United States Department of Agriculture
Natural Resources Conservation Service

Part 650 Engineering Field Handbook
National Engineering Handbook

Chapter 3 Hydraulics

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Chapter 3 – Hydraulics

650.0300 Introduction

The basic hydraulic principles in this chapter apply to soil and water conservation measures involving water flow in pipes, open channels, weirs, and orifices. The intent of this chapter is to help the designer develop a better understanding of hydraulics and use the equations and exhibits contained in the chapter. This chapter includes sections on conversion of units, hydrostatics (water at rest), hydrokinetics (water in motion), pipe flow, open-channel flow, weir flow, orifice flow, and flow measurement.

650.0301 Unit Conversion

A. Equations for calculating hydraulic properties of engineering systems use systems of units that are compatible with each other. As a result, the equations are valid only when correct units are used. When a value for a variable in the equation is known, but the unit does not match the units in the rest of the equation, it is necessary to convert to the correct system of units.

B. This chapter uses the English foot-pound-second system unless otherwise specified. Sometimes, it is necessary to convert to other units, which involves the use of numerical conversion constants. Exhibit A in section 650.0311 presents some frequently used constants.

C. Example – Converting gallons per minute (gpm) to cubic feet (ft^3)

(1) A stock water tank needs to contain 1 day's flow from a spring flowing at a rate of 3 gallons per minute. Determine the capacity, Y, of this stock water tank in cubic feet (ft^3).

$$3\ gallons\ per\ minute\ per\ day = Y\ cubic\ feet$$

(2) Solution: To equate cubic feet to gallons, it is necessary to introduce a factor to convert cubic feet to gallons to make the expression a valid equation. The following expresses another way to write 3 gallons per minute per day:

$$3\ \frac{\left(\frac{gal}{min}\right)}{day} = 3\ \frac{gal}{min} \cdot \frac{day}{1} = 3\ \frac{gal \cdot day}{min}$$

(3) Converting gallons to cubic feet and expressing the time factor in common units yields:

$$3\ \frac{gal \cdot day}{min} \times \frac{60\ min}{hr} \times \frac{24\ hr}{day} \times \frac{1\ ft^3}{7.48} = 577.5\ ft^3, use\ 578\ ft^3$$

(4) All units on the left cancel except cubic feet, leaving 3 gal/min/day equals 578 ft^3, which gives the following conversion equation:

$$X\ gpm \cdot day \times \frac{192.5\ ft^3}{1\ gpm \cdot day} = Y\ ft^3$$

D. Example - Converting acre-feet per hour (ac-ft/hr) to gallons per minute (gpm)

(1) Determine how many gallons per minute are in 1 acre-foot per hour (acre-ft/h)

$$1\ acre \cdot foot\ per\ hour = Y\ gallons\ per\ minute$$

(2) Solution: Step-by-step analysis results in a valid conversion equation consistent in both units and dimensions:

$$\frac{1\ ac \cdot ft}{hr} \times \frac{43{,}560\ ft^2}{ac} \times \frac{1\ hr}{60\ min} \times \frac{7.48\ gal}{ft^3} = 5{,}430\ \frac{gal}{min}$$

E. Example – Converting cubic feet per second-day (cfs-day) to acre-feet (ac-ft)

(1) Determine the number of acre-feet in 1 cubic foot per second per day (ft^3/s/d).

$$1\ cubic\ foot\ per\ second\ per\ day = Y\ acre \cdot feet$$

(2) Solution:

$$1\ \frac{ft^3 \cdot day}{sec} \times \frac{1\ ac}{43{,}560\ ft^2} \times \frac{24\ hr}{day} \times \frac{60\ min}{hr} \times \frac{60\ sec}{min} = 1.9835\ ac \cdot ft$$

or

$$X\ cfs \cdot day \times \frac{1.9835\ ac \cdot ft}{1\ cfs \cdot day} = Y\ ac \cdot ft$$

F. The benefits of using the conversion approach as illustrated are that it:

(1) provides fewer conversion errors;
(2) saves time during original and check computations; and
(3) provides accurate conversion factor selection from standard tables and other sources

650.0302 Hydrostatics

A. Hydrostatics, or fluid statics, deals with fluids at rest or motionless fluids.

B. Pressure-density-height relationships

(1) The fundamental equation of fluid statics relates pressure, density, and depth. Unit pressures in a fluid vary with the depth and fluid unit weight, as expressed by equation 3–1:

$$p = \gamma h\ or\ h = \frac{p}{\gamma} \qquad \text{(eq. 3-1)}$$

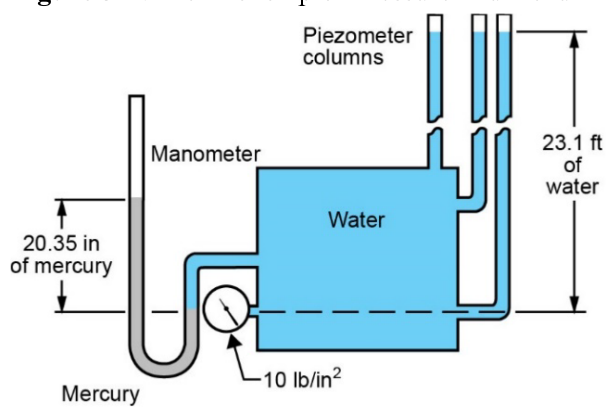
where: p = pressure per unit of area; lb/ft^2
γ = unit weight of the fluid, lb/ft^3
h = depth of submergence, or head, ft

(2) Equation 3–1 shows that pressure at any point in a liquid of given density depends upon the height of the liquid above that point. This vertical height, or head, of the liquid is an indication of pressure. Therefore, units such as inches of mercury (in Hg) or feet of water often describe pressure. Figure 3–1 illustrates pressure and head schematically in the manometers and piezometers.

(3) Example – Calculating height of water in piezometers and height of mercury in manometers

(i) If the tank in figure 3–1 is filled with water until the pressure gage reads 10 pounds per square inch (lb/in2), calculate the height of the water surface in the piezometers and the mercury in the manometer using equation 3–1.

Figure 3-1: Relationship of Pressure and Head



(ii) Solution: Water (unit weight, γ_w, of 62.4 lb/ft3)

$$h = \frac{p}{\gamma_w} = \frac{10 \;\; lb/in^2}{62.4 \;\; lb/ft^3} \times \frac{144 \; in^2}{1 \; ft^3} = 23.1 \; ft$$

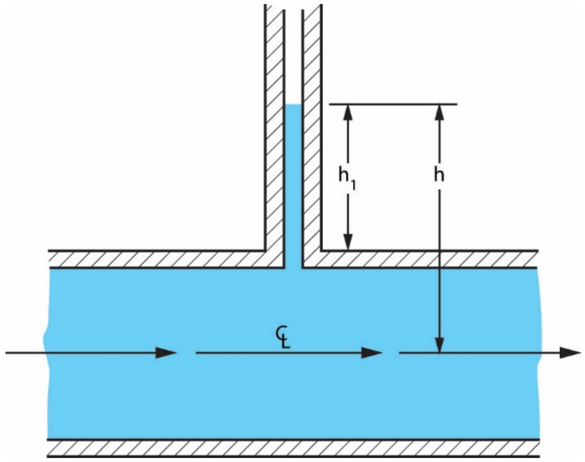
(iii) Solution: Mercury (unit weight, γ_{Hg}, of 849 lb/ft3):

$$h = \frac{p}{\gamma_{Hg}} = \frac{10 \;\; lb/in^2}{849 \;\; lb/ft^3} \times \frac{144 \; in^2}{1 \; ft^3} \times \frac{12 \; in}{1 \; ft} = 20.35 \; in$$

(4) Piezometer

(i) Figure 3–2 shows a piezometer tube connected to a pipe in which the liquid is under pressure. The height (h_1) is a measure of pressure at the wall of the pipe if the opening is at right angles to the wall and free of any roughness or projection into the moving liquid.

Figure 3-2: Piezometer Tube in Pipeline



(ii) The pressure at the wall of the pipe is $p_1=\gamma h_1$ and at the centerline $p=\gamma h$.

(iii) Piezometers also measure water pressure in drainage investigations and earth dam foundation studies. Such a piezometers are vertical, unperforated, small-diameter pipes driven into the soil so that underground water cannot flow freely along the outside of the pipe; and water can enter only at the bottom end. The lower end of the piezometer is at the level where pressure readings are desired. The pressure head is the height that water rises above the bottom of the vertical piezometer pipe.

(iv) Piezometers differ from observation wells used to determine the level of the water table. In an observation well, water freely enters the hole at any depth, thus connecting the various water bearing strata in the soil profile. In a properly installed piezometer, water only enters through the bottom end.

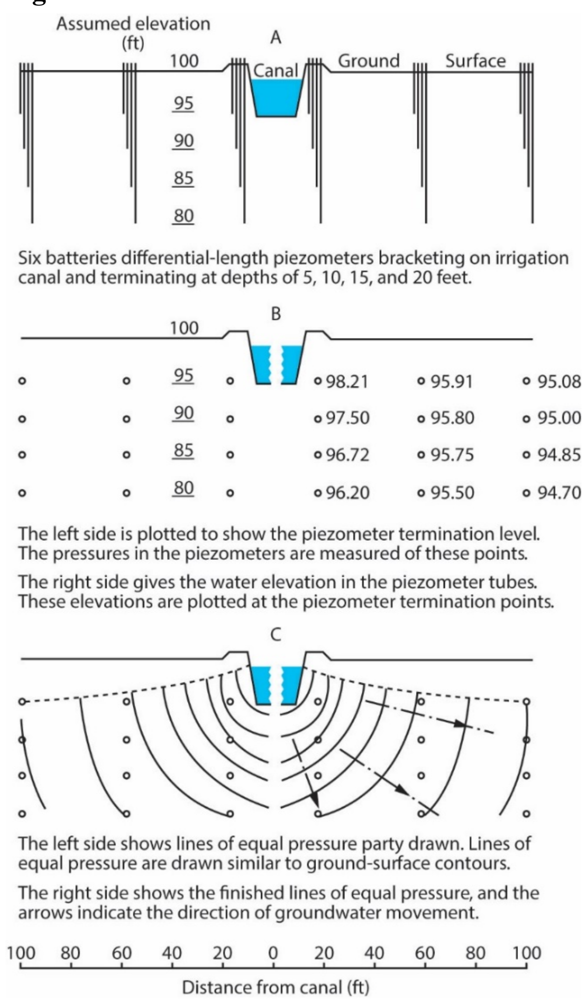
(v) It is possible to investigate canal leakage using piezometers of different lengths as illustrated in figure 3–3. In this example, investigators used sets of four piezometers 5, 10, 15, and 20 feet in length installed at right angles to the axis of the canal 15, 60, and 100 feet from the canal centerline (fig. 3–3(A)). The first objective is to obtain hydrostatic pressures at a large number of points under the water table adjacent to the canal.

(vi) In figure 3–3(B), small circles show the position of the bottom end of each piezometer. On the right-hand side of the sketch, the number beside each circle is the piezometer water level elevation. On the left side, the numbers are the elevations of the bottom ends of the piezometers. Note that water level elevations are greater than the elevations of the bottoms of the corresponding piezometers. The water surface elevation in the piezometer is written at the point of the bottom of the piezometer, not where the water surface is located.

(vii) The third step is to draw contours of equal hydrostatic pressures, as in figure 3–3(C). These pressure lines are drawn in the vertical plane in much the same manner as ground surface contours are drawn in the horizontal plane. Water moves through the soil from high to low pressures at right angles to the pressure contours. The plotted water-table contours in figure 3–3(C) from the piezometers indicate seepage from the canal.

(viii) If artesian pressure exists in the soil profile, deep piezometers show a higher water surface elevation than shorter piezometers, and the groundwater contours indicate upward water pressure.

Figure 3-3: Piezometer Data Plots to Determine Equipotential Lines

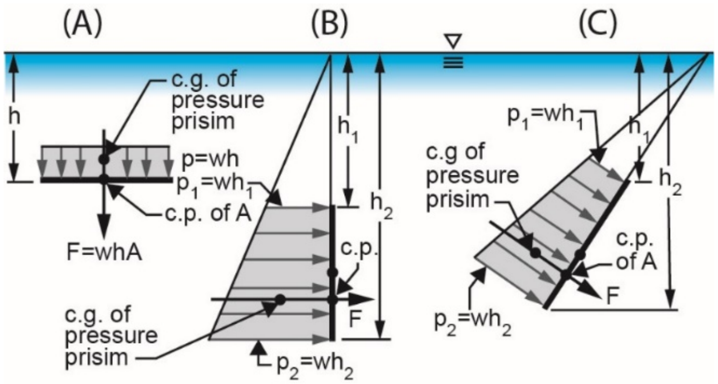


C. Forces on Submerged Plane Surfaces

(1) Calculating the size, direction, and location of forces on submerged surfaces is essential in the design of dams, bulkheads, wetland water control structures, and other hydraulic structures used in soil and water conservation projects. Computing unit and total pressures on submerged horizontal planes (fig. 3–4(A)) is simple because the pressure is uniform over the area. For vertical planes (fig 3–4(B)) and inclined planes (fig. 3–4(C)), the pressure varies with depth, as shown by equation 3–1, producing the typical pressure diagrams and the resultant forces of figure 3–4(B) and figure 3–4(C).

(2) The shaded area multiplied by a unit of length equals volume, known as the pressure volume. The resultant force, F, is the pressure volume and passes through its center of gravity (c.g.). The resultant force also passes through the center of pressure (c.p.).

Figure 3-4: Pressure on Submerged Surfaces



(3) Pressure diagrams. Pressure diagrams usually simplify the analysis of structures under pressure. Unit pressure varies directly with head, so pressure diagrams are generally triangles, trapezoids, or rectangles. The magnitude, direction, and position of the force are essential to understanding forces due to water pressure. The total force in the pressure diagram is usually represented by a single force through the center of pressure in the same direction as the unit pressures. Exhibit B in section 650.0311 gives the commonly used pressure diagrams and methods of computing the hydrostatic load and center of pressure.

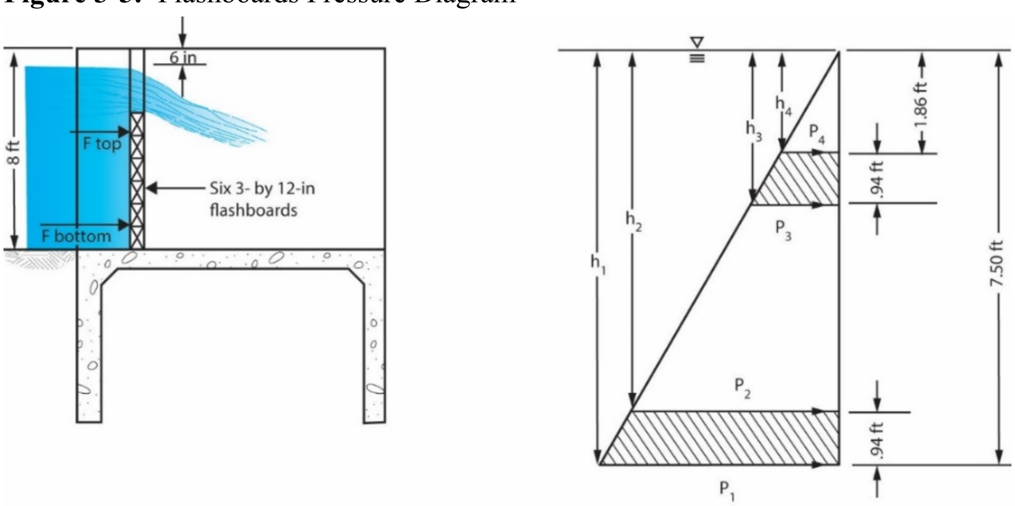
(4) Example – Pressure on flashboards

(i) For a flashboard dam built with six 3- by 12-inch flashboards, what is (a) the load per foot on the bottom board, (b) the total load on the bottom board if it is 6 feet long, and (c) the load per foot on the top board?

(ii) Solution:

- First draw the pressure diagram, figure 3–5. Finished 12-inch boards are 11.25 inches or 0.94 foot wide.

Figure 3-5: Flashboards Pressure Diagram



- Then, using equation 3–1, determine the pressure at p_1, p_2, p_3, and p_4:

$$p = \gamma_w h = (unit\ weight\ of\ water) \cdot (depth\ of\ water)$$
$$p_1 = (62.4\,lb/ft^3)(7.5\,ft) = 468.0\ \ lb/ft^2$$
$$p_2 = (62.4\,lb/ft^3)(7.5\,ft - 0.94\,ft) = 409.3\ \ lb/ft^2$$
$$p_3 = (62.4\,lb/ft^3)(1.86\,ft + 0.94\,ft) = 174.7\ \ lb/ft^2$$
$$p_4 = (62.4\,lb/ft^3)(1.86\,ft) = 116.1\ \ lb/ft^2$$

- From figure 3–4, the hydrostatic load is:

$$F = \gamma hA = pA = (unit\ pressure) \cdot (area)$$

- The load on the bottom flashboard is:

$$\left(\frac{468.0\,lb\ +\ 409.3\,lb}{2}\right) \times (0.94\,ft) = 412.3\,lb/ft$$

- The total load on the 6-foot long bottom flashboard is:

$$(412.3\,lb/ft) \times (6\,ft) = 2{,}474\,lb$$

- The load on the top flashboard is:

$$\left(\frac{174.7\,lb + 116.1\,lb}{2}\right) \cdot (0.94\,ft) = 136.7\,lb/ft$$

- The solution is the same for stoplogs in a water control structure.

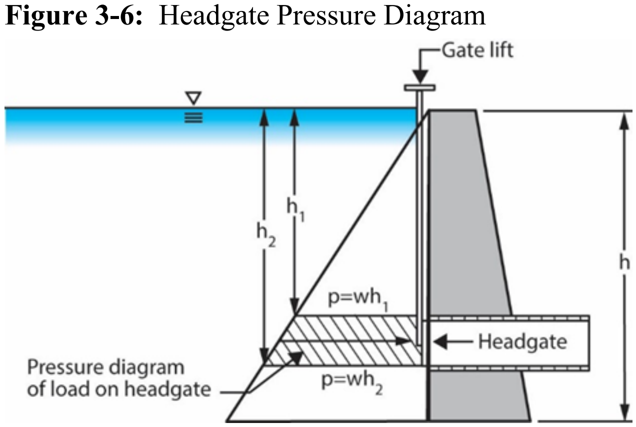
(5) Example – Pressure on headgate

(i) What is the total water load, F, on the 36-inch-wide by 24-inch high headgate shown in figure 3–6 when h_1 is 9 feet?

(ii) Solution:

$$F = \gamma_w \times \left(\frac{h_1 + h_2}{2}\right) \times area$$
$$h_2 = 9\,ft + 2\,ft = 11\,ft$$
$$F = 62.4\frac{lb}{ft^3} \times \left(\frac{9\,ft + 11\,ft}{2}\right) \times (2\,ft \times 3\,ft) = 3{,}744\,lb$$

Figure 3-6: Headgate Pressure Diagram



D. Buoyancy and flotation

(1) The principles of buoyancy and flotation state:

(i) A body submerged in a fluid is buoyed up by a force equal to the weight of fluid displaced by the body.

(ii) A floating body displaces its own weight of the fluid in which it floats.

(2) Buoyancy

(i) A submerged body is acted on by a vertical, buoyant force equal to the weight of the displaced water (eq. 3–2).

$$F_b = V\gamma_w \qquad \text{(eq. 3-2)}$$

where: F_b = buoyant force, lb
V = volume of the body, ft^3
γ_w = unit weight of water, lb/ft^3

(ii) If the unit weight of the body is greater than the unit weight of water, there is a downward force equal to the difference between the weight of the body and the weight of the water displaced; therefore, the body will sink.

(3) Flotation

(i) If the body has a unit weight less than water, the body will float with part of its volume below and part above the water surface in a position so that (eq. 3–3):

$$W = V \times \gamma_w \qquad \text{(eq. 3-3)}$$

where: W = weight of the body, lb
V = volume of the body below the water surface; i.e., the volume of the displaced water, ft^3
γ_w = unit weight of water, lb/ft^3

(4) Submergence affects hydraulic structures. Therefore, it is important to analyze hydraulic structures to ensure the submerged weight of the structure is adequate to resist flotation. Net weight of submerged porous materials differs, depending upon whether air fills the voids (unsaturated) or water fills the voids (saturated). The next example shows that the unsaturated timber is lighter than water and will float; however, the saturated timber will sink because it is heavier than water.

(5) Example—Analysis of submergence on a hydraulic structure

(i) A timber and rock riprap structure is completely submerged in a riparian wetland under normal flood flows. Materials, weights, and volumes of the materials are as shown in figure 3-7. Determine the net weight of 1 cubic yard of the wetland structure when unsubmerged, submerged with unsaturated timber, and submerged with saturated timber.

Figure 3-7: Material, % volume of structure, and unit weight

Material	% volume of structure	Unit weight
Timber	12	55 lb/ft^3—air 73 lb/ft^3—saturated
Rock riprap, 30% voids	88	150 lb/ft^3—solid stone

(ii) Solution:

- Step 1: Compute the cubic feet of timber, solid stone, and voids per cubic yard of structure:
  - Timber: $0.12 \times 27\ \text{ft}^3 = 3.24\ \text{ft}^3$
  - Solid stone: $0.7 \times 0.88 \times 27\ \text{ft}^3 = 16.63\ \text{ft}^3$
  - Voids: $0.3 \times 0.88 \times 27\ \text{ft}^3 = 7.13\ \text{ft}^3$

  (Total volume = $3.24\ \text{ft}^3 + 16.63\ \text{ft}^3 + 7.13\ \text{ft}^3 = 27\ \text{ft}^3 = 1\ \text{yd}^3$)
- Step 2: Compute the net weight of 1 cubic yard of the wetland structure (computations shown in figure 3-8).

Figure 3-8: Net weight of materials per cubic yard when unsubmerged, submerged and unsaturated, and submerged and saturated

Material	Net weight of materials per yd^3		
	Unsubmerged	Submerged	
		Timber Unsaturated	Timber Saturated
Timber	$3.24\ \text{ft}^3 \times 55 \frac{\text{lb}}{\text{ft}^3} =$ 178 lb	$3.24\ \text{ft}^3 \times \left(55 - 62.4 \frac{\text{lb}}{ft^3}\right) =$ − 24 lb	$3.24\ \text{ft}^3 \times \left(73 - 62.4 \frac{\text{lb}}{ft^3}\right) =$ 34 lb
Stone	$16.63\ \text{ft}^3 \times 150 \frac{\text{lb}}{\text{ft}^3} =$ 2,495 lb	$16.63\ \text{ft}^3 \times \left(150 - 62.4 \frac{\text{lb}}{\text{ft}^3}\right) =$ 1,457 lb	$16.63\ \text{ft}^3 \times \left(150 - 62.4 \frac{\text{lb}}{\text{ft}^3}\right) =$ 1,457 lb
Total net weight	**2,673 lb**	**1,433 lb**	**1,491 lb**

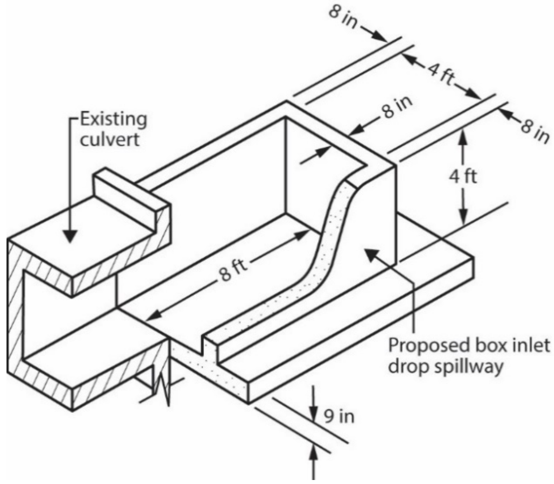
(6) Example - Determination of spread footing size for a hydraulic structure

(i) A box inlet drop spillway for a 4- by 4-foot highway culvert (fig. 3-9) is needed for grade stabilization. Using a factor of safety of 1.5, determine if the proposed structure is safe from flotation. Compute the required size of spread footing if the proposed size is subject to flotation.

(ii) The design assumptions are:

- Soil is saturated to the lip of the box and has a buoyant weight of 50 pounds per cubic feet.
- There is no frictional resistance between the walls of the box and surrounding soil.
- Unit weight of concrete is 150 pounds per cubic feet.
- Unit weight of water is 62.4 pounds per cubic feet.

Figure 3-9: Box-Inlet Drop Spillway for Highway Culvert



(iii) Solution:

- Step 1: First, determine the weight (W) of the box, assuming the slab does not extend beyond the walls.
  - End wall:

    $$4\,ft\ \times 4\,ft\ \times 0.67\,ft \times 150\ lb/ft^3 = 1,608\,lb$$
  - 2 sidewalls:

    $$4\,ft \times 8.67\,ft \times 0.67\,ft \times 150\ lb/ft^3 \times 2 = 6{,}971\,lb$$
  - Floor slab:

    $$5.33\,ft\ \times 8.67\,ft\ \times 0.75\,ft \times 150\ lb/ft^3 = 5{,}199\,lb$$
  - Total weight (W):

    $$1{,}608\,lb + 6{,}971\,lb + 5{,}199\,lb = 13{,}778\,lb$$
- Step 2: Next, determine the buoyant force (F_B) pushing the box up using equation 3–2.

  $$F_B = V \times \gamma_w = (5.33ft\ \times 4.75\,ft \times 8.67\,ft) \cdot \frac{62.4\,lb}{ft^3} = 13{,}697\,lb$$
- Step 3. Determine the uplift factor of safety. From equation 3–3, the proposed structure will float if W is less than or equal to F_B (F_B has been substituted from equation 3–2 for $V\gamma_w$).

  $$Uplift\ factor\ of\ safety = \frac{W}{F_B} = \frac{13{,}778\,lb}{13{,}697\,lb} = 1.01$$

  Since 13,778 pounds is greater than 13,697 pounds, the box will not float; however, the uplift factor of safety is only 1.01 and is below the required value of 1.5.
- Step 4: Determine the required weight of box to keep it from floating:

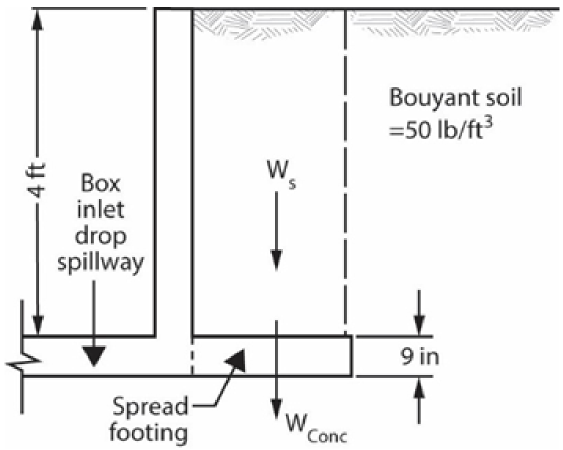
  $$W = (required\ uplift\ factor\ of\ safety) \times F_B$$
  $$= 1.5\ \times 13{,}697\,lb = 20{,}546\,lbs$$

- Step 5: Determine the additional weight to add to box to keep if from floating:

  $$20{,}546\ lb - 13{,}788\ lb = 6{,}758\ lb$$

  A spread footing around three sides of the box will provide the additional weight, along with the weight of the earth load on the footing, as shown in figure 3–10.

Figure 3-10: Spread Footing for Box Culvert



- Step 6: Determine the weight per square foot of spread footing:

  $$W_{soil} + W_{concrete} =$$

  $$\left(4\ ft\ \times 50\ \frac{lb}{ft^3}\right) + \left(0.75\ ft \times \left(150\ \frac{lb}{ft^3} - 62.4\frac{lb}{ft^3}\right)\right) =$$

  $$200\ \ lb/ft^2 + 65.7\ \ lb/ft^2 = 265.7\ \ lb/ft^2$$

- Step 7: Determine the required footing area:

  $$Required\ footing\ area = \frac{6{,}768\ lb}{265.7\ \ lb/ft^2} = 25.5\ ft^2$$

- Step 8: Use trial and error to size the footing.
  - Try a 1-foot spread footing:

    $$Footing\ area_{1-foot} =$$

    $$\big((2 \times 8.67\ ft) + 7.33\ ft\big) \times 1 = 24.7\ ft^2$$

  - Check the footing weight and uplift factor of safety for the structure

$$Footing\ weight = 24.7 ft^2 \times 265.7\ 2\ lb/ft^2 = 6{,}563\ lb$$

  - Check the uplift factor of safety for the 1-foot spread footing

    $$Uplift\ factor\ of\ safety_{1-foot} = \frac{W}{F_B} =$$

    $$\frac{13{,}778\ lb + 6.563\ lb}{13{,}687\ lb} = 1.49$$

- Since the uplift factor of safety for the 1-foot spread footing is less than the required factor of safety of 1.5, repeat the process with a larger spread footing.
- Try a 1.25-foot spread footing:

$$Footing\ area_{1-foot}\ =$$
$$\big((2 \times 8.67\ ft) + 7.83\ ft\big) \times 1.25\ ft\ = 31.5\ ft^2$$
$$Footing\ weight =$$
$$31.5 ft^2 \times 265.7\ lb/ft^2 = 8{,}370\ lb$$
$$Uplift\ factor\ of\ safety_{1-foot} = \frac{W}{F_B} =$$
$$\frac{13{,}778\ lb + 8{,}370\ lb}{13{,}687\ lb} = 1.62$$

- The spread footing needs to be more than 1 foot and less than 1.25 feet to meet the minimum factor of safety (1.5) for uplift.

650.0303 Hydrokinetics

A. Hydrokinetics deals with fluids in motion. Change in motion occurs as the result of a force applied to the fluid body (water in motion).

B. Flow continuity

(1) When the flow through a section of channel or pipe is constant, the flow is steady. If steady flow occurs at all sections in a reach, the flow is continuous. This is known as continuity of flow as expressed by equation 3–4:

$$Q = a_1 v_1 = a_2 v_2 = a_3 v_3 = a_n v_n \qquad \text{(eq. 3-4)}$$

where: Q = discharge in, ft^3/s
a = cross-sectional area, ft^2
v = mean velocity of flow, ft/s
1, 2, 3, n = subscripts denoting different cross sections

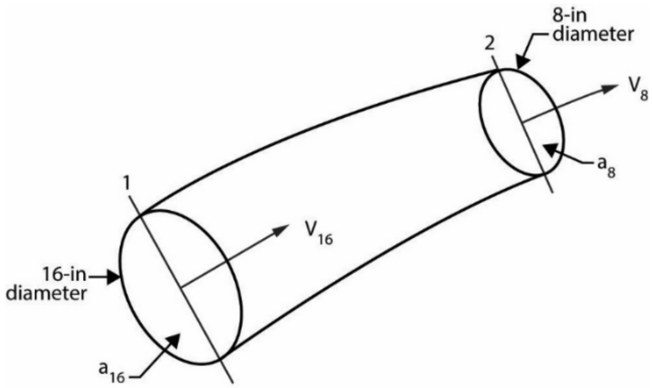
(2) Unsteady flow analysis is more complex than steady flow analysis and is beyond the scope of this chapter.

(3) Example – Computation of average velocity in a pipe

(i) Flow through the tapered pipe shown in figure 3–11 is 10 cubic feet per second.

- Calculate the average velocity at sections 1 and 2 with diameters of 16 and 8 inches, respectively.

Figure 3-11: Tapered Pipe



- Solution:
  - From equation 3–4:

$$Q = a_{16}v_{16} = a_8v_8$$

$$v_{16} = \frac{Q}{a_{16}} = \frac{Q}{\frac{\pi}{4}{d_{16}}^2} = \frac{10\ {ft^3}/_{s}}{\frac{3.1416}{4}\left(16\ in/12\ \frac{in}{ft}\right)^2} = 7.16\ ft/s$$

  - Similarly:

$$v_8 = Q_{a8} = \frac{10\ {ft^3}/_{s}}{\frac{\pi}{4}\left(\frac{8\ in}{12\ in}\right)^2} = 28.6\ ft/s$$

  - Or, based on the ratio of cross-sectional areas:

$$v_8 = v_{16} \times \left(\frac{a_{16}}{a_8}\right) = v_{16}\left(\frac{\frac{\pi}{4}{d_{16}}^2}{\frac{\pi}{4}{d_8}^2}\right) = 7.16\frac{ft}{s} \times \left(\frac{16\ in}{18\ in}\right)^2$$
$$= 28.6\ ft/s$$

C. Conservation of energy

(1) Hydrokinetics considers three forms of energy: potential energy (elevation), kinetic energy (velocity), and pressure energy

(i) Potential energy—Potential energy is the ability to do work because a mass of water is higher than a given datum. A mass of weight, W, at an elevation, z feet, has potential energy of W_z foot-pounds with respect to the datum. The elevation head, z, is a linear quantity in feet and a measure of energy in foot-pounds per pound.

(ii) Kinetic energy—Kinetic energy is a function of the velocity of the fluid:

$$\frac{Wv^2}{2g}$$

where: W = weight of the water, lb
v = velocity, ft/s
g = acceleration due to gravity, 32.2 ft/s^2

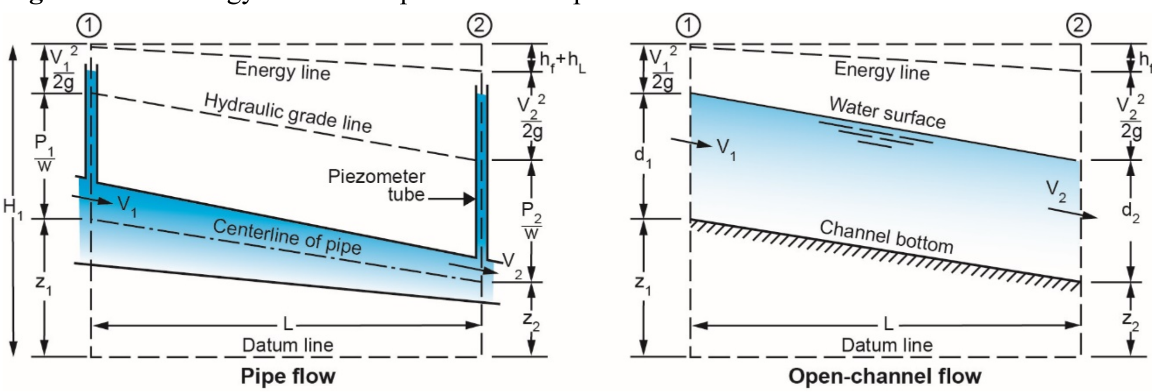
When W equals 1 pound, the kinetic energy is $v^2/2g$ and called the velocity head.

(iii) Pressure energy - Pressure energy is acquired by contact with other masses and is transmitted to or through the liquid mass under consideration. As such, a mass of water does not have pressure energy, but transmits it. Pumps or other applied forces supply pressure energy. The pressure head (h= p/w) also expresses energy per unit weight in foot-pounds per pound.

(2) These three forms of energy are interchangeable. Figure 3-12 illustrates the relationship between the three forms of energy in pipe and open-channel flow.

(3) The total head, H_1, is a vertical distance and represents the value of the total energy in the system at section 1 of figure 3-12. This total head is the sum of the velocity head equivalent to the kinetic energy, the pressure head equivalent to the energy due to pressure, and the elevation head equivalent to the energy due to position.

Figure 3-12: Energy Forms in Pipe Flow and Open-Channel Flow



(4) In channel flow, the velocity head is the elevation difference between the energy grade line and the water surface in the channel.

(5) In pipe flow, the velocity head is the elevation difference between the energy grade line and the elevation to which water would rise in a piezometer tube. The pipe may be slightly lowered or raised without changing the flow rate. If the entrance end of the pipe is lowered, the elevation head is reduced, but the pressure head is increased a corresponding amount. Conversely, if the entrance end of the pipe is raised, the elevation head increases and the pressure head decreases. If flow and pipe diameter remain constant, then velocity head is also constant.

D. Bernoulli Principle

(1) Bernoulli's principle applies the law of conservation of energy to fluid flow. In frictionless flow, the sum of kinetic energy, pressure energy, and elevation energy is equal at all sections along the stream. If velocity head, pressure head, and elevation head are measured at one station in a frictionless pipe or open channel, the total at that station would equal the total at a second station downstream in the same frictionless pipe or open channel, as shown in figure 3–12.

(2) In practice, friction and other energy losses must be considered, and the energy equation becomes equation 3–5:

$$\frac{{v_1}^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{{v_2}^2}{2g} + \frac{p_2}{\gamma} + z_2 + h_f + h_L \qquad \text{(eq. 3-5)}$$

where: v = mean velocity of flow, ft/s
p = unit pressure, ft-lb/ft3
γ = unit weight of water, lb/ft3
g = acceleration of gravity, 32.2 ft/s2
z = elevation head, ft
h_f = friction loss, ft
h_L = all head losses other than friction between stations 1 and 2 ,such as bends, ft

(3) The continuity equation (eq. 3–4) and the energy equation (eq. 3–5) are the two basic equations used to solve problems in water flow.

E. Hydraulic and energy gradients.

The hydraulic gradient in open-channel flow is the water surface, and in pipe flow, it is the elevation to which the water would rise in a piezometer tube along the pipe. The energy grade line is above the hydraulic grade line, a distance equal to the velocity head. In open-channel and pipe flow, the slope, or fall, of the energy grade line for a length of channel or pipe represents the loss of energy due to friction. Together, the hydraulic and energy grade lines reflect the loss of energy by friction and conversion between the three energy forms.

650.0304 Pipe Flow

A. Pipe flow exists when a closed conduit of any form is flowing full. In pipe flow, the conduit cross section fixes the cross-sectional area of flow and the water surface is not exposed to the atmosphere. The internal pressure within 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 culverts, drop inlets, regular and inverted siphons, and various types of pipelines.

C. Section 650.03030described the concept of flow continuity and the Bernoulli principle. This section defines laminar and turbulent flow, describes the commonly used discharge equations, and outlines the hydraulics of pipelines and culverts.

D. Laminar and turbulent flow

(1) Flow is divided into laminar flow and turbulent flow. Laminar flow occurs when individual particles of water move in parallel layers. The velocities of these layers are not necessarily the same; however, the mean flow velocity varies directly with the slope of the hydraulic grade line. Water flow through soils, for example, is laminar.

(2) In turbulent flow, the particles follow unpredictable paths. The main velocity is in the direction of flow, but there are transverse velocity components. The mean flow velocity varies with the square root of the slope of the hydraulic grade line. Most pipe flow is turbulent.

(3) Friction loss—The energy loss from turbulence at the interface of the conduit and flowing water is friction loss. The rate of friction head loss is constant for a straight conduit flowing full with constant cross section and uniform roughness, so the energy grade line slopes downward in the direction of flow equal to the friction head loss per foot of conduit. The two most commonly used equations in conservation practice design are Manning's and Hazen-Williams.

E. Manning's equation

(1) The general form of Manning's equation is equation 3–6. It can be adapted for various situations using the variables defined.

$$v = \frac{1.486}{n} r^{2/3} \cdot s^{1/2} \quad \text{(eq. 3-6)}$$

where: v = velocity, ft/s
n = Manning's roughness coefficient, dimensionless
r = hydraulic radius, ft
s = head loss, ft/ft of conduit

(2) The hydraulic radius, r, is defined as the flow area divided by the wetted perimeter.

$$r = a/p$$

where: r = hydraulic radius, ft
a = flow area, ft2
p = wetted perimeter, ft

(3) For a pipe flowing full, the hydraulic radius is one-fourth the pipe diameter (*d/4*).

(4) Starting with equation 3–6, solve for s: multiply the numerator and denominator of the right side of the equation by 2g, and substitute (H_f/L) for s.

$$H_f = \frac{29.164 n^2 L v^2}{2 g r^{4/3}}$$

where: H_f = friction head loss, ft
n = Manning's roughness coefficient, dimensionless
L = conduit length, ft
v = velocity, ft/s
g = acceleration of gravity, 32.2 ft/s2
r = hydraulic radius, ft = *A/p* = *d/4* for round pipe

(5) The equation can be simplified by creating a variable, K_c as shown in equation 3–7.

$$K_c = \frac{29.164 n^2}{r^{4/3}} \quad \text{(eq. 3-7)}$$

where: K_c = head loss coefficient for any conduit, dimensionless
n = Manning's roughness coefficient, dimensionless
r = hydraulic radius, ft
g = acceleration of gravity, 32.2 ft/s^2
L = conduit length, ft
v = velocity, ft/s

(6) Then the equation takes the form of equation 3–8.

$$H_f = K_c L \left(\frac{v^2}{2g}\right) \quad \text{(eq. 3-8)}$$

(7) To adapt equation 3–7 for circular pipes with the diameter expressed in inches instead of feet, r becomes (d/4), and d becomes d_i as in equation 3–9.

$$K_p = \frac{5{,}087n^2}{(d_i)^{4/3}} \qquad \text{(eq. 3-9)}$$

Exhibits C and D in section 650.0311 provide tables for values of K_p and K_c for the usual ranges of variables.

(8) References, such as King's Handbook of Hydraulics, provide several working forms of Manning's formula:

$$H_f = 2.87n^2 \frac{Lv^2}{d^{4/3}}$$

$$H_f = 4.66n^2 \frac{LQ^2}{d^{16/3}}$$

$$d = \left(\frac{2.159Qn}{s^{1/2}}\right)^{3/8}$$

$$d_i = \left(\frac{1{,}630Qn}{s^{1/2}}\right)^{3/8}$$

(9) Designers typically use software such as U.S. Department of Agriculture (USDA) Natural Resources Conservation Service's (NRCS) Engineering Field Tools Hydraulics or other commercial software to compute Manning's equation for typical pipes.

(10) The table in figure 3-13 lists a range of suggested values for Manning's *n* for typical pipe materials.

Figure 3-13: Values of Manning's n for pipe flow

Pipe description	n
Cast-iron, coated	0.010–0.014
Cast-iron, uncoated	0.011–0.015
Steel, riveted and spiral	0.013–0.017
Annular corrugated metal	0.021–0.025
Helical corrugated metal, 6- to 24-in dia.	0.014–0.020
Neat cement surface	0.010–0.013
Concrete	0.010–0.017
Vitrified sewer pipe	0.010–0.017
Clay, common drainage tile	0.014–0.017
Corrugated plastic tubing, 3–to 6-in dia.	0.015
Corrugated plastic tubing, 8- to 10-in dia.	0.017
Corrugated plastic tubing, 12- to 15-in dia.	0.018
Corrugated plastic tubing, 18- to 24-in dia.	0.020
PVC	0.009–0.011
HDPE, smooth wall interior	0.012

(11) Exhibits E through L in section 650.0311 provide a graphical solution for solving Manning's equations for pipe diameters 4 to 96 inches and *n* values from 0.009 to 0.025.

F. Hazen-Williams Equation

(1) The Hazen-Williams equation applies to turbulent flow, but not laminar flow. Turbulent flow is the predominant flow regime for conservation practice designs. Equation 3-10 presents the generally-used form of the Hazen-Williams equation:

$$v = 1.318Cr^{0.63}s^{0.54} \qquad \text{(eq. 3-10)}$$

where: v = velocity, ft/s
C = Hazen-Williams roughness coefficient
r = hydraulic radius, ft
s = head loss, ft/ft

(2) The hydraulic radius and slope are the same as in Manning's equation. The new term, C, is the Hazen-Williams roughness coefficient. Since Q = av, equation 3–10 may be converted to discharge in any conduit:

$$Q = 1.318aCr^{0.63}s^{0.54}$$

where: a = cross-sectional flow area, ft2

(3) Area (a) and hydraulic radius (r) can be described in terms of the inside pipe diameter in inches, d_i. Equation 3-11 is the general formula for discharge in circular pipes.

$$\frac{Q}{C} = 0.0006273{d_i}^{2.63} \cdot s^{0.54} \qquad \text{(eq. 3-11)}$$

where: d_i = pipe diameter, in

(4) Solutions of equation 3–11 for standard pipes ranging from 1 to 12 inches in diameter and a wide range in slopes may be made by using exhibits M through O in section 650.0311.

(5) Figure 3-14 lists values of C for different types of pipe.

Figure 3-14: Values of Hazen-Williams C

Pipe description	C
PVC	150
Smooth-wall HDPE	150
Welded steel pipe, new	130
Welded steel pipe, old	80
Galvanized steel pipe—dia < 4 in	80
Galvanized steel pipe—dia 4 to 12 in	100
Galvanized steel pipe—dia > 12 in	110
Concrete, very smooth, excellent joints	140
Concrete, smooth, good joints	120
Concrete, rough	110
Vitrified clay	110
Corrugated metal pipe	60

(6) Regardless of the designer's preference of equations, it is important to check results against the applicable conservation practice design standards.

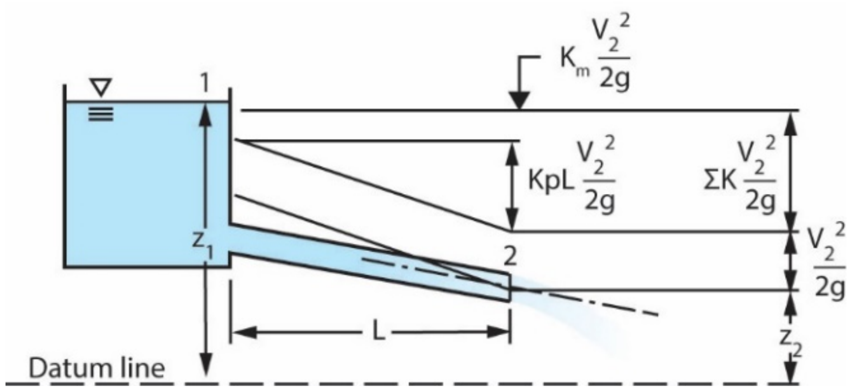
G. Other Losses

(1) In addition to friction head losses, there are other energy losses from turbulence created by changes in velocity and flow direction. To easily incorporate them into Bernoulli's energy equation, such losses are expressed in mean velocity head at a specific cross section of the pipe.
(2) These losses are known as minor losses, regardless of their magnitude. In long pipelines, the entrance and bend losses are a small part of the total loss and in such cases can be ignored. In relatively short structures such as culverts, drop inlets, and siphons, good design requires an estimate of these minor losses. When minor losses amount to 5 percent or more of the total head loss, they should be included in the flow calculations.
(3) As velocities increase, minor losses grow in importance. The 0.5 times ($v^2/2g$) entrance loss at 3 feet per second is only 0.07 foot. When the velocity increases to 30 feet per second, the 0.5 times ($v^2/2g$) entrance loss becomes a 7-foot head loss.
(4) Data on minor losses are found in exhibits P and Q in section 650.0311 of this chapter, and Title 210, National Engineering Handbook, Part 634, "Hydraulics" (210-NEH-634) - (publication date to be determined).

H. Pipeline Hydraulics

(1) The common pipe flow condition in conservation practices is free flow discharge, shown in figure 3–15.

Figure 3-15: Pipe Flow Energy Relationships



(2) The general pipe flow equation is derived from the Bernoulli's equation and continuity principles.
(3) Equating the energy in figure 3–13, using equation 3–5:

$$0 + 0 + z_1 = \frac{V^2}{2g} + 0 + z_2 + \sum\left(K\frac{v^2}{2g}\right)$$

where: z_1 = elevation head at section 1
z_2 = elevation head at section 2
$v^2/2g$ = velocity head at section 2
$\sum(K\, v^2/2g)$ = sum of the minor head losses and pipe friction losses

(4) Let $H_L = z_1 – z_2$. Then:

$$H_L = z_1 - z_2 = \frac{v^2}{2g}(1 + \Sigma K)$$

Rearrange and solve for v:

$$v = \sqrt{\frac{2gH}{1 + K_m + (K_{p \cdot L})}}$$

From the continuity principle, the pipe flow equation becomes equation 3–12:

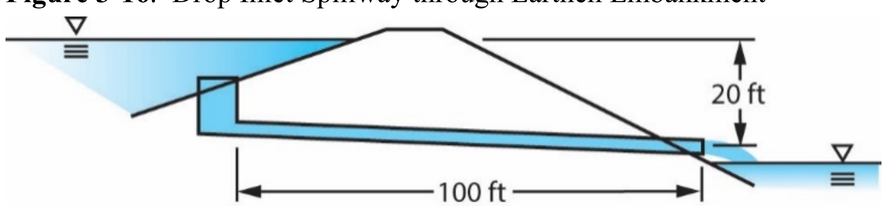
$$Q = a\sqrt{\frac{2gH}{1+K_m+(K_{p \cdot L})}} \qquad \text{(eq. 3-12)}$$

where: Q = flow, ft²/s
a = pipe area, ft²
g = acceleration of gravity, 32.2 ft/s²
H = elevation head difference, ft
K_m = coefficient of minor losses, dimensionless
K_p = pipe friction coefficient, dimensionless
L = pipe length, ft

(5) Example – Flow through a drop-inlet spillway conduit

(i) Determine the flow of a drop-inlet spillway conduit when the water surface in the upstream pool is 20 feet higher than the outlet (fig. 3–16). Assume full barrel flow. The conduit barrel is a 24-inch-diameter concrete pipe with Manning's n = 0.013. Set K_m equal to 1.0 for bend and entrance losses.

Figure 3-16. Drop Inlet Spillway through Earthen Embankment



(ii) Solution: Using equation 3–12:

$$Q = a\sqrt{\frac{2gH}{1 + K_m + (K_{p \cdot L})}}$$

$$a = \frac{\pi}{4}d^2 = \frac{3.14 \times (2\,ft)^2}{4} = 3.14 ft^2$$

and equation 3–9 (Exhibit C(a) and C(b) in section 650.0311 provides additional K_p values):

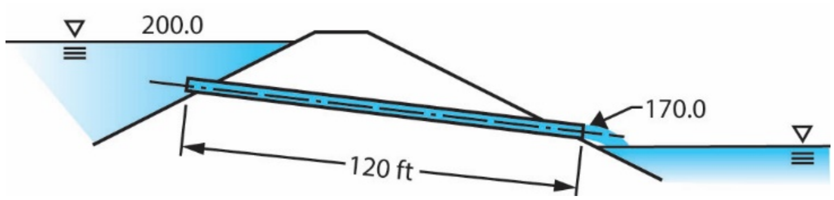
$$K_p = \frac{5{,}087n^2}{(d_i)^{4/3}} = \frac{5{,}087 \times (0.013)^2}{(24\,in)^{4/3}} = 0.0124$$

$$Q = 3.14\,ft^2 \times \sqrt{\frac{2 \times 32.2\ \,ft/s^2 \times 20\,ft}{1 + 1 + (0.0124 \times 100\,ft)}} = 63\,ft^3/s$$

(6) Example – Required pipe diameter under full-flow conditions

(i) A corrugated metal pipe with a hooded inlet through a wetland dike (fig. 3–17) needs to discharge 130 cubic feet per second when the wetland water surface is at elevation 200.0 feet and the centerline of the pipe outlet is elevation 170.0 feet. Determine the required pipe diameter under full-flow conditions. Use Manning's *n* equals 0.024.

Figure 3-17. Corrugated Metal Pipe with Hooded Inlet through a Wetland Dike



(ii) Solution:

- Step 1: Select a diameter, and determine the discharge using equation 3–12.

$$Q = a\sqrt{\frac{2gH}{1 + K_m + (K_{p \cdot L})}}$$

- Step 2: Find K_m for entrance loss and bends. Since the hooded inlet has no bends, K_b equals 0. From exhibit P in section 650.0311, select the entrance loss for an inward projecting pipe, hooded inlet: K_e equals 1.0.

$$K_m = K_b + K_e = 0 + 1.0 = 1.0$$

- Step 3: Determine K_p from equation 3–9 for a 36-inch pipe.

$$K_{p=}\frac{5{,}087 \times (0.024)^2}{(36\ in)^{4/3}} = 0.0246$$

- Step 4: Insert the values into equation 3–12 for a 36-inch pipe.

$$Q_{36\ in} = 7.07\ ft^2\sqrt{\frac{2 \times 32.2\ ft/s^2 \times (200\ ft - 170\ ft)}{1 + 1.0 + (0.0246 \times 120)}} = 140\ ft^3/s$$

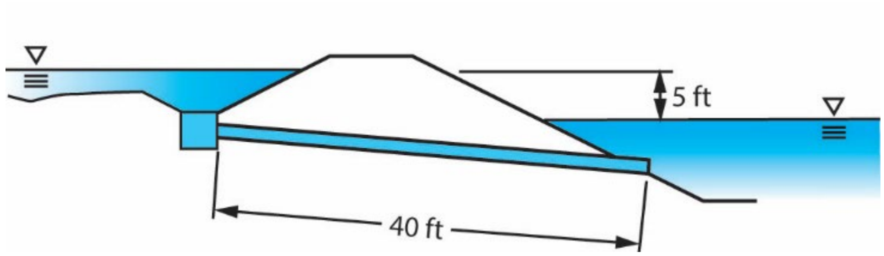
- Trial 1
  -- Diameter=36 in
  -- Area of CMP pipe=7.07 ft^2
  -- K_p=0.0246
  -- K_m=1.0
  -- Q=140 ft^3/s
- Trial 2
  -- Diameter=30 in
  -- Area of CMP pipe=4.91 ft^2
  -- K_p=0.0314
  -- K_m=1.0
  -- Q=90 ft^3/s

- The capacity of a 30-inch pipe is only 90 cubic feet per second. Since the design discharge is 130 cubic feet per second, use a 36-inch pipe with 140 cubic feet per second capacity in the wetland dike design.

(7) Example – Discharge through a field diversion

(i) An 8-inch-diameter field diversion discharges below the water surface of a nearby stream (fig. 3–18). The 40-foot corrugated plastic pipe (n = 0.020) flows full with 5 feet of head. Determine the discharge. The minor losses (entrance coefficient and bend coefficient) are 1.0.

Figure 3-18: Field Diversion Discharging Below the Water Surface of a Stream



(ii) Solution:

- Equation 3–12 also applies to submerged flow, as well as free outlet flow.

$$Q = a\sqrt{\frac{2gH}{1 + K_m + (K_{p \cdot L})}}$$

$$a = \frac{\pi}{4}d^2 = \frac{3.14 \times \left(\frac{8\ in}{12\ in/ft}\right)^2}{4} = 0.349\ ft^2$$

- Using equation 3–9:

$$K_p = \frac{5{,}087\ n^2}{{d_i}^{4/3}} = \frac{5{,}087\ \times 0.020^2}{8^{4/3}} = 0.1272$$

- Insert into equation 3–12 for full pipe flow:

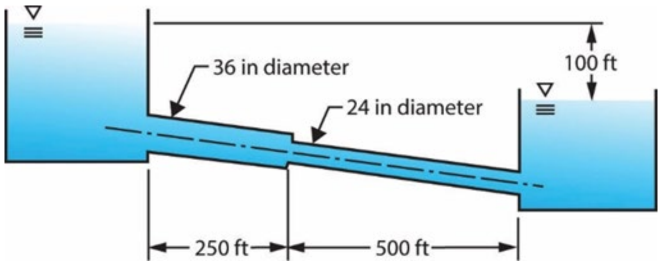
$$Q = 0.349\ ft^2\sqrt{\frac{2 \times 32.2\ ft/s^2 \times 5\ ft}{1 + 1 + (0.1272 \times 40)}} = 2.4\ ft^3/s$$

- The 8-inch field diversion discharges 2.4 cubic feet per second into the stream.

(8) Example – Pipeline flow between irrigation reservoirs

(i) A steel pipeline connects two irrigation reservoirs (fig. 3–19). The pipeline has 250 feet of 36-inch and 500 feet of 24-inch pipe. The upper reservoir water surface is 100 feet higher than the lower reservoir water surface. The entrance coefficient is 1, the contraction coefficient is 0.25, and Manning's n is 0.011. What is the flow in the pipeline between the irrigation reservoirs under these conditions?

Figure 3-19: Steep Pipeline Connecting Two Irrigation Reservoirs



(ii) Solution:

- To use equation 3–12, the loss coefficients need be expressed in terms of a single-size pipe or equivalent pipe diameter. In terms of the 24-inch pipe, the coefficients are multiplied by the ratio, C, based on the square of the pipe areas.

$$C_{36} = \left(\frac{area\ of\ 24\ in\ diameter\ pipe}{area\ of\ 36\ in\ diamater\ pipe}\right)^2 = \left(\frac{3.14\ ft^2}{7.07\ ft^2}\right)^2 = 0.197$$

$$C_{24} = \left(\frac{area\ of\ 24\ in\ diameter\ pipe}{area\ of\ 24\ in\ diamater\ pipe}\right)^2 = \left(\frac{3.14\ ft^2}{3.14\ ft^2}\right)^2 = 1.000$$

- The friction loss coefficient, K, is multiplied by the equivalent pipe diameter, C, to get the adjusted loss coefficient.

$$K_{adj} = \left(K_p L\right)C = KC$$

- The table in figure 3-20 shows the determination of the adjusted loss coefficient:

Figure 3-20: Sum of the loss coefficients

Item	K_p (exhibit C In section 650.0311)	Length (ft)	K	C	K_{adj} Adjusted loss coefficient
Entrance	—	—	0.5	0.197	0.099
36-in pipe	0.00518	250 ft	1.295	0.197	0.255
Contraction	—	—	0.25	0.197	0.049
24-in pipe	0.00889	500 ft	4.45	1.000	4.45
Exit	—	—	1.0	1.000	1.0
				TOTAL:	5.853

- Discharge:

$$Q = 3.14\ ft^2 \sqrt{\frac{(2)(32.2\ ft/s^2)(100\ ft)}{1 + 5.853}} = 963\ cfs$$

I. Culvert Hydraulics

(1) In general, culverts are conduits passing under roads, railroads, walking trails, or other structures designed to move traffic. They are designed to convey water from one side of the structure to the other (fig. 3–21). Common culvert shapes are circular, rectangular, oval, and arched. Culverts are made of metal, concrete, plastic, brick, stone, or wood.

Figure 3-21: Typical Road Culvert in Virginia

(2) There are two major types of culvert flow:
  (i) flow under inlet control
  (ii) flow with outlet control

(3) For each type, different factors and formulas are used to compute the hydraulic capacity of the culvert. Under inlet control, the slope, the roughness and diameter of the culvert barrel, the inlet shape, and the amount of headwater or ponding at the entrance must be considered. Outlet control also considers tailwater elevation in the outlet channel and the culvert length.

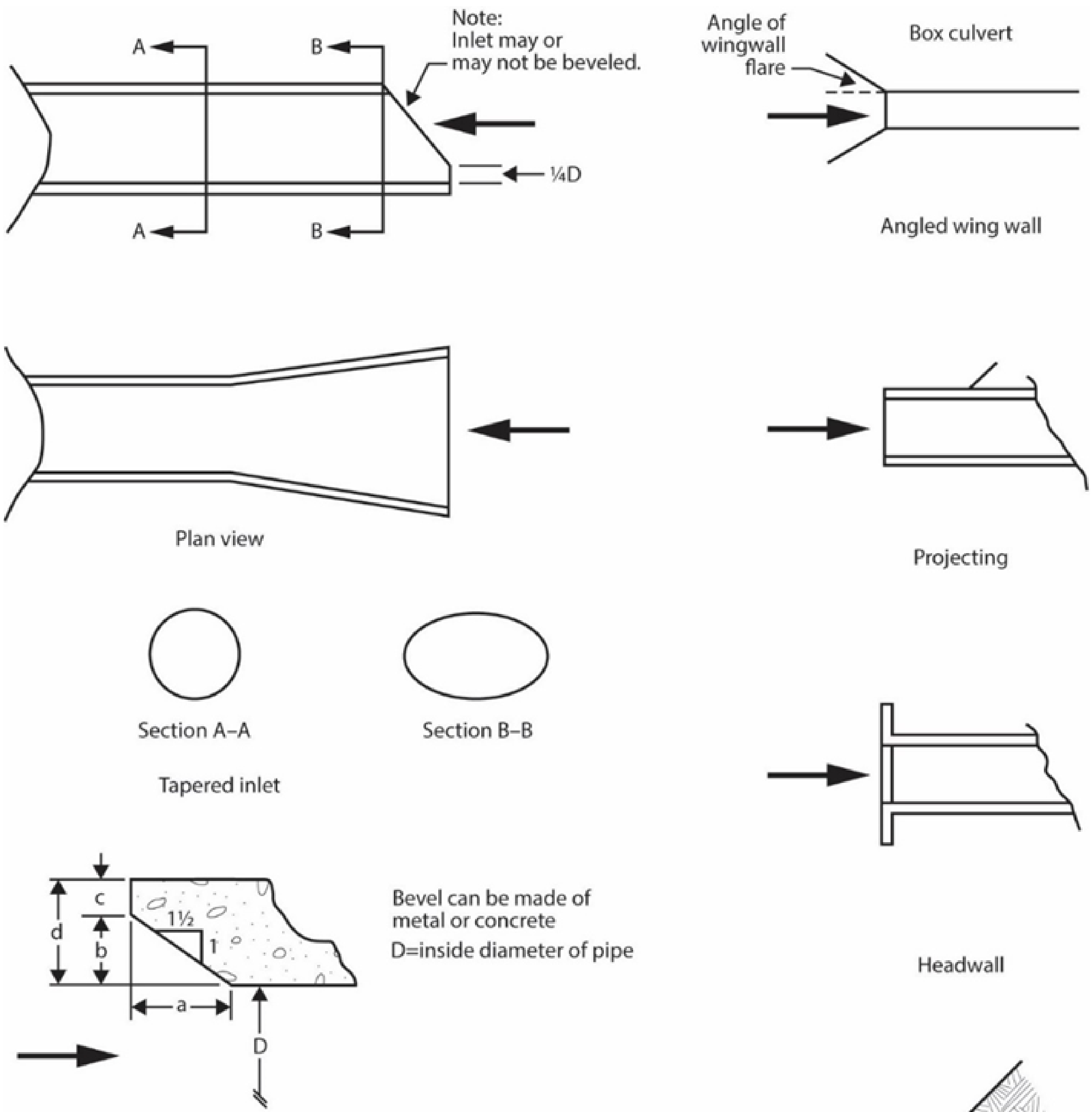
(4) To determine whether flow through the culvert is inlet controlled or outlet controlled, the designer computes both inlet-controlled flow and outlet-controlled flow. The smallest of these two computed values dictates the culvert flow regime.

(5) In situations where the culvert has an inefficient, projecting inlet without a headwall and there is very little roadfill covering the culvert, the culvert is likely to experience inlet control while the upstream water surface is below the road surface. Outlet control is more common for culverts with efficient, headwall inlets and the upstream water surface can reach a much greater depth over the culvert inlet before overflowing the road surface. Also, a steeper sloping culvert is more likely to experience inlet control than one installed on a flatter slope.

(6) "Hydraulic Design of Highway Culverts" (2012) documents the Federal Highway Administration (FHWA) research-based procedures for culvert design. The nomographs in exhibits R through AA in section 650.0311 were developed from FHWA research.

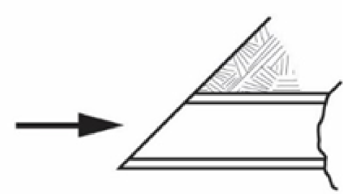
(7) Culvert inlet types—Hydraulic Design of Highway Culverts (2005) describes various inlet types. Figure 3-22 illustrates some of these.

Figure 3-22: Types of Culvert Inlets



Bevel	$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$
A	0.042	0.063	0.042	0.083
B	0.083	0.125	0.042	0.125

Bevels A and B



Mitered

(i) Tapered—This inlet is a type of improved entrance that can be made of concrete or metal. The larger diameter of the inlet gradually reduces to the diameter of the culvert.

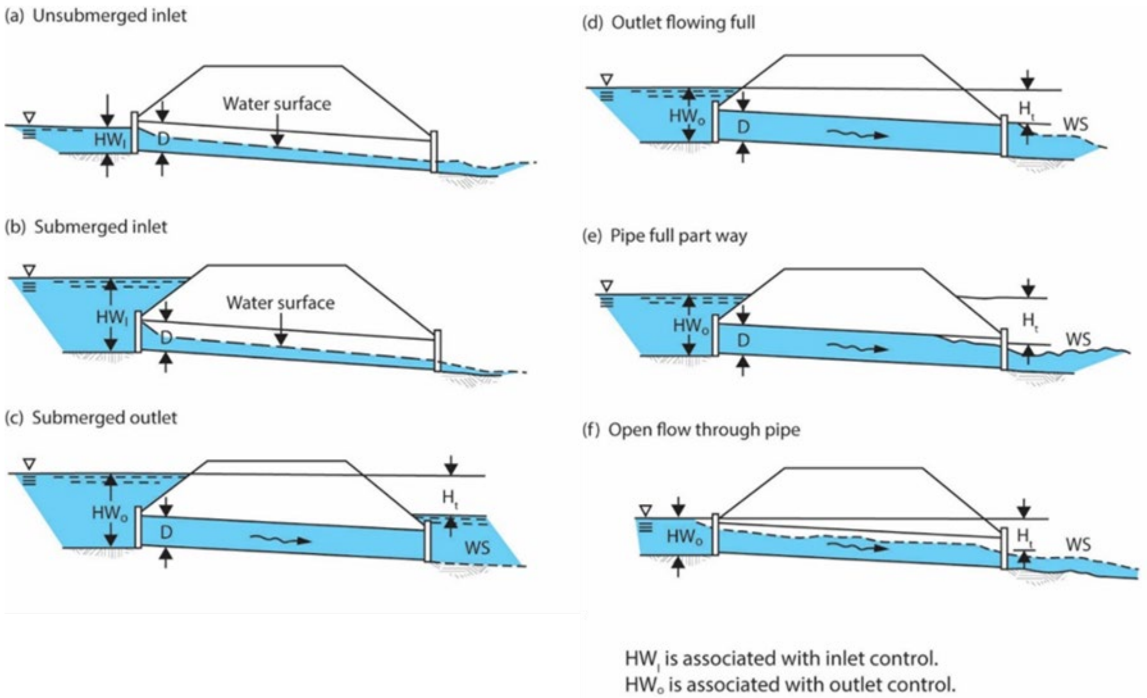
(ii) Bevel A and B—These bevels, a type of improved entrance, can be formed of concrete or metal. The square edge of a culvert inlet has been formed to an angled entrance to improve flow conditions into the culvert.
(iii) Angled wingwall—Similar to headwall, but at an angle with culvert (fig. 3–23).
(iv) Projecting—The culvert barrel extends from the embankment.
(v) Headwall—A concrete or metal structure placed around the entrance of the culvert. Headwalls considered are those giving a flush or square edge with the outside edge of the culvert barrel. No distinction is made for wingwalls with skewed alignment.
(vi) Mitered—The end of the culvert barrel is on a miter or slope to conform with the fill slope. All degrees of miter are treated alike since research data on this type of inlet are limited.

(8) Headwater is measured from the culvert invert midway into the mitered section.
(i) End section—The common prefabricated end made of either concrete or metal and placed on the inlet or outlet ends of a culvert. The closed portion of the section, if present, is not tapered (not illustrated).
(ii) Grooved edge—The bell or socket end of a standard concrete pipe is an example of this entrance (not illustrated).

Figure 3-23: Culvert inlet with angled wingwall (Photo courtesy of USDA-NRCS)

(9) Culvert flow–inlet control—Under inlet control, the culvert discharge is controlled at the culvert entrance (fig. 3–24) by the depth of headwater (HW_1) and the entrance geometry of the culvert, including the barrel shape and cross-sectional area, and the type of inlet edge, shape of headwall, and other losses. With inlet control, the entrance acts as an orifice, so that flow in the culvert barrel does not become pressure flow. Figure 3–24(a) and (b) represent two types of inlet-controlled flow. Headwater depth is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth + velocity head).

Figure 3-24: Culvert flow conditions



(10) Example – Discharge capacity of a concrete pipe culvert

(i) Determine the discharge capacity of an existing 42-inch concrete pipe culvert. The allowable headwater depth (HW) upstream is 8.0 feet, and the culvert slope is 0.02 foot per foot. The culvert has a projecting entrance and no backwater from downstream flow. Assume inlet control.

(ii) Solution:

- Using exhibit R in section 650.0311, compute HW/D ratio.

$$\frac{HW}{D} = \frac{8\,ft\ \times 12\ in/ft}{42\ in} = 2.3$$

- At 2.3 on scale 3, projecting entrance, draw a horizontal line to scale 1. From this point on scale 1, draw a connecting line between it and 42-inch-diameter on the diameter scale. On the discharge scale, read Q equal 130 cubic feet per second.
- If the culvert slope is steeper than the neutral slope, inlet controls. Otherwise, if the culvert slope is less than neutral slope, then outlet controls. Check the culvert slope to see if outlet controls.

$$s_0 > s_n$$

where: s_o = culvert slope

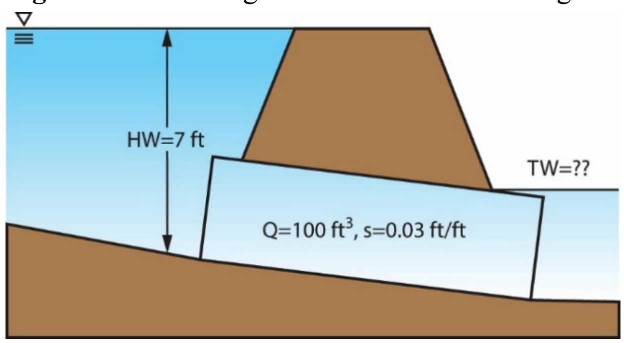
s_n = neutral slope - slope at which friction head loss equals elevation head

- Manning's *n* for concrete pipe equals 0.012 for Q equals 130 cubic feet per second and D equals 42 inches (exhibit H in section 650.0311). The minimum pipe slope to carry 130 cubic feet per second in a 42-inch-diameter pipe is 0.014 foot per foot. The installed culvert slope is greater than the neutral slope (0.020 ft/ft > 0.014 ft/ft). Therefore, the 42-inch-diameter culvert is in inlet control.

(11) Example – Required diameter of a corrugated metal pipe culvert stream crossing

(i) Determine the required diameter of a corrugated metal culvert for a proposed stream crossing in figure 3–25. The design discharge is 100 cubic feet per second, the maximum headwater depth is 7.0 feet, and the culvert will be placed on a 0.03 foot per foot slope. There will be no backwater from downstream flow. The culvert entrance will be mitered to conform to the embankment slope.

Figure 3-25: Corrugated Metal Culvert Through Steam Crossing Embankment



(ii) Solution:

- There is no direct solution, so check several different sizes. The initial trial size can be obtained from exhibit L in section 650.0311 for n equals 0.025, slope equals 0.03 foot per foot, and Q equals 100 cubic feet per second. From exhibit U in section 650.0311, start with a 36-inch-diameter culvert.
- Exhibit U in section 650.0311 applies to corrugated metal culverts with inlet control. Starting from the 36-inch diameter on the diameter scale, draw a line through 100 cubic feet per second on discharge scale to scale 1, then move horizontally to scale 2 (mitered to conform to slope). Read the HW/D ratio on scale 2 as 3.8.

$$HW = \frac{HW}{D} \times D = 3.8 \times 3.0 = 11.4\ ft$$

- The required headwater is greater than the design headwater, so try a larger diameter. The table in figure 3-26 summarizes the trials.

Figure 3-26: Summary of Trial Solutions

Culvert diameter, D (in)	Exhibit U (section 650.0311) scale 2 HW/D ratio	Required HW (ft)
36	3.8 D	11.4
42	2.2 D	7.7
48	1.45 D	5.8

- The 48-inch-diameter corrugated pipe required HW is less than the 7.0 feet maximum design HW for the stream crossing. Exhibit L in section 650.0311 shows the neutral slope for a 48-inch culvert is 0.018 foot per foot. Since the installed culvert slope is greater than the neutral slope, (0.03 ft/ft > 0.018 ft/ft) inlet flow controls. Therefore, since a 48-inch-diameter culvert is the smallest culvert with a 100 cubic feet per second capacity with a required headwater depth less than 7 feet, use a 48-inch-diameter culvert.

(12) Culvert flow–Outlet control

(i) Within the culvert barrel, outlet flow conditions produce flow in one of three ways:

- full pipe flow for the entire length of the culvert barrel, or full flow
- full pipe flow for part of the culvert barrel length
- open flow for the entire length of the culvert barrel

(ii) Figure 3–24(c), (d), (e), and (f) show various types of outlet control flow.

(iii) The equation and graphs for solving head loss give accurate results for the conditions shown in figure 3–25(c), (d), (e), and (f). For the fourth condition shown in figure 3–24(f), the accuracy decreases as the head decreases. The head H_t (fig. 3–24(c) and (d)) or the energy required to pass a given discharge through the culvert flowing in outlet control with the barrel flowing full throughout its length consists of three major parts: velocity head H_v, entrance loss H_e, and friction loss H_f, all expressed in feet.

(iv) Equation 3–13 describes the relationship of three terms (fig. 3–27(a)).

$$H_t = H_v + H_f + H_e \qquad \text{(eq. 3-13)}$$

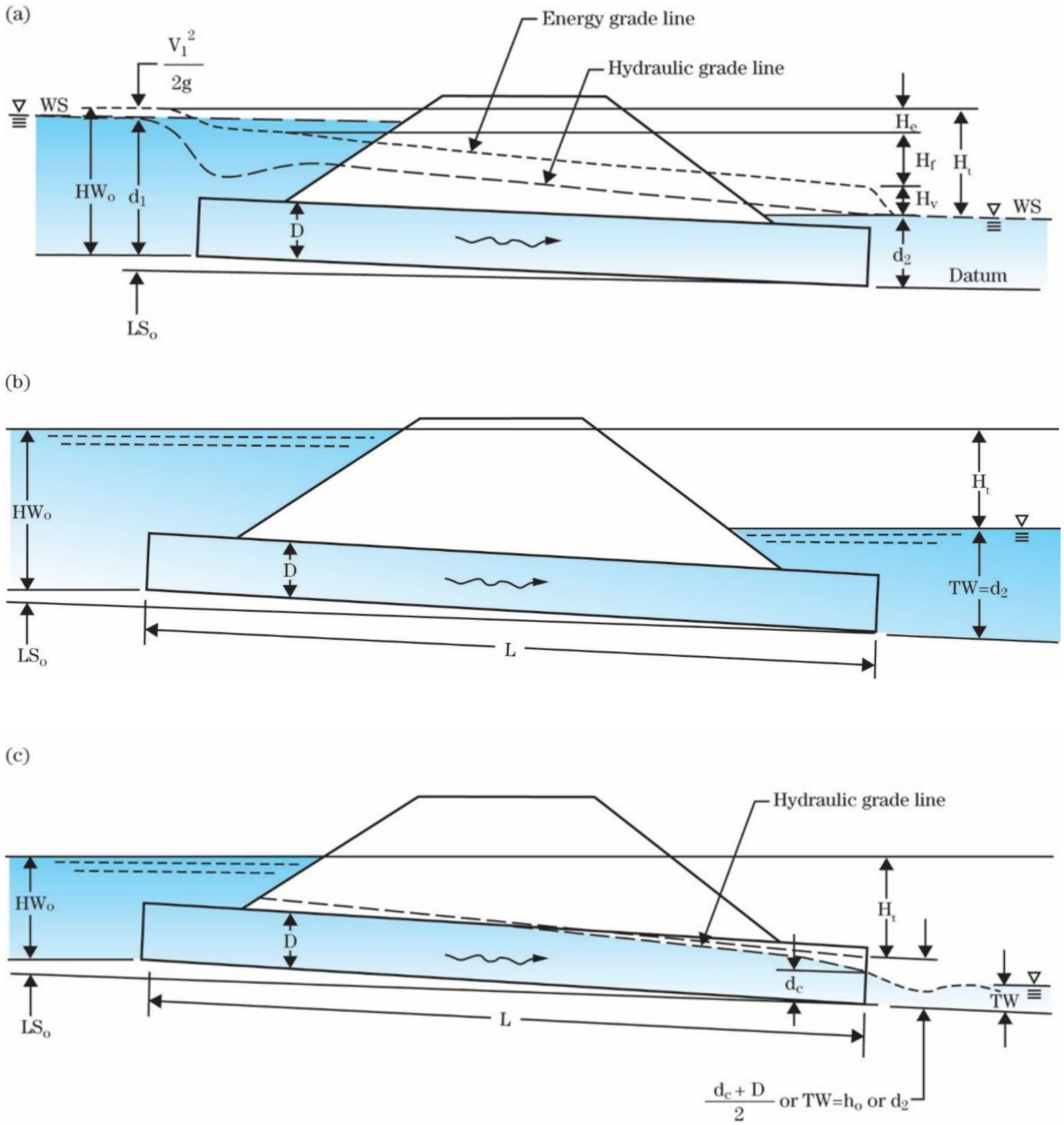
where: H_t = total head, ft

H_v = $(v^2)/2g$ when v is the average velocity in the culvert barrel, ft

H_e = entrance loss, which depends on the inlet geometry. The loss is expressed as a coefficient K_e (exhibit Q in section 650.0311) times the barrel velocity head, ft

H_f = friction loss, ft

Figure 3-27: Elements of Culvert Flow



(v) Equation 3–14 shows how the entrance loss is computed.

$$H_e = K_e \frac{v^2}{2g} \quad \text{(eq. 3-14)}$$

(vi) Equation 3–15 shows how the barrel friction loss is computed.

$$H_f = \frac{29n^2L}{r^{1.33}} \times \frac{v^2}{2g} \quad \text{(eq. 3-15)}$$

where: H_f = friction loss, ft
n = Manning's roughness coefficient
L = length of culvert barrel, ft
v = velocity in culvert barrel, ft/s
g = acceleration of gravity, 32.2 ft/s^2
r = hydraulic radius, ft

(vii) Substituting in equation 3–13 provides equation 3–16 for the total head.

$$H_t = \left(1 + K_e + \frac{29n^2L}{r^{1.33}}\right) \times \frac{v^2}{2g} \quad \text{(eq. 3-16)}$$

(viii) Figure 3–27(a) illustrates the terms of equations 3–13 through 3–19, hydraulic grade line, energy grade line, and headwater depth, HW_I.

(ix) Equation 3–17, shows that H_t is derived by equating the total energy upstream from the culvert to the energy just inside the culvert outlet.

$$H_t = d_1 + \frac{v^2}{2g} + Ls_0 - d_2 = H_v + H_e + H_f \quad \text{(eq. 3-17)}$$

where: v_1 = velocity in the approach section, ft/s

s_o = slope of the channel bed, ft/ft

(x) From figure 3–27(a), the culvert headwater is expressed in equation 3–18.

$$HW_0 = H_t + d_2 - Ls_o \quad \text{(eq. 3-18)}$$

(xi) If the velocity head in the approach section ($v_1^2/2g$) is low, it can be ignored and HW_0 becomes the elevation difference between the upstream water surface and the culvert inlet invert.

(xii) The depth (d_2) for culverts flowing full is equal to the culvert height in figure 3–25(d), or the tailwater depth (TW), whichever is greater as shown by figure 3–27(b).

(xiii) When the culvert barrel flows partially full for part of the barrel length, the hydraulic grade line passes through a point where the water breaks with the top of the culvert and, if extended as a straight line, will pass through the plane of the outlet end of the culvert at a point above critical depth. Essentially, this point is halfway between d_c and the crown of the culvert, or equal to:

$$\frac{d_c + D}{2}$$

or TW, whichever is greater.

(xiv) Setting h_o equal to d_2 as defined above, outlet control can be written as equation 3–19:

$$HW_0 = H_t + h_0 - Ls_0 \quad \text{(eq. 3-19)}$$

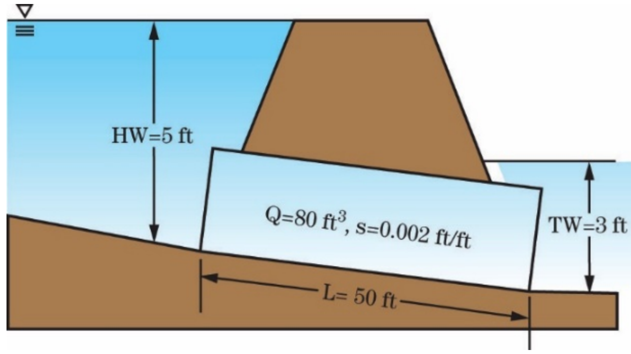
(xv) This equation was used to develop the nomographs shown in exhibits W through AA in section 650.0311, which can be used to develop stage discharge curves for the approach section to culverts flowing with outlet control.

(xvi) Entrance loss coefficients for various types of entrances when flow is in outlet control are given in table 3–4.

(xvii) Example – Minimum diameter culvert for access road stream crossing

- A planned access road crosses a stream as shown in figure 3–28. The proposed crossing is a 50-foot concrete culvert, n equals 0.012. The design flow is 80 cubic feet per second and the tailwater depth is 3 feet. The planned culvert slope is 0.002 foot per foot. The access road surface is 5 feet above the stream channel, so the maximum headwater depth is 5 feet. There is no headwall planned for the culvert; the socket end of the culvert will project from the fill on the upstream end. Determine the minimum diameter of the culvert for these conditions.

Figure 3-28: Access Road Stream Crossing with Concrete Culvert



- Solution:
  - From equation 3–18:

    $$HW_0 = H_t + d_2 - Ls_o$$

  - Rearrange to solve for H_t on the left side of the equation:

    $$H_t = HW_0 - h_0 + Ls_0$$
    $$= 5\,ft - 3\,ft + (50\,ft \times 0.002\ \,ft/ft) = 2.1 ft$$

  - From the table in figure 3-29 the entrance coefficient for a concrete pipe projecting from the fill with socket end upstream, K_e equals 0.2.
  - Enter exhibit X in section 650.0311, Head for concrete pipe culverts flowing full - n equals 0.012, and draw a line between H equals 2.1 feet on the head scale and Q equals 80 cubic feet per second on the discharge scale. This first line passes through the turning line. On the length scale for K_e equals 0.2, draw a second line from the 50-foot mark through the previously marked point on the turning line to the pipe diameter scale. This second line intersects the diameter scale at approximately 39 inches. Since 39 inches is not a standard culvert size, use the next larger standard pipe size, 42-inch-diameter pipe.

Figure 3-29: Entrance loss coefficients for pipe culverts (USACE HEC-RAS 5.0 Hydraulic Reference Manual, Feb. 2016, Tables 6-3 and 6-4, Entrance loss coefficients for pipe culverts)

Culvert Type	**Coefficient, K_e**
Corrugated metal pipe or pipe-arch	
End projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square-edged	0.5
Mitered end conforming to fill slope	0.7
End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Circular concrete pipe	
No headwall	
End projecting from fill, square-edged end	0.5
End projection from fill, socket end (groove-end)	0.2
Headwall or headwall and wingwalls	
Square-edged end	0.5
Socket end of pip (groove-end)	0.2
Rounded end, radius of D/12	0.2
No headwall, special end treatment	
Mitered end conforming to fill slope	0.7
Precast end-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side slope-tapered inlet	0.2
Reinforced concrete box	
No wingwalls, headwall parallel to embankment	
Square-edged on three edges	0.5
Beveled on three sides or rounded on three-edged, radius of D/12	0.2
Wingwalls, 30° to 75° to barrel	
Square-edged crown	0.4
Beveled top edge or rounded crown edge, radius of D/12	0.2
Wingwalls, 10° to 25° to barre	
Square-edged crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged crown	0.7
Side- or slope-tapered inlet	0.2

(13) Culvert planning considerations—There are several factors that must be taken into account when designing a culvert. Some of the more common are described briefly.

(i) Downstream erosion control.—Culverts generally have higher velocities than the adjacent natural stream because the culvert flow area is smaller than the stream cross section. As a result, the higher velocity at the culvert exit will often scour the downstream streambed or streambanks. Appropriate erosion-control measures may be needed downstream of the culvert outlet.

(ii) Fish passage.—Some culverts block fish from passing through them. Proper culvert design and selection can enhance fish passage. For more information, see 210-NEH-654, Chapter 14 (210-NEH-654-14), Technical Supplement 14N, “Fish Passage and Screening Designs”, and Conservation Practice Standards Codes 396, “Fish Passage, and 578, Stream Crossing. The USDA Forest Service FishXing software can be used for evaluating and designing culverts for fish passage.

(iii) Tidal wetland considerations.—In coastal areas, roads are often constructed parallel to the coast and across tidal wetlands. Culverts through these roads affect the inland tidal wetland water surface elevations. Proper culvert design may restore the hydrologic wetland function.

(iv) Materials and aesthetics.—The proposed culvert should meet any local requirements such as material type, end treatment, or minimum diameter. Local ordinances may specify the use of specific materials in order to control the appearance of culverts once they are installed.

650.0305 Open-channel Flow

A. The critical difference between open-channel flow (figs. 3–30 and 3–31) and pipe flow is that open-channel flow has a free water surface, whereas pipe flow has no free water because water fills the entire conduit. Open-channel flow calculations are complex because the water surface elevation and the cross-sectional area may vary with respect to time. In addition, flow depth, discharge, and slopes of the channel bottom and water surface are interdependent. Channel cross sections vary from semicircular to the irregular forms of natural streams. The channel surface may vary from that of polished metal used in testing flumes to rough, irregular riverbeds. Moreover, the roughness in an open channel varies with the position of the free water surface.

Figure 3-30: Open-channel Flow in Maryland Stream

Figure 3-31: Open-channel Flow in New Mexico Irrigation Canal

B. Therefore, the proper selection of friction coefficients carries greater uncertainty for open channels than for pipes. In general, the analysis of open-channel flow is more empirical than that of pipe flow, but the empirical method is the best available.

C. Types of Channel Flow

(1) Open-channel flow can be classified according to the change in flow depth with respect to the time interval being considered and the channel cross-sectional area occupied by the flow. The classification is as follows:

(i) steady flow

(ii) uniform flow

(iii) nonuniform flow

- gradually varied flow
- rapidly varied flow

(iv) Unsteady flow

(v) unsteady uniform flow (rare)

(vi) unsteady varied flow

- gradually varied unsteady flow
- rapidly varied unsteady flow

(2) Steady Flow and Unsteady Flow: Based on Time Interval

(i) Open-channel flow is steady if the flow depth at a given cross section does not change or can be assumed to be constant during the time interval being considered. The flow is unsteady when the flow depth at a given cross section changes with time. In many open-channel problems, it is only necessary to study flow behavior under steady conditions. If, however, the change in flow condition with respect to time is of major concern, the flow should be treated as unsteady. In floods and surges, for instance, which are typical examples of unsteady flow, the stage of flow changes instantaneously as the waves pass by, and the time element becomes important in the design of control structures.

(ii) Uniform flow and nonuniform flow: based on channel space used

- Open-channel flow is uniform when the flow depth is constant at every section of the channel. A uniform flow may be steady or unsteady, depending on whether the depth changes during the time period being considered.
- Steady uniform flow is the basic type of flow treated in open-channel hydraulics. The depth of the flow does not change during the time interval under consideration. Unsteady uniform flow means that the water surface fluctuates from time to time while remaining parallel to the channel bottom, which is practically impossible.
- Flow is nonuniform if the depth of flow changes along the length of the channel. Nonuniform flow may be either steady or unsteady. Nonuniform flow may be classed as either rapidly or gradually varied. The flow is rapidly varied when the depth changes abruptly over a comparatively short distance; otherwise, it is gradually varied. Examples of rapidly varied flow are the hydraulic jump and the hydraulic drop. Figure 3-32 shows various types of flow.

D. Channel Cross Section Elements

(1) The elements of cross sections of an open channel required for hydraulic computation are:

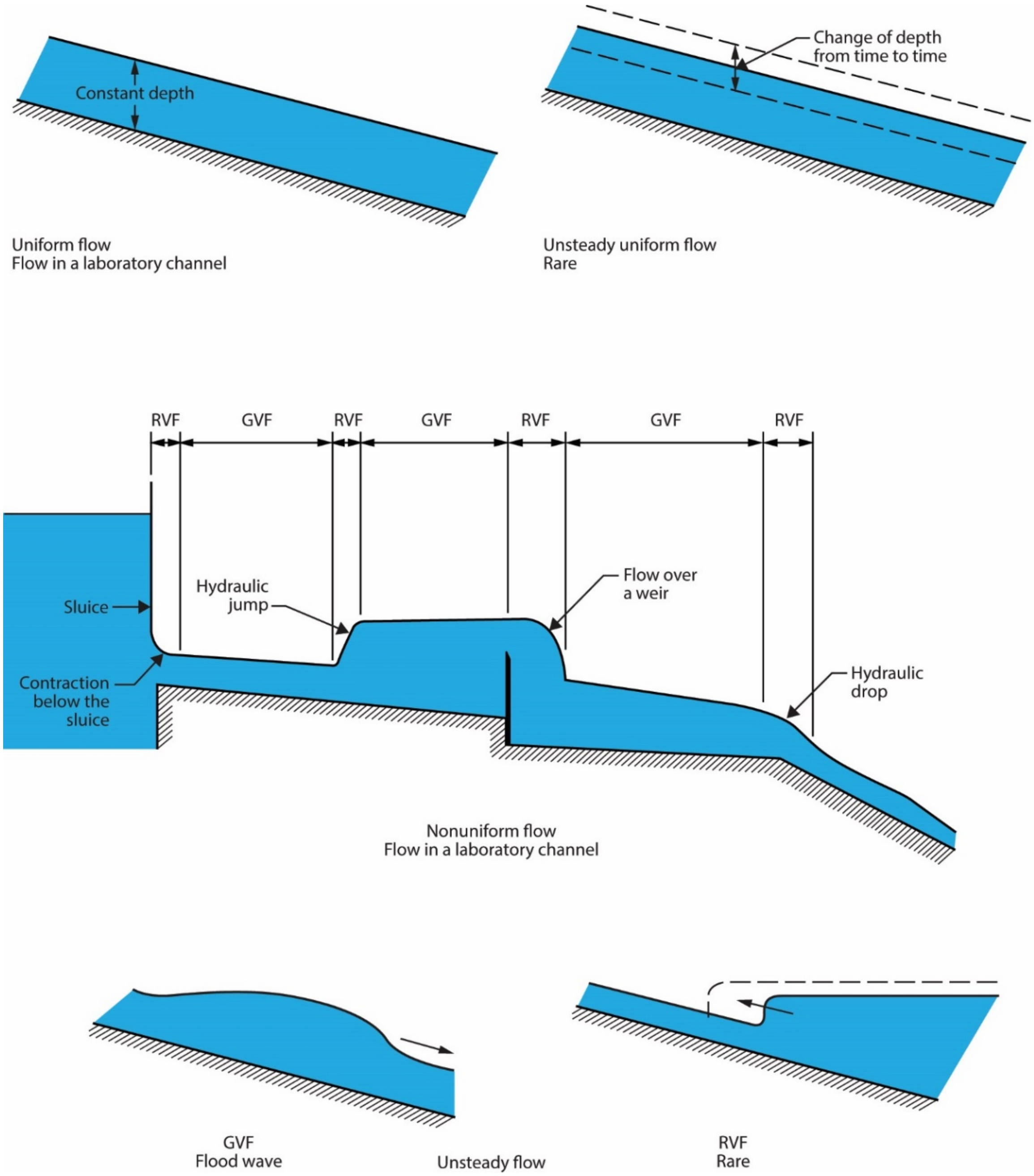
(i) Cross-sectional flow area, a, ft^2

(ii) Wetted perimeter, which is the length of the boundary of the cross section in contact with the flow, *p*, ft

(iii) Hydraulic radius, which is the cross-sectional area of the stream divided by the wetted perimeter, *r*, ft.

(2) Exhibit BB in section 650.0311 gives general formulas for determining area, wetted perimeter, hydraulic radius, and top widths of trapezoidal, rectangular, triangular, circular, and parabolic sections. In addition, software such as the USDA-NRCS Engineering Field Tools Hydraulics or HEC–RAS provide cross section properties.

Figure 3-32: Types of Open-channel Flow



E. Manning's Equation

(1) Most open-channel formulas express mean velocity of flow as a function of the roughness of the channel, the hydraulic radius, and the slope of the energy gradient. Manning's equation is the most commonly used open channel formula; it is simple and provides flow velocities consistent with experimental data (eq. 3–20).

$$v = \frac{1.486}{n} \cdot r^{2/3} \cdot s^{1/2} \qquad \text{(eq. 3-20)}$$

where: v = mean velocity of flow, ft/s
r = hydraulic radius, ft
s = slope of the energy gradient, ft/ft
n = coefficient of roughness, dimensionless

(2) Since Q = av, as defined by equation 3–4, Manning's equation may also be written for discharge (eq. 3–21).

$$Q = a\frac{1.486}{n} \cdot r^{2/3} \cdot s^{1/2} \qquad \text{(eq. 3-21)}$$

where: a = cross-sectional area, ft^2
Q = discharge, ft^3/s

(3) Coefficient of roughness, *n*. The *n* value is influenced by several factors, as shown in the table in figure 3-33.

(4) Estimating the *n* value for natural streams is more difficult than it is for pipes and lined channels. The *n* value of an earthen natural or constructed channel varies seasonally and annually; it is not a fixed value. The annual growth of vegetation, uneven sediment, and debris accumulation in the channel, bank erosion, and sloughing, as well as other factors alter the *n* value from year to year.

(5) Additionally, Manning's *n* tends to decrease with increasing in-channel stage; therefore, varying Manning's *n* with stage should be considered. Modeling software, such as HEC–RAS, provides vertical variation in *n* values when developing hydraulic computational models. These factors should be evaluated with respect to the kind of channel, degree of maintenance, seasonal requirements, season of the year when the design storm normally occurs, and other considerations as a basis for selecting the value of *n*. Generally, turbulent conditions increase retardance. Roughness coefficient estimation for natural channels and excavated channels is more difficult than for lined channels.

(6) These references provide additional information on determining roughness coefficients for streams:

(i) Roughness Characteristics of Natural Channels (USDOI–USGS water supply paper 1849, Harry H. Barnes Jr., 1967)

(ii) Determination of the Manning Coefficient from Measured Bed Roughness in Natural Channels (USDOI–USGS water supply paper 1898–B, J.T. Limerinos, 1970)

(iii) Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains (USGS water supply paper 2339, G.J. Arcement, Jr., and V.R. Schneider, 1989)

(iv) Guide for Selecting Roughness coefficient *n*-values for Channels guide (USDA–SCS, G.B. Faskin, 1963)

Figure 3-33: *n*-Value Factors for streamflow

***n* value factor**	**Effect**
Physical roughness	The streambed and streambank materials, as well as surface irregularity, have a significant impact. Fine soil particles on smooth, uniform surfaces result in relatively low *n* values. Coarse materials, such as gravel or boulders, and pronounced surface irregularities result in higher *n* values.
Vegetation	The height, density, distribution, and type of vegetation affect flow resistance in the stream. During the year, flow resistance increases as vegetation grows and foliage develops, and then the flow resistance declines when vegetation goes dormant.
Cross section	Gradual and uniform changes in cross section size will not significantly affect *n*. However, abrupt changes or alternating small and large sections significantly affect *n*.
Channel alignment	Large radius curves without frequent changes in direction of curvature offer low resistance to flow. Severe meandering increases flow resistance.
Deposition or scour	These processes tend to increase *n* because the cross section is changing.
Obstructions and debris	Logjams and debris deposits increase the *n* value depending on the number, type, and size of the obstructions.

F. Specific Energy in Channels

(1) The specific energy equation is used to solve many open channel problems, such as water surface profiles upstream of culverts and channel junctions and the water surface profile in a chute spillway.

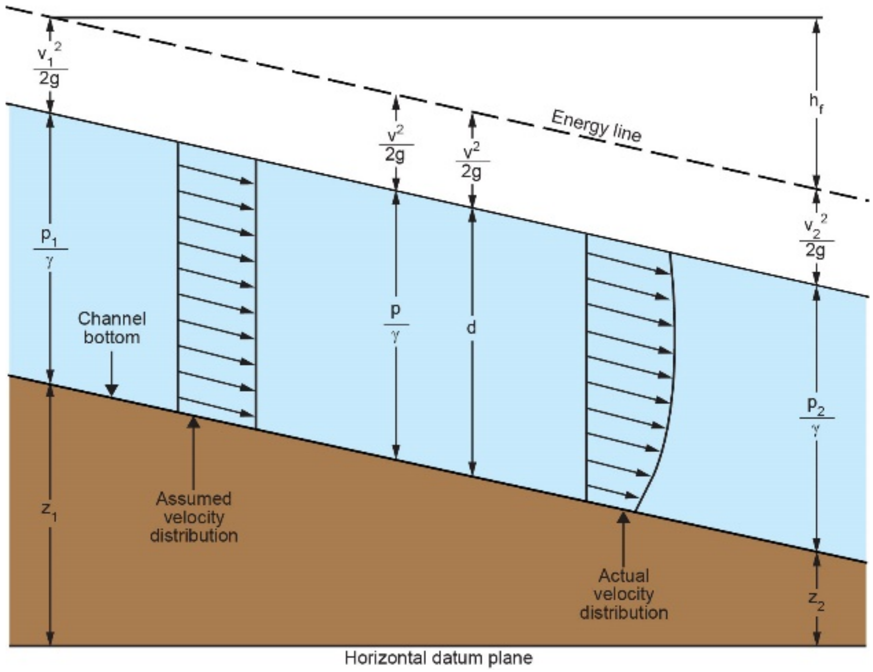
(2) Figure 3–34 shows a channel in uniform flow. For a given slope, roughness, cross section, and rate of flow, the depth may be calculated from the Manning equation.

(3) Assuming a uniform velocity distribution, the Bernoulli equation for this reach is:

$$\frac{{v_1}^2}{2g}+\frac{p_1}{\gamma}+z_1=\frac{{v_2}^2}{2g}+\frac{p_2}{\gamma}+z_2+h_f+h_L$$

where: v = velocity, ft/s
g = acceleration of gravity, 32.2 ft/s^2
p = pressure, lb/ft^2
γ = unit weight of water, lb/ft^3
z = elevation, ft
h_f = friction head, ft
h_L = head losses other than friction, ft

Figure 3-34: Channel Energy Relationships



(4) This shows that energy is lost as flow occurs. However, the distance from channel bottom to energy line remains constant and is given by equation 3–22.

$$H_e = \frac{p}{\gamma} + \frac{v^2}{2g} = d + \frac{v^2}{2g} \qquad \text{(eq. 3-22)}$$

where: H_e = specific energy, ft, which for open-channel flow is the sum of the water depth and the velocity head.

d = flow depth, ft

G. Critical Flow

(1) Critical flow can be thought of as a dividing point between subcritical (tranquil) and supercritical (rapid) flow. At this dividing point, certain relationships between specific energy and discharge and between specific energy and flow depth exist. Two conditions describe critical flow:

(i) The discharge is maximum for a given specific energy head.

(ii) The specific energy head is minimum for a given discharge.

(2) In other words, for a given channel section there is one, and only one, critical discharge for a given specific energy head. Any discharge greater or less than that requires additional specific energy.

(3) General equation for critical flow.

(i) The general equation for critical flow in any channel (eq. 3–23):

$$\frac{Q^2}{g} = \frac{a^3}{T} \qquad \text{(eq. 3-23)}$$

where: Q = discharge, ft^3/s

g = acceleration of gravity, 32.2 ft/s^2

a = flow cross-sectional area, ft^2

T = flow top width, ft

(ii) Rearranging terms yields:

$$\frac{Q^2}{a^2} = \frac{ag}{T}$$

(iii) By definition, discharge is the product of the cross-sectional area and the flow velocity:

$$Q = av$$

(iv) Correspondingly, velocity is discharge divided by the area. Square both sides of the equation:

$$v^2 = \frac{Q^2}{a^2}$$

(v) Velocity squared is equivalent to these two terms:

$$\frac{Q^2}{a^2} = v^2 = \frac{ag}{T}$$

(vi) From figure 3–35, Specific Energy Schematic, the specific energy is the sum of the depth and velocity head:

$$H_e = d + \frac{V^2}{2g}$$

(vii) Substitute (ag/T) for v^2:

$$H_e = d + \frac{a}{2T}$$

(viii) The median flow depth, d_m, can be defined as the area, a, divided by top width, *T*:

$$d_m = \frac{a}{T}$$

(ix) Therefore,

$$H_e = d + \frac{d_m}{2}$$

where: Q = total discharge, ft^3/s
a = cross-sectional area, ft^2
d = depth of flow to the bottom of the section, ft
d_m = a/T mean depth of flow, ft
g = acceleration of gravity, 32.2 ft/s^2
H_e = specific energy head, i.e., the energy head referred to the bottom of channel, ft
T = top width of the stream, ft
v = mean velocity of flow, ft/s

(4) Figure 3–35, the specific energy diagram, shows the relationships between discharge, energy, and depth. Figure 3-36 lists critical flow terms and their definitions.

Figure 3-35: Specific Energy Schematic

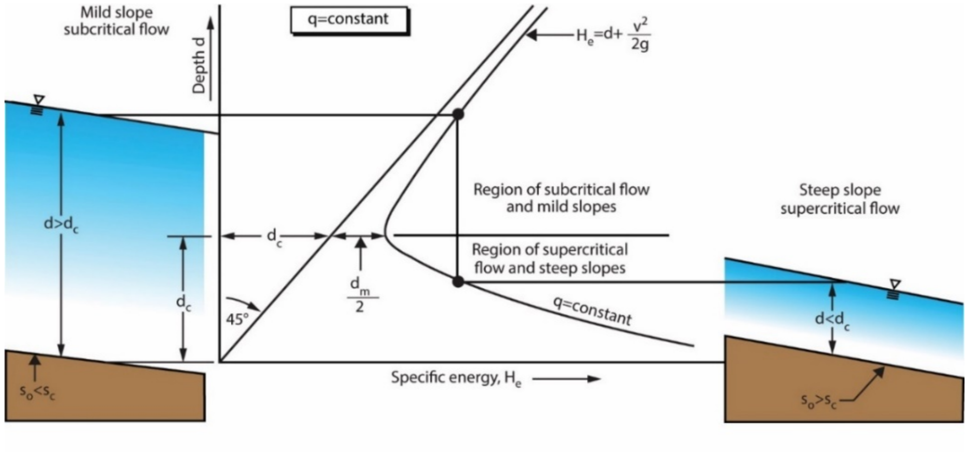


Figure 3-36: Critical Flow Condition Definitions

Term	Definition
Critical discharge	The maximum discharge for a given specific energy or a discharge that occurs with minimum specific energy.
Critical depth	The depth of flow at which the discharge is maximum for a given specific energy, or the depth at which a given discharge occurs with minimum specific energy.
Critical velocity	The mean velocity when the discharge is critical.
Critical slope	The slope which will sustain a given discharge at uniform, critical depth in a given channel.
Subcritical flow	Flow conditions where depth is greater than critical and velocity is less than critical.
Supercritical flow	Flow conditions where depth is less than critical depth and velocity is greater than critical velocity.

(5) The specific energy curve shows the variation of specific energy with flow depth for a constant Q in a channel of a given cross section. Similar curves for any discharge at a section of any form may be obtained from equation 3–22. This curve illustrates several key points:

(i) In a specific energy diagram, pressure head and velocity head are shown graphically (fig. 3–35). The pressure head (depth in open-channel flow) is represented by the horizontal scale as the distance from the vertical axis to the line along which H_e equals d to the curve of constant Q.

(ii) For any discharge, there is a minimum specific energy, and the depth of flow corresponding to this minimum specific energy is the critical depth. For any specific energy greater than this minimum, there are two depths, sometimes called alternate depths, of equal energy at which the discharge may occur. One of these depths is in the subcritical range and the other is in the supercritical range.

(iii) At depths of flow near critical, a minor change in specific energy causes a much greater change in depth.
(iv) Through most of the subcritical flow range, the velocity head is relatively small compared to specific energy at a given discharge. Therefore, any change in depth approximately equals the change in specific energy.
(v) Through the supercritical range, the velocity head for any discharge increases rapidly as depth decreases, and changes in depth are associated with much greater changes in specific energy.

(6) Instability of critical flow—Critical slope, or s_c is that slope that will sustain a given discharge in a given channel at uniform, critical depth. The instability of uniform flow at or near critical depth is usually defined in terms of critical slope, s_c.
(i) The critical slope, s_c, is computed using equation 3–24.

$$s_c = 14.56\frac{n^2 d_m}{r^{4/3}} \qquad \text{(eq. 3-24)}$$

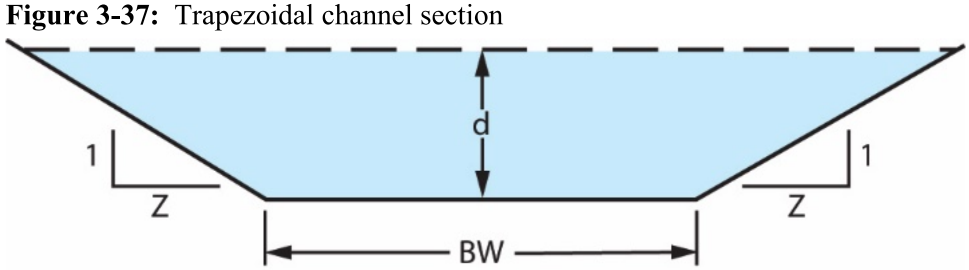
where: s_c = critical slope, ft/ft
n = roughness coefficient, dimensionless
d_m = median flow depth, ft
r = hydraulic radius, ft

(ii) Uniform flow near critical depth is unstable because of the unique relationship between energy head and flow depth that is readily disturbed by minor changes in energy. Uniform flow at or near critical depth exhibits an unstable wavy surface caused by changes in depth resulting from minor energy changes. This unstable slope range is between 70 and 130 percent of critical slope.
(iii) Uniform flow designs near critical depth should be avoided due to the unstable flow. If the design must include uniform flow near critical depth, then the design should include additional freeboard for the expected wave height. When topography governs the channel slope, varying the design channel width will force flow conditions into either subcritical stable or supercritical stable.

H. Open-channel Flow Problems

(1) Determining discharge for a trapezoidal channel
(i) For the trapezoidal channel shown in figure 3–37, determine Q in cubic feet per second and v in feet per second, given b (denoted as BW in figure 3-37) equals 8 feet, d equals 2.5 feet, the side slopes equal 2:1 (Z=2), n equals 0.04 and s equals 0.006 feet per feet.

Figure 3-37: Trapezoidal channel section



(ii) Solution:

- From exhibit BB in section 650.0311, find equations for cross-sectional area, *a*, wetted perimeter, *p*, and hydraulic radius, *r*. Then solve Manning's equation for velocity and use the continuity equation to determine the discharge.
  - Cross-sectional area, *a*:

$$a = bd + zd^2 = (8\,ft \times 2.5\,ft + 2(2.5\,ft)^2 = 32.5\,ft^2$$

  - Wetted perimeter, *p*:

$$p = b + 2d\sqrt{z^2+1} = 8\,ft + (2ft \cdot 2.5\,ft \cdot \sqrt{(2\,ft)^2+1}$$
$$= 19.2\,ft$$

  - Hydraulic radius, *r*:

$$r = \frac{a}{p} = \frac{32.5\,ft^2}{19.2\,ft} = 1.69\,ft$$

  - Velocity, *v*:

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$
$$= \frac{1.486}{0.04}(1.69\,ft)^{1/2}(0.006\ \ ft/ft)^{1/2} = 4.1\,ft/s$$

  - Discharge, *Q*:

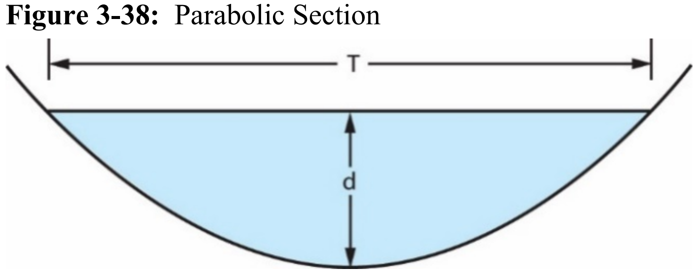
$$Q = va = 4.1\,ft/s \cdot 32.5\,ft^2 = 133\ \ ft^3/s$$

- The trapezoidal channel flow is 133 cubic feet per second.

(2) Determining discharge for a parabolic channel

(i) For the parabolic channel section shown in figure 3–38, determine *Q* in cubic feet per second and *v* in feet per second given d equals 1.2 feet, *T* equals 30 feet, *n* equals 0.05 and *s* equals 0.02 feet per foot.

Figure 3-38: Parabolic Section



(ii) Solution:

- From exhibit BB in section 650.0311, find equations for *a*, *p*, and *r*. Then solve Manning's equation for velocity and use the continuity equation to determine the discharge.
  - Cross-sectional area, *a*:

$$a = \frac{2dT}{3} = \frac{2 \cdot (1.2\,ft)(30\,ft)}{3} = 24\,ft^2$$

  - Wetted perimeter, *p*:

$$p = T + \frac{8d^2}{3T} = (30\,ft) + \frac{8 \cdot (1.2\,ft)^2}{3 \cdot (30\,ft)} = 30.1\,ft$$

  - Hydraulic radius, *r*:

$$r = \frac{a}{p} = \frac{24\,ft^2}{30.1\,ft} = 0.80\,ft$$

  - Velocity, *v*:

$$v = \frac{1.486}{n} r^{2/3} s^{1/2} = \frac{1.486}{0.05} (0.80\,ft)^{1/2} (0.02\ ft/ft)^{1/2}$$
$$= 3.6\,ft/s$$

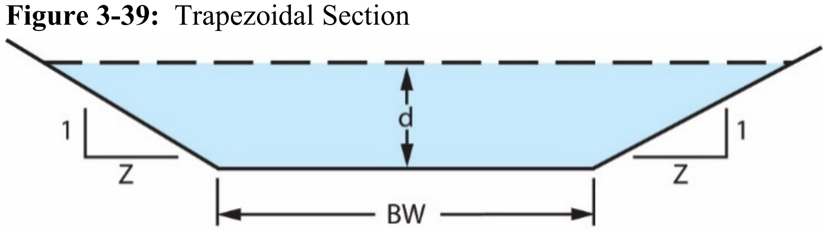
  - Discharge, *Q*:

$$Q = va = 3.6\,ft/s \cdot 24\,ft^2 = 86\ ft^3/s$$

(3) Determining flow depth and velocity for a trapezoidal channel

(i) Determine the flow depth and velocity for a trapezoidal channel (fig. 3–39) with a 15-foot bottom width (BW) and side slopes of 2:1 on 0.0009 foot per foot slope for a design discharge of 300 cubic feet per second. Use a Manning's *n* of 0.02.

Figure 3-39: Trapezoidal Section



(ii) Solution:

- Assume a flow depth, *d*, then compute *a*, *p*, and *r*, from exhibit BB in section 650.0311. Then find *v*, and compute *Q*. Figure 3-40 shows the computation results.
- From the table of trial computations, for a discharge of 300 cfs, the depth of flow is approximately 3.4 feet and velocity is 4.1 feet per second. These iterations can be performed using the USDA-NRCS Engineering Field Tools Hydraulic software.

Figure 3-40: Determination of discharge for a range of flow depths in a trapezoidal channel

Trial	d, ft	a, ft^2	p, ft	f, ft	v, ft/s	Q, cfs
1	3.0	63.00	28.42	2.22	3.79	239
2	3.1	65.72	28.86	2.28	3.86	254
3	3.2	68.48	29.31	2.34	3.92	269
4	3.3	71.28	29.76	2.40	3.99	284
5	3.4	74.12	30.21	2.45	4.06	301

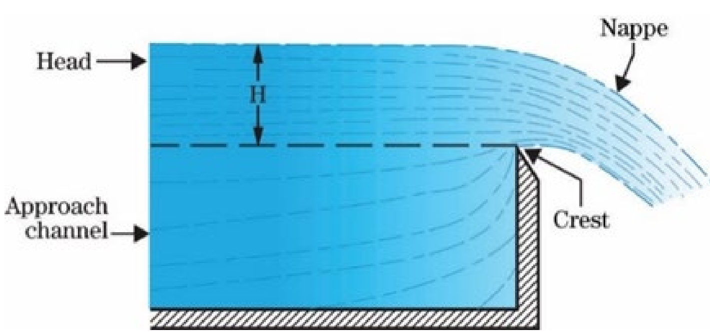
650.0306 Weir Flow

A. A weir is a notch of regular form through which water flows (fig. 3–41). The structure containing the notch is also called a weir. The edge over which the water flows is the crest. The two basic types of weirs are sharp-crested weirs and broad-crested weirs.

Figure 3-41: Weir in New Mexico Irrigation Ditch

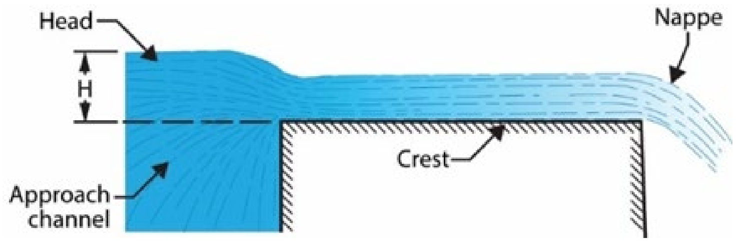
B. The sharp-crested weir is used to measure the channel or stream flow. In a sharp-crested weir, the flow springs clear of the crest, as shown in figure 3–42.

Figure 3-42: Sharp-crested Weir



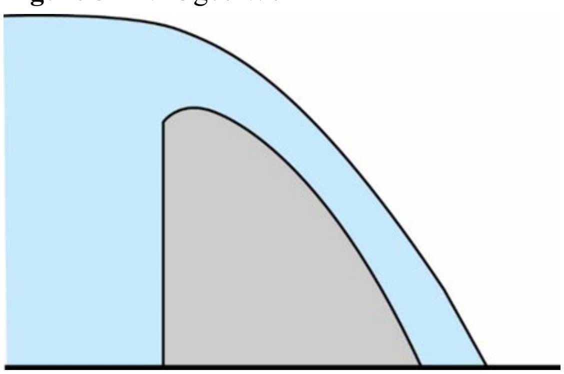
C. Most hydraulic structures have broad-crested weirs (fig. 3–43). The crest is horizontal and long in the direction of flow so flow maintains contact with the crest rather than springing clear. The level crest in an earthen auxiliary spillway of a flood control dam is an example of a broad-crested weir.

Figure 3-43: Broad-crested Weir



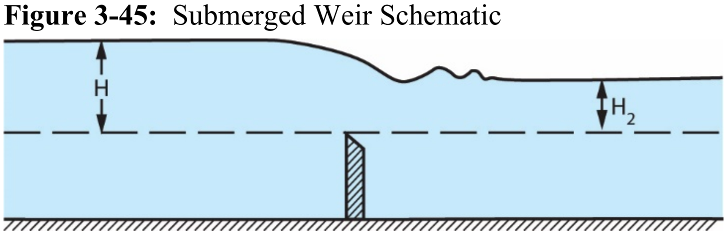
D. An additional type of weir is the ogee (fig. 3–44). The downstream section of an ogee crest is parabolic shaped to mimic the underneath side of the nappe jet passing over a sharp-crested weir. Ogee weirs are commonly found in large hydraulic structures where passing the design flow at minimum head is important. For more information on ogee crests, see U.S. Department of Interior Bureau of Reclamation, Design of Small Dams, third edition.

Figure 3-44: Ogee Weir



E. If the overflowing sheet of water or nappe discharges into the air, as in figures 3–42 and 3-43, the weir has free discharge. If the discharge is partially submerged under water, as shown in figure 3–45 below, the weir is submerged or drowned.

Figure 3-45: Submerged Weir Schematic



F. Basic Weir Equation

(1) The typical weir equation is equation 3–25.

$$Q_w = C_w L_w (H_w)^{3/2} \qquad \text{(eq. 3-25)}$$

where: Q_w = weir discharge, ft^3/s
C_w = weir coefficient, dimensionless
L_w = weir length, ft
H_w = weir head, ft

(2) Adjustments to the weir equation include effects of end contractions, velocity of approach and submergence.

G. Contractions

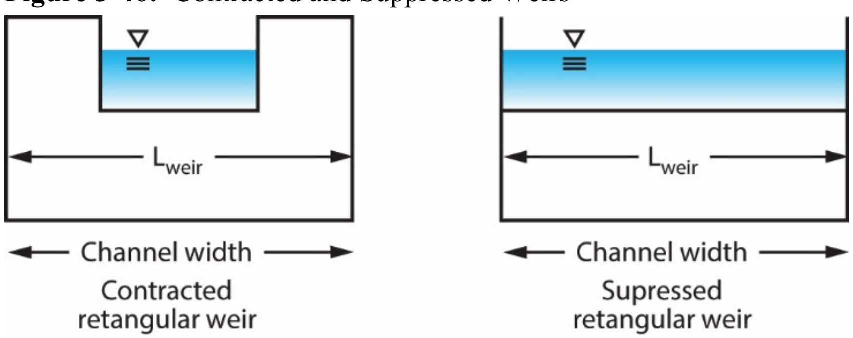
(1) When the length of the weir is narrower than the channel, the weir is contracted (fig. 3-46). If the weir extends the full width of the approach and exit channels, the weir is suppressed.

(2) End contractions reduce the effective length of a weir. To allow for end contractions, the weir length in the basic equation is adjusted according to equation 3–26.

$$L_e = L_w - (0.1 \times N \times H_w) \qquad \text{(eq. 3-26)}$$

where: L_e = effective weir length, ft
L_w = measured weir length, ft
N = number of contractions (N can be 1 or 2)
H = weir head, ft

Figure 3-46: Contracted and Suppressed Weirs



(3) Equation 3–26 is generally applied only for sharp-crested weirs and not for broad-crested weirs because for most broad-crested weirs, the end contractions are fully or partially suppressed. For drop spillways, the basic formula can be used without modifying for contraction effect.

H. Velocity of Approach

The velocity of approach is the average velocity in the approach channel upstream of the weir measured at a distance $3H_w$ upstream from the weir. The velocity head is added to the measured head to determine the discharge. Equation 3–27 is the weir equation adjusted for the velocity of approach.

$$Q_w = C_w L_w \left(H_w + \frac{v^2}{2g}\right)^{3/2} \qquad \text{(eq. 3-27)}$$

where: Q_w = weir discharge, ft^3/s
C_w = weir coefficient, dimensionless
L_w = weir length, ft
H_w = weir head, ft
v = velocity of approach, ft/s
g = acceleration of gravity, 32.2 ft/s^2

I. Weir Coefficients

Values of the weir coefficient, C_w, vary with the type of crest used (fig. 3-47). The weir coefficient for broad-crested weirs commonly used in soil conservation work is 3.1.

Figure 3-47: Weir Coefficients (Source: HEC-RAS Hydraulic Reference Manual, version 4.1, January 2010)

Weir crest Shape	Typical Coefficient Range
Broad crested	2.6–3.1
Ogee crested	3.2–4.1
Sharp crested	3.2–3.3

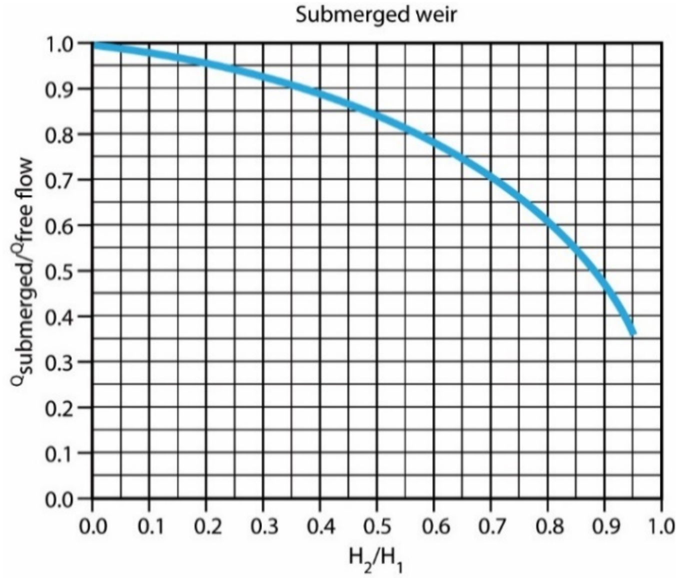
J. Submerged Weir Flow

(1) When a weir is submerged, as in figure 3-45, the discharge will be less than for the same upstream head in free-flow conditions. This submergence flow reduction is expressed as the ratio of the upstream and downstream head over the weir in the Villemonte equation (eq. 3-28) and is shown graphically in figure 3-48.

$$\frac{Q_{submerged}}{Q_{free\ flow}} = \left(1 - \left(\frac{H_2}{H_1}\right)^{3/2}\right)^{0.385} \qquad \text{(eq. 3-28)}$$

(2) The reduction in discharge for broad-crested weirs is less than 10 percent when the submergence ratio H_2/H_1 is less than 0.4.

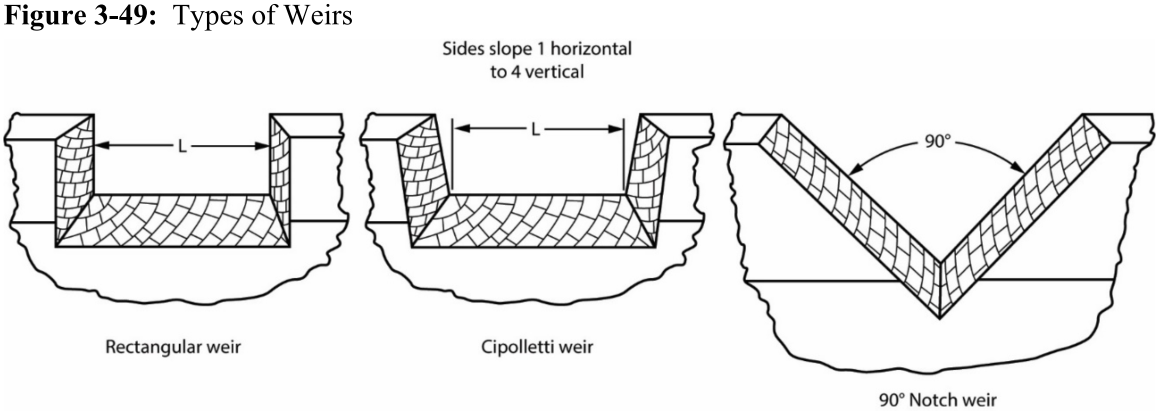
Figure 3-48. Weir Submergence Effect on Discharge



K. Use of Weirs to Measure Open-channel Flow

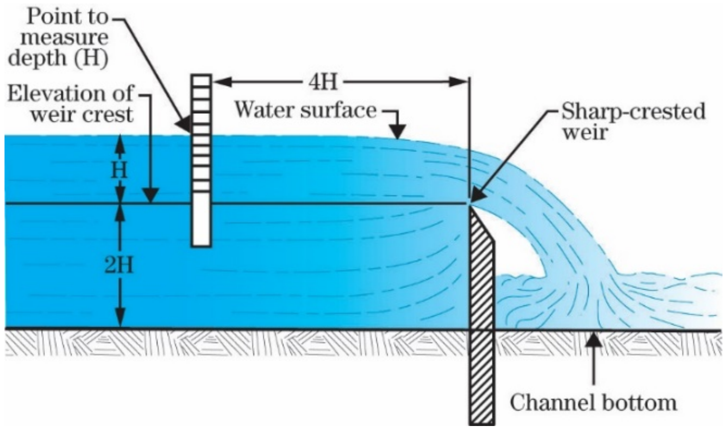
(1) Sharp-crested weirs are used extensively for measuring the flow of water. The most common types, rectangular, Cipolletti, and the 90-degree V-notch weirs, are shown in figure 3-49.

Figure 3-49: Types of Weirs



(2) For accurate discharge estimates, the weir should be properly set and the weir head measured upstream of the crest so that the reading is not affected by the downward curve of the water (fig. 3–50). The weir should be at right angles to the stream where the channel is straight, free from eddies, and of sufficient width to produce full-end contractions. The crest of the weir must be level for the rectangular and Cipolletti types. The bottom of the notch must be above the channel bottom, a height equal to twice the maximum head, preferably more. If a weir is to continue to give reliable results, it must be maintained to preserve these conditions.

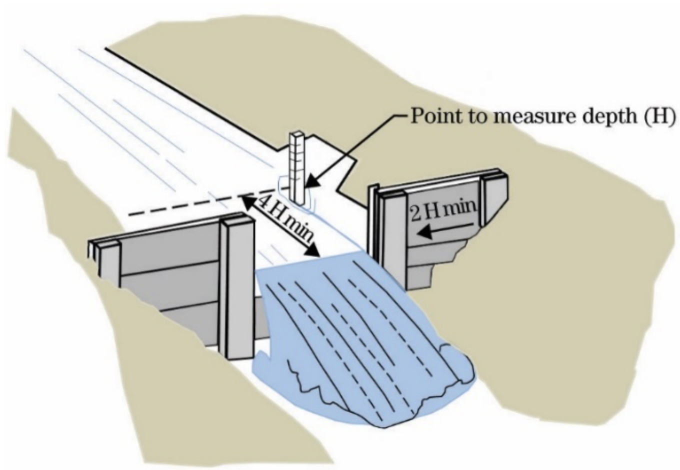
Figure 3-50: Profile of Sharp-crested Weir Used to Measure Flow



(3) Rectangular contracted weir

(i) A rectangular contracted weir has its crest and sides so far removed from the bottom and sides, respectively, of the weir box or channel in which it is set, that full contraction, or reduced area of flow, is developed (fig. 3–51).

Figure 3-51: Rectangular Contracted Weir



(ii) For a sharp-crested rectangular weir, the weir coefficient, C_w, is 3.33. If the weir is contracted:

$$Q = 3.33{H_w}^{3/2}(L - 0.2H_w)$$

where: Q = discharge, ft^3/s

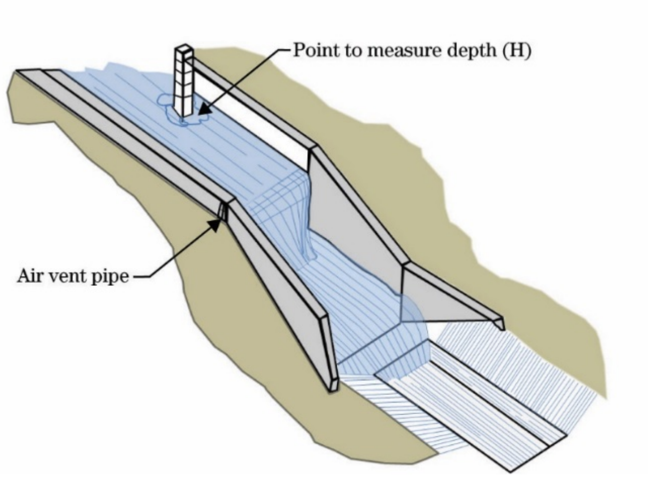
H_w = weir head measured at a minimum of $4H_w$ upstream of the weir, ft

L_w = measured weir length, ft

(4) Rectangular suppressed weir

(i) A rectangular suppressed weir crest is sufficiently far from the bottom of the approach channel that the crest contraction is fully developed. The sides of the weir coincide with the sides of the approach channel that extend downstream beyond the crest and prevent lateral expansion of the nappe. Figure 3-52 illustrates a suppressed weir in a flume drop.

Figure 3-52: Suppressed Weir in a Flume Drop Structure



(ii) This type of weir should be aerated beneath the overflowing sheet at the weir crest. This can be accomplished by venting the underside of the nappe to the atmosphere at both sides of the weir.

(iii) The sharp-crested rectangular suppressed weir equation is:

$$Q_w = 3.33 L_w {H_w}^{3/2}$$

(iv) And the equation can be modified for the velocity of approach in equation 3–29.

$$Q_w = 3.33 L_w (H_w + h_v)^{3/2} - {h_v}^{3/2} \qquad \text{(eq. 3-29)}$$

where: Q_w = discharge, ft^3/s
H_w = weir head, ft
L_w = measured weir length, ft
h_v = velocity head, ft = (approach velocity)^2/2g

(iv) Example - Rectangular suppressed weir

- Assume the gabion drop structure shown in figures 3–53 and 3–54 has a 10-foot-wide crest. The gabion drop structure is a broad-crested weir. Use C_w = 3.1 (fig. 3-47). Compute weir flow when the upstream water surface is 2 feet above the structure crest for free flow condition, and a submerged condition (fig. 3–55) when the downstream water surface is 0.9 foot above the weir crest.

Figure 3-53: Broad-crested Weir in Idaho Gabion Drop Structure

Figure 3-54: Gabion Drop Structure Schematic

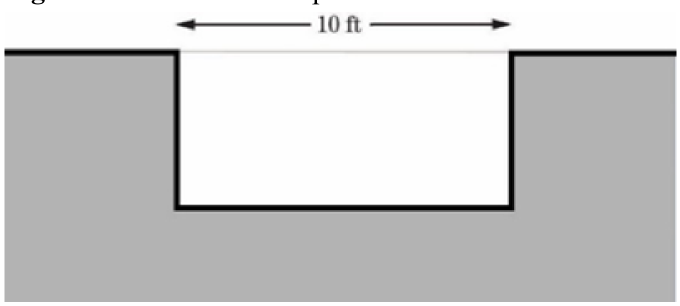
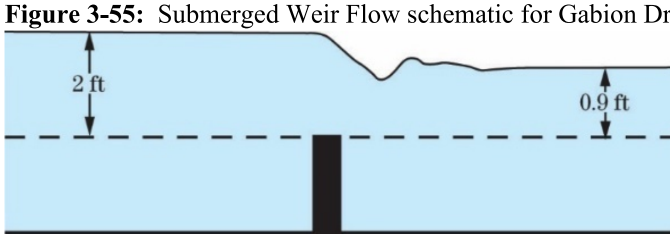


Figure 3-55: Submerged Weir Flow schematic for Gabion Drop Structures



- Solution:
  - For free-flow conditions:

$$Q_w = C_w L_w (H_w)^{3/2} = 3.1(10\ ft)(2\ ft)^{3/2} = 88\ ft^3/s$$

  - For the submerged condition (fig. 3–56), using equation 3–28.

$$Q_{submerged} = Q_{free\ flow}\left(1 - \left(\frac{H_2}{H_1}\right)^{3/2}\right)^{0.385}$$

$$= 88\ cfs\left(1 - \left(\frac{0.9\ ft}{2\ ft}\right)^{3/2}\right)^{0.385} = 77\ cfs$$

- The same result can be obtained graphically using figure 3-48.

$$\frac{H_1}{H_2} = \frac{0.9\,ft}{2\,ft} = 0.45$$

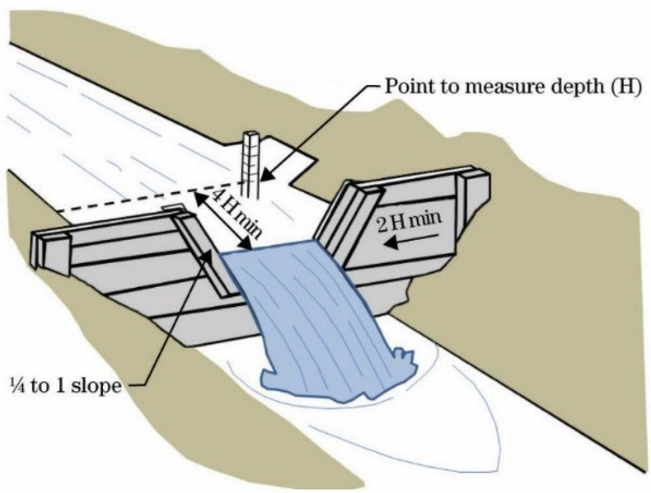
- Find 0.45 on the horizontal axis, draw a line up to the point where it intersects the curve, and draw a line to the vertical axis. For $H1/H_2$=0.45, the discharge ratio $Q_{submerged}/Q_{freeflow}$=0.875.

$$Q_{submerged} = Q_{free\ flow} \cdot Submergence\ ratio$$
$$= (88\,cfs) \cdot 0.875 = 77\,cfs$$

(3) Cipolletti weir

(i) A Cipolletti weir (fig. 3–56) is trapezoidal in shape. Its thin plate crest and sides are sufficiently far from the bottom and sides of the approach channel as to develop full contraction of flow at the nappe. The sides incline outwardly at a slope of 1 to 4.

Figure 3-56: Cipolletti Weir



(ii) Since the Cipolletti weir is a contracted weir, it should be installed accordingly; however, the discharge will have essentially the same value if the end contractions of the weir were suppressed. The effect of end contractions in reducing discharge has been overcome by sloping the sides of the weir.

(iii) The Cipoletti weir equation for full contractions is equation 3–30.

$$Q_w = 3.367\, L_w\, {H_w}^{3/2} \qquad \text{(eq. 3-30)}$$

where: Q_w = discharge, ft^3/s
L_w = measured weir length, ft
H_w = weir head, ft

The weir length should be at least $3H_w$ and preferably $4H_w$ or longer.

(4) V-notch weir

(i) The crest of the 90-degree V-notch weir consists of a thin plate with the sides of the notch inclined 45 degrees from vertical (fig. 3–57). This weir has a contracted notch and all conditions for accuracy stated for the standard contracted rectangular weir apply. The minimum distance from the side of the weir to the channel bank should be measured horizontally from the point where the maximum water surface intersects the edge of the weir. The minimum bottom distance should be measured between the point of the V-notch and the channel bottom.

(ii) The V-notch weir is especially useful for measuring small discharges.

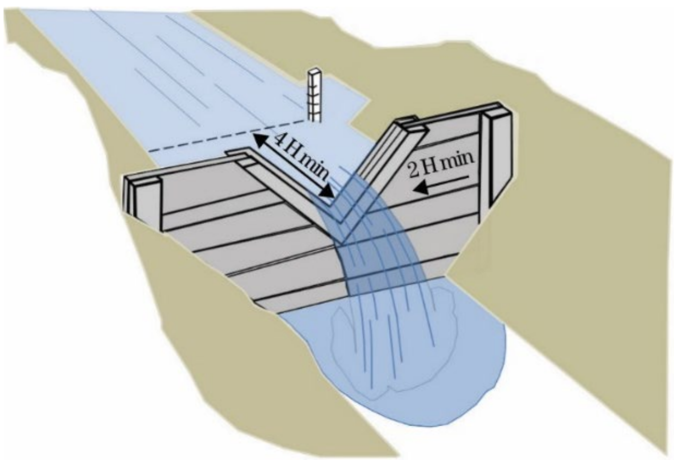
(iii) The V-notch weir equation is equation 3–31.

$$Q_w = 2.52\, H_w^{\ 2.47} \qquad \text{(eq. 3-31)}$$

where: Q_w = discharge, ft^3/s

H_w = weir head above the point of the V-notch, ft

Figure 3-57: 90-degree V-notch Weir



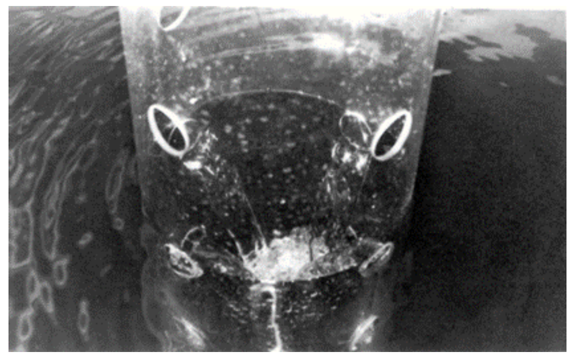
650.0307 Orifice Flow

A. An orifice is a hole of regular form through which water flows. For example, the irrigation water flows through the orifice in the sprinkler shown in figure 3–58.

B. The round holes in the clear riser in figure 3–59 function as orifices.

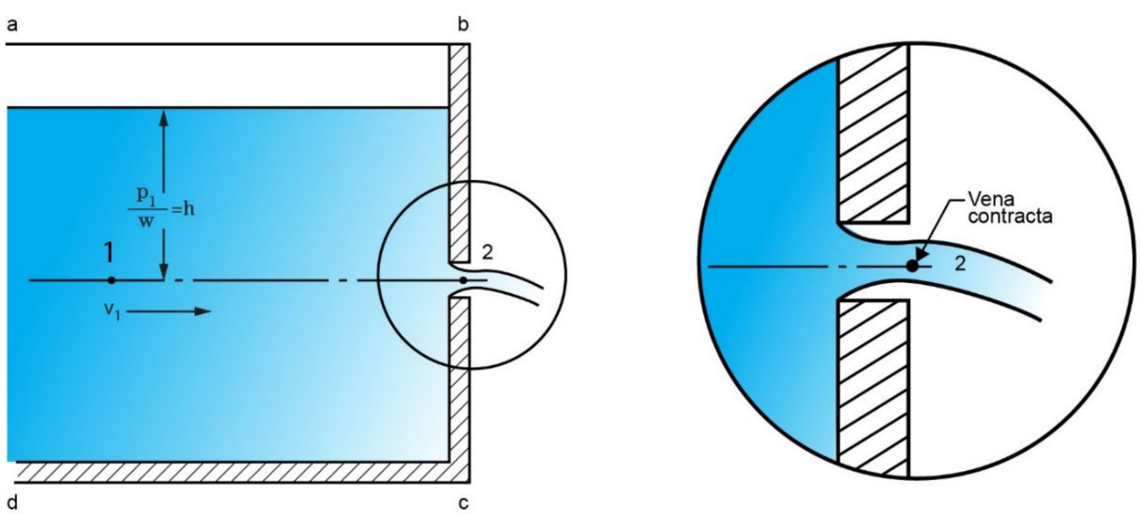
Figure 3-58: Irrigation Sprinkler in New Mexico

Figure 3-59: Orifice Jet Inside a Clear Round Riser



C. Figure 3-60 shows a schematic of an orifice and illustrates flow through a sharp-edged orifice where the flow springs clear from the upstream orifice edge and does not cling to the orifice walls or downstream edge. An orifice discharging into the air has free discharge, whereas a submerged orifice discharges under water. Orifices may be circular, square, rectangular, or of any other regular form.

Figure 3-60: Orifice Flow Schematic



D. Orifice Equation

(1) The Bernoulli equation written from point 1 to point 2 illustrated in figure 3–60 is:

$$\frac{{v_1}^2}{2g} + \frac{p_1}{\gamma} = \frac{{v_2}^2}{2g} + \frac{p_2}{\gamma} + h_1$$

(2) Rearrange to solve for v_2.

$$v_2 = \sqrt{2g\left(\frac{p_1}{\gamma} - \frac{p_2}{\gamma} + \frac{{v_1}^2}{2g} - h_1\right)}$$

(3) Point 2, where the jet has ceased to contract, is called the vena contracta. Its pressure is equal to that of the surrounding fluid. For free discharge into the atmosphere, p_2 is zero on the gage scale. For large tanks, v_1 is negligible. Replacing p_1/γ with h and dropping the subscript of v_2, the equation becomes:

$$v = \sqrt{2g(h - h_1)}$$

(4) Neglecting energy losses, the equation for the theoretical velocity, v_t, becomes

$$v_t = \sqrt{2gh}$$

E. Orifice Coefficients

(1) The energy loss may be accounted for by applying a coefficient of velocity, C_v, to the theoretical velocity in equation 3–32.

$$v = C_v\sqrt{2gh} \qquad \text{(eq. 3-32)}$$

where: v = velocity, ft/s
g = acceleration of gravity, 32.2 ft/s^2
h = orifice head, ft

(2) The area at the vena contracta, a_2, is smaller than the orifice area, a. The ratio between the two areas is the coefficient of contraction, C_c.

$$C_c = \frac{a_2}{a}$$

(3) The orifice discharge is the product of the velocity and the area at the vena contracta.

$$Q = a_2 v_2 = C_c a C_v \sqrt{2gh}$$

(4) The product of C_c, and C_v, is the coefficient of discharge, C_d, in equation 3–33.

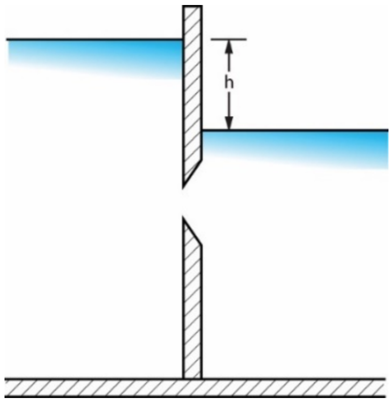
$$Q = C_d a\sqrt{2gh} \qquad \text{(eq. 3-33)}$$

where: Q = discharge, ft^3/s
C_d = orifice coefficient of discharge, dimensionless
a = orifice area, ft^2
g = acceleration of gravity, 32.2 ft/s^2
h = orifice head, ft

F. Submerged Orifices

(1) The submerged orifice in figure 3–61 is the type most often used in soil conservation work.

Figure 3-61: Submerged Orifice

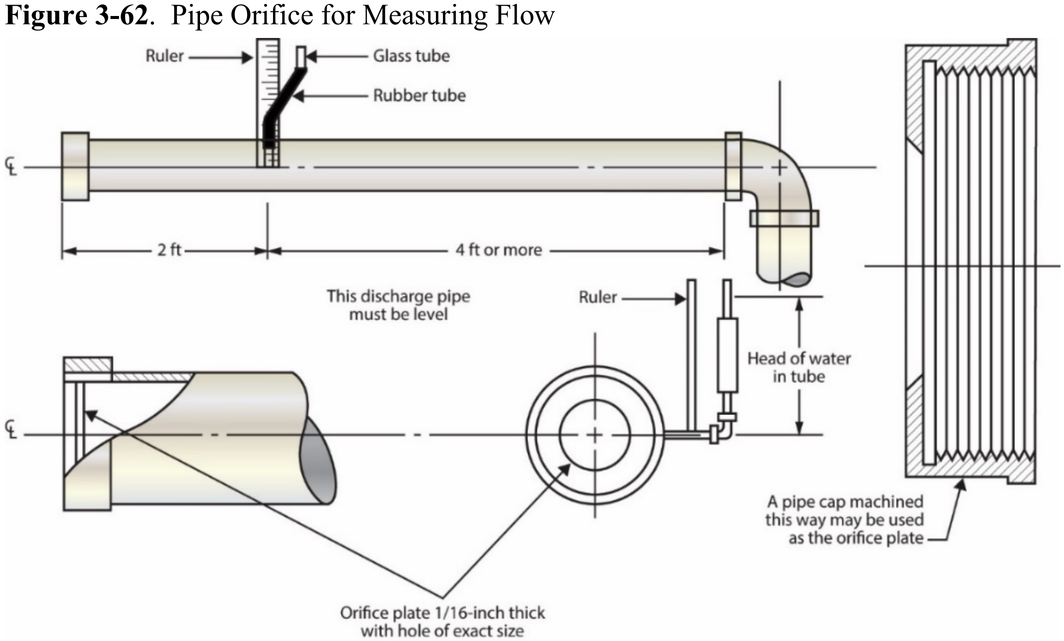


(2) As in the free discharge orifice, equations 3–29 and 3–33 also apply to the submerged orifice. The coefficient of discharge for submerged orifices is approximately the same as for free discharge orifices. Discharge coefficients for specific orifice flow applications can be found in manufacturers' publications.

G. Measuring Pipe Flow Using Orifices

(1) A pipe orifice (fig. 3–62) is usually a circular orifice near or at the end of a horizontal pipe. The orifice head is measured with a manometer at points upstream and downstream from the orifice. For a further description of this type of orifice, refer to King's Handbook of Hydraulics.

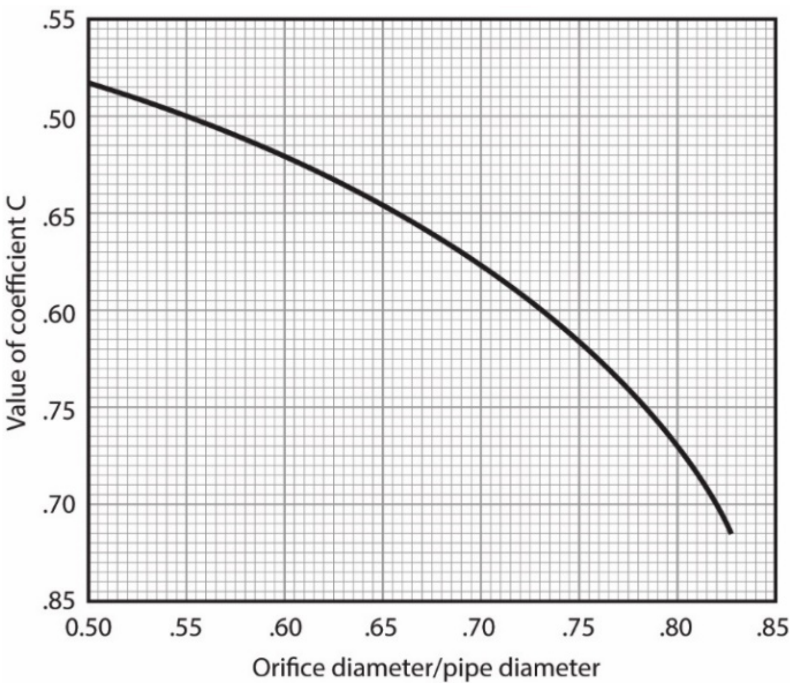
Figure 3-62. Pipe Orifice for Measuring Flow



(2) A sharp-edged orifice commonly used in measuring flow within a range of 50 to 2,000 gallons per minute has a circular orifice located at the end of a level section of pipe. Place a transparent tube manometer 2 feet upstream from the orifice. The pipe 4 feet upstream of the manometer should be free of elbows, valves, or other fittings. The ratio of the orifice diameter to the pipe diameter should be between 0.50 and 0.83. The orifice should induce full pipe full conditions. Measure the head in the manometer with a ruler or tape measure.

(3) Discharge through the orifice can be computed by equation 3–33. The orifice coefficient can be obtained from figure 3–63.

Figure 3-63: C_d, Coefficient for Orifice in End of Pipe



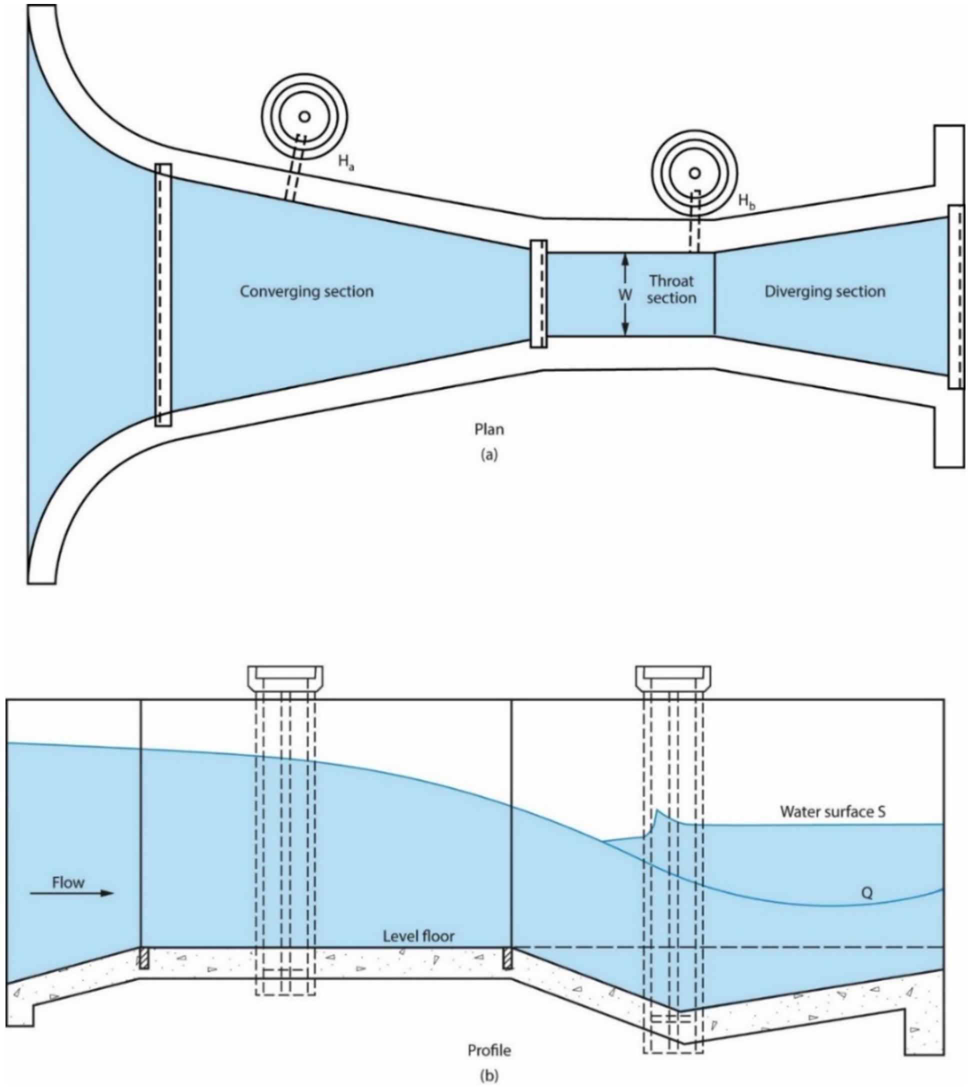
650.0308 Flow Measurement

A. Parshall Flume

(1) With a Parshall flume (fig. 3–64), the discharge is obtained by measuring the loss in head caused by forcing a stream of water through the throat section of the flume, which has a depressed bottom.

(2) Since the velocity of approach has little effect on the measured discharge accuracy, the Parshall flume requires no pond above it. The flume uses a small amount of head and does not require corrections to the free-flow discharge even with moderate submergence. Floating trash tends to pass through the flume, as do fine-grained sediments. Generally, only head, H_a, needs to be recorded for determining the discharge. If the flow is extremely submerged, then both H_a and H_b should be recorded. The current edition of the U.S. Department of Interior, Bureau of Reclamation (USDI BOR) Water Measurement Manual (2001) provides additional information about Parshall flumes.

Figure 3-64: Parshall Flume



B. Flat-bottomed Trapezoidal Flumes

Flat-bottomed trapezoidal flumes conform to typical shapes of small canals. Generally, the cross section of the canal matches the entrance section of the flume. Sedimentation is less of a concern, and the canal can drain dry between uses since the floor of the flume is flush with the bottom of the canal. For more information, see USDA Agricultural Research Service Bulletin 41–140 (1968) and the current edition of the USDI, BOR, Water Measurement Manual, chapter 8 (2001).

C. Current Meter

(1) Basically, the current meter is a wheel having several cups or vanes. Current flow rotates the wheel and the speed of the rotating wheel indicates the current velocity, based on the manufacturer's rating table.

(2) A reference point is established on one bank of the stream, and a measuring tape is stretched across the stream for horizontal distance. Soundings and current-meter readings are taken at prescribed depths at regularly spaced horizontal intervals, usually from 2 to 10 feet, depending on the stream width. Readings also should be made where abrupt changes in velocity or flow depth occur.

(3) A common method used to determine mean velocity requires readings at two points in each vertical: 0.2 and 0.8 of the sounded depth measured from the water surface. The average reading becomes the mean velocity in the vertical. Where depth is too shallow to obtain two readings, a single reading at 0.6 depth represents the mean velocity.

(4) The discharge of each segment of stream between adjacent verticals is the product of the cross-sectional area and the mean velocity. If d_1 and d_2 represent the flow depths at two adjacent verticals, v_1 and v_2 the respective mean velocities in these verticals, and W the distance between the verticals, then the discharge in that part of the cross section is computed from equation 3–34.

$$Q = W \times \left(\frac{d_1+d_2}{2}\right) \times \left(\frac{v_1+v_2}{2}\right) \qquad \text{(eq. 3-34)}$$

where: Q = discharge, ft^3/s
W = distance between adjacent verticals, ft
d_1 = flow depth at vertical 1, ft
d_2 = flow depth at vertical 2, ft
v_1 = mean velocity at vertical 1, ft/s
v_2 = mean velocity at vertical 2, ft/s

(5) The total discharge of the stream is the sum of such computations for the individual segments. See USDI, BOR, Water Measurement Manual, chapter 10 (2001) for more information regarding the use of current meters.

D. Water-stage Recorder

(1) A water-stage recorder combines a clock and an instrument that records the rise and fall of a water surface with respect to time. Water-stage recorders are common at permanent stream-gaging stations.

(2) Water-stage recorders are desirable under the following conditions:

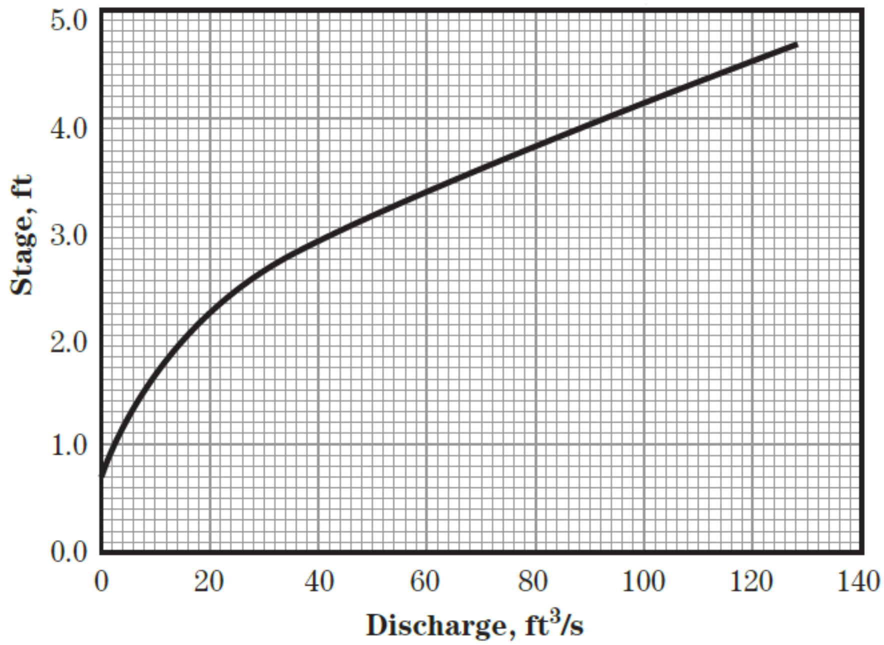
(i) The flow in the stream or channel fluctuates rapidly, and occasional staff-gage readings would not provide a satisfactory discharge estimate.

(ii) The gaging station is remote or consistent observers are unavailable.

(iii) There is a need for continuous records of flow for legal or technical purposes.

(3) By the combined use of the stage-discharge curve (fig. 3–65) and the water-stage recorder, a hydrograph of the flow in a stream or channel may be plotted. A hydrograph is developed by plotting discharge on a vertical scale against time plotted on a horizontal scale. The area beneath the curve represents the volume of water passing the gaging station during any selected time period.

Figure 3-65. Example of stage-discharge for unlined irrigation canal



E. Measurement by Floats

(1) The flow velocity of a stream may be approximated using surface floats and channel cross sections. Select a straight, uniform reach with minimum surface waves. Calm days provide better surface velocity measurements compared to windy days, when the floats may blow off-course.

(2) Divide the stream into segments and determine the average depth for each segment. The segments should be narrower in the outer edges of the stream than in the central section. Lay out float courses in the middle of the strips or the previously defined segments. The mean velocity of a strip in a straight, regular-shaped channel is approximately 85 percent of the surface float velocity but can vary from 80 to 95 percent.

(3) Multiply the cross-sectional area of each strip by the adjusted mean velocity to determine the discharge. The sum of the discharges of the strips is the total discharge. On small streams, rather than dividing the stream into segments, a number of float runs can be made and an average used for the surface velocity of the stream.

F. Slope-area Method

(1) The slope-area method uses the water surface slope in a uniform channel reach and average cross-sectional area to estimate discharge. The selected reach should be straight and at least 200 feet and preferably 1,000 feet long. The reach should be free of rapids, abrupt falls, sudden contractions, or expansions.

(2) The discharge is computed from Manning's equation 3–21.

$$Q = \frac{1.486}{n} a r^{2/3} \sqrt{s}$$

where: Q = flow, ft^3/s
n = Manning's roughness coefficient, dimensionless
a = cross-sectional flow area, ft^2
r = hydraulic radius, ft
s = water surface slope, ft/ft

(3) The water surface slope is the difference in the water surface elevation from the upstream to downstream end divided by the reach length. To develop a stage-discharge curve, place gage points inside stilling wells on each channel bank and the stream centerline. The gage points should reference a common datum. For irregular cross sections, the area and the wetted perimeter at several cross sections should be measured and a mean value used for computing hydraulic radius.

(4) The discharge determined by the slope-area method is approximate because selecting the roughness factor n is not precise. If the water surface elevations are rapidly fluctuating, the slope and areas should be measured simultaneously.

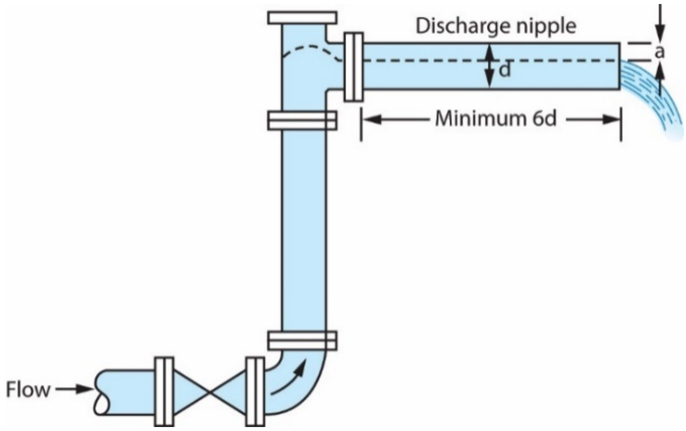
G. California Pipe Method

(1) The California pipe method, developed by B.R. Vanleer (1922, 1924), is used to measure the discharge from the open end of a partially filled horizontal pipe discharging freely into the air. This method can also be adapted to the measurement of discharge in small open channels where such flow can be diverted through a horizontal pipe flowing partially full and discharging freely into the air.

(2) The method has four requirements for accurate results:
(i) discharge pipe must be level and be at least six times the pipe diameter in length
(ii) discharge pipe must discharge partially full
(iii) discharge pipe must discharge freely into air
(iv) velocity of approach must be a minimum

(3) Figure 3–66 illustrates one method of meeting these requirements.

Figure 3-66. Measuring Flow by California Pipe Method



(4) Other designs may be possible. With such an arrangement, the only measurements necessary are the inside diameter of the pipe and the vertical distance from the upper inside surface of the pipe to the surface of the flowing water at the outlet end of the pipe. The discharge may then be computed by equation 3–35.

$$Q = 8.69\left(1 - \frac{a}{d}\right)^{1.88} d^{2.48} \qquad \text{(eq. 3-35)}$$

where: Q = flow, ft^3/s
a = distance from top inside surface of pipe to water surface, ft
d = pipe diameter, ft

(5) This equation was developed from experimental data for pipes 3 to 10 inches in diameter. NRCS testing showed that for depths greater than half of the diameter of the pipe (or *a/d* less than 0.5) the discharges did not follow the equation. Use care whenever the pipe is less than half full, or in other words, whenever *a/d* is less than 0.5.

H. Coordinate Method

(1) In this method, coordinates of the jet flowing from the end of a pipe are measured. The flow from pipes may be measured whether the pipe is discharging horizontally as shown in figure 3-67, vertically upward as shown in figure 3-68, or at an angle. Generally, the discharge pipe is set in a horizontal position for measurement purposes, rather than at an angle. Coordinate methods are used to measure the flow from flowing wells (discharging vertically) and from small pumping plants (discharging horizontally). See the USDI BOR, Water Measurement Manual, chapter 14 for additional information on these methods.

(2) This method has limited use because it is difficult to accurately measure the jet coordinates. Only use this method when other measurement methods are not practical and a 10 percent error is acceptable.

(3) To measure the flow from pipes discharging horizontally (fig. 3–67), measure the horizontal and vertical distance from the inside top of the pipe to the top surface of the jet. These horizontal and vertical distances are the X and Y coordinates. The discharge pipe must be level and long enough to permit the water to flow smoothly as it exits the pipe. Exhibits DD through GG in section 650.0311 give discharge values for pipe diameters up to 6 inches where the X coordinate is 0, 6, 12, or 18 inches.

(4) To measure the flow from pipes discharging vertically upward, measure the inside pipe diameter (*d*) and height of the jet above the pipe outlet (*H*) (fig. 3–68). Exhibit CC in section 650.0311 gives discharge values for pipe diameters up to 12 inches and jet heights up to 40 inches.

Figure 3-67: Coordinates for Horizontal Discharge Approximation

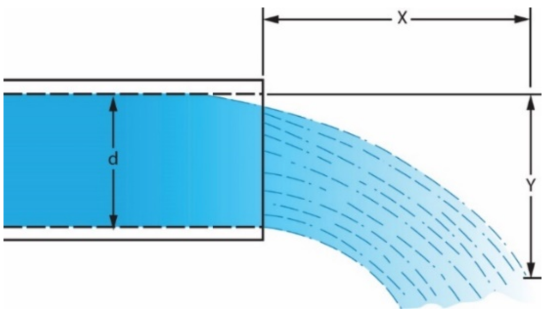
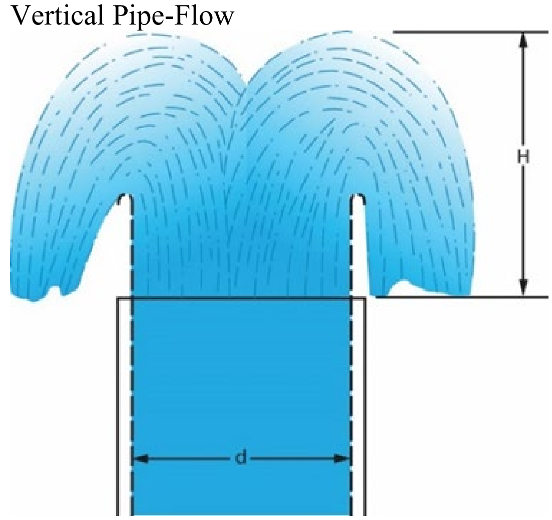


Figure 3-68: Coordinate Methods Data for Vertical Pipe-Flow



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T. In addition to these references, material from the following references was incorporated into the original 1984 document:

(1) Hydraulics Correspondence Course by George A. Lawrence, State Conservation Engineer, State of Utah.
(2) Utah State Engineering Handbook, Section 5, Hydraulics.
(3) Vinnard, J.K. 1948. Elementary fluid mechanics. John Wiley and Sons, Inc.

650.0310 Acknowledgements

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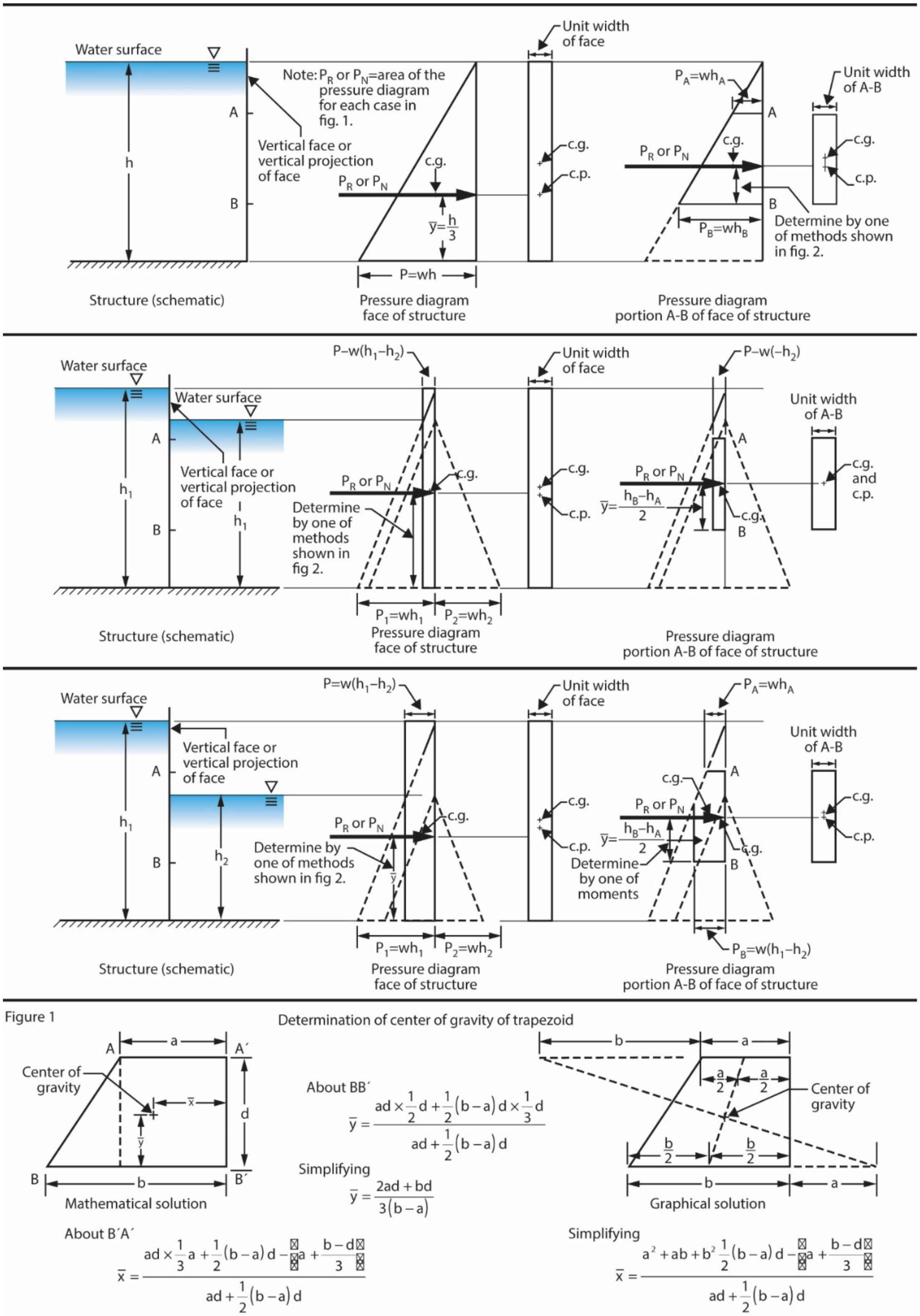
650.0311 Exhibits

A. Unit Conversions

Base unit	Equivalent units
Time	
1 day (d)	24 hours (h)
	1,440 minutes (min)
	86,400 seconds (s)
Length	
1 mile	5,280 feet (ft)
	1,760 yards (yd)
Area	
1 ft^2	144 square inches (in^2)
1 yd^2	9 ft^2
1 acre	46,560 ft^2
1 mi^2	640 acres
	27,878,400 ft^2
Volume	
1 gallon (gal)	231 in^3
	0.1337 ft^3
1 million gal	3.0689 acre-ft
1 ft^3	1,728 in^3
	7.48 gal
1 acre-ft	43,560 ft^3
	325,850 gal
	12 acre-inch (acre-in)
Flow	
1 gal/min	0.00223 ft^3/s
	1,440 gal/d
1 million gal/d	1.547 ft^3/s (cfs)
	695 gal/min
	448.8 gal/min
	646,300 gal/d
	0.992 acre-in/h
	1.983 acre-ft/d
Weight	
1 gal water	8.33 pounds (lb)
1 ft^3 water	62.4 lb

B(a). Pressure Diagrams for Hydrostatic Loads



B(b). Pressure Diagrams for Hydrostatic Loads

Figure 2

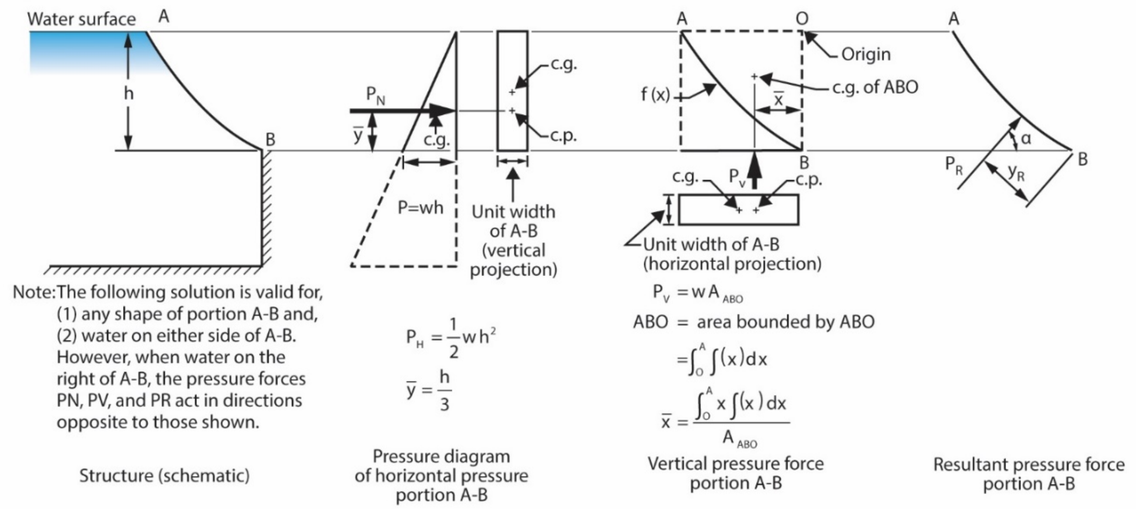


Figure 3

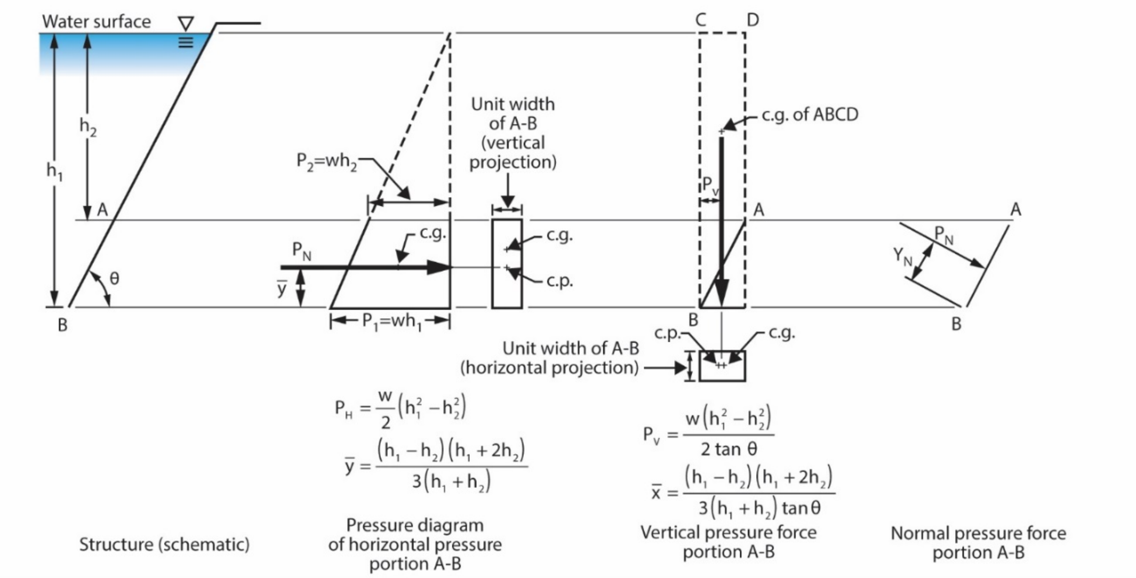


Figure 4

Symbols and definitions

c.g.	Center of gravity of area, as indicated.
c.p.	Center of pressure; i.e., point of action of a pressure force, or a component of a pressure face, of a structure.
n	Height of water above a point, as indicated in structure (schematic), or as indicated by subscript.
p	Intensity of pressure of a point indicated by subscript, or of bottom of structure or portion of structure if no subscript is used.
P_H	Horizontal component of pressure force per foot width.
P_V	Vertical component of pressure force per foot width.
P_N	Normal pressure force per foot width.
P_R	Resultant pressure force per foot width.
w	Weight of water per cubic foot.
$\bar{x}, \bar{y}$	Coordinates of c.g. of pressure diagram.
Y_N	Distance form given point perpendicular to line of action of P_N.
Y_R	Distance from given point perpendicular to line of action of P_R.

C(a). Head Loss Coefficient K_p for Circular Pipe Flowing Full

$$K_p = \frac{5{,}087\, n^2}{(d_i)^{4/3}}$$

Head loss coefficient, K_p, for circular pipe flowing full

Pipe diam, in	Flow area, ft^2	Manning's coefficient of roughness, n							
		0.009	0.01	0.011	0.012	0.013	0.014	0.015	0.016
6	0.196	0.0378	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194
8	0.349	0.0258	0.0318	0.0385	0.0458	0.0537	0.0623	0.0715	0.0814
10	0.545	0.0191	0.0236	0.0286	0.0340	0.0399	0.0463	0.0531	0.0604
12	0.785	0.0150	0.0185	0.0224	0.0267	0.0313	0.0363	0.0417	0.0474
15	1.23	0.0111	0.0138	0.0166	0.0198	0.0232	0.0270	0.0309	0.0352
18	1.77	0.00873	0.0108	0.0130	0.0155	0.0182	0.0211	0.0243	0.0276
21	2.41	0.00711	0.00878	0.0106	0.0126	0.0148	0.0172	0.0198	0.0225
24	3.14	0.00595	0.00735	0.00889	0.0106	0.0124	0.0144	0.0165	0.0188
30	4.91	0.00442	0.00546	0.00660	0.00786	0.00922	0.0107	0.0123	0.0140
36	7.07	0.00347	0.00428	0.00518	0.00616	0.00723	0.00839	0.00963	0.0110
42	9.62	0.00282	0.00348	0.00422	0.00502	0.00589	0.00683	0.00784	0.00892
48	12.6	0.00236	0.00292	0.00353	0.00420	0.00493	0.00572	0.00656	0.00747
54	15.9	0.00202	0.00249	0.00302	0.00359	0.00421	0.00488	0.00561	0.00638
60	19.6	0.00175	0.00217	0.00262	0.00312	0.00366	0.00424	0.00487	0.00554
72	28.3	0.00138	0.00170	0.00205	0.00245	0.00287	0.00333	0.00382	0.00435
84	38.5	0.00112	0.00138	0.00167	0.00199	0.00234	0.00271	0.00311	0.00354
96	50.3	0.00094	0.00116	0.00140	0.00167	0.00196	0.00227	0.00260	0.00296

$$H_l = K_p L \frac{v^2}{2g}$$

Example: Compute head loss in 300 ft of 24-in diameter pipe when the full flow is 30 ft^3/s and Manning's n is 0.015.

$$v = \frac{Q}{a} = \frac{30\ ft^3/s}{3.14\ ft^2} = 9.55\ ft/s$$

$$\frac{v^2}{2g} = \frac{\left(9.55 \frac{ft}{s}\right)^2}{2\ \times 32.2\ ft/s^2} = 1.42\ ft$$

$$H_l = K_p L \frac{v^2}{2g} = 0.0165 \times 300 \times 1.42 = 7.03\ ft$$

C(b). Head Loss Coefficient K_p for Circular Pipe Flowing Full

$$K_p = \frac{5{,}087\, n^2}{(d_i)^{4/3}}$$

Head loss coefficient, K_p, for circular pipe flowing full

Pipe diam, in	Flow area, ft²	Manning's coefficient of roughness, n								
		0.017	**0.018**	**0.019**	**0.020**	**0.021**	**0.022**	**0.023**	**0.024**	**0.025**
6	0.196	0.1348	0.1512	0.1684	0.1866	0.2058	0.2258	0.2468	0.2688	0.2916
8	0.349	0.0919	0.1030	0.1148	0.1272	0.1402	0.1539	0.1682	0.1831	0.1987
10	0.545	0.0682	0.0765	0.0852	0.0944	0.1041	0.1143	0.1249	0.1360	0.1476
12	0.785	0.0535	0.0600	0.0668	0.0741	0.0817	0.0896	0.0980	0.1067	0.1157
15	1.23	0.0397	0.0446	0.0496	0.0550	0.0606	0.0666	0.0727	0.0792	0.0859
18	1.77	0.0312	0.0349	0.0389	0.0431	0.0476	0.0522	0.0570	0.0621	0.0674
21	2.41	0.0254	0.0284	0.0317	0.0351	0.0387	0.0425	0.0464	0.0506	0.0549
24	3.14	0.0212	0.0238	0.0265	0.0294	0.0324	0.0356	0.0389	0.0423	0.0459
30	4.91	0.0158	0.0177	0.0197	0.0218	0.0241	0.0264	0.0289	0.0314	0.0341
36	7.07	0.0124	0.0139	0.0154	0.0171	0.0189	0.0207	0.0226	0.0246	0.0267
42	9.62	0.0101	0.0113	0.0126	0.0139	0.0154	0.0169	0.0184	0.0201	0.0218
48	12.6	0.00843	0.00945	0.0105	0.0117	0.0129	0.0141	0.0154	0.0168	0.0182
54	15.9	0.00720	0.00808	0.00900	0.00997	0.0110	0.0121	0.0132	0.0144	0.0156
60	19.6	0.00626	0.00702	0.00782	0.00866	0.00955	0.0105	0.0115	0.0125	0.0135
72	28.3	0.00491	0.00550	0.00613	0.00679	0.00749	0.00822	0.00898	0.00978	0.0106
84	38.5	0.00400	0.00448	0.00499	0.00553	0.00610	0.00669	0.00731	0.00796	0.00864
96	50.3	0.00334	0.00375	0.00418	0.00463	0.00510	0.00560	0.00612	0.00667	0.00723

$$H_l = K_p L \frac{v^2}{2g}$$

Example: Compute head loss in 300 ft of 24-in diameter pipe when the full flow is 30 ft³/s and Manning's n is 0.015.

$$v = \frac{Q}{a} = \frac{30\ ft^3/s}{3.14\, ft^2} = 9.55\, ft/s$$

$$\frac{v^2}{2g} = \frac{\left(9.55\,\frac{ft}{s}\right)^2}{2\ \times 32.2\ \ ft/s^2} = 1.42\, ft$$

$$H_l = K_p L \frac{v^2}{2g} = 0.0165 \times 300 \times 1.42 = 7.03\, ft$$

D. Head Loss Coefficient K_c for Square Conduit Flowing Full

$$K_c = \frac{29.16n^2}{(r)^{4/3}}$$

Conduit width and height ft	Flow area ft²	Manning's coefficient of roughness						
		0.010	**0.011**	**0.012**	**0.013**	**0.014**	**0.015**	**0.016**
2	4	0.00735	0.00889	0.01058	0.01242	0.01440	0.01653	0.01881
2.5	6.25	0.00546	0.00660	0.00786	0.00922	0.01070	0.01228	0.01397
3	9	0.00428	0.00518	0.00616	0.00723	0.00839	0.00963	0.01096
3.5	12.25	0.00348	0.00422	0.00502	0.00589	0.00683	0.00784	0.00892
4	16	0.00292	0.00353	0.00420	0.00493	0.00572	0.00656	0.00746
5	25	0.00217	0.00262	0.00312	0.00366	0.00424	0.00487	0.00554
6	36	0.00170	0.00205	0.00245	0.00287	0.00333	0.00382	0.00435
7	49	0.00138	0.00167	0.00199	0.00234	0.00271	0.00311	0.00354
8	64	0.00116	0.00140	0.00167	0.00196	0.00227	0.00260	0.00296
9	81	0.00099	0.00120	0.00142	0.00167	0.00194	0.00223	0.00253
10	100	0.00086	0.00104	0.00124	0.00145	0.00168	0.00193	0.00220
12	144	0.00067	0.00082	0.00097	0.00114	0.00132	0.00152	0.00173

$$H_l = K_c L \frac{v^2}{2g}$$

Example: Compute discharge in 250 ft of 3-ft by 3-ft box conduit, when the head loss is 2.25 ft and Manning's n is 0.014.

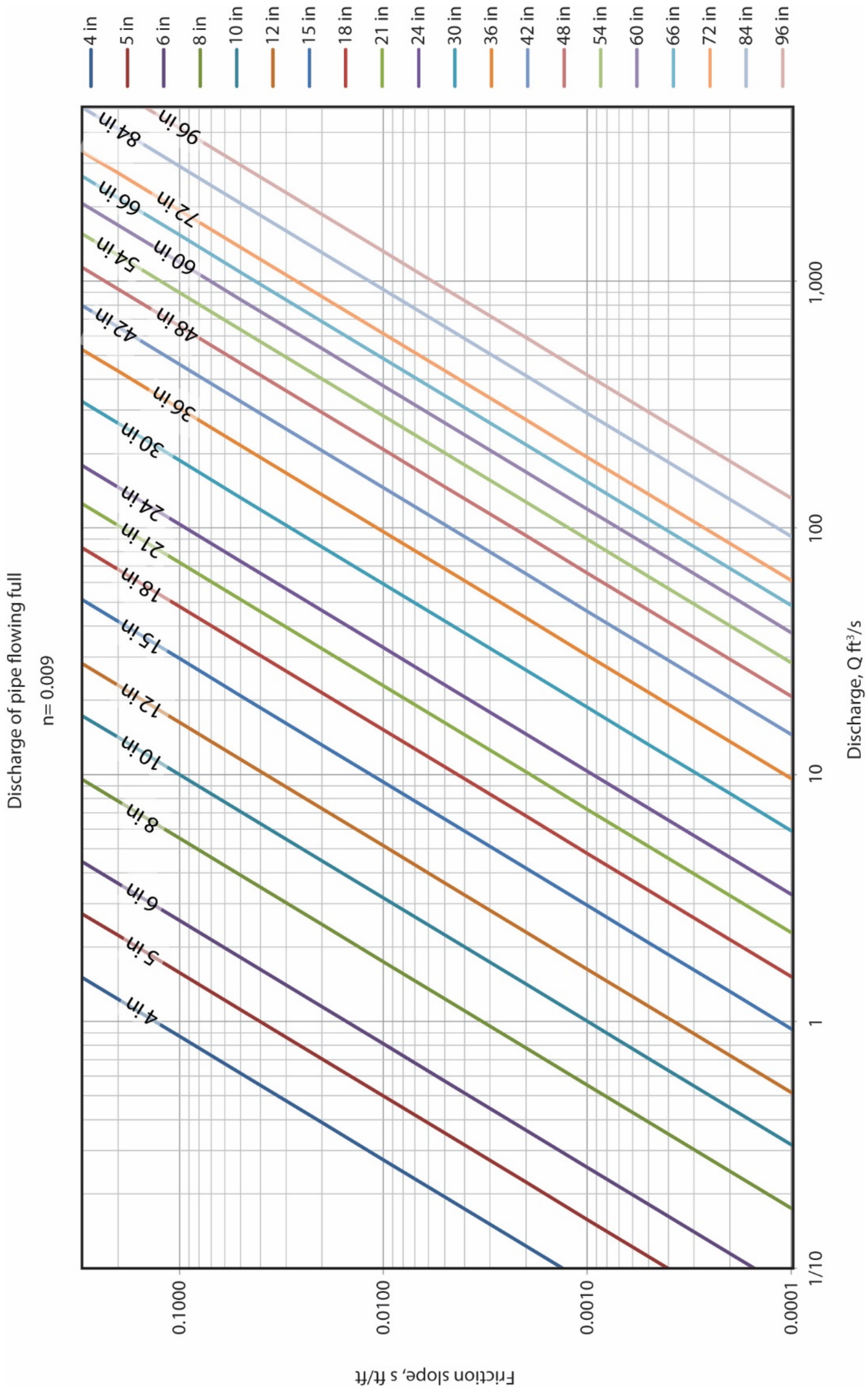
$$H_l = K_c L \frac{v^2}{2g}$$

$$v = \sqrt{2 \times 32.2\ ft^3/s\ \times 1.073\ ft} = 8.31\ ft/s$$

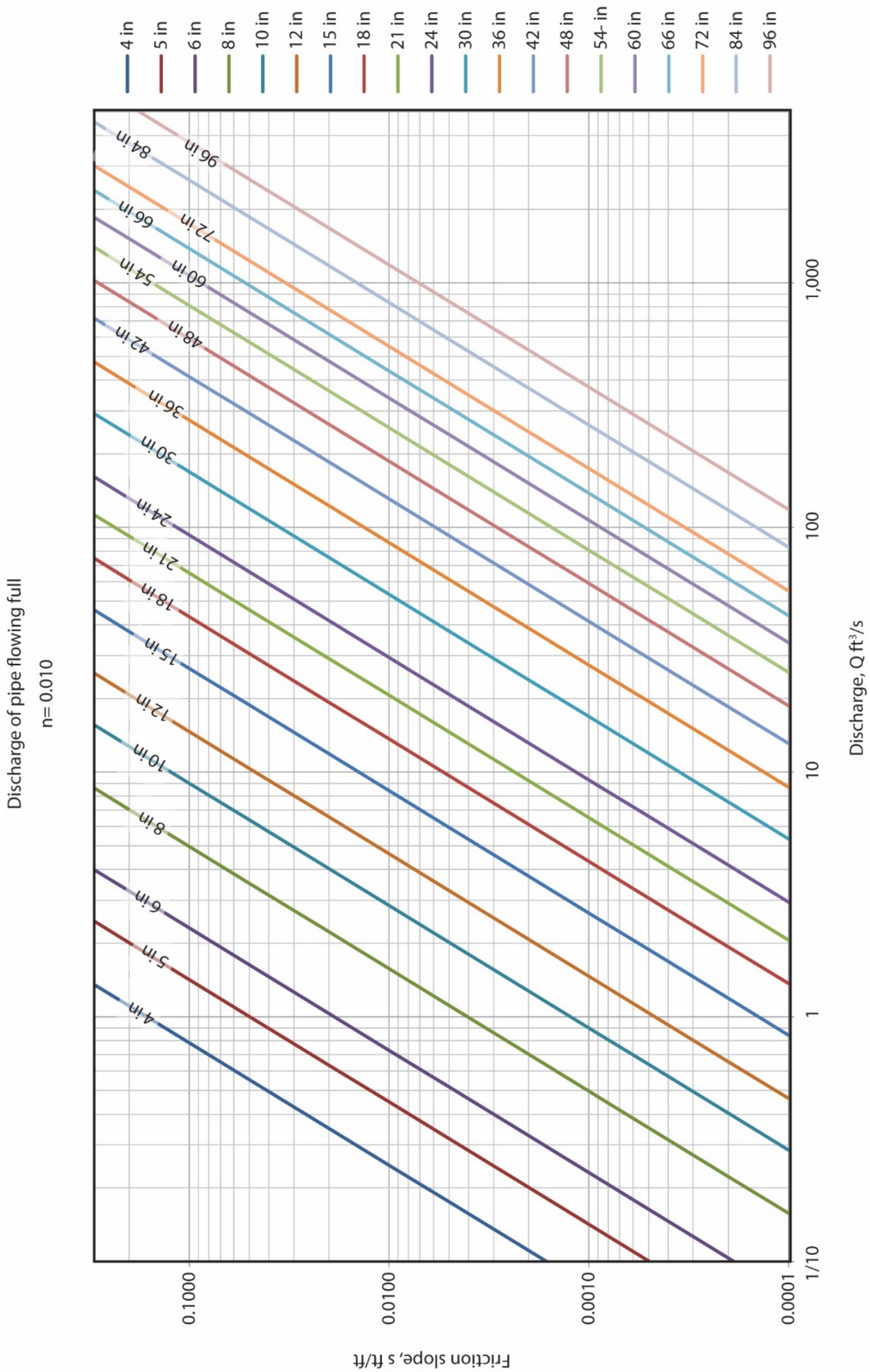
$$\frac{v^2}{2g} = \frac{H_l}{K_c L} = \frac{2.25\ ft}{0.00839 \times 250\ ft} = 1.073\ ft$$

$$Q = va = 8.31\ ft/s \times 9\ ft^2 = 75\ ft^3/s$$

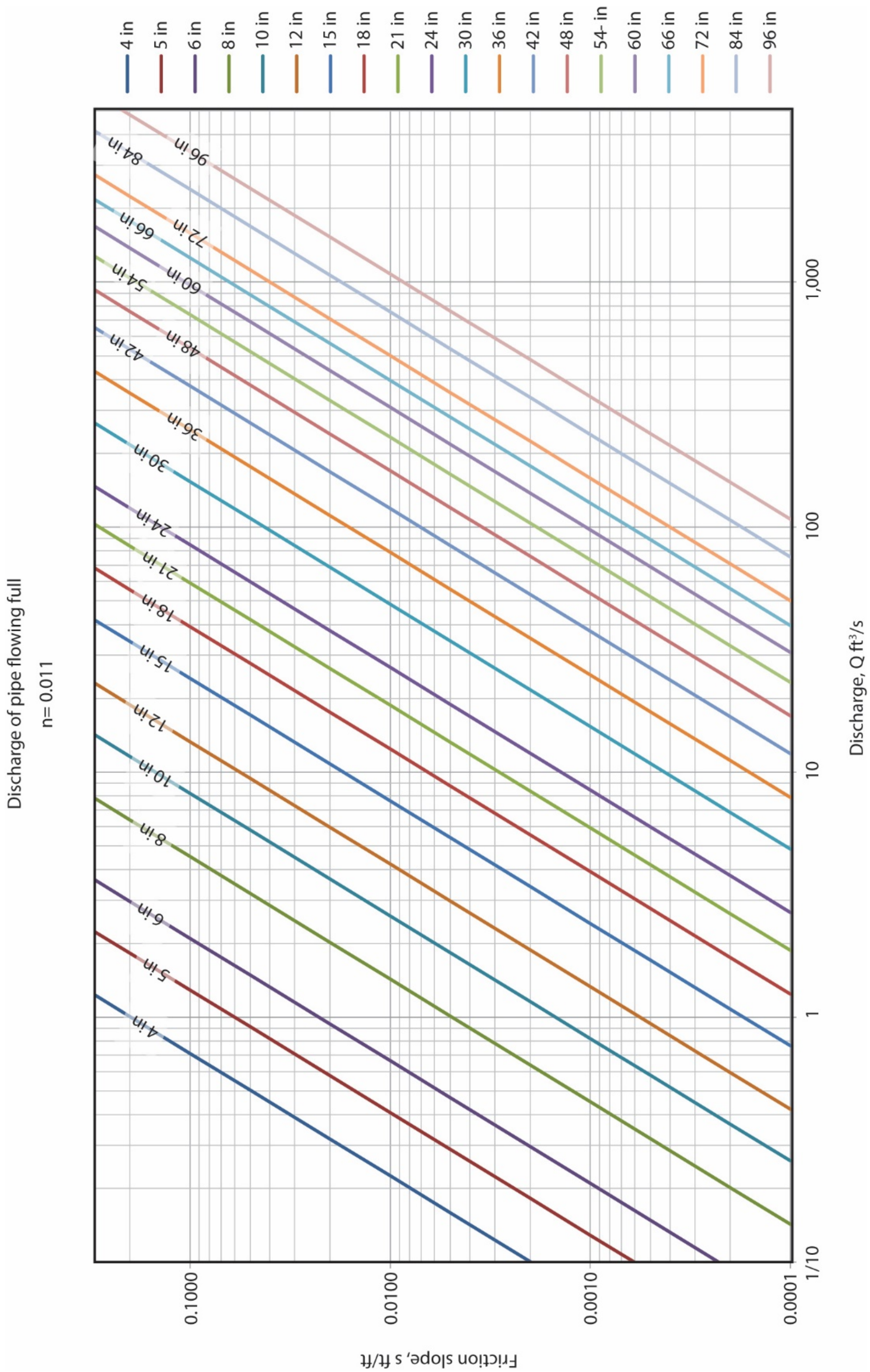
E. Discharge of Pipe Flowing Full n = 0.009



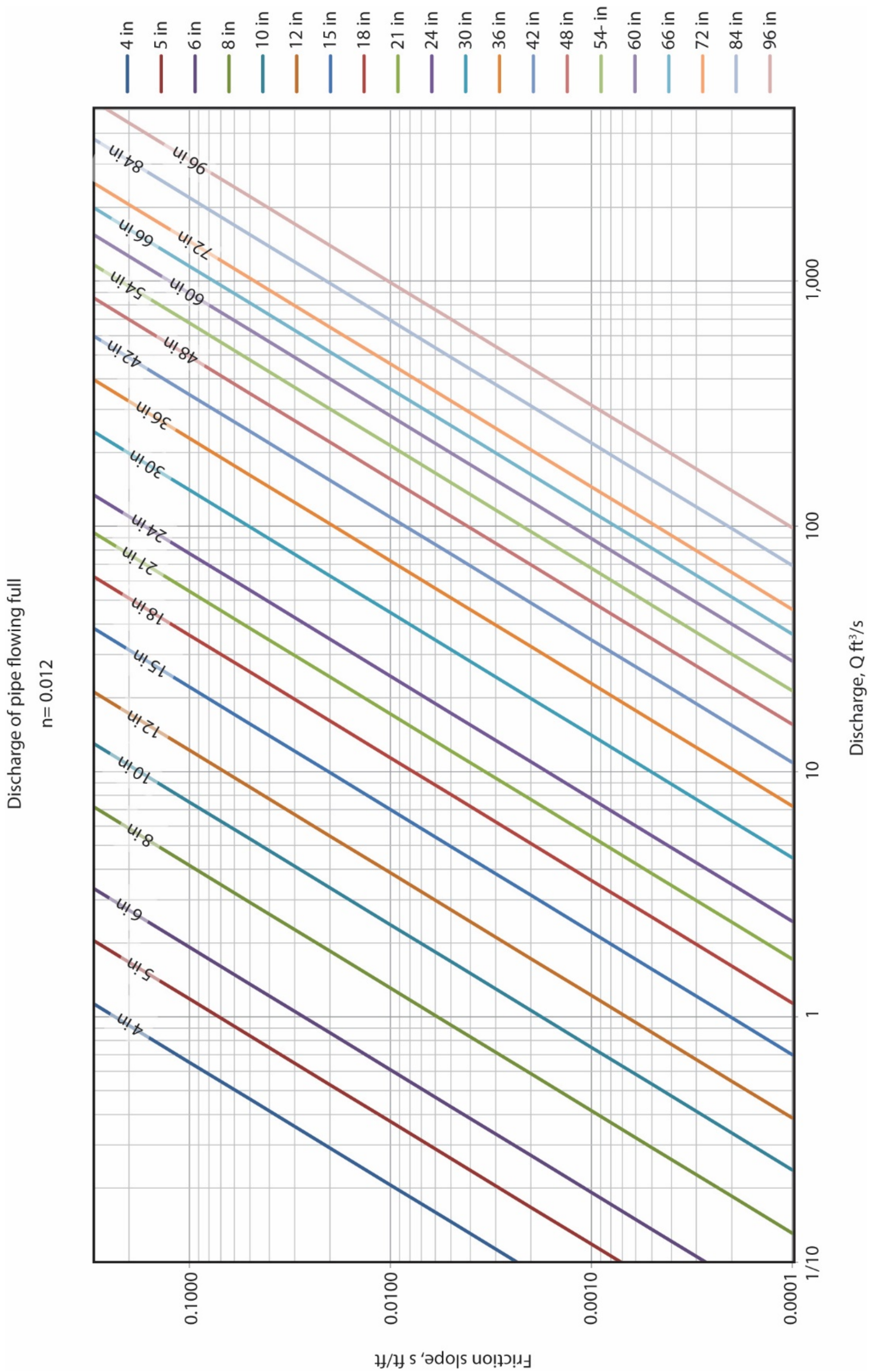
F. Discharge of Pipe Flowing Full n = 0.010



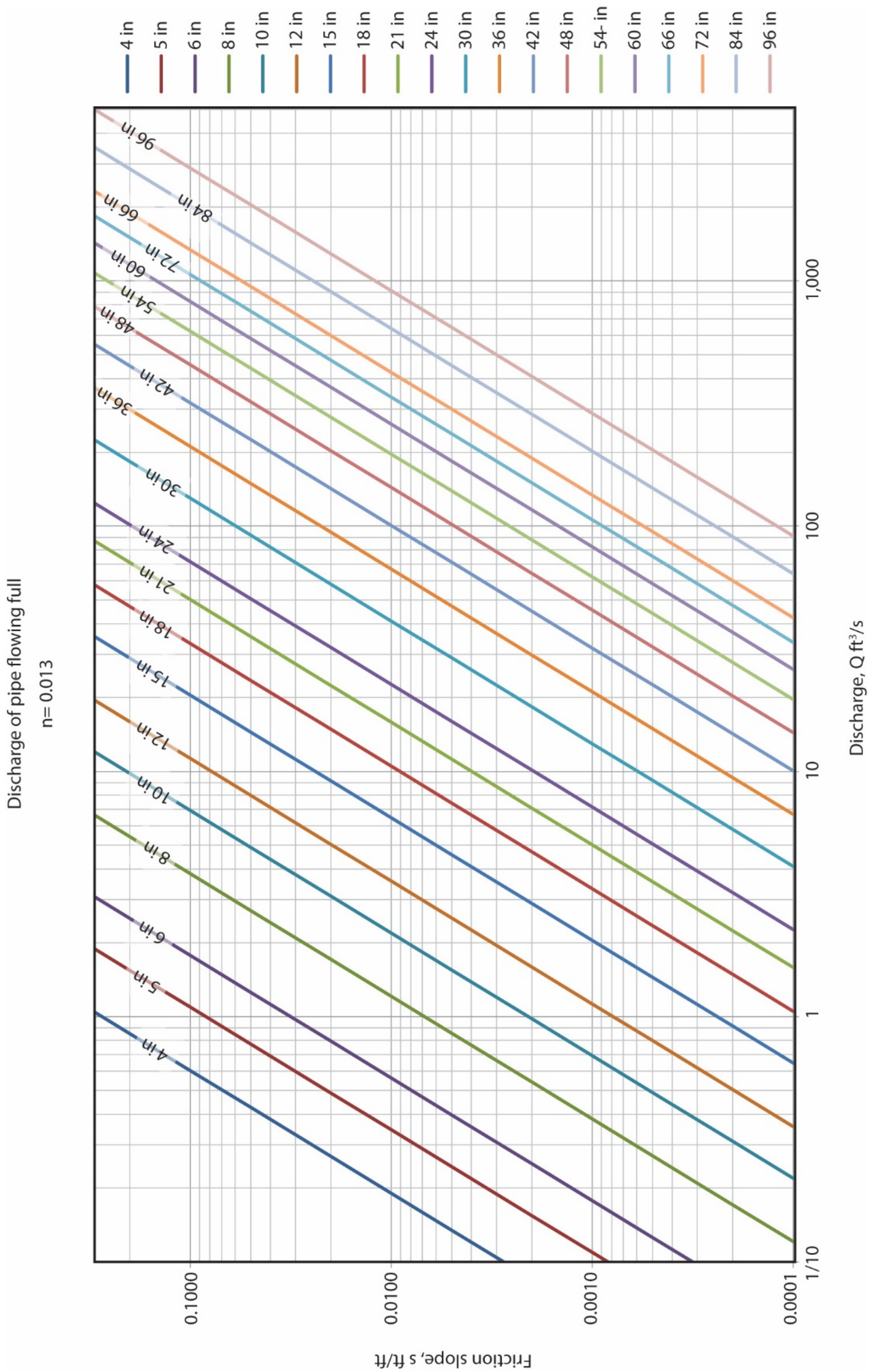
G. Discharge of Pipe Flowing Full n = 0.011



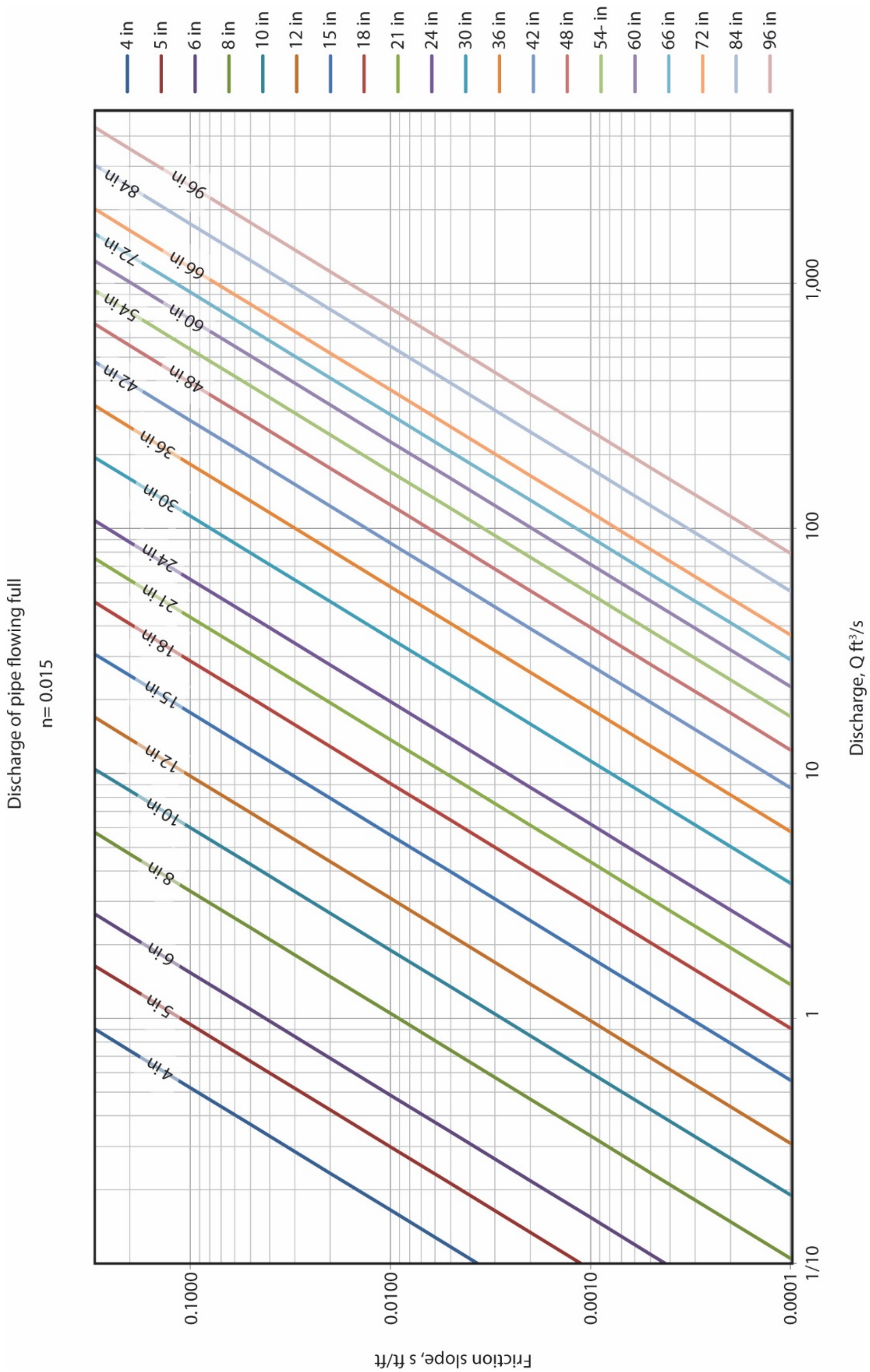
H. Discharge of Pipe Flowing Full n = 0.012



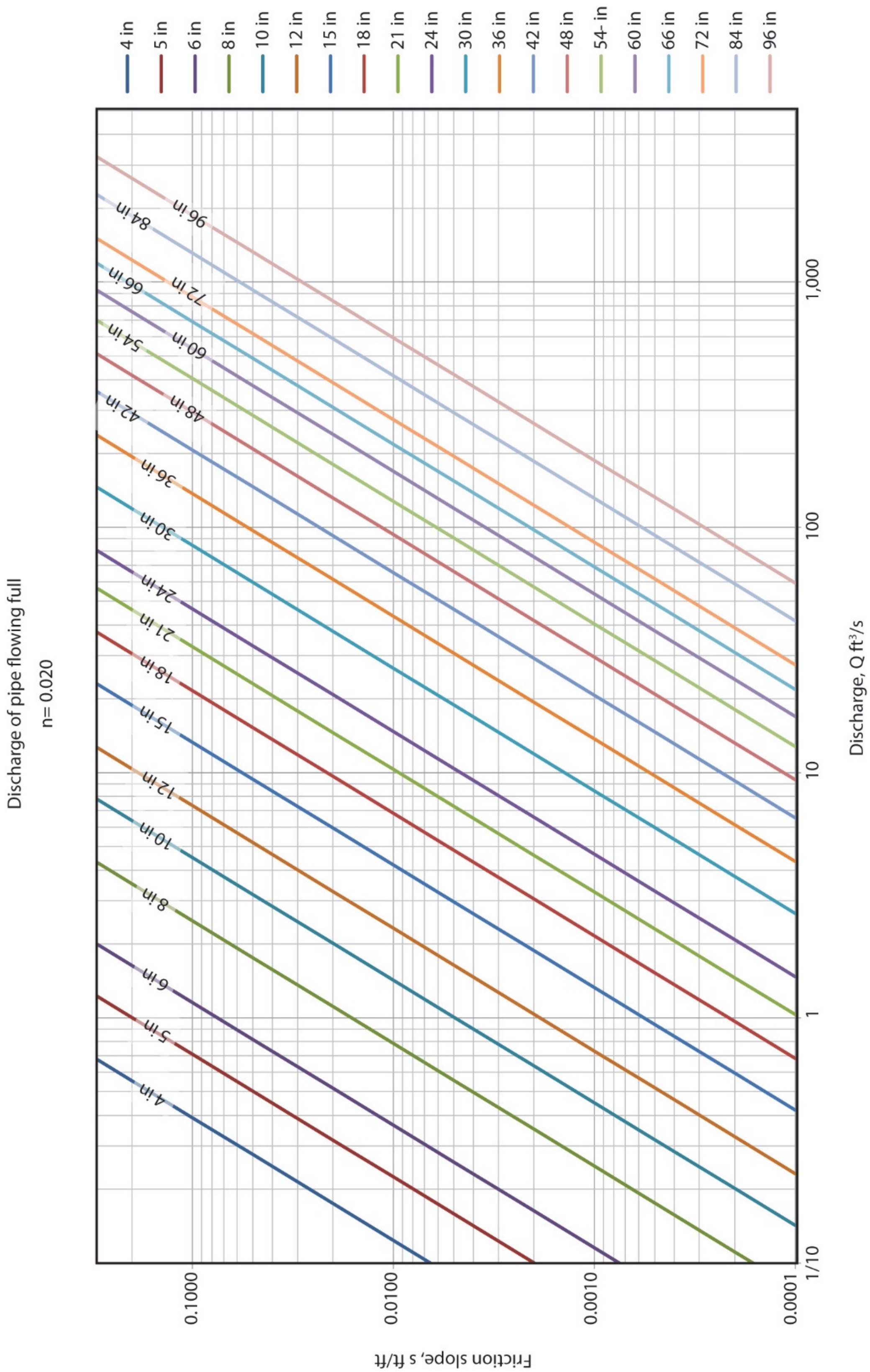
I. Discharge of Pipe Flowing Full n = 0.013



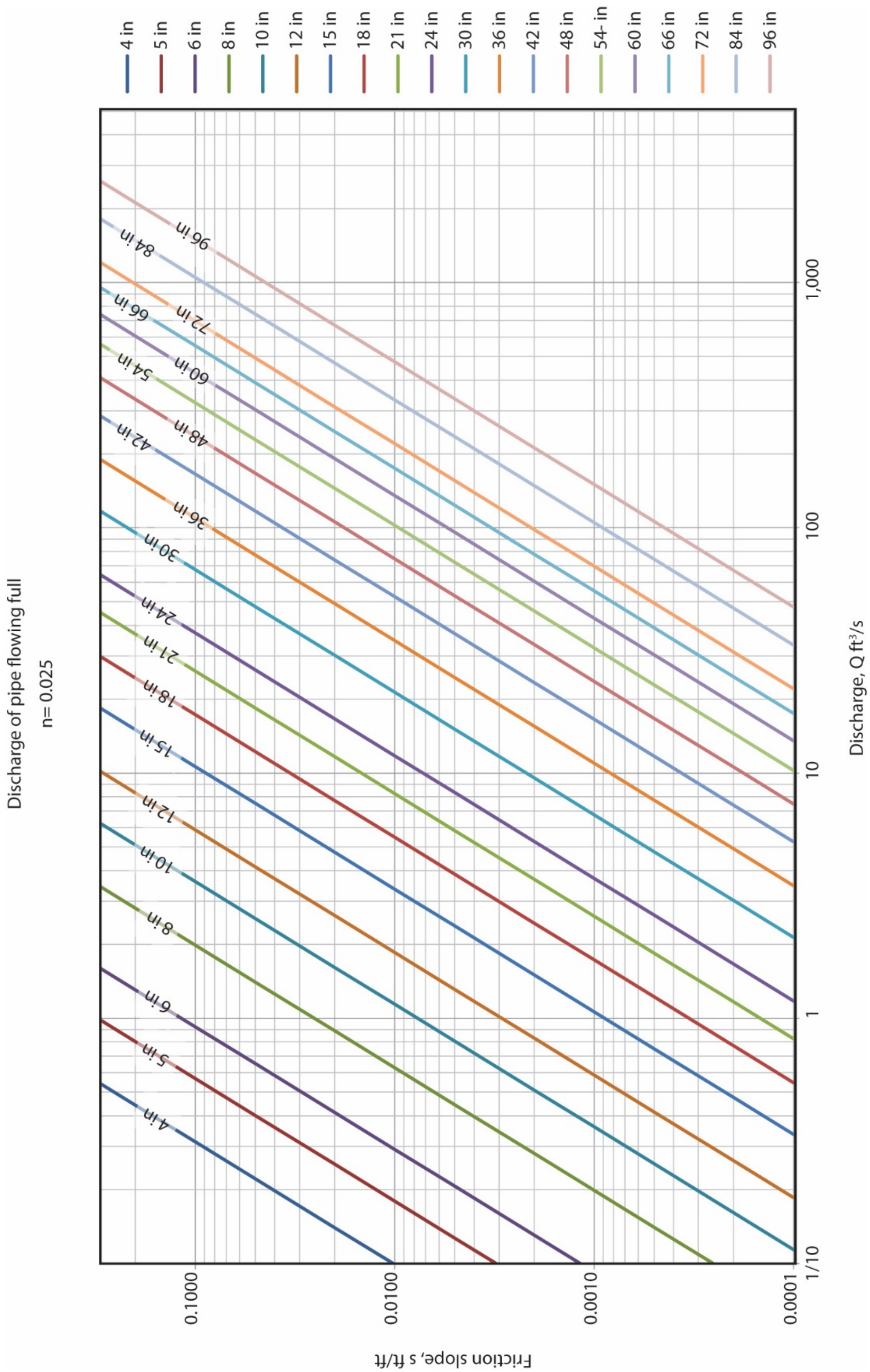
J. Discharge of Pipe Flowing Full n = 0.015



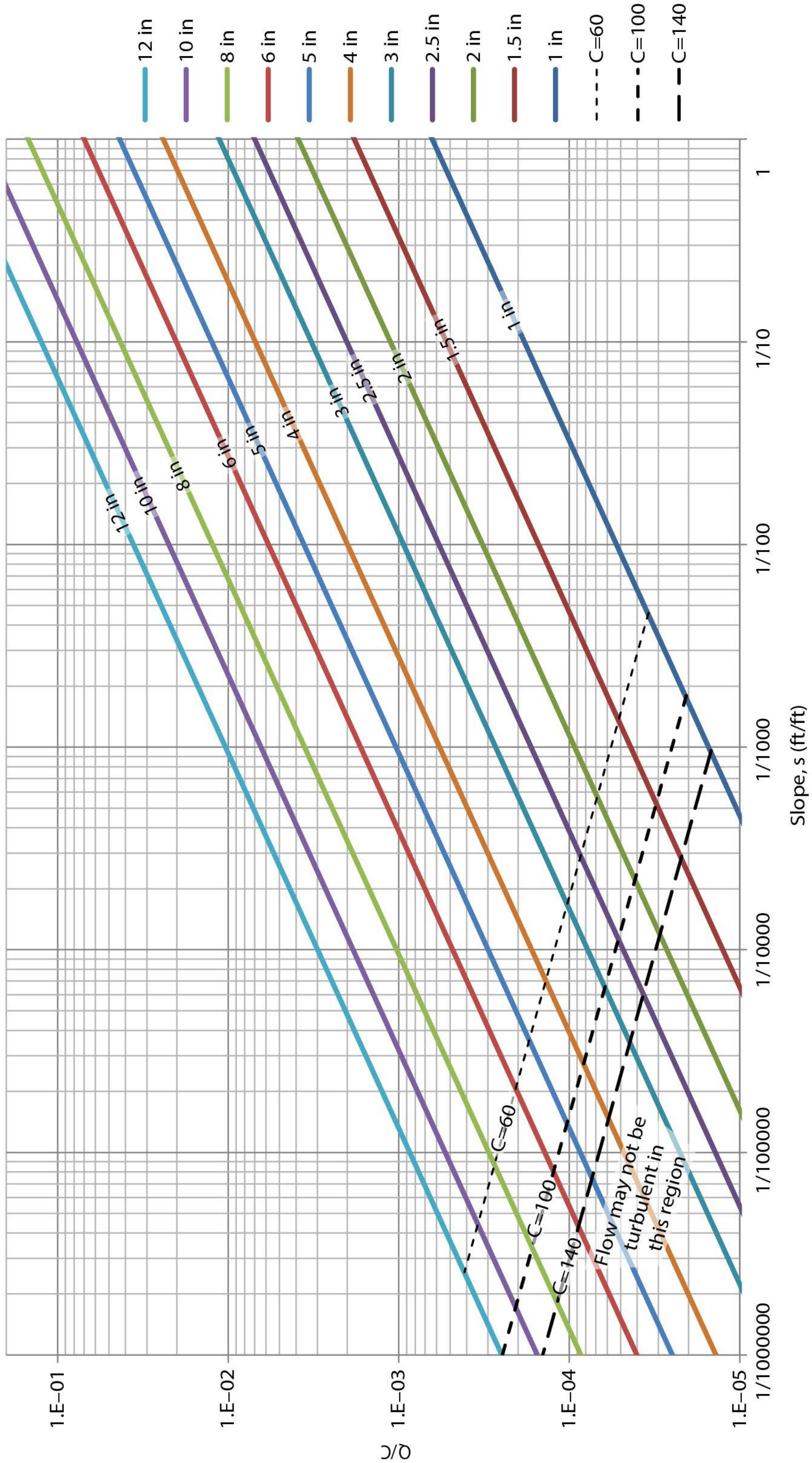
K. Discharge of Pipe Flowing Full n = 0.020



L. Discharge of Pipe Flowing Full n = 0.025



M. Hazen Williams for Round Pipes



N. Hazen-Williams Head Loss for 100 ft. of 1-4 in SDR 26 PVC Pipe C = 150

$$s_{ft/100\,ft} = \left[\left(\frac{Q_{gal/min}}{42.23 \times \left(d_1{}^{2.63}\right)}\right)^{\left(\frac{1}{0.54}\right)}\right] \times 100 \quad \textit{for PVC pipe, C=150}$$

Q (gal/ min)	Friction head loss for 100 ft SDR 26 PVC pipe, Hazen-Williams C = 150						
	1 1.195 in	1 1/4 1.532 in	1 1/2 1.754 in	2 2.193 in	2 1/2 2.655 in	3 3.230 in	4 4.154 in
2	0.15	0.04	0.02	0.01			
4	0.53	0.16	0.08	0.03	0.01		
6	1.13	0.34	0.17	0.06	0.02	0.01	0.00
8	1.93	0.58	0.30	0.10	0.04	0.02	0.00
10	2.92	0.87	0.45	0.15	0.06	0.02	0.01
15	6.18	1.84	0.95	0.32	0.13	0.05	0.01
20	10.52	3.14	1.62	0.55	0.22	0.08	0.02
25	15.91	4.74	2.45	0.83	0.33	0.13	0.04
30	22.30	6.65	3.44	1.16	0.46	0.18	0.05
35		8.85	4.58	1.54	0.61	0.23	0.07
40		11.33	5.86	1.97	0.78	0.30	0.09
45		14.09	7.29	2.46	0.97	0.37	0.11
50		17.12	8.86	2.98	1.18	0.45	0.13
60		24.00	12.42	4.18	1.65	0.63	0.19
70			16.52	5.57	2.19	0.84	0.25
80			21.15	7.13	2.81	1.08	0.32
90				8.86	3.49	1.34	0.39
100				10.77	4.25	1.63	0.48
120				15.10	5.95	2.29	0.67
140				20.09	7.92	3.05	0.89
160					10.14	3.90	1.15
180					12.61	4.85	1.43
200					15.33	5.90	1.73
250					23.17	8.92	2.62
300						12.50	3.67
350						16.63	4.88
400						21.29	6.25
450							7.78
500							9.45
550							11.28
600							13.25
700							17.63
800							22.57

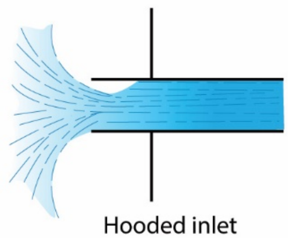
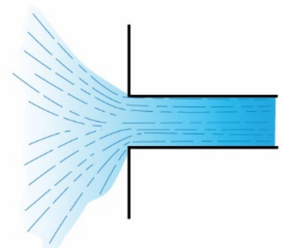
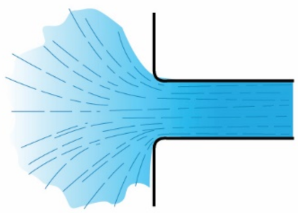
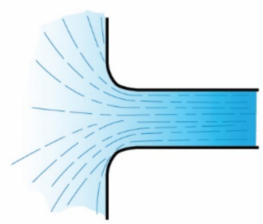
O. Hazen-Williams Head Loss for 100 ft. of 5-12 in SDR 26 PVC Pipe C = 150

$$s_{ft/100\,ft} = \left[\left(\frac{Q_{gal/min}}{42.23\times\left(d_1{}^{2.63}\right)}\right)^{\left(\frac{1}{0.54}\right)}\right] \times 100 \;\; \textit{for PVC pipe, C=150}$$

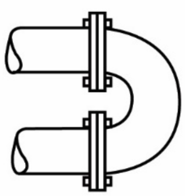
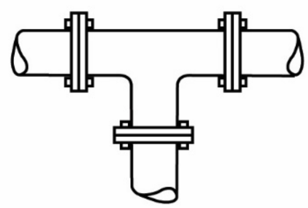
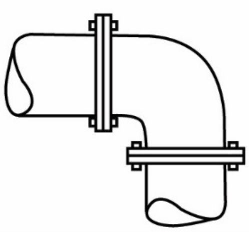
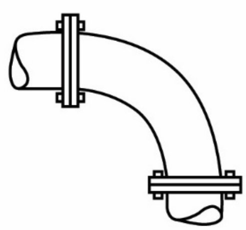
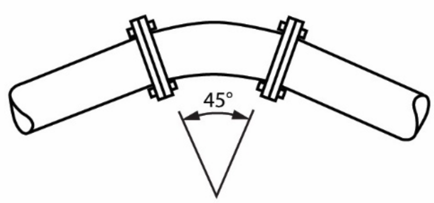
Q (gal/ min)	Friction head loss for 100 ft SDR 26+ PVC pipe, Hazen-Williams C=150				
	5 5.135 in	6 6.115 in	8 7.961 in	10 9.924 in	12 11.770 in
100	0.17	0.07	0.02	0.01	
120	0.24	0.10	0.03	0.01	
140	0.32	0.14	0.04	0.01	0.01
160	0.41	0.17	0.05	0.02	0.01
180	0.51	0.22	0.06	0.02	0.01
200	0.62	0.26	0.07	0.02	0.01
250	0.93	0.40	0.11	0.04	0.02
300	1.31	0.56	0.15	0.05	0.02
350	1.74	0.74	0.21	0.07	0.03
400	2.23	0.95	0.26	0.09	0.04
450	2.77	1.18	0.33	0.11	0.05
500	3.37	1.44	0.40	0.14	0.06
550	4.02	1.72	0.47	0.16	0.07
600	4.72	2.02	0.56	0.19	0.08
700	6.28	2.68	0.74	0.25	0.11
800	8.04	3.43	0.95	0.32	0.14
900	10.00	4.27	1.18	0.40	0.18
1,000	12.15	5.19	1.44	0.49	0.21
1,100	14.50	6.19	1.71	0.59	0.26
1,200	17.03	7.27	2.01	0.69	0.30
1,300	19.75	8.44	2.33	0.80	0.35
1,400	22.66	9.68	2.68	0.92	0.40
1,500		11.00	3.04	1.04	0.45
1,600		12.39	3.43	1.17	0.51
1,700		13.86	3.84	1.31	0.57
1,800		15.41	4.26	1.46	0.64
1,900		17.04	4.71	1.61	0.70
2,000		18.73	5.18	1.77	0.77
2,500			7.84	2.68	1.17
3,000			10.98	3.75	1.64
3,500			14.61	4.99	2.18
4,000			18.71	6.40	2.79
5,000				9.67	4.21
6,000				13.55	5.90

P. Head Loss Coefficients for Pipe Entrances and Bends

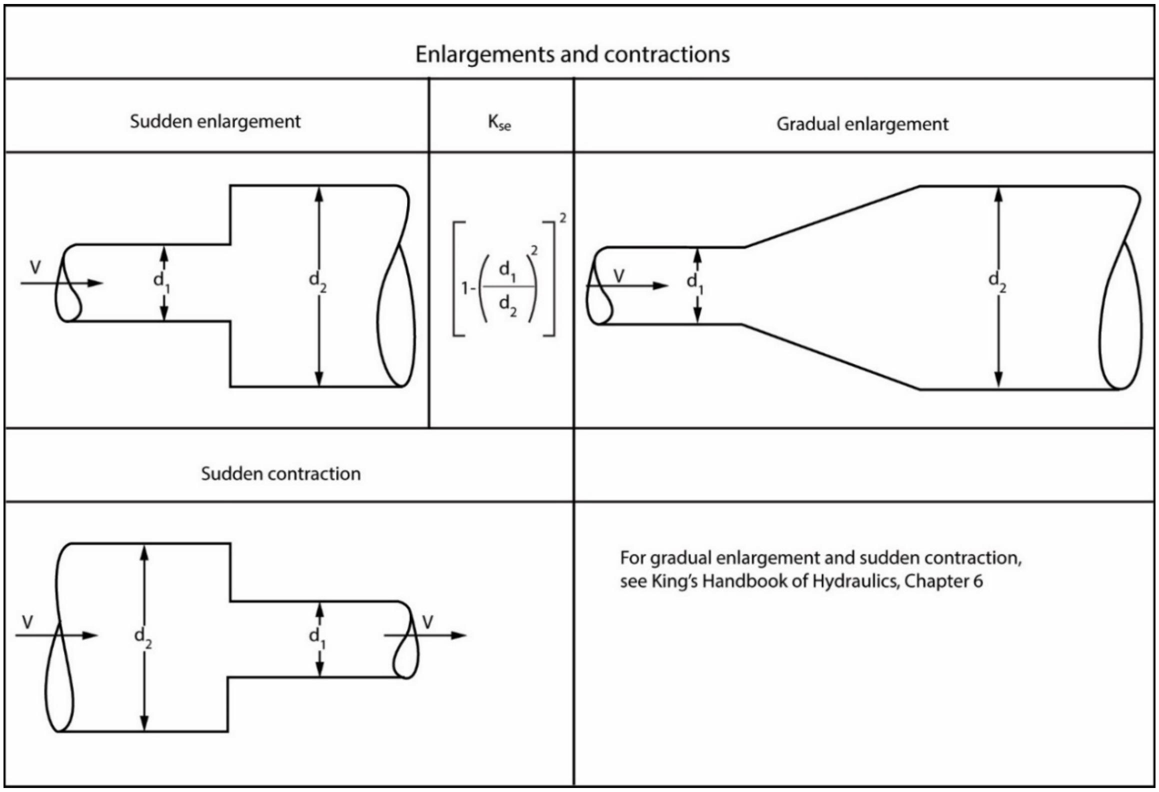
Pipe entrances

Pipe entrance		K_e
Inward projecting pipe		0.78
Hooded inlet		1.00
Sharp-cornered		0.50
Slightly rounded		0.23
Bell mouth		0.04

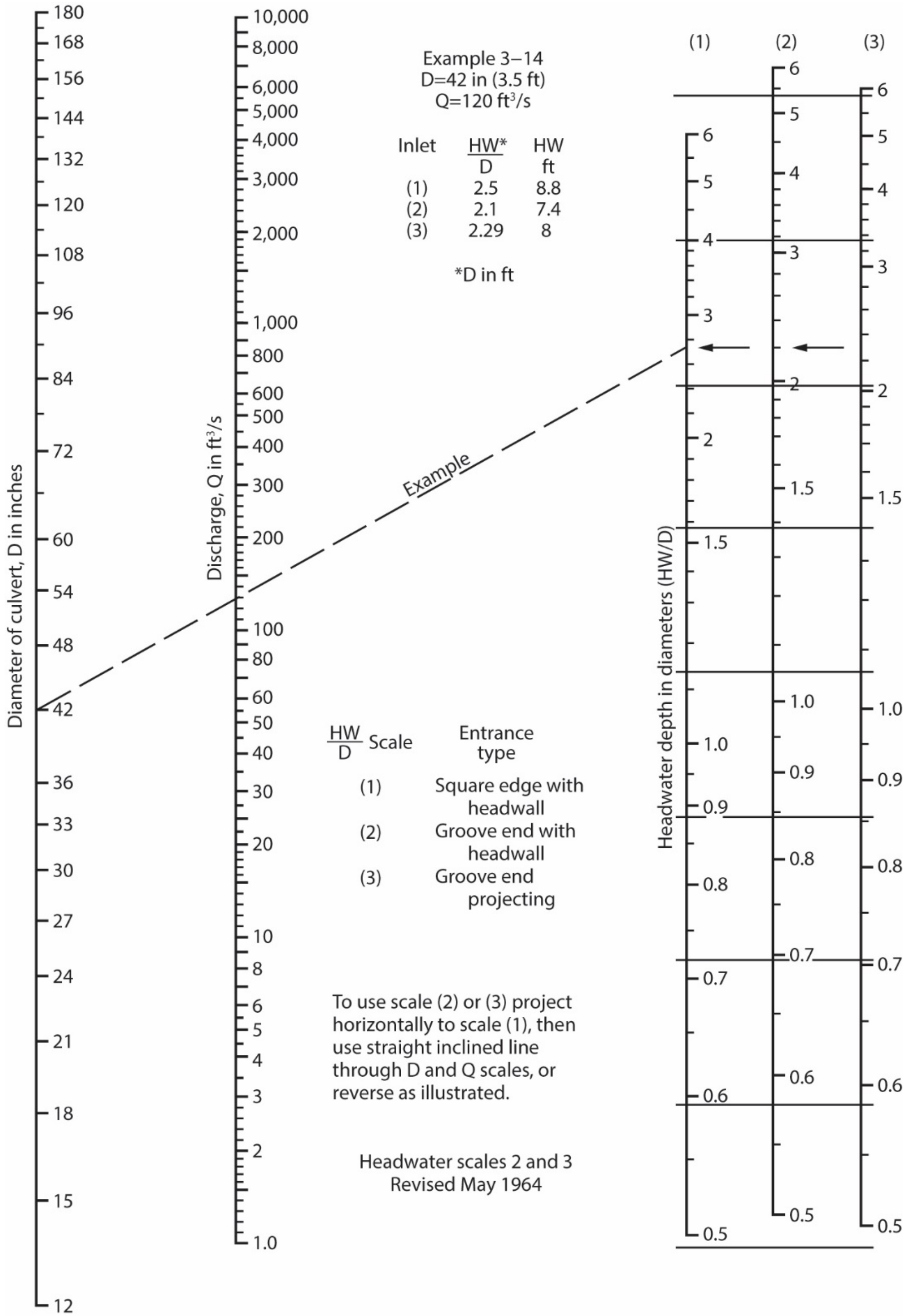
Pipe bends

Pipe bend		Coefficient	Value
Return bend		K_{RB}	2.20
Standard tee		K_{ST}	1.80
Standard 90° elbow		K_{90}	0.90
Long radius elbow		K_{LR}	0.60
45° Elbow		K_{45}	0.42

Q. Headwater Depth for Enlargements and Contractions

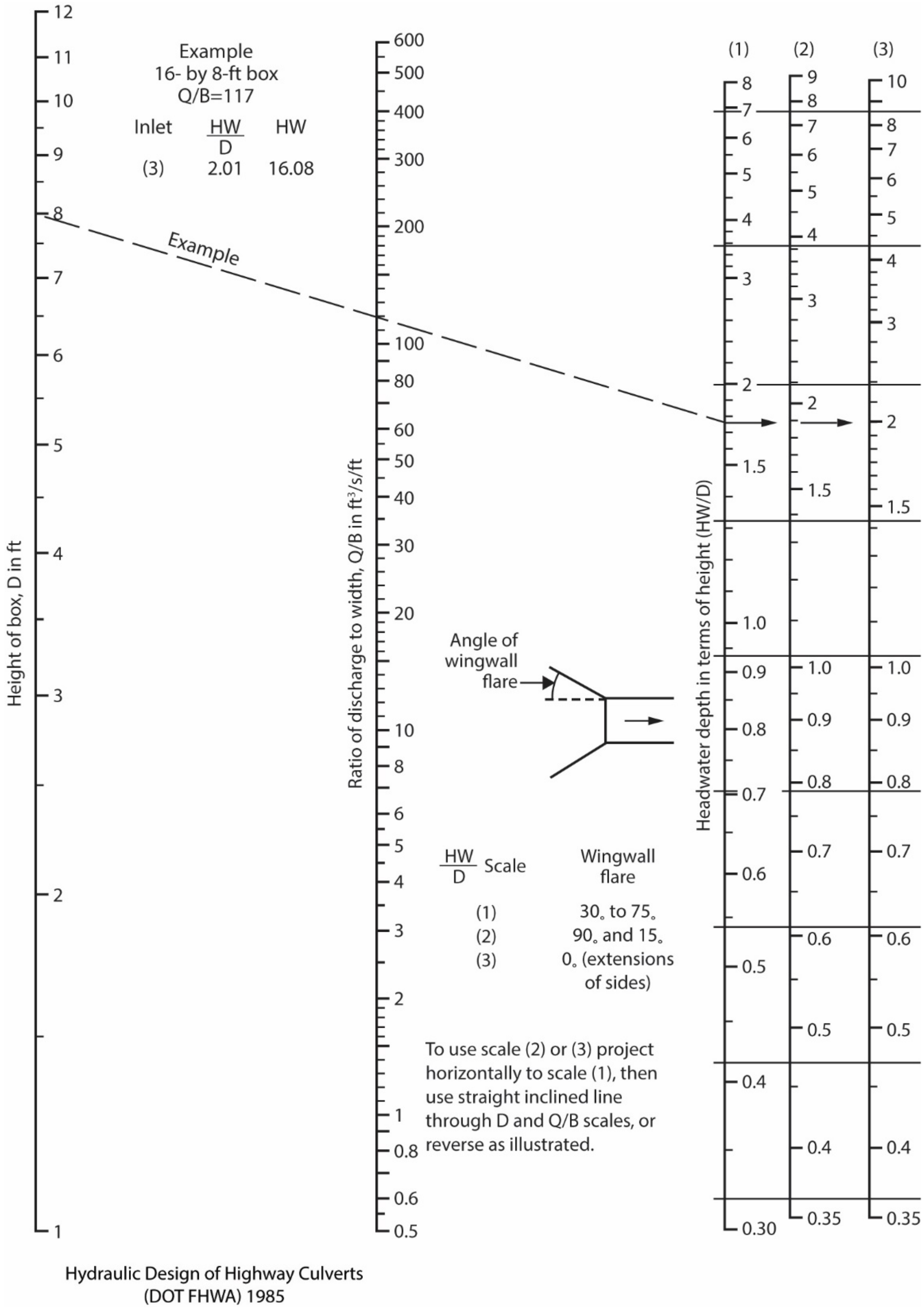


R. Headwater Depth for Concrete Pipe Culverts with Inlet Control

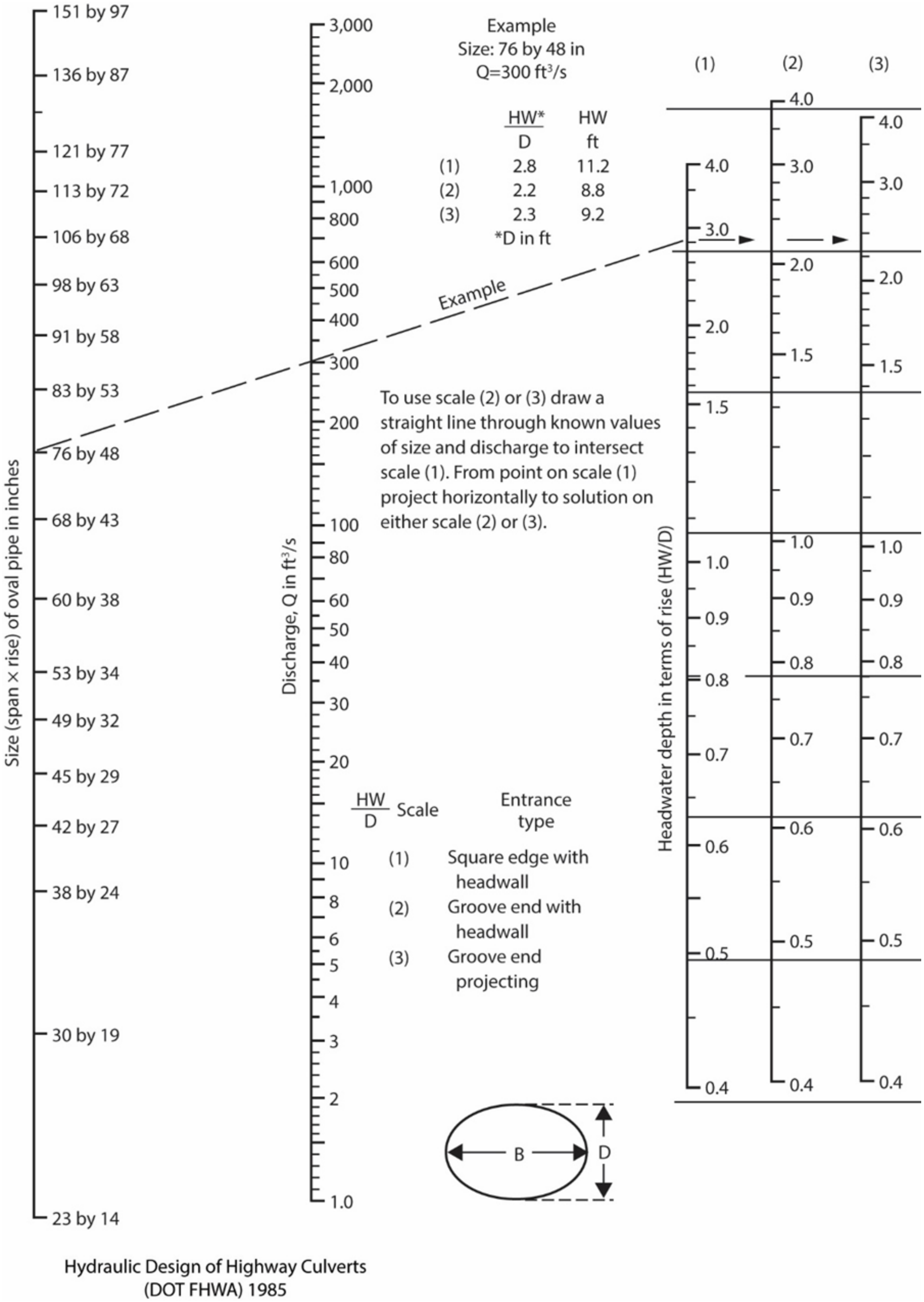


Hydraulic Design of Highway Culverts
(DOT FHWA) 1985

S. Headwater Depth for Box Culverts with Inlet Control

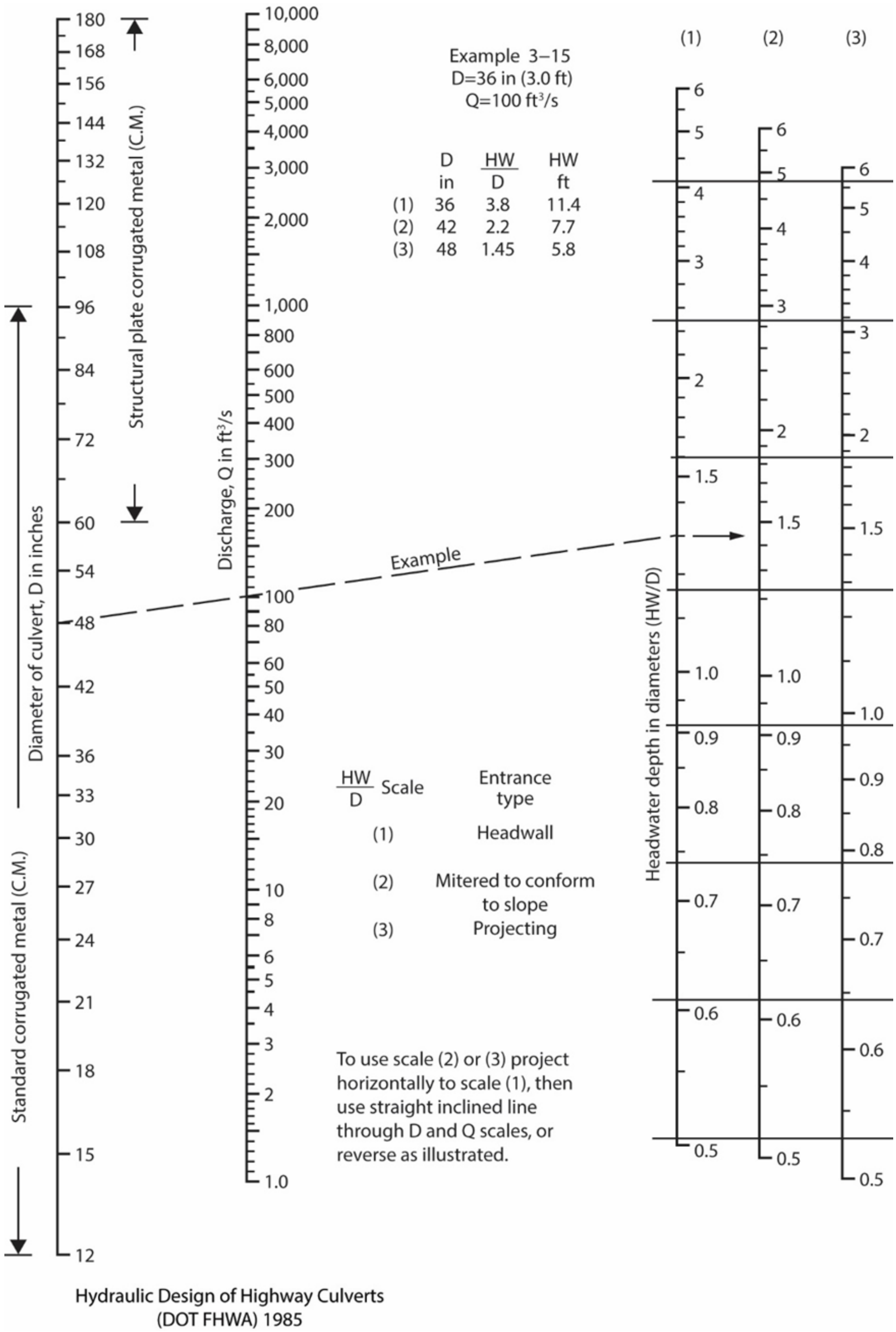


T. Headwater Depth for Oval Concrete Pipe Culverts Long Axis Horizontal with Inlet Control

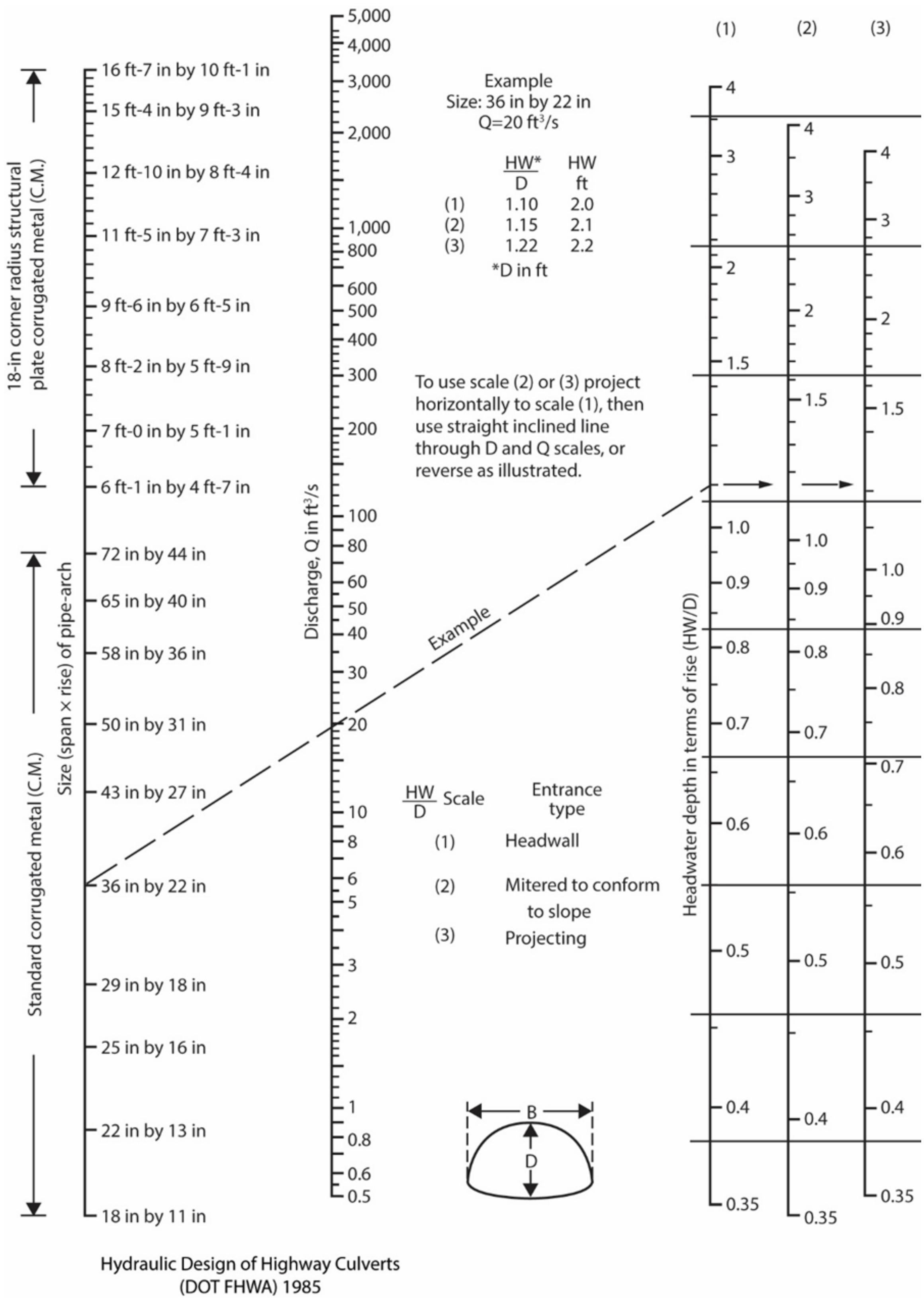


Hydraulic Design of Highway Culverts
(DOT FHWA) 1985

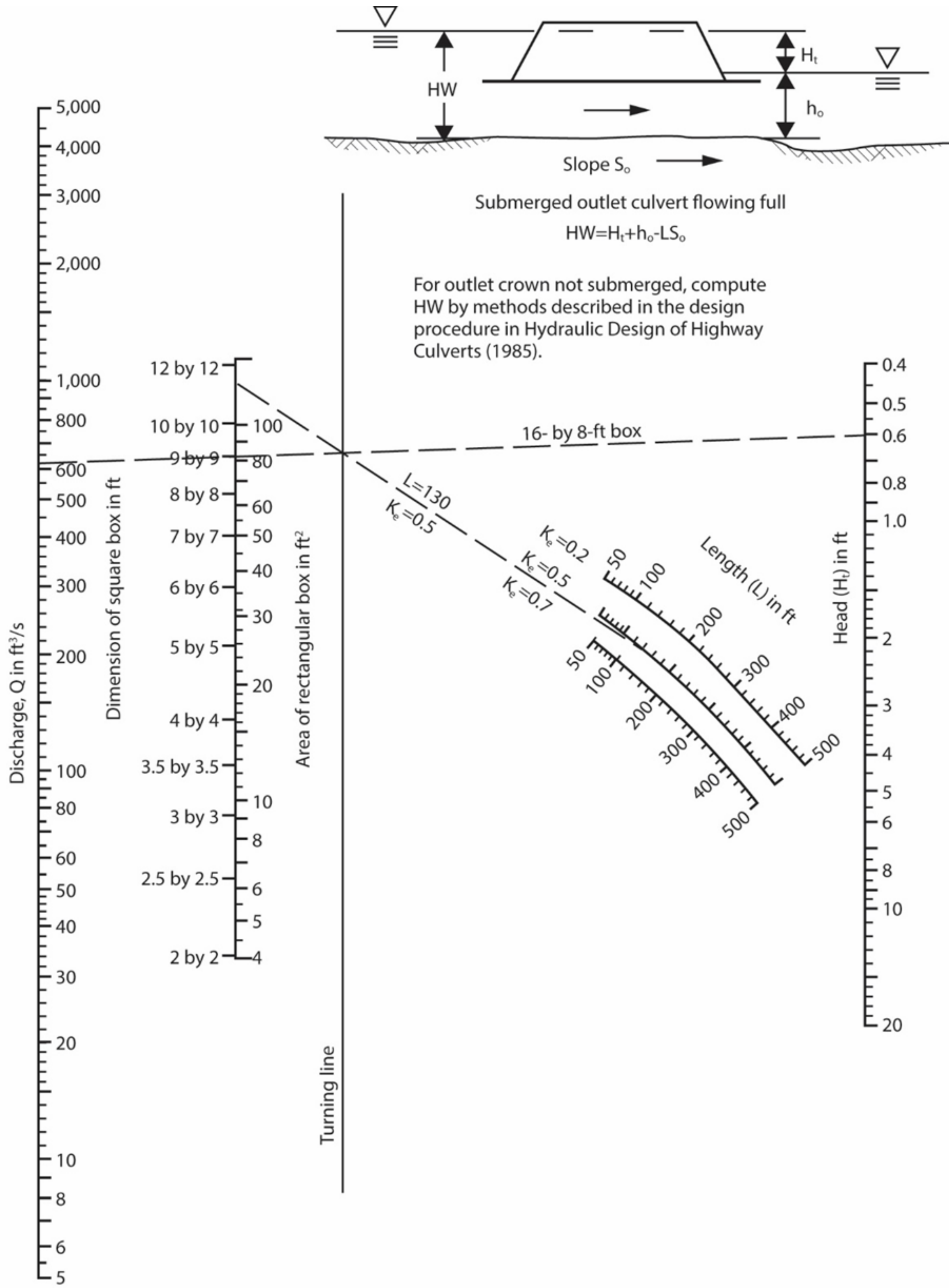
U. Headwater Depth for Corrugated Metal Culverts with Inlet Control



V. Headwater Depth for Corrugated Metal Pipe Arch Culverts with Inlet Control

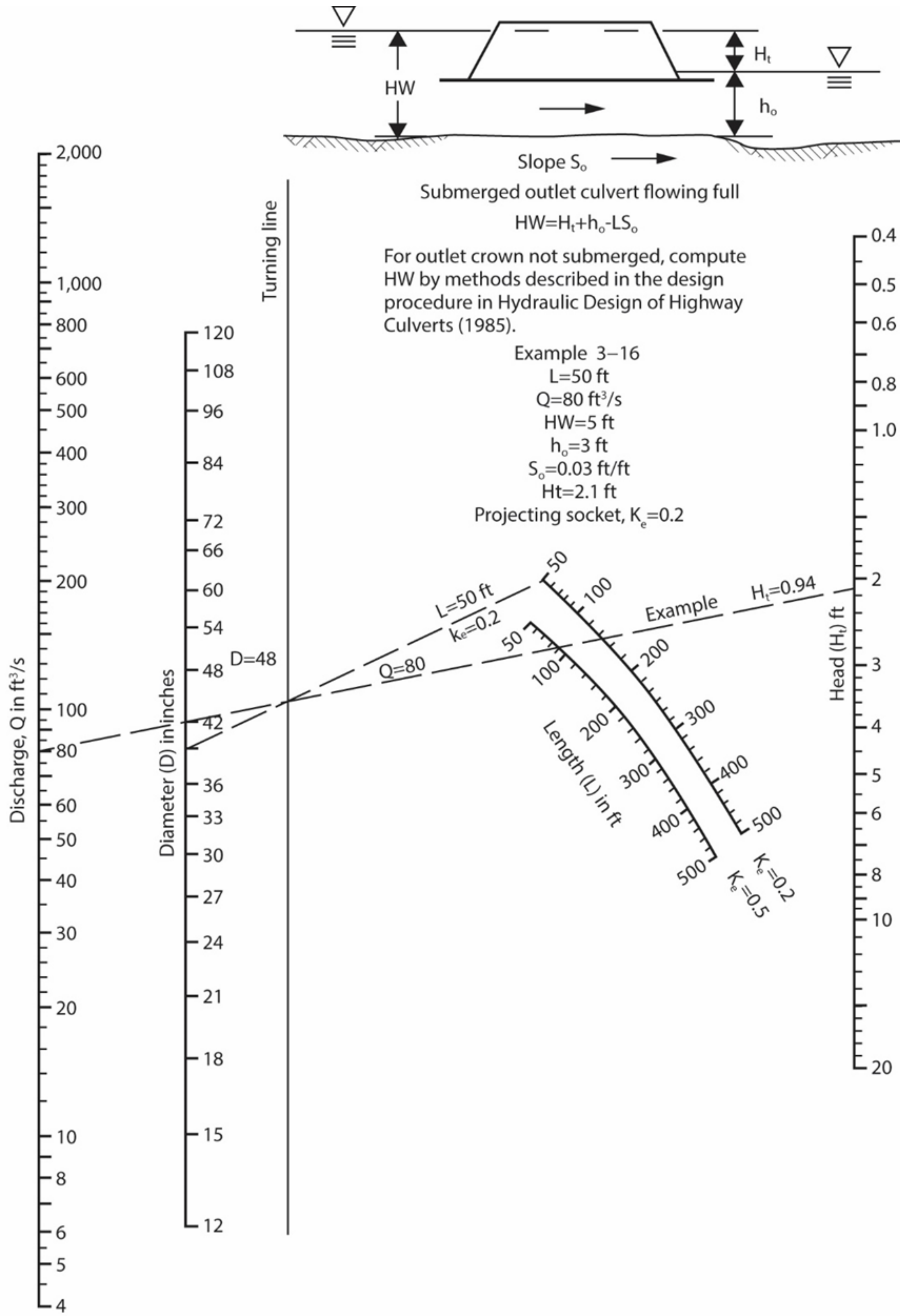


W. Head for Concrete Box Culverts Flowing Full, n = 0.12



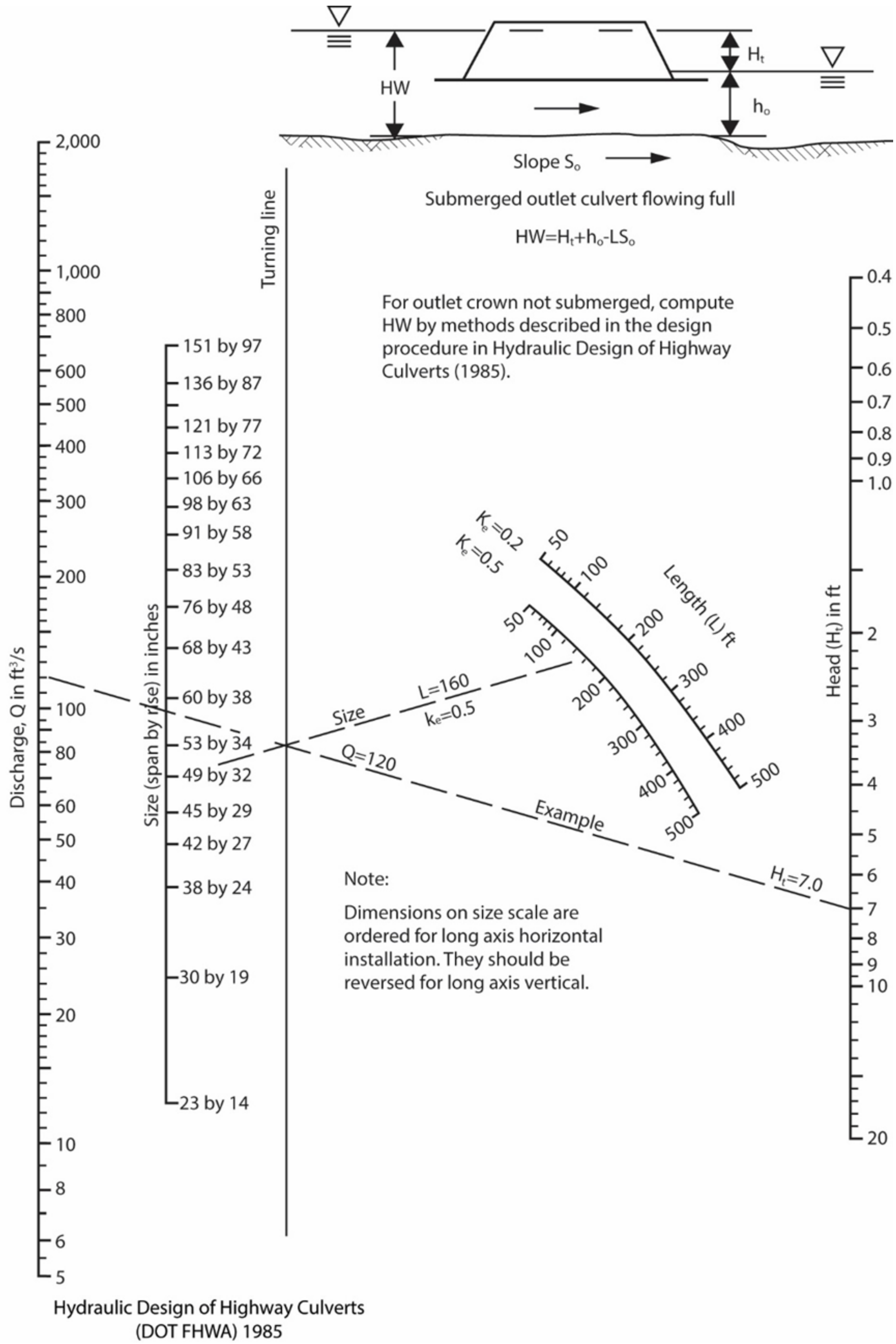
Hydraulic Design of Highway Culverts
(DOT FHWA) 1985

X. Head for Concrete Pipe Culverts Flowing Full, n = 0.12



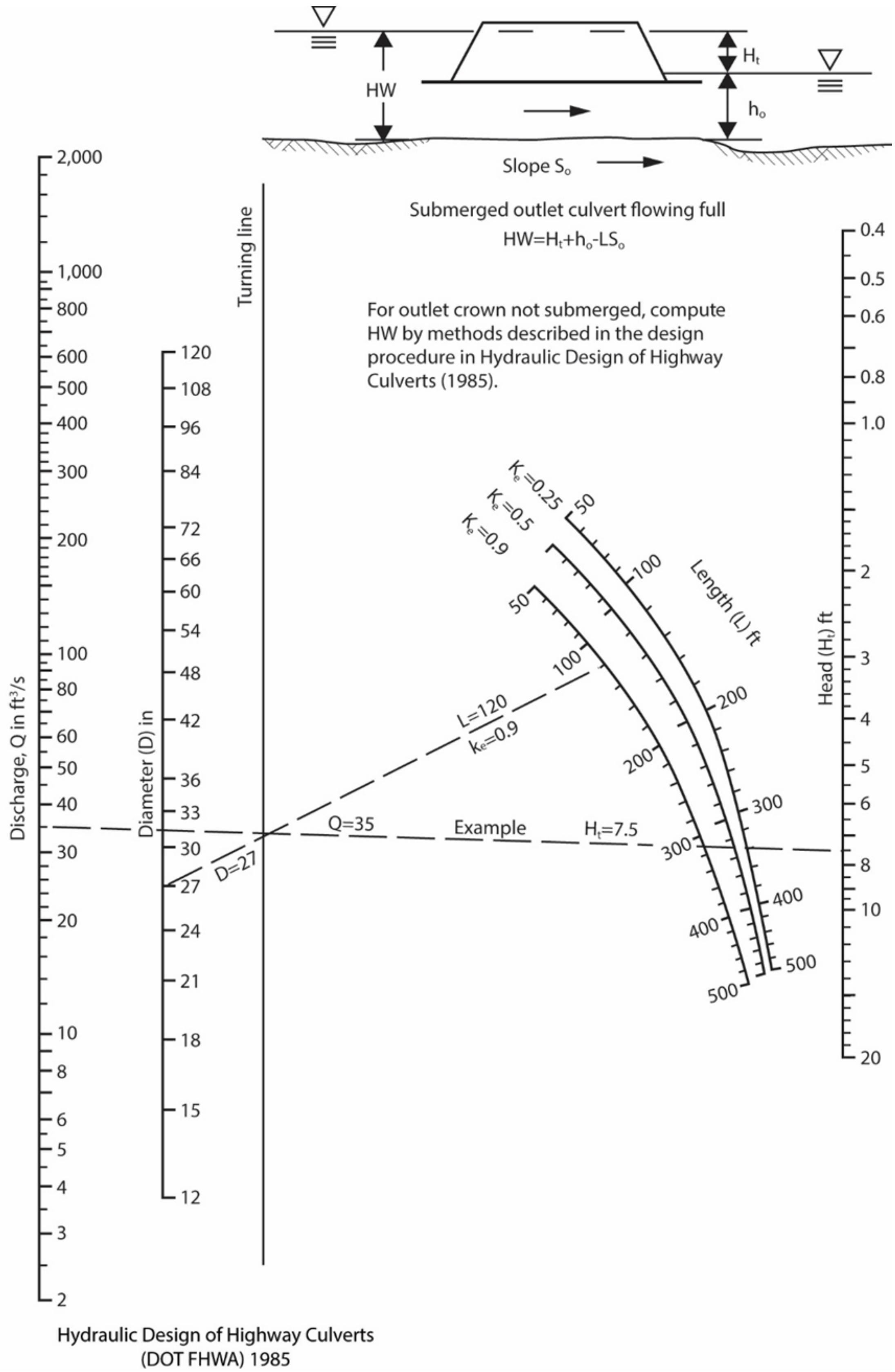
Hydraulic Design of Highway Culverts (DOT FHWA) 1985

Y. Head for Oval Concrete Pipe Culverts Long Axis Horizontal or Vertical Flowing Full, n = 0.12

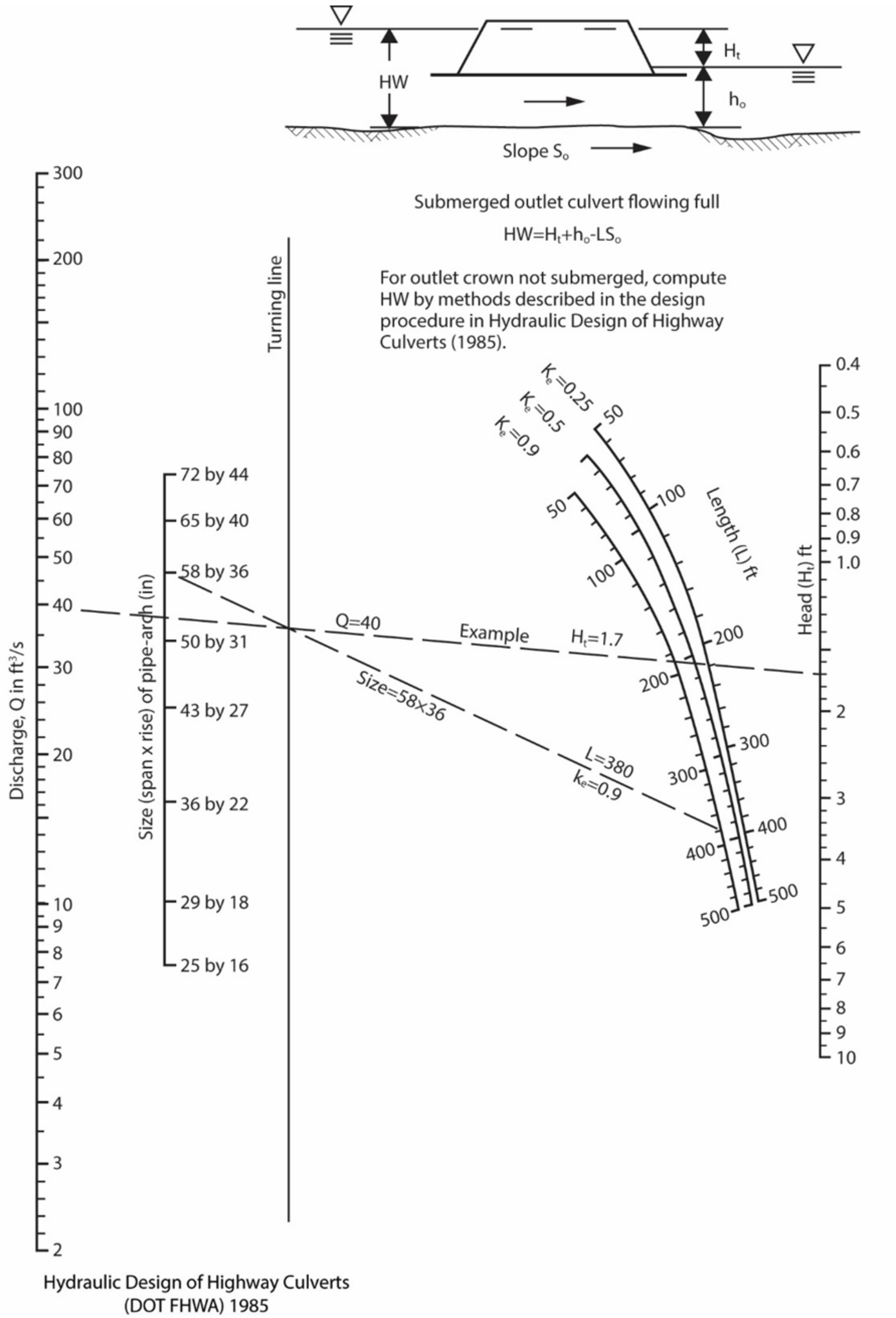


Hydraulic Design of Highway Culverts (DOT FHWA) 1985

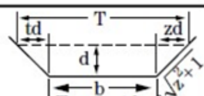
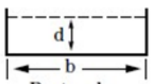
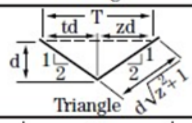
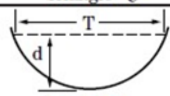
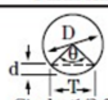
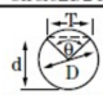
Z. Head for Standard Corrugated Metal Pipe Culverts Flowing Full, n = 0.024



AA. Head for Standard Corrugated Pipe Arch Culverts Flowing Full, n = 0.024



BB. Elements of Channel Sections

Section	Area a	Wetted perimeter P	Hydraulic radius r	Top width T
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+d^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2d\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$	$\frac{2dT^2}{3T^2+8d^2}$	$\frac{3d}{2d}$
 Circle≤1/2 full 2	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$
 Circle≥1/2 full 3	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$

1 Satisfactory approximation for the interval $0<d/T\leq 0.25$
When $d/T>0.25$, uses $p=1/2 \quad 1\sqrt{16d^2+T^2} + T^2/8d \sinh^{-1} 4d/T$

2 $\theta=4\sin^{-1}\sqrt{d/D}$

3 $\theta=4\cos^{-1}\sqrt{d/D}$

Insert θ in degrees in above equations

CC. Flow of Water from Vertical Pipes (a)

Jet height (in)	Pipe diameter							
	2-in std	3-in std	4.0-in OD well casing	4-in std	5.0-in OD well casing	5-in std	6-in OD well casing	6-in std
2.0	28	57	75	86	103	115	137	150
2.5	31	69	95	108	132	150	182	205
3.0	34	78	112	128	160	183	225	250
3.5	37	86	124	145	183	210	262	293
4.0	40	92	135	160	205	235	295	330
4.5	42	98	144	173	225	257	320	365
5	45	104	154	184	240	275	345	395
6	50	115	169	205	266	306	385	445
7	54	125	186	223	293	336	420	485
8	58	134	202	239	315	360	450	520
9	62	143	215	254	335	383	480	550
10	66	152	227	268	356	405	510	585
12	72	167	255	295	390	450	565	650
14	78	182	275	320	420	485	610	705
16	83	195	295	345	455	520	655	755
18	89	208	315	367	480	555	700	800
20	94	220	333	386	510	590	740	850
25	107	248	377	440	580	665	830	960
30	117	275	420	485	640	740	925	1,050
35	127	300	455	525	695	800	1,000	1,150
40	137	320	490	565	745	865	1,075	1,230

Discharge curves in Utah Engineering Experiment Station Bulletin 5, Measurement of Irrigation Water, June 1955

Jet height (in)	Pipe diameter					
	8-in OD well casing	8-in std	10-in OD well casing	10-in std	12-in OD well casing	12-in std
2.0	200	215	265	285	330	355
2.5	275	290	357	385	450	480
3.0	340	367	450	490	570	610
3.5	405	440	555	610	705	755
4.0	465	510	660	725	845	910
4.5	520	570	760	845	980	1,060
5	575	630	840	940	1,120	1,200
6	670	730	1,000	1,125	1,370	1,500
7	750	820	1,150	1,275	1,600	1,730
8	810	890	1,270	1,420	1,775	1,950
9	870	955	1,360	1,550	1,930	2,140
10	925	1,015	1,450	1,650	2,070	2,280
12	1,010	1,120	1,600	1,830	2,300	2,550
14	1,100	1,220	1,730	2,000	2,530	2,800
16	1,180	1,300	1,870	2,140	2,720	3,000
18	1,265	1,400	2,000	2,280	2,900	
20	1,335	1,480	2,100	2,420		
25	1,520	1,670	2,380	2,720		
30	1,690	1,870	2,650	3,000		
35	1,820	2,020	2,850			
40	1,970	2,160				

Discharge curves in Utah Engineering Experiment Station Bulletin 5, Measurement of Irrigation Water, June 1955

DD. Flow of Water from Horizontal Pipes – X=0 in

Flow of water from horizontal pipes (gal/min) X = 0 inch					
Y(in)	Pipe diameter				
	2 in	3 in	4 in	5 in	6 in
0.20		67.7	180	308	
0.30		66.5	175	303	530
0.40		65.1	171	298	518
0.50		63.6	166	293	506
0.60	18.3	62.0	161	287	494
0.70	17.6	60.4	156	282	482
0.80	16.7	58.4	150	277	470
0.90	15.4	55.7	145	271	458
1.00	13.7	53.1	139	265	446
1.20	9.5	46.9	128	251	422
1.40	6.0	40.5	115	237	398
1.60		31.9	102	221	373
1.80		24.0	90	205	347
2.00		17.3	77	187	321
2.20		11.8	64	167	295
2.40		7.3	52	147	270
2.60			41	127	246
2.80			32	108	223
3.00			24	90	200
3.30			13	65	167
3.60				45	137
3.90				29	111
4.20					86
4.50					64
4.80					45

Purdue Engineering Experiment Bulletin 32, Measurement of pipe flow by the coordinate method, August 1928.

EE. Flow of Water from Horizontal Pipes – X=6 in

Flow of water from horizontal pipes (gal./min), X = 6 inches					
Y (in)	Pipe diameter				
	2 in	3 in	4 in	5 in	6 in
0.24	177	310	548		
0.36	146	274	503	969	1,243
0.48	126	247	462	857	1,113
0.60	111	229	435	772	1,019
0.72	100	215	404	705	947
0.84	92	202	377	646	889
0.96	85	193	355	606	844
1.08	79	184	337	574	808
1.20	75	175	319	543	772
1.80	60	139	265	449	660
2.40	51	119	229	390	583
3.00	45	105	206	350	525
3.60	40	94	188	314	476
4.20	37	86	169	278	431
4.80	35	79	151	238	386
5.40	32	71	133	193	332
6.00	30	63	116	150	247
6.60	27	50	99	112	
7.20	25	38	83		
7.80	23	29	69		
8.40	20				

Purdue Engineering Experiment Bulletin 32, Measurement of pipe flow by the coordinate method, August 1928.

FF. Flow of Water from Horizontal Pipes – X=12 in

Flow of water from horizontal pipes (gal/min), X = 12 inches					
Y (in)	Pipe diameter				
	2-in	3-in	4-in	5-in	6-in
0.96	157	319	570	1,014	
1.08	148	305	548	974	1,315
1.20	139	292	530	925	1,257
1.80	114	247	444	763	1,055
2.40	99	215	395	655	929
3.00	87	193	359	583	844
3.60	79	176	332	530	772
4.20	73	161	305	489	718
4.80	68	149	287	458	673
5.40	63	140	269	426	633
6.00	60	132	256	404	597
6.60	57	126	242	386	574
7.20	54	120	233	368	548
7.80	52	114	224	355	525

GG. Flow of Water from Horizontal Pipes – X=18 in

Flow of water from horizontal pipes (gal/min), X = 18 inches					
Y (in)	Pipe diameter				
	2 in	3 in	4 in	5 in	6 in
1.80	166	346	624	1,014	1,400
2.40	144	305	557	907	1,261
3.00	129	274	503	826	1,153
3.60	117	251	462	754	1,068
4.20	109	233	431	700	992
4.80	101	220	404	655	934
5.40	95	206	382	615	884
6.00	89	197	364	579	839
6.60	84	187	346	548	799
7.20	81	180	332	521	763
7.80	77	172	319	498	732
8.40	75	166	305	476	705