nomenclature (see fig 4-32 and 4-33).

Figure 4-32: Output table of Junction information for EPANET example

Network Table - Nodes

Node ID	Demand GPM	Head ft	Pressure psi
Junc 2	0.00	245.00	62.83
Junc 3	0.00	242.87	59.74
Junc 4	0.00	240.74	56.65
Junc 5	150.00	238.62	54.86
Junc 6	200.00	234.73	51.88
Junc 7	300.00	228.61	44.90
Resvr 1	-650.00	95.00	0.00

Figure 4-33: Output table of links (Pipelines) for EPANET example.

Network Table - Links

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Friction Factor
Pipe 2	650.00	2.66	2.13	0.016
Pipe 3	650.00	2.66	2.13	0.016
Pipe 4	650.00	2.66	2.13	0.016
Pipe 5	500.00	3.19	3.88	0.016
Pipe 6	300.00	3.40	6.12	0.017
Pump 1	650.00	0.00	-150.00	0.000

(3) What they do show is the operating characteristics of the system and how it will perform. The reports can be used to build a design and include the appropriate data.

D. Friction, velocities, pressure, flows can be found in the EPANET program for each node and line. This is useful and can be transferred to other programs for graphing or documentation. Printing reports and explaining system hydraulics by the user may be the best way to show results. Another option with the program is to inject chemicals (chemigation and fertigation) and look at aging and distribution with varying flow parameters. Concentrations, timing and direction can be quickly shown in real time system operation.

E. EPANET can be used with multiple pumps, loops and varying flow requirements. It is easier to use than most spreadsheets, more flexible and files that can be transferred to others who can easily view and use.

634.0408 WaterCAD

A. Bentley® WaterCAD is a commercial software program written to do hydraulic calculations in piping systems. This tool has been available through a limited number of licenses to NRCS staff, or it can be purchased (see disclaimer page 634-4.i.)

B. This program is more robust than EPANET and provides added flexibility in hydraulic analysis. There are examples and lessons on how to use the program as a part of the software package. With more ability in a program requires more challenges for using. The following example is given to provide a bit of insight on its use.

(1) Example 4-30 Use of WaterCAD for a single pipeline. In example 4-29, inputs from the EPANET section will be used to show the WaterCAD program. The first thing after opening the tool is to set up the system parameters with Tools, Options that are appropriate. The Drawing tab in the Options table is also a good item to check to set the scale and text alignment. Lastly the Labeling tab in this menu has the names of the different codes for each of the items that can be inserted such as pipes, valves, and pumps.

(2) One option to make the project easier to follow is to insert a background image. This can be from many different formats including jpeg, dxf, shp, jif, or sid. There can also be many layers that can be turned on and off as needed. Multiple images can be placed side by side to sequence a series of views, such as for a long pipeline. Switching images off and on can help in layout and in building a longer plan view of a system into a complete file. Look under the View, Background Layer to start a new file and add to your background drawing. The scale of the background can be changed by adjusting the XY coordinates of the image. Be sure and adjust the image the same in both dimensions. It can be moved by adjusting the 0:0 location.

(3) Next look at the inputs for the example. In this case the drawing from the EPANET example above will be used as the layout. Building this data into WaterCAD is like inputting into EPANET. Using the different symbols on the left of the screen can guide you to the type of item to select. First start with the reservoir , which is a quarter of the way down from the top of the screen. After clicking on it, place it on the screen where it belongs. Every time you left click on the mouse a reservoir will be placed until you either choose a new item or select the edit arrow at the top of the list. To select and edit an item after it is on the screen, left click and it will turn red and you can delete, edit or move the item.

(4) Place a pump next to the reservoir and connect it to the reservoir with a pipe. After that either enter junctions or pipes. Using the pipe button, will include junctions, if needed. For example, when entering a pipe to connect the reservoir to the pump a junction will be shown. By clicking on the reservoir, the line will show a junction which can be ignored and clicking on the pump will link the pipe. This way the suction side of the pump can be any length and size, which is not as easy to do in EPANET.

(5) While still in the pipe mode and connected to the pump, move to the right and for every mouse click a pipe and junction will be placed. This can continue for as many lengths as necessary. The length will be scaled based on the distance you move. Having a background image will provide a good basis and scale for the layout. An example of a junction could be a change in grade, T in the pipe, outlet for a sprinkler, valve or turnout.

(6) After the example has been entered and it looks like the EPANET example, it is necessary to edit the information to the actual conditions. To do this simply click on the item you want to edit, and a properties screen will open. This is where pipe, junctions, reservoirs, pumps can be edited. You can move across the system and choose items in whatever order you want. Pipe type, size, roughness factors and length may be entered. The junctions may also be edited by clicking on the points and changing data in the properties table. Information that needs changing includes elevation, flow out of a point, or valve information.

(7) This program is much more robust than described here, this is just a quick overview to introduce the subject.

634.0409 Appurtenances in Pipelines and Networks

A. In this section many appurtenances commonly used in pipelines and pipe networks are described.

B. Air Vacuum and Release Valves

(1) Entry and entrapment of air in a pipeline during filling and operation of a pipeline can cause development of air pockets. Air pockets tend to restrict and reduce flow or increase pumping costs in pipelines. Air pockets that build-up in pumped pipelines can reduce flows by 5 to 15%. In low-head, gravity-driven flow, air pocket build-ups can reduce flow up to 50%. On the other hand, a lack of air entry can cause pipe collapse if a vacuum develops behind flowing water as a pipeline empties. Air valves are of four types:

(2) Air/vacuum relief valves, also known as kinetic air valves, large orifice air valves, vacuum breakers, low-pressure air valves, and air relief (not release) valves. Large volumes of air are discharged before a pipeline is pressurized, especially at pipe filling. Large quantities of air are admitted when the pipe drains and at the appearance of water column separation.

(3) Continuous acting air and vacuum valves, also known as double orifice air valves, or combination air valves, fill the functions of the air/vacuum relief valves and air release valves, admitting and releasing large quantities of air when needed, and releasing air continuously when the lines are pressurized (figure 4-34).

Figure 4-34: Continuous acting air valve

(4) Air release valves are also known as automatic air valves, small orifice air valves, continuous acting air vents, and pressure air valves. These vents continue to discharge air, usually in smaller quantities, after the air vacuum valves close, as the line is pressurized.

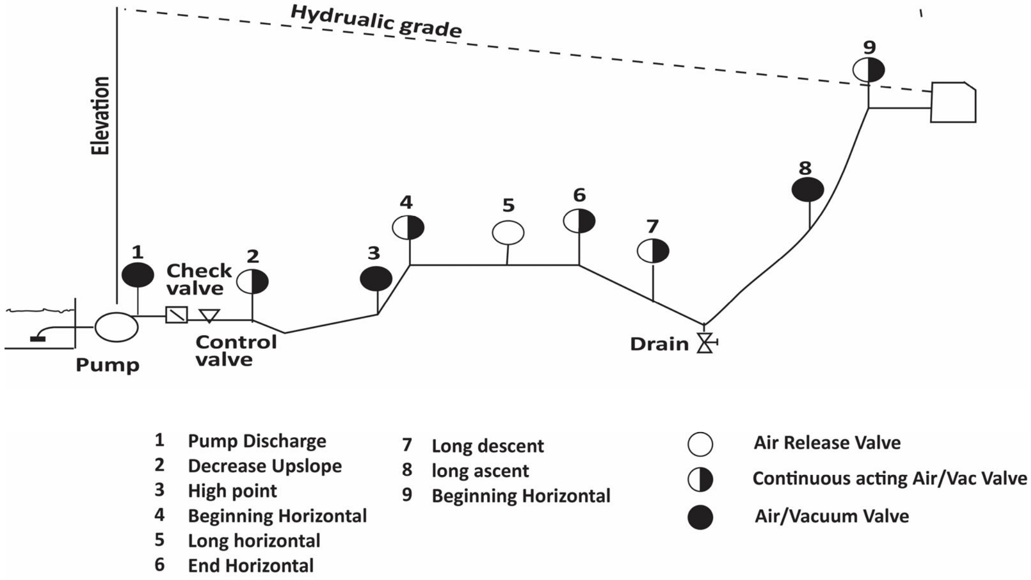
(5) Vacuum relief valves are large orifice valves for vacuum relief only. These valves allow air to enter the pipeline when draining.

(6) Guidelines for placement of air valves:

(i) Pipelines in general (figure 4-35) - It is important to establish a smooth pipeline grade and not follow the terrain or an excessive number of air valves will be needed. The designer must balance the cost of air valve locations with the cost of additional excavation. The high points and grade changes that are less than 1 pipe diameter are typically ignored because the flow will flush accumulated air downstream. Air valves should be placed at the following locations:

- Place continuous-acting air and vacuum valves at all high points. The large orifice is necessary to discharge air at pipe fill-up, and for air intake during pipe drainage or when the pipe bursts. The small orifice releases air that continually collects at the high points.
- On long horizontal runs air release valve should be placed at 1250-ft to 2500-ft (380 m to 760 m) intervals on pipelines. Best practice is to alternate air release and combination air valves
- Long descents: combination air valve at 1250-ft to 2500-ft (380 m to 760 m) intervals.
- Long ascents: air/vacuum valve at 1250-ft to 2500-ft (380-m to 760-m) intervals.
- Decrease in an up slope: air/vacuum valve.
- Increase in a down slope: combination air valve.
- Locate an air release valve at the end of the line

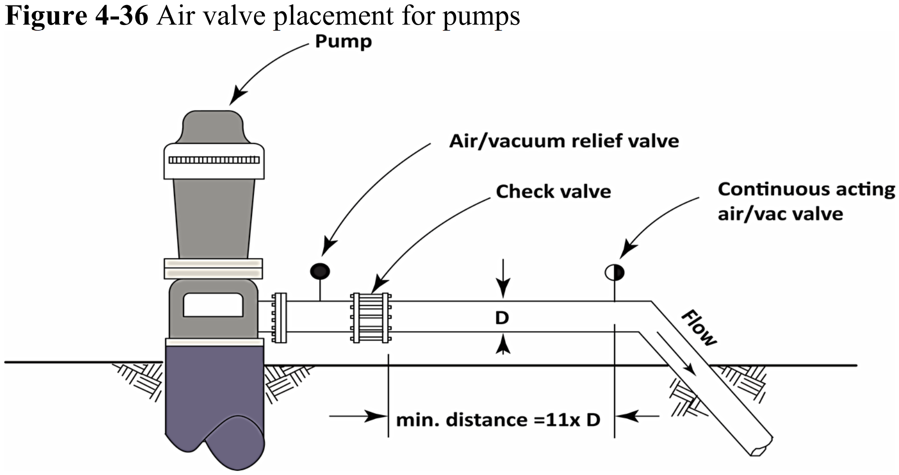
Figure 4-35 Air valve placement a long a pipeline



(ii) Locate an air-and-vacuum or vacuum valves to allow air to enter behind the water as a pipeline is emptied. Vacuum valves are generally not needed on pipelines of less than 3-inch diameter for protection from collapse; however, they may be needed to insure complete drainage of the line. On the other hand, vacuum valves are important on low head plastic irrigation pipe and large diameter steel pipe (>24”) to prevent collapse of the line.

(iii) Air is usually present in pumps and in suction pipes. When pumps are started and during operation, air will continue to enter through the pump, fittings, and related accessories. A combination air/vacuum relief and continuous acting air valve should be placed directly after the pump and before the pump check valve (figure 4-36).

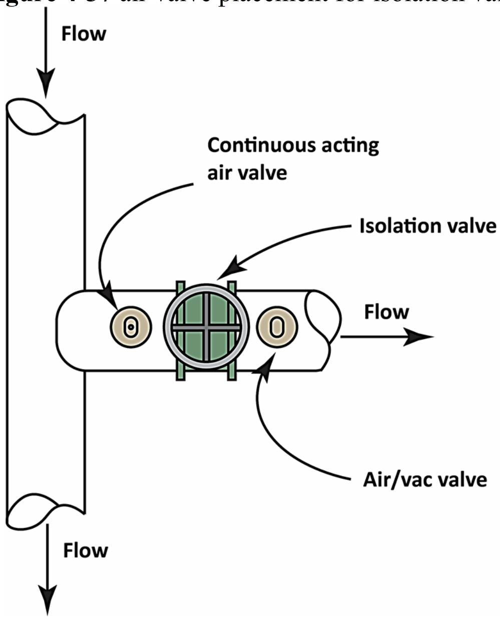
Figure 4-36 Air valve placement for pumps



(iv) For isolating valves, a continuous acting air valve should be placed upstream of the valve to release air pressure when the valve is closed and protect from air-bubble induced cavitation. It also provides slow end-of-pipe air release for air trapped by the closed valve when filling the pipe (figure 4-37).

(v) Downstream of an isolation valve, a continuous acting air/vacuum relief is needed to prevent collapse during draining of the pipe and to aid in filling operation when the valve is open (figure 4-37).

Figure 4-37 air valve placement for isolation valves



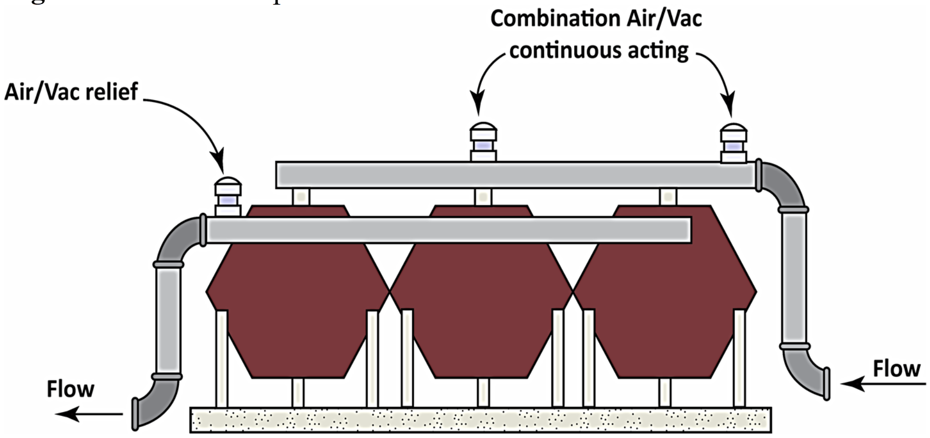
(vi) Air valve placement at flow meters or metering valves

- One of the most important places to locate air valve in irrigation systems is before water meters and automatic metering valves. When air rushes through a water meter or the metering component of an automatic metering valve, it spins the velocity measuring spinning accelerator with great speed, registering very high volumes of flow. The metering instruments cannot distinguish between air and water. The crops may only receive a small portion of the water that the meters registered and supposedly delivered. Thus, creating severe, often irreversible damage to crops.
- When water lubricates and cools the spinning parts of flow meter. Air provides little lubrication and the spinning part tend to heat. If the spinning element is made of plastic, it may even melt. Placing an air vent before a metering device protects it from this phenomenon.

(vii) Filter heads

- Combination Air/Vacuum Relief and Continuous Acting Air valves should be installed at the inflow side of filter heads to prevent air from entering the filters
- On top of the filters to release air from within and to enable draining and backwashing.
- At the outflow side to release remaining air and to prevent vacuum conditions and suction of filter media out of the filter.

Figure 4-38 Air valve placement on filter stations



(viii) Vacuum protection in microirrigation especially subsurface drip irrigation, is essential, even at very low negative pressure, back siphoning of dirt into the emitters, and causing other damage to pipe and accessories. Air intake should be equal to maximum drainage flow rate (maximum slope). To prevent suction even at low negative pressures, air intake should be determined at low negative pressure, like 1.45 psi (0.1 atmosphere).

Figure 4-39 Air valve placement on a microirrigation manifold

(ix) The need for air valves should also be evaluated in these other common areas of air accumulation

- Road crossings where larger diameter pipes dip down and then rise up again in sharp slopes. If the crossing is wide and/or deep, air valves should be installed on the top elbows at both sides of the crossing. If the crossing is narrow and shallow, placing one combination air vent on the upstream top elbow may be sufficient.
- Areas of pressure change such pressure reducing valves or sudden enlargements can cause turbulence and allow air to be released from the water to the pipe. A combination Air/Vacuum relief and continuous acting air valve should be installed very close and downstream from the accessory or disturbance.
- Control heads and other areas of disturbance or air accumulation.

(x) Avoid oversizing air release valves to lessen the possibility of water hammer. See section 634.0410 for more information on water hammer

(7) Valve sizes may be determined for a pipeline by calculating: (1) flow for both filling and emptying the pipeline, (2) pipe collapse pressure, and (3) valve intake or discharge at a prescribed pressure across the valve as illustrated in example 4-31.

(i) Example 4-31 - Air valve sizing. Given: 18” diameter, 80 psi, SDR 51, PVC pipe, on a slope of 0.05 ft/ft. Note that SDR is the pipe outside diameter divided by the wall thickness.

- Determine flow for draining the line using the Hazen-Williams formula (equation 4-26). C_{HW} for PVC pipe is 150:

$$Q = 0.432 \times 150 \times 1.5^{2.63} \times 0.05^{0.54} = 37.34cfs$$

- Air vent valves must re-enter air at the same rate as the calculated water flow of 37.34 cfs.
- Determine pipe collapse pressure using equation for PVC pipe given in ASTM in this case DD2241:

$$P_c = \frac{2E}{(1-v^2)} \times \frac{1}{(\mathrm{SDR})[(\mathrm{SDR}-1)]^2}$$

where: Pc = collapse pressure of PVC pipe (psi)
E = Young’s modulus of elasticity (400,000 psi)
ν = Poisson’s ratio (0.33)
SDR= Standard Dimension Ratio (d_o/t)
t = wall thickness (in.)
d_o= outside diameter of pipe (in.)

$$P_c = \frac{2(400{,}000)}{(1-0.33^2)} \times \frac{1}{(51)[(51-1)]^2} = 7\ psi$$

- Use the collapse pressure determined above or 5 psi, whichever is lower (standard design practice).
- Determine orifice diameter, using the orifice flow equation (equation 4-49):

$$Q = C_d A\sqrt{2gh} \qquad \text{(eq. 4-49)}$$

where: C_d = flow coefficient = 0.6
A = orifice area
ω_{air} = specific weight of air =0.0764 lbs/ft^3
P_c = (5 lbs/in^2) (144 in^2/ ft^2) = 720 psf
h = P_c/ω_{air}

- Rearranging the orifice formula,

$$A = \frac{37.34}{0.6\sqrt{2(32.2)(720/0.0764)}} = 0.079ft^2$$

$$A = 0.08ft^2 \times 144\frac{in^2}{ft^2} = 11.37in^2$$

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 11.37}{\pi}} = 3.80\ inches, orifice\ diameter$$

- Determine the size of orifice required when filling the pipe. When filling a line, good practice is not to exceed *1 fps* (note: not all systems provide that much control of filling velocity, see next part of this example). Using a filling velocity of *1 fps* and the *18inch* diameter pipe:

  $$Q = AV = (\pi/4) \times (1.5\text{ft})^2 \times 1\, ft/s = 1.76cfs$$

  Therefore, air must be discharged from the line at a rate of *1.76 cfs*

- Size the orifice for *2 psi* pressure across the valve (standard design practice). Using the orifice formula, as before:

  $$P_c = (2lbs/in^2)(144in^2/ft^2) = 288\, psf$$

  $$A = \frac{1.76}{0.6\sqrt{2(32.2)(288/0.0764)}} = 0.006ft^2$$

  $$= 0.004ft^2 * 144\, in^2/ft^2 = 0.85\, in^2$$

  $$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4\times 0.85}{\pi}} = 1.04\, in \text{ orifice diameter.}$$

- Alternative determination of orifice size required when filling the pipe. When filling a line, with an electric powered pumping plant not equipped with special controllers or valves, it may not be possible to limit filling velocity to *1 fps*. The electric motor is either on or off which might produce filling velocities of up to *5 fps*. Using a filling velocity of *5 fps*:

  $$Q = AV = (\pi/4)(1.5)(5) = 5.89cfs$$

  Therefore, air must be discharged from the line at a rate of *5.89 cfs*

- Size the orifice for *2 psi* pressure across the valve (standard design practice). Using the orifice formula, as before:

  $$P_c = (2\, lbs/in^2)(144\, in^2/ft^2) = 288psf$$

  $$A = \frac{5.89}{0.6\sqrt{2(32.2)(288/0.0764)}} = 0.02ft^2$$

  $$= (0.0\, ft^2)(144\, in^2/ft^2) = 2.87in^2$$

  $$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{(4)(2.87)}{\pi}} = 1.91in \text{ orifice diameter}$$

(ii) Comparing orifice sizes of *3.80 in*. diameter for vacuum relief and *1.04 or 1.01 in*. diameter for air release shows that the *3.80 in*. diameter orifice should be used if an air- and-vacuum valve, or continuous-acting air-and-vacuum valve, is to be used. If individual air release and vacuum relief valves are used, then they each may be of the size determined.

(8) Additional information on the operation and settings for air valves is available from the valve manufacturers.

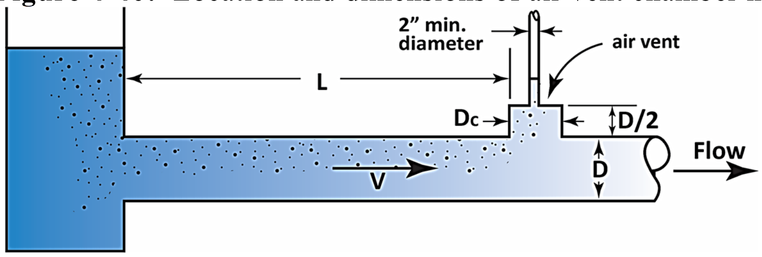
C. Air Vents

(1) Air vents act similarly to air vacuum or air release valves; however, they allow for the release of large volumes of air near pumps or water intakes, pressure boxes, and check valves. Air vents are often used in low-head pipelines. The figure below shows a simple air vent consisting of a cylindrical chamber of diameter D_c located at a distance L from a water intake where air entrainment occurs. The height of the air

vent chamber should be at least one-half the diameter of the pipe (*D/2*), and the chamber's area should be at least half of the mainline pipe cross sectional area (fig 4-40). For a cylindrical chamber, the minimum diameter is, therefore,

$$D_c = \sqrt{\frac{D^2}{2}} \qquad \text{(eq. 4-50)}$$

Figure 4-40: Location and dimensions of air vent chamber near a water intake.



(2) The minimum length L, measured from the pipe entrance is given by, Roberson, et.al. (1956) Using units of the International System (SI), the corresponding equation is:

$$L = 5.77 \times V \times D \qquad \text{(eq. 4-52)}$$

where V is in m/s, D in m, and L in m.

(3) Using units of the English System (ES), the corresponding equation is (4) Example 4-32 Air vent chamber sizing. A *12-in* diameter pipeline is to carry a discharge *Q = 2.5 cfs*. Determine the minimum diameter, height, and location of an air vent chamber from a water intake where air entrainment occurs.

(i) The pipe diameter is *D = 1* ft, and the flow velocity can be calculated from 634.0301equation 3-04:

$$V = \frac{4\,Q}{\pi D^2} = \frac{4(2.5cfs)}{3.1416(1ft^2)} = 3.18fps$$

(ii) The chamber should have a height of D/2 = 0.5 ft = 6 in, and a minimum diameter of:

$$D_c = \sqrt{\frac{D^2}{2}} = \sqrt{\frac{(1ft)^2}{2}} = 0.707ft$$

(iii) The distance from the water intake to the air vent should be at least:

$$L = 1.76 \times V \times D = 1.76 \times 3.18 \times 1 = 5.6ft$$

D. Pressure Control Valves

(1) Pressure relief valves (or pressure safety valves) are used in pipeline systems to protect against excessive pressure. They are designed to open and discharge small amounts of water at a preset pressure limit. Excess pressure opens the valve and some water is released (fig 4-41).

Figure 4-41: Spring operated pressure relief valve

(i) Example 4-33 – Pressure relief valve selection. Given: *8"* diameter, *80psi, SDR 51*, plastic irrigation pipe. Pipe inside diameter is *7.84in = 0.65ft*. Design pipeline velocity is *5fps*.

(ii) Select a pressure relief valve to protect against possible surge pressure at the end of the pipeline. The valve is set to relieve pressure at *80psi*, which is the pipe pressure rating. Size the pressure relief valve to pass the full flow at *150%* of the pipe pressure rating:

(iii) First, determining the pipeline flow:

$$Q = AV = (\pi/4)(0.65^2)(5) = 1.66 cfs$$

(iv) Determining minimum orifice diameter, using orifice equation (equation 4-49):

$$Q = C_d A\sqrt{2gH}$$

where: Cd = flow coefficient = 0.6
A = orifice area (ft^2)
P = (150%/100) (80 lbs/in2) (144 in^2/ ft^2) = 17280 psf
ω_w = specific weight of water = 62.4 lbs/ft^3
h = P /ω_w

(v) Rearranging the orifice formula,

$$A = \frac{Q}{C_d\sqrt{2gh}} = \frac{1.66}{0.6\sqrt{2 \times 32.2 \times (17280/62.4)}} = 0.0207 ft^2$$

$$= 0.0207 ft^2 \times 144\, in^2/ft^2 = 2.98 in^2$$

$$D = \sqrt{4A/\pi} = \sqrt{4 \times 2.98/\pi} = 1.95\ in, orifice\ diameter$$

(vi) Review of manufacturer's literature indicate that pressure relief valves with *2-in* and larger orifices are available. If, for safety concerns, the designer wishes to reduce the valve escape velocity, larger-size valves are available or multiple valves may be used. Find the dimensions of the valve opening using the continuity equation:

(2) Pressure reducing valves are used to reduce downstream pressure to protect and assure proper function of certain components. They maintain a constant downstream pressure regardless of the upstream pressure. The reducing valves are usually hydraulic controlled and are adjustable over a wide range of pressures and flows (figure 4-42).

Figure 4-42 A hydraulic control globe type pressure reducing valve on an irrigation mainline

(3) Similar to pressure reducing valve a pressure regulating valve (PRV) are generally smaller scale spring operated devices used for regulating the pressure in sprinkler (figure 4-43) or microirrigation systems.

Figure 4-43 Pressure regulating valve on a center pivot

(i) Example 4-34 – Pressure reducing valve selection. Given: *3"* diameter, *SDR 41, PVC* pipe. Pipe inside diameter is *3.33in = 0.278ft*. The pipe is a *500ft* sprinkler lateral, laid on a *2.5%* uphill slope. The lateral's design velocity is 5*fps*. A minimum of *30-psi* pressure is needed at the last (end-of-line) sprinkler.

(ii) Select a pressure reducing valve to maintain a maximum of *20%* pressure variation at the sprinklers located along the lateral.

(iii) First, determine the pipeline flow:

$$Q = AV = (\pi/4) \times 0.278^2 \times 5 = 0.302cfs$$

(iv) Determine friction losses for the lateral using the Hazen-Williams formula (equation 4-28). C_{HW} for the PVC pipe lateral is *130*.

$$h_f = 4.732\left(\frac{Q}{C_{HW}}\right)^{1.85}\frac{L}{D^{4.87}} = 4.732\left(\frac{0.303}{130}\right)^{1.85}\frac{500}{(0.278)^{4.87}}$$
$$= 16.27ft$$

The friction loss of *16.27 ft* is equivalent to (*16.27ft/2.31 lb/in²) =7.0 psi*.

(v) Determine minimum lateral pressure needed upstream of the first sprinkler. This pressure is the sum of the pressure needed at the last sprinkler plus the lateral elevation difference plus the friction losses (local losses neglected).

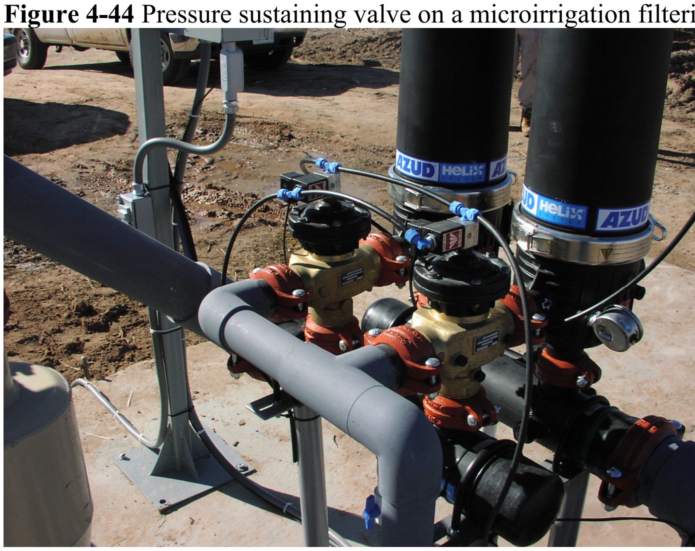
$$Elev.Diff. = \left(0.025\,{}^{ft}/_{ft}\right) \times 500ft = 12.5ft, \text{ equivalent to 5.4 psi}$$

$$P = 30 + 5.4 + 7 = 42.4psi$$

(v) At the first sprinkler, the pressure needs to be reduced from about *43 psi* to between 30 - *36 psi (1.2 x 30psi*). Review of manufacturer's literature indicates that a 2-in pressure reducing valve is available to meet the need for reduced pressure. To maintain a maximum of *20%* sprinkler pressure variation and 30 psi at lateral end, pressure reducing valves are needed at about the first half of the lateral's sprinklers due to lower elevation and less friction loss.

(4) Pressure-sustaining valves consist of the basic valves and a three-way pressure-sustaining pilot (figure 4-44). Pressure is sustained at the upstream side of the valve to a preset level, while the valve outlet drains excessive pressure to maintain the preset inlet pressure. Pressure is maintained constant, regardless of upstream fluctuating pressure and flow rate. Pressure sustaining valves are used on hilly terrain to maintain pressure in elevated areas and many other applications where sustained pressure is necessary.

Figure 4-44 Pressure sustaining valve on a microirrigation filtering station

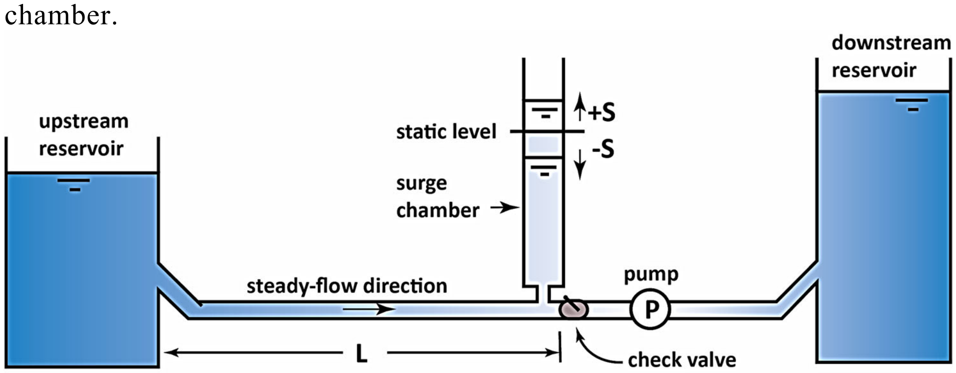


(5) Details on the operation of pressure relief, reducing, and sustaining valves are available from the valve manufacturers. The pipeline system pressure range and capacity are parameters needed for valve selection.

E. Surge/Air Chambers

(1) Surge or air chambers are vertical chambers, typically of cylindrical cross-section and open to the atmosphere, attached to pipelines to allow pressure relief during hydraulic transients. They perform a similar role as pressure relief valves, except that instead of releasing water flow to another pipeline, they allow the water surface level in the chamber to increase in response to a local pressure increase. During a hydraulic transient (unsteady flow) the water surface in the chamber will oscillate until a steady state is recovered. Figure 4-45 shows a schematic of a surge chamber in a pump-pipeline system. The surge chamber in this case would help dissipate a pressure wave that may result after a sudden failure of the pump. The system also includes a check valve that shuts off flow as the flow reverses.

Figure 4-45: Schematic of pump-pipeline system protected with a surge chamber.



(2) For the situation illustrated in the figure 4-40, if the cross-sectional area of the surge chamber is A_c and that of the pipeline is A, the maximum height S of the water surface in the chamber above its steady-state level (i.e., the amplitude of the oscillation) is calculated as:

$$S = \frac{Q}{A_c}\left(\frac{\pi}{2}\right)\sqrt{\frac{A_C L}{A(g)}} \qquad \text{(eq. 4-53)}$$

where Q is the steady-state discharge, L is the length of pipeline between the upstream reservoir and the check valve, and g is the acceleration of gravity *(= 9.81* m/s^2 *= 32.2* ft/s^2*).*

(3) The time in seconds required for the water level in the chamber to first reach its maximum height (one quarter of the period of oscillation) is calculated as, Tullis (1989):

$$T = \left(\frac{\pi}{2}\right)\sqrt{\frac{A_C L}{A(g)}} \qquad \text{(eq. 4-54)}$$

(4) The water in the chamber will oscillate until friction in the pipe, and in the chamber, slows down the oscillation and produces a complete flow shut off in the pipe upstream of the check valve. Notice that in the absence of a check valve, flow reversal will occur through the pump which could affect its operation after starting up.

(5) Example 4-35 – Surge chamber calculation. The pipeline in figure 4-40 is a *1-ft* diameter pipeline and the length from the supply reservoir to the check valve is *300 ft*. If the pipe carries a steady-state discharge of *4 cfs*.

(i) Determine the maximum water surface increase in a surge chamber of cross-sectional area $A_c = 0.785\ ft^2$. If the static level at the chamber is $H_0 = 500\ ft$, how high will the water level reach in the surge tank? What is the time T required to reach the maximum water elevation?

(ii) With $D = 1.0\ ft$, the pipe area is:

$$A = \frac{\pi D^2}{4} = \pi \frac{(1^2)}{4} = 0.785 ft^2$$

(iii) With L = 300 ft, Q = 4 cfs, g = 32.2 ft/s^2, then the rise in water elevation is:

$$S = \frac{Q}{A_c}\left(\frac{\pi}{2}\right)\sqrt{\frac{A_c L}{A(g)}} = \frac{4\,{}^{ft^3}/_{s}}{0.785 ft^2}\left(\frac{\pi}{2}\right)\sqrt{\frac{0.785 ft^2 (300 ft)}{0.785 ft^2 \left(32.2\,{}^{ft}/_{s^2}\right)}}$$

$$= 24.43 ft$$

(iv) Thus, the maximum water level is:

$$H = H_0 + S = 500 ft + 24.34 ft = 524.43 ft$$

(v) The time required to reach the maximum water level:

$$T = \left(\frac{\pi}{2}\right)\sqrt{\frac{A_c L}{A(g)}} = \left(\frac{3.1416}{2}\right)\sqrt{\frac{0.785 ft^2 (300 ft)}{0.785 ft^2 \left(32.2\,{}^{ft}/_{s^2}\right)}} = 4.79 s$$

F. Check Valves

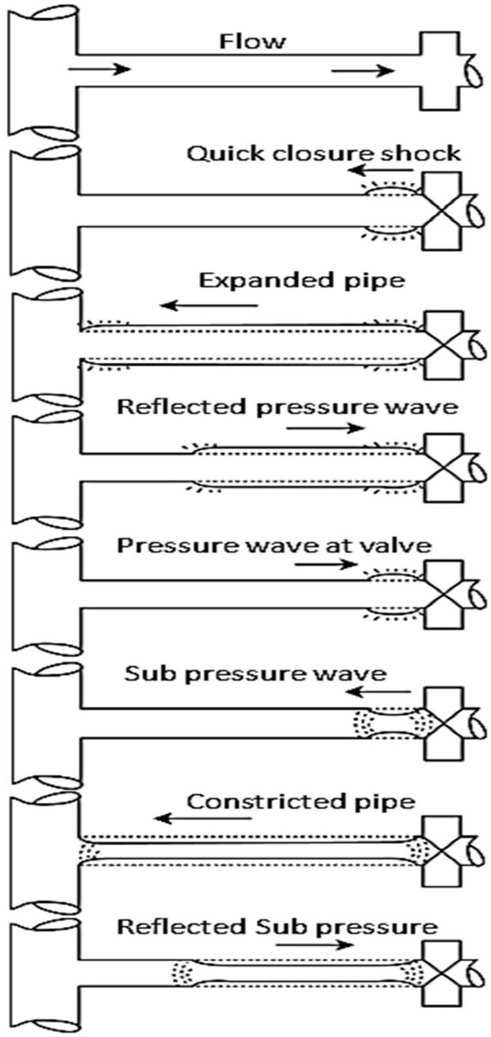
(1) Check valves (figure 4-46) are valves that allow flow only in one direction. The pressure of the flowing water opens the valves in the preferred direction. A sudden change in flow direction, such as the case of a pump failure, immediately closes the valve avoiding a reverse flow. The lack of a check valve leaves a pump unprotected against backflow, which may damage the pump and affect its operational efficiency. Details on the operation of check valves can be obtained from the valve manufacturers.

Figure 4-46: Check valve installed on a pumping plant

634.0410 Hydraulic Transients (Water Hammer)

A. The term hydraulic transient refers to a pipeline flow, and its attendant pressure, changing rapidly with time. A hydraulic transient in a pipeline can be originated by the closing of a valve, a pump failure, or a sudden increase in pipeline demand. Water hammers can easily be produced by a sudden closure of a valve. In this case, a pressure wave originates at the valve that travels through the pipe at speeds of the order of 1,440 m/s or 4,700 ft/s. The pressure wave reflects at the other end of the pipeline, be it a reservoir or a main line connection, and travels back towards the valve where pressure builds up again. The process repeats many times until friction dampens the pressure wave and the water flow finally stops (figure 4-47). Typically, a hammering noise is produced by the transient, thus the name water hammer. Pipelines, appurtenances, and tanks subject to water hammer may suffer deformation and damage in joints and other locations. In designing pipelines, care should be taken to prevent the presence of the water hammer phenomenon by providing check valves, pressure relief valves, and surge chambers in the pipeline systems.

Figure 4-47 Cycle of a hydraulic transient created by a sudden valve closing



B. Valve closure hydraulic transients can be minimized by closing the valve slowly. Unexpected hydraulic transients, such as a pump failure, will likely produce a high-magnitude pressure wave in pipelines with its attendant vibration. To prevent damage to the pipeline in such a case the pipeline should be provided with valves or surge chambers. The pipeline will be able to better survive a sudden hydraulic transient if it is buried, or otherwise anchored securely.

C. The formal analysis of hydraulic transients in a pipeline includes the simultaneous solution of two partial differential equations involving the discharge Q and the total head H at different locations x along the pipeline and at different times t. This type of analysis requires complex computer programming, considered beyond the scope of this handbook.

D. The closing of a valve is a common cause of hydraulic transients. If closing a valve changes the flow velocity in a pipe from V to V_f, the instantaneous increase in pressure near the valve, Δp, is calculated by using the equation:

$$\Delta p = \left(V - V_f\right)\sqrt{\frac{\omega/g}{\frac{1}{E_v}+\frac{D}{t(E)}}} \qquad \text{(eq. 4-55)}$$

Where ω is the specific weight of water (typically, ω = 62.4 lb/ft^3), g is the acceleration of gravity (= *32.2 ft/s^2*), E_v is the bulk modulus of elasticity of water, t is the pipe wall thickness, and E is the modulus of elasticity for the pipe material. A typical value of the modulus of elasticity for water is E_v = *311,000 psi* at 60ºF (see 634.0107, Exhibit D, Figure 1-9). Modulus of elasticity values for common pipeline materials include steel (*E = 30,000,000 psi*), cast iron (*E = 15,000,000 psi*), concrete (*E = 3,000,000 psi*) and PVC (*E = 400,000 psi*, with some variation by manufacturing class).

E. For additional information on water hammer analyses see Finnemore and Franzini (2002) or USDA (2005).

F. Example 4-36 – Pressure increases with sudden closing of a valve. Consider a steel pipe (*E = 30,000,000 psi*) with a wall thickness *t = 0.25 in = 0.25in/12 in/ft =0.02 ft*, and a diameter *D = 0.5 ft.*

(1) Closing a valve in a pipe that reduces the flow velocity from *V = 2.5 fps* to *Vf = 0.5 fps* for water at 60ºF in such a pipeline produces a pressure increase of:

$$\Delta p = (2.5fps - 0.5fps)\sqrt{\frac{\dfrac{62.4\,{}^{lb}/_{ft^3}}{32.2\,{}^{ft}/_{s^2}}}{\dfrac{1}{(311{,}000 lb/in^2)144\,{}^{lb}/_{ft^2}}+\dfrac{0.5ft}{(0.02ft)(30{,}000{,}000 lb/in^2)\left(144\,{}^{lb}/_{ft^2}\right)}}}$$

$$= 16{,}604\,{}^{lb}/_{ft^2} = \frac{16{,}604\,lb/ft^2}{144\,psi/lb/ft^2} = 115psi$$

(2) The typical value for atmospheric pressure is *14.7 psi*, thus, an increase of *115 psi* represents almost eight times the atmospheric pressure. Pipe should have adequate pressure ratings or otherwise be protected to withstand hydraulic transients. **This surge pressure is in addition to the operation pressure**.

(3) For more information on Calculating surge pressures refer to NEH Part 636, Chapter 52, Structural design of Flexible Conduits.

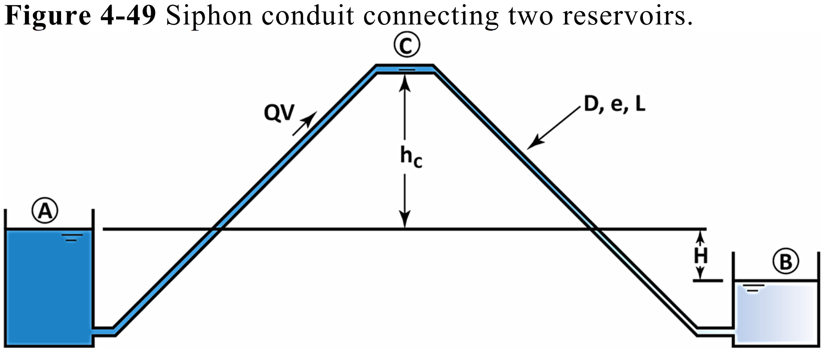
634.0411 Cavitation

A. When the local absolute pressure in a closed conduit (by its elevation and the hydraulic characteristics of the system) falls below the vapor pressure of water, vapor bubbles, or cavities, can form in the flow. These cavities are then carried with the flow towards zones of higher pressure where they implode (collapse onto themselves) producing a characteristic noise, vibration, and even pitting of the pipeline walls at the point of bubble collapse. Cavitation conditions are to be avoided because of their damaging effects, since cavitation can cause pitting and perforations in pipes, and even collapsing of pipes if the pressure drop at a given location is excessively large. Locations where cavitation may occur are the high point of a pipeline, or the suction side of a pump in a pipeline system. Cavitation is undesirable for pump operation since it can reduce pump efficiency and cause deterioration of the impeller (figure 4-48). An example of cavitation analysis in a siphon is presented figure 4-49. Cavitation was introduced in section 634.0104 Physical Properties of Water (5), in relation to the vapor pressure of water.

Figure 4-48 A cavitated pump impeller

B. Figure 4-49 illustrates a conduit delivering water from reservoir (A) to reservoir (B). Such an arrangement, where the middle part of the conduit raises above the free surface of both reservoirs is called a siphon. To start the siphon, it is necessary to fill the conduit with water by using, for example, a vacuum pump at point (C). Once the water flow is established the vacuum pump can be removed and the system continues flowing as long as the free surface at reservoir (A) is higher than that at reservoir (B). Siphon systems can be set up to overcome a hill separating two reservoirs or to discharge water downstream over a dam.

Figure 4-49 Siphon conduit connecting two reservoirs.



C. Example 4-37 – Cavitation at high point of a siphon. Suppose that the siphon has a diameter $D = 1.0\,ft$, a length $L = 200\,ft$, and the material is smooth enough that the roughness can be taken as $e = 0$. Let the difference of elevation between the reservoirs be $H = 20\,ft$, and the elevation difference of point (C) above the level of reservoir (A) be $h_C = 10\,ft$. Local loss coefficients are $K_e = 0.5$ and $K_d = 1.0$ for the entrance and discharge points at reservoirs (A) and (B), respectively.

(1) Find (a) The discharge in the siphon. (b) Determine the absolute pressure at high point (C), located halfway through the conduit, and check if cavitation is likely to occur. Assume that the water temperature is 60^oF and use an atmospheric pressure $p_{atm} = 14.7\,psi$. (c) What is the maximum value of h_C to prevent cavitation from taking place at point (C)?

(i) Writing the energy equation between points (A) and (B), with $V_A = V_B = 0$, using gage pressures $P_A = P_B = 0$, taking $z_A = H$ and $z_B = 0$, and including friction (using the Darcy-Weisbach equation to estimate friction losses) and local losses ($\sum K = 1.5$), the resulting equation is:

$$H = \frac{8Q^2}{\pi^2 g D^4}\left(f\left(\frac{e}{D}\frac{4Q}{\pi \nu D}\right)\frac{L}{D} + \sum K\right)$$

634.0107D figure 1-9 gives the kinematic viscosity, $\nu = 1.217x10\text{-}5\,ft^2/s$, for water at 60^oF. A spreadsheet application gives $Q = 14.73\,cfs$.

(ii) To determine the absolute pressure P_C at point (C) write the energy equation between points (A) and (C). $L_{A\text{-}C} = 200\,ft/2 = 100\,ft$:

$$z_A + \frac{p_A}{\omega} + \frac{V_A^2}{2g} - h_f - \sum h_L = z_C + \frac{p_C}{\omega} + \frac{V_C^2}{2g}$$

With $V_A = 0$, gage pressure $p_A = 0$, and elevations:

$$z_A = H = 20ft$$

$$z_C = H + h_C = 20ft + 10ft = 30ft$$

The flow velocity at point (C) is the constant value:

$$V = \frac{4\,Q}{\pi D^2} = \frac{(4)(14.73\ ft^3/s)}{3.1416\ (1.0\,ft)^2} = \frac{18.75ft}{s}$$

And the Reynolds number is:

$$R_e = \frac{VD}{\nu} = \frac{(18.75\,ft/s)(1.0ft)}{1.217x10^{-5}\,ft^2/s} = 1.54x10^6$$

(iii) With this value of the Reynolds number (indicating turbulent flow) and relative roughness $e/D = 0$, the Swamee-Jain equation (equation 4-19) produces the following friction factor:

$$f = \frac{0.25}{log^2\left(0.27\left(\frac{e}{D}\right) + 4.62\left(\frac{\nu D}{Q}\right)^{0.9}\right)} = \frac{0.25}{log^2\left(0 + 4.62\left(\frac{1.217x10^{-5}\,{ft^2}/{s}\,(1.0ft)}{14.73\,{ft^3}/{s}}\right)^{0.9}\right)}$$

$$= 0.0108$$

(iv) The friction losses (using the Darcy-Weisbach equation) and local head losses (entrance losses) are calculated as follows:

$$h_f = f\left(\frac{L}{d}\right)\frac{V^2}{2g} = (0.0108)\left(\frac{\frac{200ft}{2}}{1.0ft}\right)\frac{\left(18.75\,{}^{ft}/_{s}\right)^2}{(2)\left(32.2\,{}^{ft}/_{s^2}\right)} = 5.9ft$$

$$\sum h_L = \sum K\frac{V^2}{2g} = (0.5)\left(\frac{(18.75\,ft/s)^2}{2(32.2ft/s^2)}\right) = 2.73ft$$

(v) Substituting values calculated above into the energy equation:

$$20ft + \frac{0}{\omega} + \frac{0}{2g} - 5.9ft - 2.73ft = 30ft + \frac{p_c}{\omega} + \frac{\left(18.75\,{}^{ft}/_{s^2}\right)^2}{2(32.2ft/s^2)}$$

(vi) Solving for the pressure head at point (C):

$$\frac{p_c}{\omega} = -24.09ft$$

And, using $\omega = 62.4\ lb/ft^3$, the gage pressure at point (C) is:

$$P_c = 62.4\,{}^{lb}/_{ft^3}\,(-24.09ft) = -1503psf = -\frac{1503}{144}psi = -10.44psi$$

(vii) Calculating the absolute pressure at point (C) and comparing to the vapor pressure, $p_v = 0.26\ psi$, for water at 60^oF (634.0107D figure 1-9):

$$(P_c)_{abs} = P_c + P_{atm} = -10.44psi + 14.7psi = 4.26psi > 0.26psi$$

Since the absolute pressure at point (C) is larger than the vapor pressure of water at a temperature of 60°F, no cavitation would occur at point (C) for the siphon system shown.

(viii) To determine the maximum height h_C to avoid cavitation at point (C), the absolute pressure at point (C) is set equal to the vapor pressure of water at 60oF, and the value of h_C from the energy equation between points (A) and (C) is calculated.

(ix) The gage pressure at point (C) is:

$$P_c = (P_c)_{abs} - P_{atm} = P_v - P_{atm} = 0.26psi - 14.7psi = -14.44psi$$

(x) The corresponding pressure head is:

$$\frac{P_c}{\omega} = \frac{-14.44psi}{62.4\,{}^{lb}/_{ft^3}} = \frac{-14.44psi\left(144\,{}^{psf}/_{psi}\right)}{62.4\,{}^{lb}/_{ft^3}} = -33.32ft$$

(xi) The velocity head, head losses, and elevation of point A are the same as in the energy equation used above. The elevation of point (C) is:

$$z_c = (h_c)_{max} + 20ft$$

Thus, the energy equation between (A) and (C) becomes:

$$20ft + \frac{0}{\omega} + \frac{0}{2g} - 5.9ft - 2.73ft =$$

$$= (h_c)_{max} + 20ft - 33.32ft + \frac{18.75\,{}^{ft}/_{s^2}}{(2)\left(32.2\,{}^{ft}/_{s^2}\right)} =$$

(xii) The resulting value of h_C is:

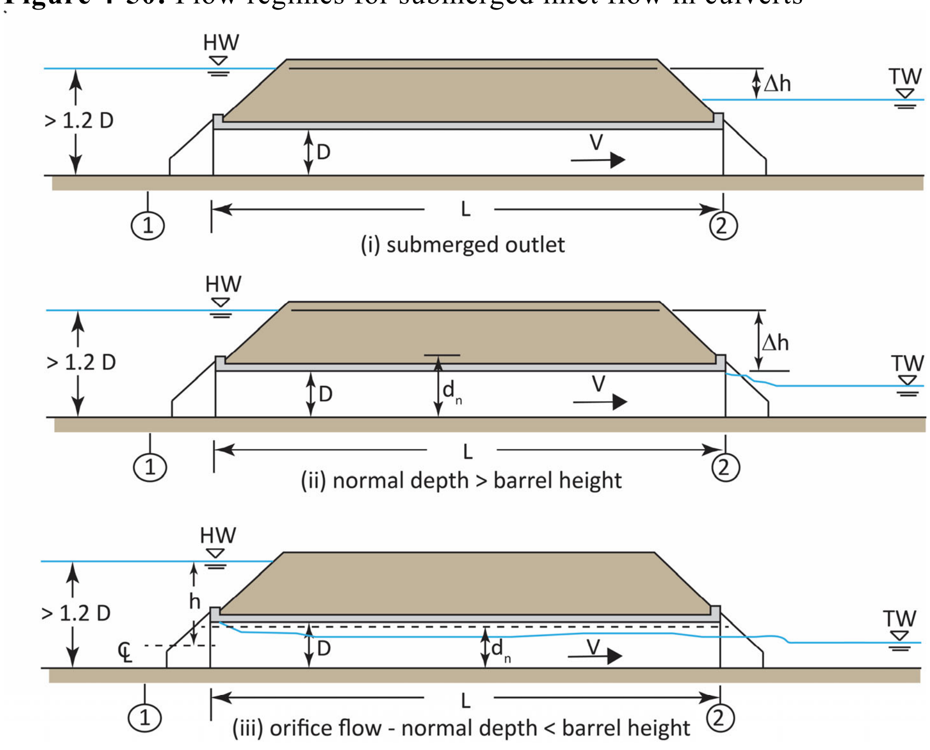
$$(h_c)_{max} = 19.23ft$$

634.0412 Culverts

A. General

(1) A culvert is a relatively short conduit connecting two open channels and typically located under a road or highway. Figure 4-50 illustrates three possible flow regimes when a culvert's inlet is submerged.

Figure 4-50: Flow regimes for submerged inlet flow in culverts



(2) The flows of figure 4-50(i) and (ii) are said to be under outlet control, while that of 4-50 (iii) is under inlet control. In figure 4-50, *V* stands for the flow velocity in the full-flowing culvert; *HW* stands for head water (that needs to be at a minimum of *1.2 D* to provide a submerged inlet) and *TW* for tailwater depths, i.e., the depths of flow upstream and downstream of the respective culverts. *D* is the diameter of the culvert, *h* is the vertical distance from the head water surface to the centerline of the culvert, and Δ*h* is the difference between the headwater and tailwater depths.

(3) Publication FHWA-NHI-01-020, HYDRAULIC DESIGN OF HIGHWAY CULVERTS, Hydraulic Design Series No. 5, 2012, available from the Federal Highway Administration (FHWA) provides detailed information for the analysis of culvert flow (FHWA, HDS 5). This publication is available at: https://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7&id=13

(4) The FHWA also provides a computer program, HY-8, for calculating culvert flow. The program is available at: http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/

B. Culvert Flow with Inlet Control

(1) This case is represented in figure 4-50 (iii). The capacity of the culvert is limited by the capacity of the culvert opening, rather than by conditions farther downstream. In such case, the open-channel normal depth of flow d_n in the culvert is less than the barrel height (height of the culvert section), and the entrance does not allow enough water into the culvert to allow filling the full barrel height. Thus, the flow is under inlet control. The discharge Q into the culvert can be calculated using the following equation (orifice flow):

$$Q = C_d A\sqrt{2gh} \qquad \text{(eq. 4-49)}$$

(2) Where A is the area of the inlet section, C_d is a discharge coefficient, g is the acceleration of gravity, and h is the vertical distance from the *HW* surface to the centerline of the inlet or the *HW* and *TW* depth difference whichever is smallest.

(3) For inlet control, the required HW head is calculated as:

$$h = \frac{Q^2}{2gC_d^2A^2} \qquad \text{(eq. 4-56)}$$

(4) Values of the discharge coefficient C_d vary from 0.62 (for a sharp-edged inlet without contraction suppression) to 1.0 (for a well-rounded inlet). Use the value of $C_d = 0.60$ as a conservative reference value. Contraction suppression refers to the absence of contracted flow at the inlet cross-section. Suppressed contraction at the bottom of the inlet occurs when the inlet invert is set at stream bed level. Partial contraction suppression can be achieved by using flared wingwalls at the inlet approach.

(5) As an alternative to equation 4-49 (general orifice flow equation), FHWA, HDS 5 includes a submerged inlet control equation, based on extensive laboratory testing.

(6) Example 4-38 – Circular culvert calculation under inlet control.

A circular culvert of diameter $D = 2\ ft$, carrying a discharge $Q = 40\ cfs$, is to be designed under inlet control. What is the required headwater height, h, above the culvert centerline if the inlet discharge coefficient is $C_d = 0.75$?

(i) The pipe area is:

$$A = \pi D^2/4 = \pi \times (2ft)^2/4 = 3.1416ft^2$$

(ii) And the HW head is:

$$h = \frac{Q^2}{2gC_d^2A^2} = \frac{(40\,ft^3/s)^2}{(2)(32.2\,ft/s^2)(0.75ft)^2(3.1416ft)^2} = 4.48ft$$

$HW\ depth = \mathrm{h}\ + 1/2\,\mathrm{D}\ = 4.48ft + 1.0ft = 5.48ft$

(iii) Checking that HW depth exceeds 1.2D:

$HW\ depth = 4.48ft + 1.0ft = 5.48ft > 1.2 \times 2ft = 2.4ft$

C. Culvert Flow with Outlet Control

(1) Figures 4-50(i) and (ii) show instances of flow with outlet control. In figure 4-44(i) the outlet is fully submerged, a condition that may result from inadequate channel capacity downstream or by an existing backwater from a connecting stream. Even if the tailwater depth is below the barrel height, as shown in figure 4-44(ii), the normal flow depth d_n within the culvert is greater than the culvert height D, and the culvert flows full.

(2) Analysis of flow for the outlet control cases of figure 4-50(i) and (ii) is approached by writing the energy equation between the free surface of the headwater section (1) and that of the tailwater section (2). The elevation of the outlet (section 2) is taken as a reference, so that $z_1 = \Delta h$, and $z_2 = 0$. Since the culvert is open to the Atmosphere at both ends, the gage pressure, $P_1 = P_2 = 0$. Also, $V_1 = 0$, and $V_2 = V$. Including head losses in the culvert, the resulting energy equation is:

$$\Delta h = h_L + \frac{V^2}{2g} \qquad \text{(eq. 4-57)}$$

(3) In the simplest case, the head losses h_L includes friction head losses h_f and entrance losses h_e. Friction head losses are calculated using Manning's equation:

$$h_f = \frac{n^2 V^2 L}{C_u^2 R_h^{4/3}} \qquad \text{(eq. 4-58)}$$

(4) Where n is Manning's resistance coefficient, C_u is the coefficient in Manning's equation ($C_u = 1.0$ for units of the SI, and $Cu = 1.486$ for English units), and R_h is the hydraulic radius in the culvert cross-section. Entrance losses are calculated using:

$$h_e = K_e\left(\frac{V^2}{2g}\right) \qquad \text{(eq. 4-59)}$$

(5) A higher value for K_e, the entrance loss coefficient, gives a higher head loss. Values of the entrance loss coefficient are presented in figure 4-11 found in section 634.0405. Also see FHWA, HDS 5 for additional values of entrance loss coefficients. Use a value of 1 for the exit loss.

(6) Substituting the sum of equations 4-58 and 4-59 for h_L in equation 4-57 produces the equation:

$$\Delta h = h_f + (K_e + 1)\frac{V^2}{2g} \qquad \text{(eq. 4-60)}$$

(7) The approach presented in FHWA, HDS 5 includes the use of a bend coefficient K_b, and an exit coefficient K_x (to replace the value of 1 in equation 4-60); the resulting equation is:

$$Q = A\sqrt{\frac{2g\Delta h}{K_e + K_p L + K_b + K_x}} \qquad \text{(eq. 4-61)}$$

(8) Where K_p is the pipe friction loss coefficient, which, in equation 4-60, is equal to:

$$K_p = \frac{2gn^2}{C_u^2 R_h^{4/3}} \quad \text{(eq. 4-62)}$$

(9) A typical culvert design consists in determining the required diameter of a culvert given the design discharge, the inlet entrance conditions, the pipe material and length, the culvert slope, and the headwater and tailwater depths. The use of Culvert Flow, USDA-NRCS Hydraulics Formula program is illustrated in the example 4-39.

(10) Example 4-39 – Circular culvert sizing. It is desired to install $L = 50\ ft$ of concrete culvert pipe, $n = 0.012$, in a drainage channel for a road crossing. The design discharge is $Q = 80\ cfs$ with a tailwater depth of $d_2 = 3.0\ ft$. The slope of the culvert is to be 0.002 ft/ft. The maximum headwater depth is $d_1 = 5.0\ ft$.

(i) Determine the required pipe diameter, if the inlet has a groove edge and a headwall.

(ii) The elevation change in 50 ft of culvert is:

$$\Delta z = S_o \times L = 0.002\, ft/ft \times 50 ft = 0.1 ft$$

(iii) Since the culvert outlet elevation is taken to be "0", the inlet elevation is 0.1 ft. Figure 4-51 shows the calculation of the discharge for culvert flow using the Culvert Flow tab in the USDA-NRCS Hydraulics Formula program for a guessed diameter of $24\ in = 2\ ft$. The result shows $Q = 27.9\ cfs$, (see figure 4-51) which is smaller than the required discharge of 80 cfs. The diameter is increased until $Q = 80\ cfs$. Figure 4-52 shows the convergence of values.

Figure 4-51: Culvert discharge calculation

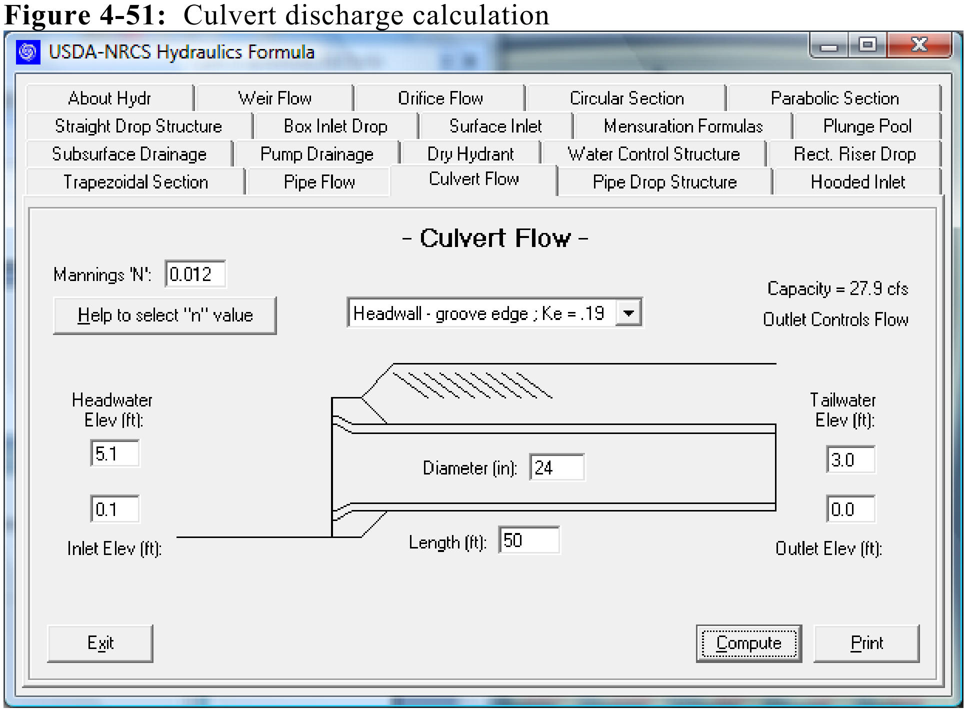


Figure 4-52: Culvert Discharge Convergence of Values

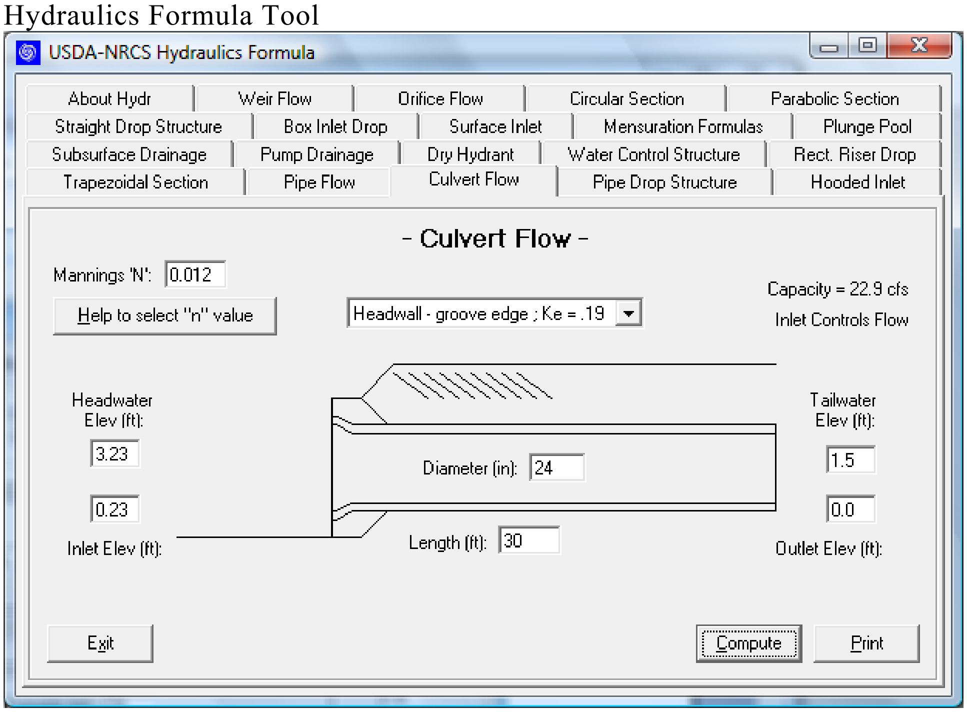
D (in)	Q (cfs)
24	27.9
36	67.1
48	105.7
42	88.0
39	78.7
40	81.8

A standard sized concrete culvert of $D = 42$ *in* will carry the required *80 cfs*.

(11) Example 4-40 Calculate the discharge in a PVC culvert with a diameter of 2 ft (24 in), and a length of 30 ft if the head wall inlet has a groove edge. The inlet invert is located 0.23 ft above the outlet invert (set as zero), the headwater depth is 3.0 ft (headwater elevation = 3.23 ft), while the tailwater depth is 1.5 ft (figure 4-53).

(i) The USDA-NRCS Hydraulics Formula program aids in the selection of the values of *Manning's n* and the entrance coefficient K_e. The discharge calculation is shown below:

Figure 4-53: Culvert discharge calculation using USDA-NRCS Hydraulics Formula Tool



(ii) Thus, the calculated discharge is $Q = 22.9$ *cfs* and the inlet controls the flow (the equations associated with inlet control apply).

(11) In some applications the culvert headwater depth is below the barrel height producing free or unsubmerged inlet conditions. The techniques of open channel flow presented in 210-634-3, section 634.0316 "Gradually Varied Flow" can be used to solve for the critical depth of flow and the headwater depth. This analysis can allow for entrance and exit losses. As an alternative, FHWA, HDS 5 includes an unsubmerged inlet control equation, based on extensive laboratory testing.

(12) An alternative for culvert flow calculations is the use of nomograms

D. Culvert flow solutions using nomograms

(1) This section includes nomograms for the calculation of culvert flow for different pipe materials. A nomogram is a graphical device used for solving equations. In a nomogram, the user pinpoints the location of two points in the scales provided and traces a straight line to a third scale. The intersection of the straight line with the third scale is the value of the unknown variable sought. Some nomograms include additional scales corresponding to different conditions for the problem, as well as turning lines to relate the solution to a third variable.

(2) The four nomograms included in this section are the same as charts in *FHWA-NHI-01- 020*, *HYDRAULIC DESIGN OF HIGHWAY CULVERTS, Hydraulic Design Series No. 5*, September 2001. *FHWA, HDS 5* contains 55 additional charts for hydraulic analyses of culverts of different types and flow conditions. Examples are included for each of the nomograms.

(3) The four nomograms for this section are described as follows:

(i) Figure 4-54. Nomogram for headwater depth for concrete pipe culverts with inlet control.

(ii) Figure 4-55. Nomogram for headwater depth for corrugated metal (CM) pipe culverts with inlet control.

(iii) Figure 4-56. Nomogram for concrete pipe culverts flowing full with outlet control, Manning's $n = 0.012$.

(iv) Figure 4-57. Nomogram for head for corrugated metal (CM) pipe culverts flowing full with outlet control, $n = 0.024$.

(4) The variables referenced in these nomograms are described as follows:

(i) D = culvert pipe diameter (in)

(ii) $\frac{HW}{D}$ = Head water depth to culvert pipe diameter ratio (dimensionless)

(iii) Q = discharge (cfs)

(iv) L = culvert length (ft)

(v) H or Δh = head (ft), the difference in elevation between the HW depth and downstream depth in a culvert

(5) Notice that variable Δh in Figure 4-50 is the same as variable H in the nomograms of Figures 4-56 and 4-57.

(6) Example 4-41. Concrete pipe culvert with inlet control

(i) Consider a concrete pipe culvert with diameter $D = 42\ in = 3.5\ ft$, carrying a discharge $Q = 120\ cfs$. Determine the required head water depth (HW) for the following conditions:

- Square edge with headwall
- Groove end with headwall
- Groove end projecting

(ii) Using Figure 4-54, first locate $D = 42\ in$ on scale (4), then $Q = 120\ cfs$ on scale (3) and trace a straight line through these two points and extend it up to scale (1). The point of intersection of the EXAMPLE line with scale (1) corresponds to the value of $HW/D = 2.5$ for a square edge with headwall. To find the values of HW/D for the other cases (groove end with headwall and groove end projecting), draw a horizontal line from the point found in scale (1) to intersect scale (2) and scale (3), respectively. The values read in scales (2) and (3) are $HW/D = 2.1$ and $HW/D = 2.2$, respectively. The value of the corresponding headwater can be calculated multiplying the HW/D value by the pipe diameter D. Thus, for the three cases required in this example, the following results are obtained:

- Square edge with headwall, $HW=(HW/D)*D = 2.5 \times 3.5\ ft = 8.75\ ft \approx 8.8\ ft$
- Groove end with headwall, $HW=(HW/D)*D = 2.1 \times 3.5\ ft = 7.35\ ft \approx 7.4\ ft$
- Groove end projecting, $HW=(HW/D)*D = 2.2 \times 3.5\ ft = 7.7\ ft$

(iii) The nomographs were computed assuming 2% culvert slopes and are accurate within 10% for determining the required inlet control headwater.

(7) Example 4-42. Corrugated metal (CM) pipe culvert with inlet control

(i) Consider a CM pipe culvert with diameter $D = 36\ in = 3\ ft$, carrying a discharge $Q = 66\ cfs$. Determine the required head water depth (HW) for the following conditions:

- Headwall
- Mitered to conform to slope
- Projecting

(ii) Using figure 4-55 first locate $D = 36\ in$ on scale (4), then $Q = 66\ cfs$ on scale (5) and trace a straight line through these two points and extend it up to scale (1). The point of intersection of the EXAMPLE line with scale (1) corresponds to the value of $HW/D = 1.8$ for a square edge with headwall. To find the values of HW/D for the other cases (mitered to conform to slope and projecting), draw a horizontal line from the point found in scale to intersect scale (2) and scale (3), respectively. The values read in scales (2) and (3) are $HW/D = 2.1$ and $HW/D = 2.2$, respectively. The value of the corresponding headwater can be calculated multiplying the HW/D value by the pipe diameter D. Thus, for the three cases required in this example, the following results are obtained:

- Headwall, $HW=(HW/D)*D = 1.8 \times 3\ ft = 5.4\ ft$
- Mitered to conform to slope, $HW=(HW/D)*D = 2.1 \times 3\ ft = 6.3\ ft$
- Projecting, $HW=(HW/D)*D = 2.2 \times 3\ ft = 6.6\ ft$

(8) Example 4-43. Concrete pipe culvert flowing full with outlet control ($n = 0.012$)

(i) Determine the diameter of a concrete pipe culvert flowing full with a length $L = 110\ ft$ with an entrance loss coefficient $k_e = 0.5$ if it is to carry a flow $Q = 70\ cfs$ with a head loss $H = 0.94\ ft$.

(ii) Using figure 4-56, first, trace a straight line from the DISCHARGE scale at point $Q = 70\ cfs$ to the HEAD scale at $H = 0.94$. Next, locate the point $L = 110\ ft$ on the LENGTH scale corresponding to $k_e = 0.5$, and trace a straight line through the intersection of the first line with the TURNING LINE, extending it all the way to the DIAMETER scale. The ending point of this second line represents the predicted value for the diameter. In this case, $D = 48\ in = 4\ ft$. Since this value is a standard value for pipe diameters, this will be the design diameter. On the other hand, if the diameter had been, say, $D = 51\ in$, then, it is recommended to use the next larger standard diameter, $D = 54\ in = 4.5\ ft$.

(9) Example 4-44. Corrugated metal (CM) pipe culvert flowing full with outlet control ($n = 0.024$)

(i) Determine the head loss H for a CM culvert with diameter $D = 27\ in$ and length $L = 120\ ft$ if it is to carry a discharge $Q = 35\ cfs$. The entrance loss coefficient is $k_e = 0.9$.

(ii) Using figure 4-57, first, trace a straight line from the DIAMETER scale at $D = 27\ in$ to the point $L = 120\ ft$ in the LENGTH scale corresponding to $k_e = 0.9$. Next, locate the point $Q = 35\ cfs$ in the DISCHARGE scale, and trace a straight line through the intersection of the first line with the TURNING LINE, extending all the way to the HEAD scale. The end of this line will show the required value of $H = 7.5\ ft$.

Figure 4-54: Nomogram for headwater depth for concrete pipe culverts ***with inlet control***.

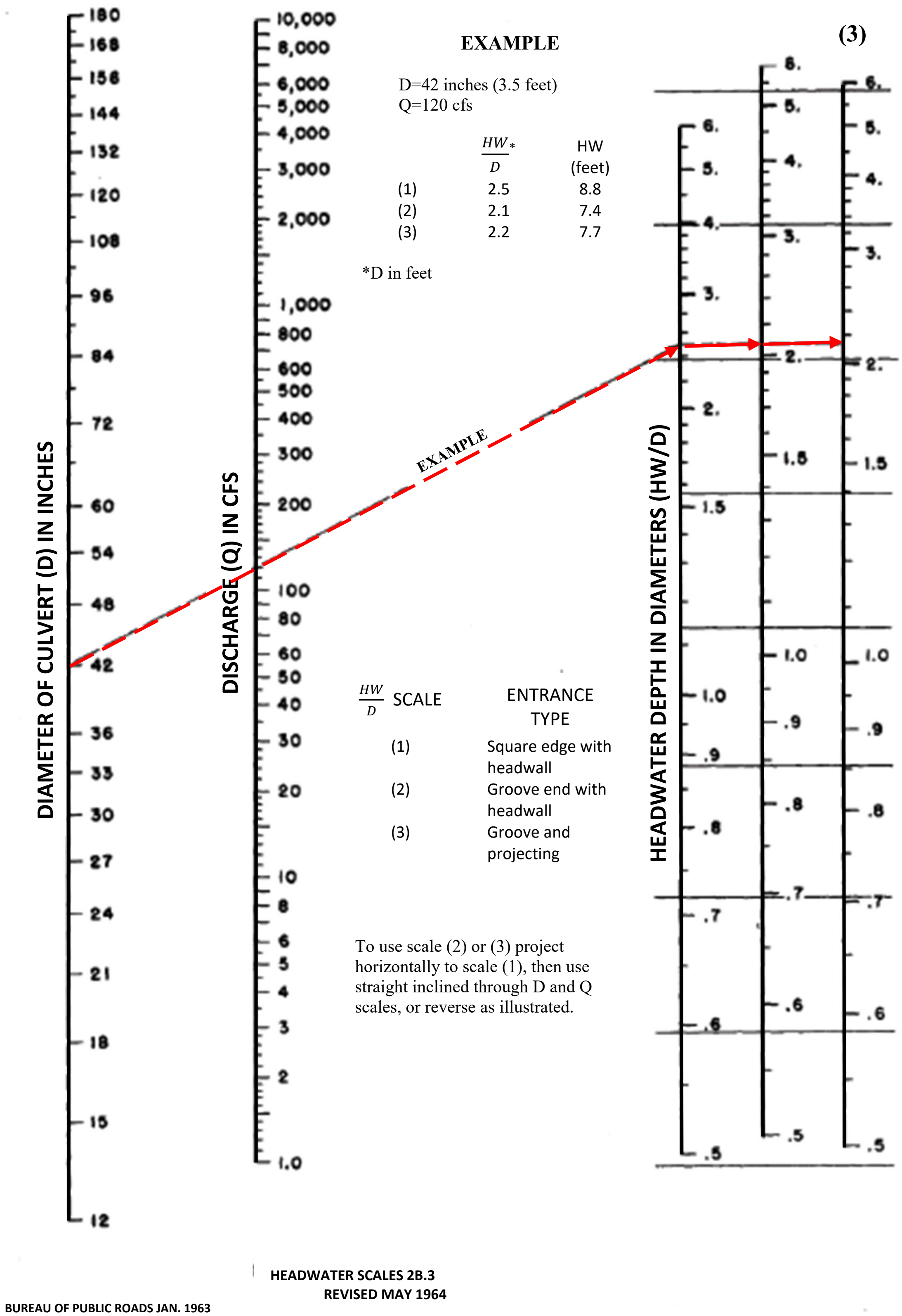


Figure 4-55: Nomogram for headwater depth for corrugated metal (CM) pipe culverts ***with inlet control***.

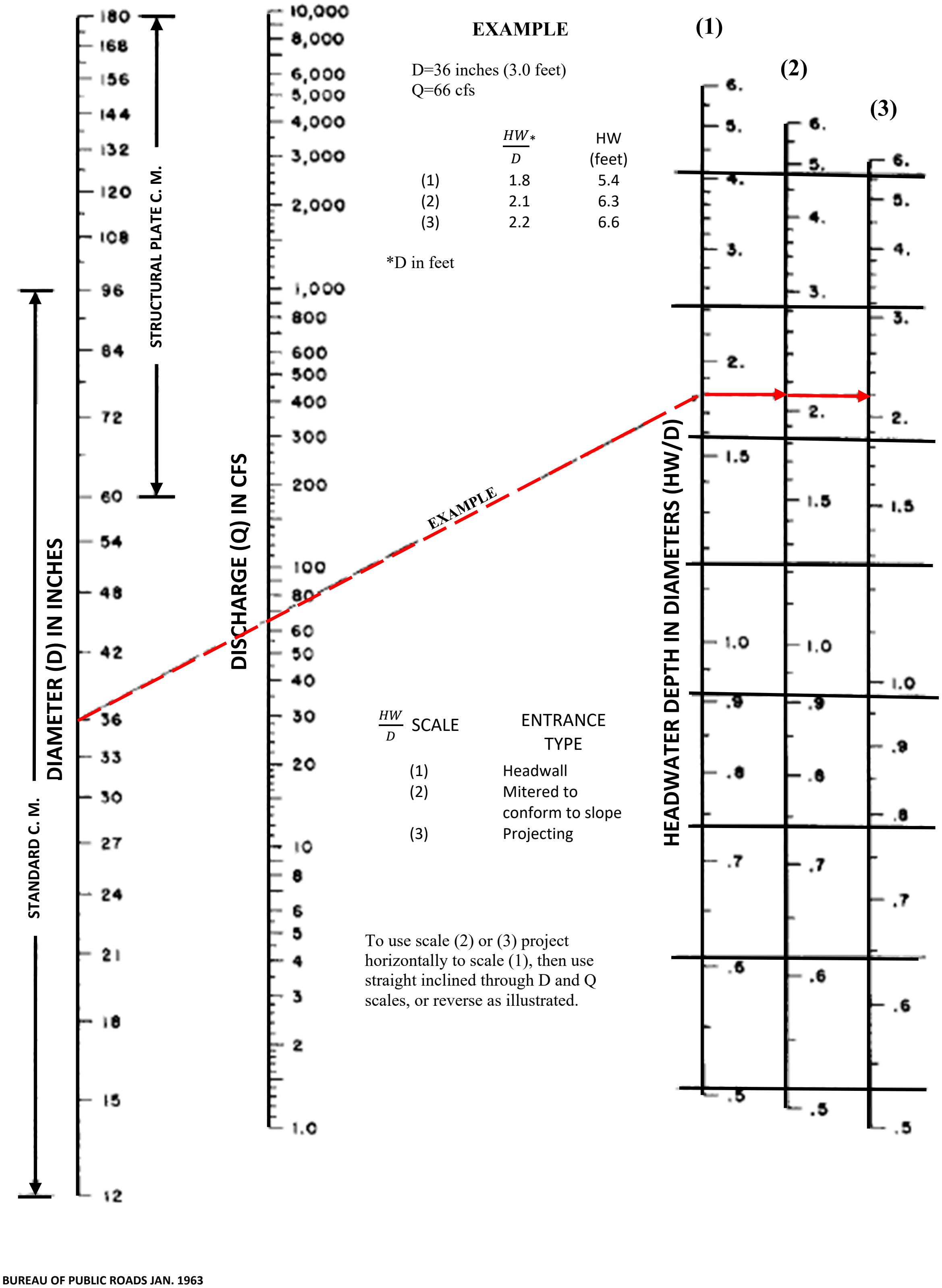
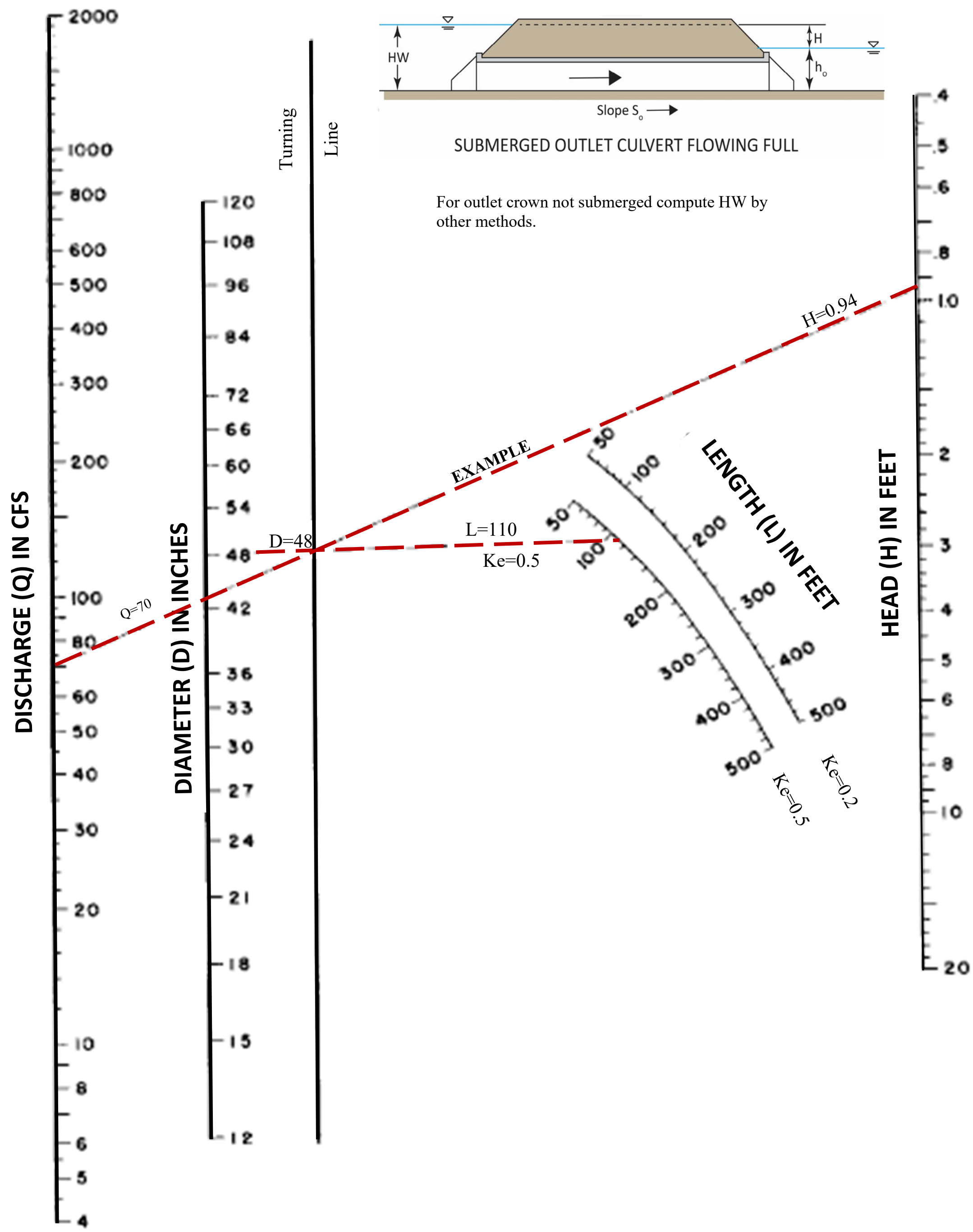
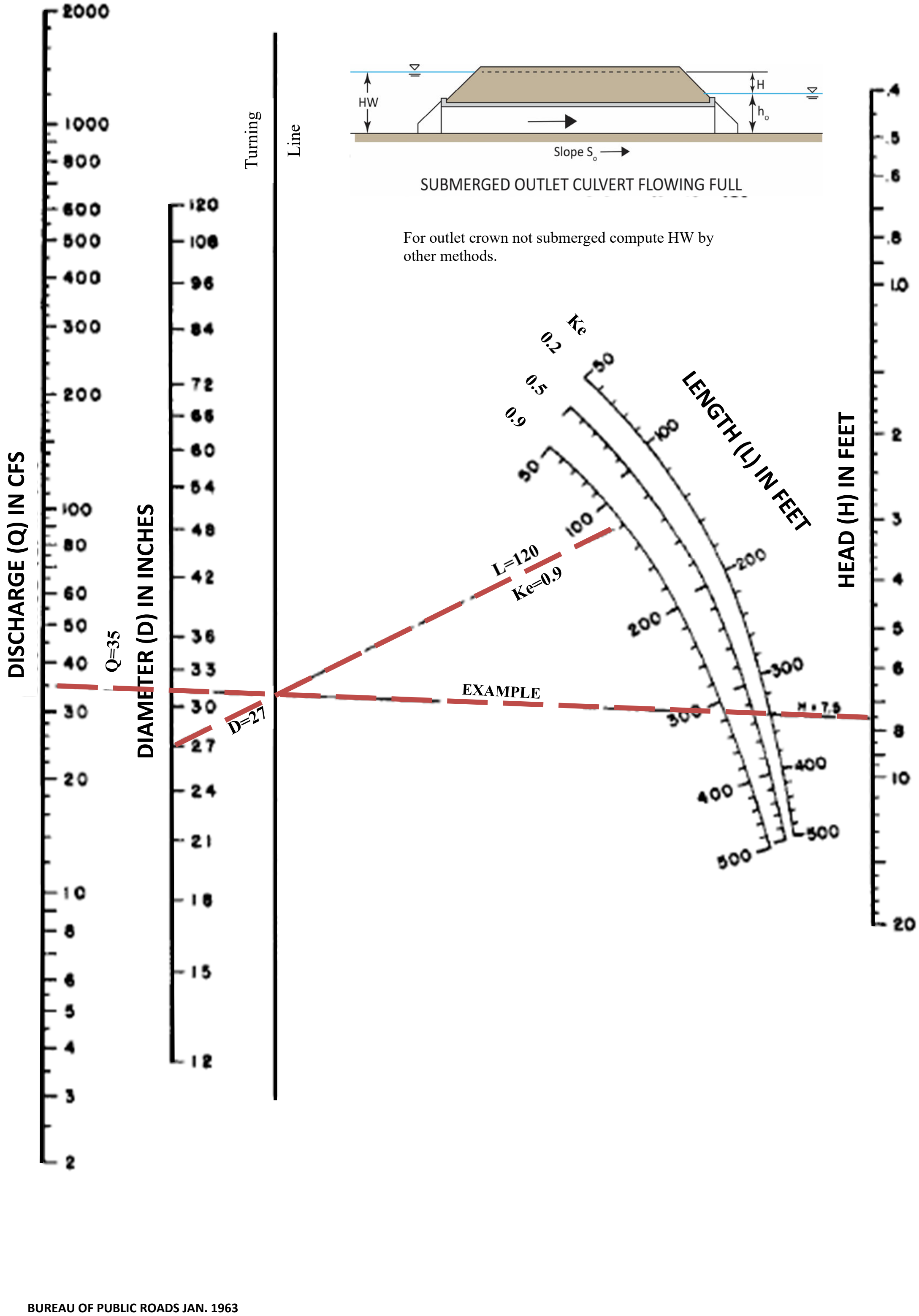


Figure 4-56 Nomogram for concrete pipe culverts flowing full ***with outlet control***, Manning's $n = 0.012$.



BUREAU OF PUBLIC ROADS JAN. 1963

Figure 4-57: Nomogram for head for corrugated metal (CM) pipe culverts flowing full ***with outlet control,*** n = 0.024.



634.0413 Sprinkler Irrigation

A. Sprinkler systems irrigate by spraying water in a desirable pattern. Sprinkler systems typically include a pump, a main pipeline (or, simply, main), lateral pipelines (or, simply, laterals), risers, and sprinkler heads.

B. Sprinkler systems can be classified as permanent, semi-permanent, portable, and continuous move. Permanent systems have permanently located main and lateral pipelines and a pumping plant. Semi-permanent systems consist of a permanent pumping plant and main pipeline to which portable lateral pipelines can be attached. Portable sprinkler systems have both portable main and lateral pipelines. Continuous move systems include center pivot systems and linear move systems.

C. Center pivot systems consist of a continuously moving lateral pipeline that rotates about a pivot point producing a circular irrigation pattern. A center pivot sprinkler system is shown figure 4-58.

Figure 4-58 Central pivot irrigation system near Grace, Idaho.

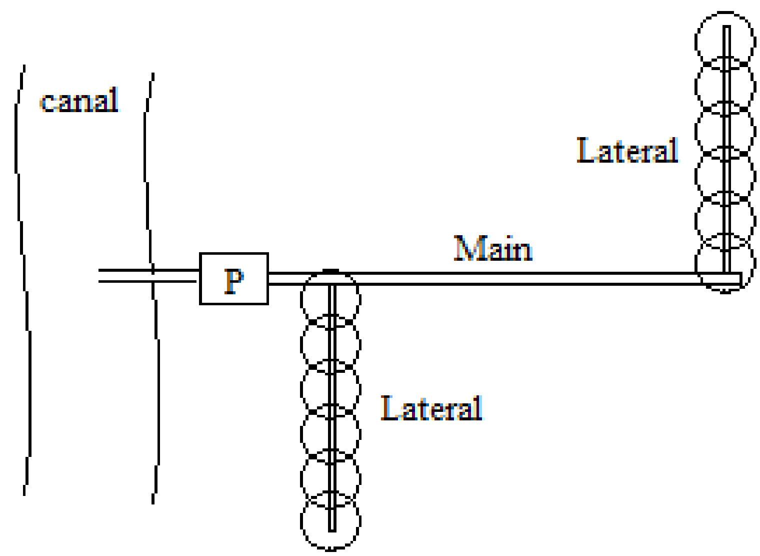
D. Sprinklers can be classified as low-pressure (6-25 psi), medium-pressure (25-60 psi), and high-pressure (> 60 psi) sprinklers. Specific characteristics for commercial sprinkler heads, such as the pressure and discharge ranges for their operation, and the wetted diameter (effective distance of water spraying) are typically provided by manufacturers. Sprinkler types that operate under higher operating pressure provide a larger wetted diameter.

E. Low-pressure spray types of sprinklers include fixed sprays, spinners, and rotating sprays. Low-pressure sprinklers have become the most common sprinkler type today.

F. A high-pressure sprinkler generally consists of one or two nozzles that rotate under the effect of a hammer blade. The water spray impinges on a hammer blade which produces an intermittent water spray and rotates the sprinkler head. A high-pressure sprinkler head is typically mounted on a 1-inch, or larger (25 mm) diameter riser attached to a supply pipeline or lateral.

G. Figure 4-59 shows a schematic for the layout of a periodic move sprinkler system (hand line or wheel line) with a pump (P), a main line, and two lateral lines. Basic information needed for the design of a sprinkler irrigation system includes a contour map of the plot to irrigate (this provides information on the ground slopes on which the pipelines will be laid), the soil characteristics (maximum application rate should not exceed the soil infiltration rate), source and quality of water available, crops to be irrigated, and local climate.

Figure 4- 59 Schematic of a sprinkler system layout



H. Some basic guidelines for sprinkler system alignment include: (1) the laterals should be perpendicular to the prevailing wind direction; (2) the main line should be as short as possible to reduce head losses; (3) the pump should be located at the center of the irrigated area if possible; and (4) provide for future expansion of the system.

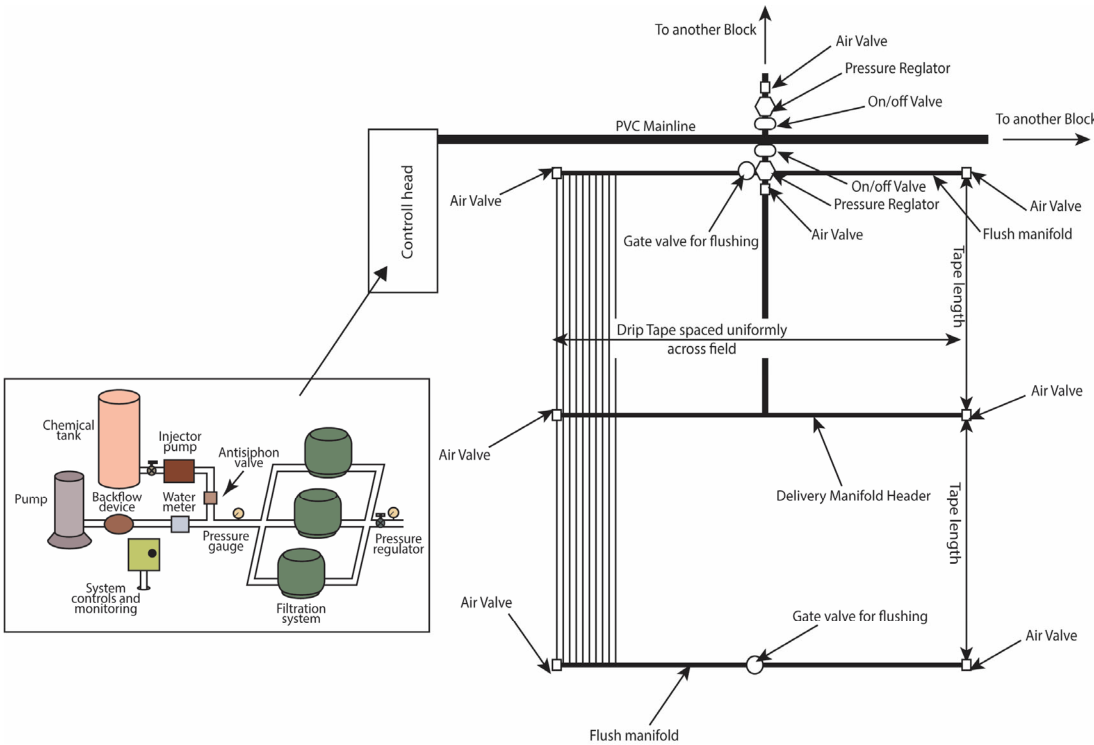
I. Detailed design guidelines and calculations for sprinkler irrigation systems are available in NRCS, Title 210, National Engineering Handbook, Part 623, Chapter 11 - Sprinkle Irrigation. This reference may be downloaded from the engineering handbooks listed at: http://directives.sc.egov.usda.gov/

634.0414 Microirrigation

A. Microirrigation (MI) is accomplished by the frequent application of small quantities of water as drops, tiny streams, or miniature spray through emitters or applicators placed along a water delivery line. MI encompasses many methods or concepts such as bubbler, drip, subsurface drip, mist and spray. The emitters are located at or near the plant root zone thus placing water only where the plant can use it. Thus, microirrigation can be very effective for all crops, especially widely spaced crops (such as orchards, melons, cucumbers).

B. Microirrigation systems include a control unit or system through which water is controlled, filtered, and possibly provided with additives. The control unit is typically located at the highest place in the field, and the pipelines laid parallel to the terrain slope. As with sprinkler irrigation systems, the main line in a microirrigation system is often divided into secondary branches (manifolds) which connect to the lateral lines with the emitters. The mains, submains, and manifolds are frequently fitted with flow or pressure regulators. Devices are usually provided at the end of the lines to flush and clean the system (fig 4-60).

Figure 4-60 Typical parts layout of a microirrigation system



C. Water is dissipated from a pipe distribution network under low pressure in a predetermined pattern (figure 4-61). The shape and design of the emitter reduces the operating pressure in the supply line, and a small volume of water is discharged at the emission point

Figure 4-61 Emitter wetted pattern on a fresh pick pea field

D. Detailed design guidelines and calculations for microirrigation systems are available in NRCS, Title 210, National Engineering Handbook, Part 623, Chapter 7 - Microirrigation. This reference may be downloaded from the engineering handbooks section of NRCS eDirectives Electronic Directives System: http://directives.sc.egov.usda.gov/

634.0415 References

A. For references see 210 NEH Part 634 Chapter 1 Section 634.0105 References