# CHAPTER 3. HYDRAULICS

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# CHAPTER 3. HYDRAULICS

## 1. GENERAL

This chapter presents the hydraulic principles that apply to the design and operation of soil and water conservation measures. It will help the technician to develop a better understanding of hydraulics and to use the equations and exhibits contained herein.

The chapter contains sections on conversion of units, principles of water at rest (hydrostatics), and principles of water in motion (hydrokinetics). It discusses the application of these principles to the flow of water in pipes, in open channels, and through weirs. Lastly, the more common methods of measuring the flow of water in open channels and pipes are covered.

## 2. CONVERSION OF UNITS

Valid equations must be expressed in corresponding units. That is, in a true equation there must be equality between both units and numbers. The chance of making conversion errors can be greatly reduced by forming the habit of thinking in terms of equality of units as well as their relative numerical values.

The foot-pound-second system is used in this chapter unless otherwise specified. Sometimes, however, it is necessary to convert to other units, which involves the use of numerical conversion constants. Some frequently used constants are given in Exhibit 3-1.

EXAMPLE 3-1

It is desired to build a stock water tank with a capacity in cubic feet that will contain one day's flow from a spring that flows at the rate of three gallons per minute.

3 gallons per minute day = Y cubic feet

Since cubic feet can not be directly equated to gallons, some unit factor having numerical value must be introduced if the expression is to be made a valid equation.

The expression 3 gallons per minute per day is a fractional expression that can be written:

$$\frac{3\text{ gal.}}{\frac{\text{min.}}{\text{day}}} = \frac{3\text{ gal.}}{\frac{\text{min.}}{\text{day}}} = \frac{3\text{ gal.}}{\text{min.}} \times \frac{\text{day}}{1} = \frac{3\text{ gal.day}}{\text{min.}}$$

Analysis shows that:

$$\frac{3\ \cancel{\text{gal}}.\cancel{\text{day}}}{\cancel{\text{min}}.} \times \frac{60\ \cancel{\text{min}}.}{1\ \cancel{\text{hr}}.} \times \frac{24\ \cancel{\text{hr}}.}{1\ \cancel{\text{day}}} \times \frac{1\text{ cu.ft.}}{7.48\ \cancel{\text{gal}}.} = 577.5\text{ cu.ft.}$$

Note that all units on the left cancel except cubic feet, thus leaving the same units on each side of the equation. The result is the following general equation for conversions between gallons per minute per day and cubic feet. If 3 gal/min/day equals 577.5 cu.ft., one gal/min/day equals 192.51 cu.ft. Then:

$$X\text{ gpm day} \times \frac{192.51\text{ cu.ft.}}{1\text{ gpm day}} = Y\text{ cu.ft.}$$

EXAMPLE 3-2

1 acre foot per hour = Y gallons per minute

Step by step analysis results in a valid conversion equation consistent in both units and dimensions:

$$\frac{1\ \cancel{\text{ac}}.\cancel{\text{ft}}.}{\cancel{\text{hr}}.} \times \frac{43{,}560\ \cancel{\text{sq}}.\cancel{\text{ft}}.}{1\ \cancel{\text{ac}}.} \times \frac{1\ \cancel{\text{hr}}.}{60\text{ min.}} \times \frac{7.48\text{ gal.}}{1\ \cancel{\text{cu}}.\cancel{\text{ft}}.} = 5431\ \frac{\text{gal.}}{\text{min.}}$$

or

$$X\ \frac{\cancel{\text{ac}}.\cancel{\text{ft}}.}{\cancel{\text{hr}}.} \times \frac{\frac{5431\text{ gal.}}{\text{min.}}}{\frac{\cancel{\text{ac}}.\cancel{\text{ft}}.}{\cancel{\text{hr}}.}} = Y\ \frac{\text{gal.}}{\text{min.}}$$

EXAMPLE 3-3

1 cubic foot per second day = Y acre feet

Analysis results in:

$$\frac{1\ \cancel{\text{cu}}.\text{ft}.\cancel{\text{day}}}{\cancel{\text{sec}}.} \times \frac{1\text{ ac.}}{43{,}560\ \cancel{\text{sq}}.\cancel{\text{ft}}.} \times \frac{24\ \cancel{\text{hr}}.}{\cancel{\text{day}}} \times \frac{60\ \cancel{\text{min}}.}{\cancel{\text{hr}}.} \times \frac{60\ \cancel{\text{sec}}.}{\cancel{\text{min}}.} = 1.9835\text{ ac.ft.}$$

$$X\ \cancel{\text{cfs}}\ \cancel{\text{day}} \times \frac{1.9835\text{ ac.ft.}}{\cancel{\text{cfs}}\ \cancel{\text{day}}} = Y\text{ ac.ft.}$$

If this approach to conversion problems is used, the results will be:

1. Freedom from conversion errors.

2. Savings in time in both original and "check" computations.

3. Accuracy of conversion factor selection from standard tables and other sources.

## 3. HYDROSTATICS

The subject of hydrostatics (fluid statics) deals with problems in which the fluid is motionless or at rest.

### PRESSURE-DENSITY-HEIGHT RELATIONSHIPS

The fundamental equation of fluid statics relates pressure, density, and depth. Unit pressures in a fluid vary directly with the depth and the unit weight of the fluid and are expressed by the equation:

$$p = wh \quad \text{or} \quad h = \frac{p}{w} \qquad \text{(Eq. 3-1)}$$

where $p$ = intensity of pressure per unit of area
$w$ = unit weight of the fluid
$h$ = depth of submergence, or head.

Equation 3-1 shows that pressure at any point in a liquid of given density depends solely upon the height of the liquid above the point. This allows the vertical height, or "head," of the liquid to be used as an indication of pressure. Thus, pressure may be quoted in such units as "inches of mercury" and "feet of water." The relationship of pressure and head is illustrated numerically in the "manometer" and "piezometer columns" of Figure 3-1.

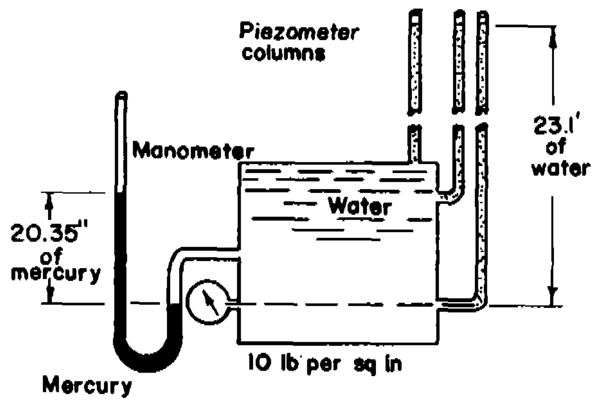



Figure 3-1 Relationship between pressure and head

If the tank is filled with water until the pressure gage reads 10 p.s.i., the height of the water surface in the piezometers and the mercury in the manometer can be calculated from Equation 3-1.

Example 3-4

Water:

$$h = \frac{p}{w} = \frac{\frac{10 \text{ lb.}}{\text{sq.in.}}}{\frac{62.4 \text{ lb.}}{\text{cu.ft.}}} \times \frac{144 \text{ sq.in.}}{1 \text{ sq.ft.}} = 23.1 \text{ feet}$$

Mercury (unit weight of 849 pounds per cubic foot):

$$h = \frac{\frac{10 \text{ lb.}}{\text{sq.in.}}}{\frac{849 \text{ lb.}}{\text{cu.ft.}}} \times \frac{144 \text{ sq.in.}}{1 \text{ sq.ft.}} \times \frac{12 \text{ in.}}{1 \text{ ft.}} = 20.35 \text{ inches}$$

Piezometer

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. The pressure at the wall of the pipe is: $p_1 = wh_1$ and that at the centerline $p = wh$.

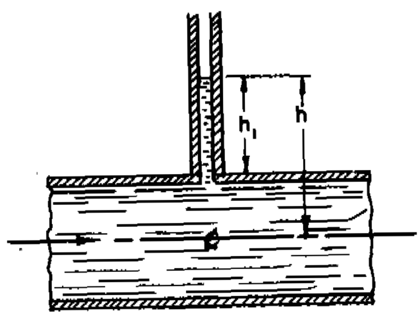



Figure 3-2 Piezometer tube in a pipeline

Piezometers are used to measure water pressure in drainage investigations and earth dam foundation studies. Such a piezometer is an unperforated small-diameter pipe, so designed and installed that after it has been driven into the soil the underground water cannot flow freely along the outside of the pipe and can enter it only at the bottom end. The piezometer is so driven that its lower end is in the stratum or at the level where the pressure is to be read. The height that water rises above the bottom of the pipe is the pressure head.

A piezometer should not be confused with an observation well which is used to determine the level of the water table. The well permits water to enter the hole at any level, thus connecting the various water bearing strata in the soil profile. The properly installed piezometer permits water to enter only at the bottom end and from only that level in the soil profile.

A typical example of the manner in which piezometers of different lengths may be used in sets to determine whether a canal is leaking is illustrated in Figure 3-3. In this example, sets of four piezometers 5, 10, 15 and 20 feet in length have been installed on a line at right angles to the axis of the canal at distances of 15, 60, and 100 feet from the centerline of the canal (Diagram A). The first objective is to obtain hydrostatic pressures at a large number of points under the water table adjacent to the canal.

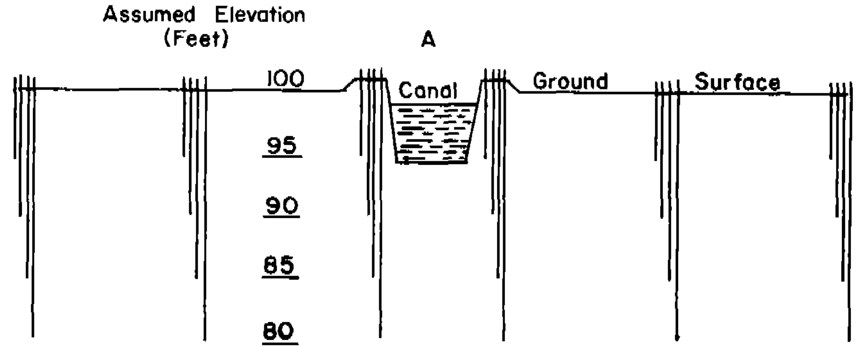
In Diagram B, which represents the same cross section shown in A, the small circles indicate the position of the bottom end of each piezometer. On the right-hand side of the sketch, the number beside each circle is the water level elevation in each piezometer. On the left side the numbers are the elevations of the bottom ends of the piezometers. Note that the 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 in the profile where the bottom of the piezometer is located, not where the water surface is located.

The third step is to draw contours of equal hydrostatic pressures, as in Diagram 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 and in a direction at right angles to the pressure contours. This example indicates seepage from the canal.

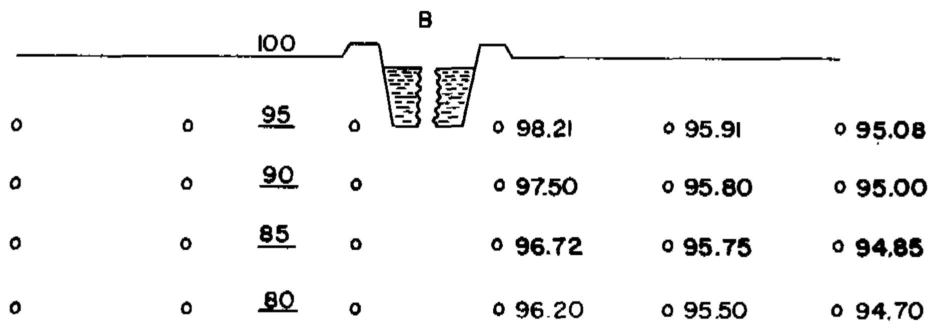
If an artesian pressure condition existed in the soil profile, the deep piezometers would show higher water surface elevations than the shorter piezometers, and the ground water contours would indicate upward water pressure.

## FORCES ON SUBMERGED PLANE SURFACES

The calculation of the size, direction, and location of the forces on submerged surfaces is essential in the design of dams, bulkheads, water control gates, etc.

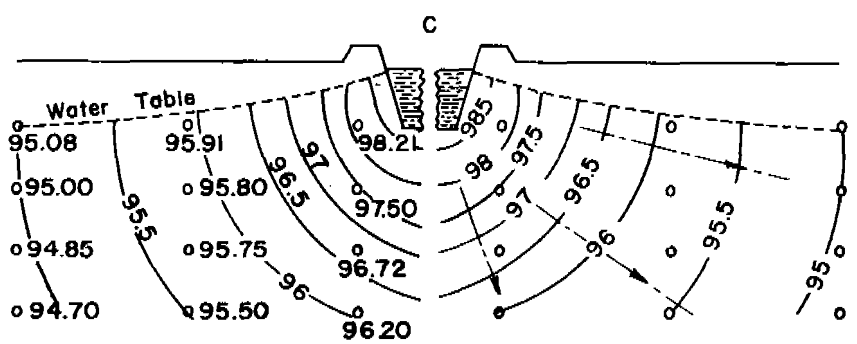



Six batteries of differential-length piezometers bracketing an irrigation canal and terminating at depths of 5, 10, 15, and 20 feet.




The left side is plotted to show the piezometer termination levels. The pressures in the piezometers are measured at these points.

The right side gives the water elevations in the piezometer tubes. These elevations are plotted at the piezometer termination points.




The left side shows lines of equal pressure partly drawn. Lines of equal pressure are drawn similar to ground-surface contours.

The right side shows the finished lines of equal pressure and the arrows indicate the direction of ground-water movement.

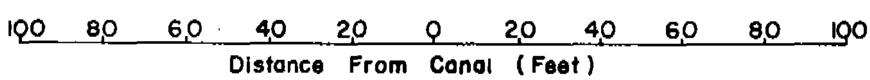



Figure 3-3 Plotting differential-length piezometer data in the determination of equipotential pressure lines

For a submerged horizontal plane, the calculation of unit and total pressures is simple because the pressure is uniform over the area. For vertical and inclined planes the pressure varies with depth, as shown by Equation 3-1, producing the typical pressure diagrams and the resultant forces of Figure 3-4.

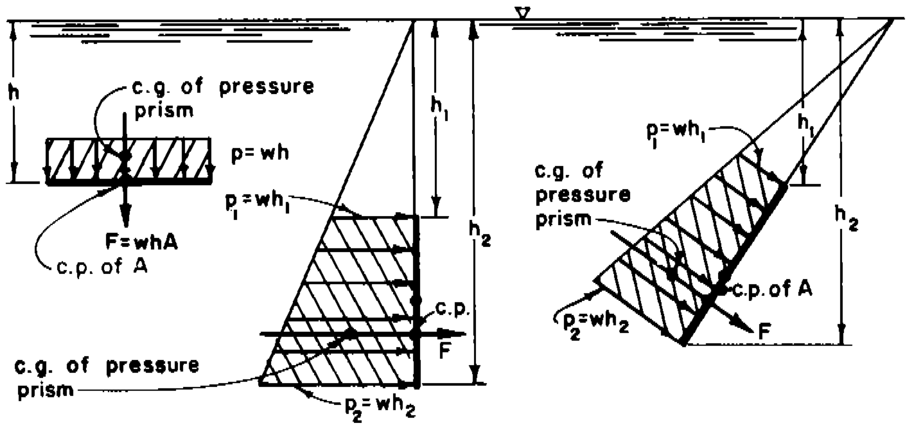



Figure 3-4 Pressure on submerged surfaces

The shaded area, when multiplied by a unit of length equals volume, which is known as the "pressure volume." The resultant force, F, is equal to the pressure volume and passes through its center of gravity (c.g.). The resultant force also passes through a point on the plane defined as the "center of pressure" (c.p.).

## Pressure Diagrams

The analysis of structures under pressure usually will be simplified by use of pressure diagrams. Since unit pressure varies directly with head, diagrams showing the variation of unit pressure in any plane take the form of triangles, trapezoids, or rectangles.

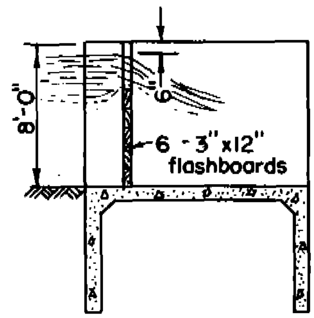
In solving problems of force due to water pressure, the magnitude, direction, and position of the force must be considered. The total force represented by the pressure diagram can, for some problems, be represented by a single force arrow through the pressure center acting in the same direction as the unit pressures.

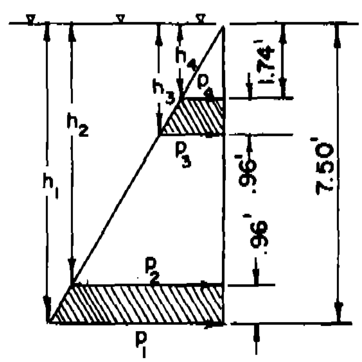
Exhibit 3-2 gives the most commonly used pressure diagrams and methods of computing the hydrostatic load and center of pressure.

Example 3-5

A flashboard type of dam is built with six 3 x 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 six feet long, and (c) the load per foot on the top board?

Solution: First draw the pressure diagram, remembering that a finished 12-inch board is 11.5-inches or 0.96-foot wide.




Then from Equation 3-1:

$$p = wh = \text{unit weight of water x depth of water}$$

$p_1 = 62.4\ (7.5) = 468.0$ lbs. per sq. ft.
$p_2 = 62.4\ (7.5 - 0.96) = 408.1$ lbs. per sq. ft.
$p_3 = 62.4\ (1.74 + 0.96) = 168.5$ lbs. per sq. ft.
$p_4 = 62.4\ (1.74) = 108.6$ lbs. per sq. ft.

And from Figure 3-4:

The hydrostatic load, F = whA = pA = unit pressure x area

then

$$(a) = \frac{468.0 + 408.1}{2} \times 0.96 = 420.5 \text{ lbs. per ft.}$$

$$(b) = 420.5\ (6) = 2523 \text{ lbs.}$$

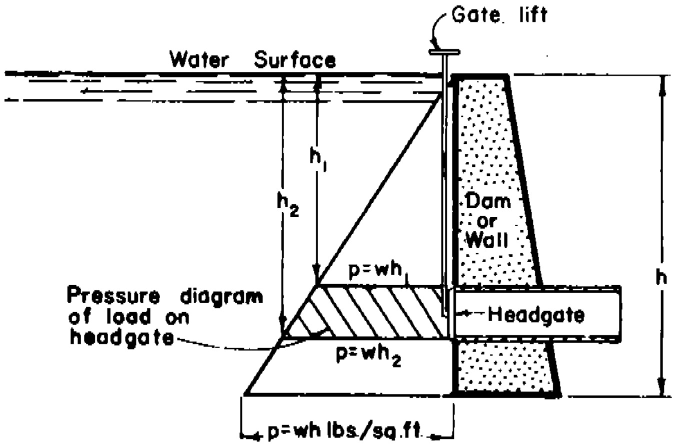
$$(c) = \frac{168.5 + 108.6}{2} \times 0.96 = 133.0 \text{ lbs. per ft.}$$

The solution would be the same for stoplogs in a water control structure.

Example 3-6

What is the total water load, F, on the headgate shown if it is 36-inches wide by 24-inches high, and $h_1$ is 9 feet?

Solution:



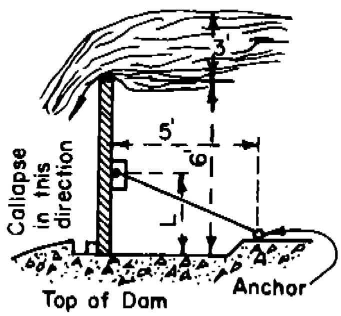

$$F = \frac{wh_1 + wh_2}{2} \text{ area}$$

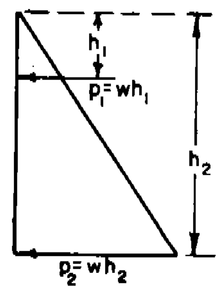
$$h_2 = 9 + 2 = 11 \text{ feet}$$

$$F = 62.4\left(\frac{9+11}{2}\right)(2)(3) = 3744 \text{ lbs.}$$

Example 3-7

This sketch shows the cross section of a collapsible flashboard with water at the maximum allowable elevation. Determine the position of the center of pressure and the pivot under the conditions shown. Experience has shown that the pivot on the gate must be 6/7 of the distance from the bottom of the flashboard to the center of pressure for the board to collapse.




Pressure Diagram

Solution:

As defined, the center of pressure is the point where a perpendicular through the center of gravity of the pressure prism strikes the area under pressure. Use Exhibit 3-2 for the solution of this problem.

First draw the pressure diagram:

$h_1 = 3$ ft. and $p_1 = 62.4(3) = 187.2$ lbs/sq.ft. $= a$

$h_2 = 3 + 6 = 9$ and $p_2 = 62.4(9) = 561.6$ lbs/sq.ft. $= b$

$d = 6$ ft.

then from Exhibit 3-2

$$\bar{y} = \frac{2ad + bd}{3(a + b)}$$

$$= \frac{2(187.2)(6) + 561.6(6)}{3(187.2 + 561.6)}$$

$$= \frac{5616.0}{2246.6} = 2.50 \text{ ft.} = \text{center of pressure}$$

and

$$L = \frac{6}{7}(2.5) = 2.14 \text{ ft.}$$

Example 3-8

It is required to determine the maximum thickness of flashboards needed in a flashboard dam. The flashboards will impound water to a depth of 7 feet, have a 6-foot span, and be of Coast Region Douglas Fir.

This can be solved by the use of Exhibit 3-3.

From chart A, thickness = 3.40 inches
From chart B, correction = 1.15

Flashboard thickness = 3.4(1.15) = 3.91 inches
Use nominal 4 x 12-inch flashboards

BUOYANCY AND FLOTATION

The familiar principles of buoyancy and flotation are usually stated, respectively:

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

2. A floating body displaces its own weight of the fluid in which it floats.

## Buoyancy

A submerged body is acted on by a vertical, buoyant force equal to the weight of the displaced water.

$$F_B = Vw \qquad \text{(Eq. 3-2)}$$

$F_B$ = buoyant force
$V$ = volume of the body
$w$ = unit weight of water

If the unit weight of the body is greater than that of water, there is an unbalanced downward force equal to the difference between the weight of the body and the weight of the water displaced. Therefore, the body will sink.

## Flotation

If the body has a unit weight less than that of water, the body will float with part of its volume below and part above the water surface in a position so that:

$$W = Vw \qquad \text{(Eq. 3-3)}$$

$W$ = weight of the body
$V$ = volume of the body below the water surface; i.e., the volume of the displaced water
$w$ = unit weight of water

A check should be made of the stability of hydraulic structures as they will be affected by submergence and whether the weight of the structure will be adequate to resist flotation.

Porous materials, when submerged, have different net weights depending upon whether the voids are filled with air or water. Note the wide variation in the possible net weight of one cubic foot of treated structural timber weighing 55 pounds under average atmospheric moisture conditions and having 30 percent voids:

| 1 cu. ft. of structural timber, 30 percent voids | Before Saturation | After Saturation |
|---|---|---|
| $W$ = weight in air, lb. | 55 | 55 + (0.30 x 62.4) = 73.72 |
| $F_B$ = buoyant force when submerged, lb. | 62.4 | 62.4 |
| $W-F_B$ = weight when submerged in water (net weight), lb. | 55 - 62.4 = - 7.4 | 73.72 - 62.4 = 11.32 |

The degree to which the factors discussed above are capable of affecting the net or stabilizing weight of a structure is illustrated by the following example:

Example 3-9

Assume a timber crib diversion dam subject to complete submergence under normal flood flows. Materials, weights and volumes are:

| Material | Percent of Volume of the Dam | Unit Weights lbs./cu. ft. |
|---|---|---|
| Timber | 12 | 55 in air |
| Timber | | 73 saturated |
| Loose stone, 30 percent voids | 88 | 150 solid stone |

Determine the net weight of one cubic yard of the dam when 1) not submerged, 2) submerged but timber not saturated, and 3) submerged with timber saturated.

1. Compute cubic feet of timber, solid stone, and voids per cubic yard of dam:

   a. Timber $0.12 \times 27 = 3.24$ cu. ft.
   b. Solid stone $0.7 \times 0.88 \times 27 = 16.63$ cu. ft.
   c. Voids $0.3 \times 0.88 \times 27 = 7.13$ cu. ft.

2. Compute the net weights of one cubic yard of dam:

| Material | Net Weights of Materials in Pounds per Cubic Yard of Dam: Not submerged | Submerged: Timber not saturated | Submerged: Timber saturated |
|---|---|---|---|
| Timber | 3.24 x 55 = 178 | 3.24(55 - 62.4) = -24 | 3.24(73 - 62.4 = 34 |
| Stone | 16.63 x 150 = 2494 | 16.63(150 - 62.4) = 1457 | 1457 |
| Effective or stabilizing weight of dam per cubic yard | 2672 | 1433 | 1491 |

Example 3-10

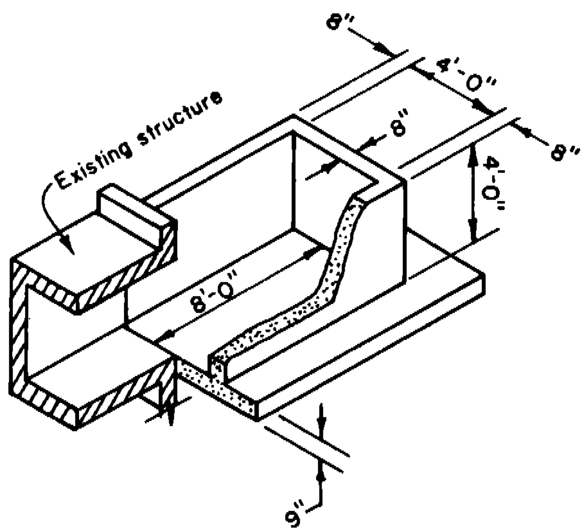
A box inlet drop spillway for a 4 x 4-foot highway culvert is to be constructed. The box inlet has been designed as shown. Determine if it is safe from flotation with a safety factor of 1.5 and if not, determine the size of spread footing required. Design assumptions are as follows:

1. The soil is saturated to the lip of the box and has a buoyant weight of 50 pounds per cubic foot.

2. There is no frictional resistance between the walls of the box and the surrounding soil.

3. Unit weight of concrete - 150 pounds per cubic foot.

4. Unit weight of water - 62.4 pounds per cubic foot.




First determine the weight (W) of the box.

| | | |
|---|---|---|
| End wall = 4' x 4' x 0.67' x 150 | = | 1,608 lbs. |
| 2 sidewalls = 4' x 8.67' x 0.67' x 150 x 2 | = | 6,907 |
| Floor slab = 5.33 x 8.67 x 0.75 x 150 | = | 5,199 |
| W | = | 13,714 lbs. |

Next determine the buoyant force ($F_B$) acting on the box by Equation 3-2:

$$F_B = Vw = \left(5.33 \times 4.75 \times 8.67\right) 62.4 = 13{,}697 \text{ lbs.}$$

From Equation 3-3, flotation will occur if W is less than or equal to $F_B$ ($F_B$ has been substituted from Equation 3-2 for Vw):

13,714 lbs. is greater than 13,697 lbs., therefore, the box will not float, but the required safety factor of 1.5 has not been accomplished.

Required weight of box: $W = 1.5\ F_B = 1.5(13{,}697) = 20{,}546$ lbs.

Additional weight to be added to box = 20,546 - 13,714 = 6,832 lbs.

This additional weight will be provided with a spread footing around three sides of the box, and the weight of the earth load on the footing.

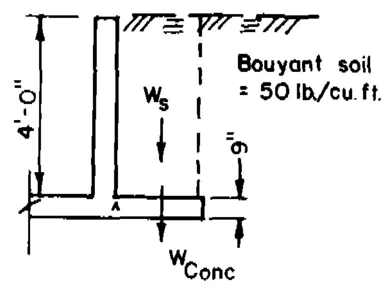
Weight per square foot of spread footing

$$= W_s + W_c = 4(50) + 0.75(150 - 62.4)$$

$$= 200 + 65.7 = 265.7 \text{ lbs.}$$

Required area of footing

$$= \frac{6832}{265.7} = 25.7 \text{ sq. ft.}$$




If a one foot wide spread footing is provided, the footing area would be 2(8.67) + 7.33 = 24.7 sq. ft. and provide 24.7(265.7) = 6550 lbs. of additional weight. The safety factor against uplift would be:

$$SF = \frac{13{,}714 + 6550}{13{,}697} = 1.48$$

Similarly, a spread footing 1'-3" wide would produce a safety factor of 1.57. By interpolation, a footing 1'-1" wide would meet the factor of safety requirement.

## 4. HYDROKINETICS

Hydrokinetics is the solution of fluid problems in which a change of motion occurs as the result of the application of a force to the fluid body (water in motion).

## FLOW CONTINUITY

When the discharge through a given cross section of a 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 and is expressed by the equation:

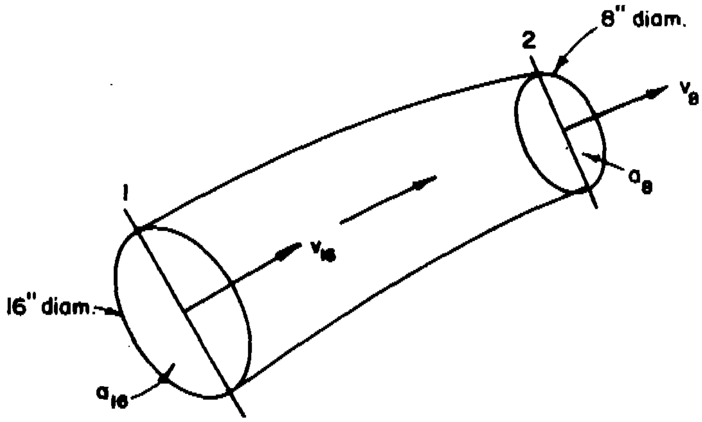
$$Q = a_1v_1 = a_2v_2 = a_3v_3 = a_nv_n \qquad \text{(Eq. 3-4)}$$

where Q = discharge in cubic feet per second
a = cross-sectional area in square feet
v = mean velocity of flow in feet per second
1, 2, 3, n = subscripts denoting different cross sections

Most of the hydraulic problems handled by field technicians deal with cases of continuous flow.

### Example 3-11

10 cfs of water flows through the tapered pipe shown below. Calculate the average velocities at sections 1 and 2 with diameters of 16 and 8 inches respectively.




from Equation 3-4

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

$$V_{16} = \frac{Q}{a_{16}} \quad \frac{Q}{\frac{\pi}{4}d^2} \text{ where d, in feet, equals } \frac{16}{12}$$

$$= \frac{10}{\frac{\pi}{4}\left(\frac{16}{12}\right)^2} = \frac{10}{\frac{3.1416\,(1.333)^2}{4}} = 7.16 \text{ fps}$$

Similarly:

$$V_8 = \frac{Q}{a_8} = \frac{10}{\frac{\pi}{4}\left(\frac{8}{12}\right)^2} = 28.64 \text{ fps}$$

or, based on the ratio of cross-sectional areas

$$V_8 = V_{16}\left(\frac{a_{16}}{a_8}\right)^2 = 7.16\left(\frac{16}{8}\right)^2 = 28.64 \text{ fps}$$

## CONSERVATION OF ENERGY

Three forms of energy are normally considered in the analysis of problems in water flow: Potential or elevation energy, pressure energy, and kinetic energy.

### Potential Energy

Potential energy is the ability to do work because of the elevation of a mass of water with respect to some datum. A mass of weight, w, at an elevation z feet, has potential energy amounting to wz foot pounds with respect to the datum. The elevation head, z, expresses not only a linear quantity in feet, but also energy in foot pounds per pound.

### Pressure Energy

Pressure energy is acquired by contact with other masses and is transmitted to or through the liquid mass under consideration. A mass of water, as such, does not have pressure energy. Pressure energy may be supplied by a pressure pump or through some other applied force. The pressure head ($h = \frac{p}{w}$) also expresses energy in foot pounds per pound.

### Kinetic Energy

Kinetic energy exists because of a velocity of motion and amounts to:

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

where W = weight of the water
v = velocity in feet per second
g = acceleration due to gravity

When W equals one pound, the kinetic energy has a value of $\frac{v^2}{2g}$.

This expression is called the velocity head. In other words, if the velocity of a stream of water is known, it is possible to compute the head which is converted from pressure energy or potential energy to create kinetic energy. This principle is extremely important in hydraulics.

Under certain conditions, the three forms of energy are interchangeable. The relationship between the three forms of energy in pipe and channel flow is shown by Figure 3-5.

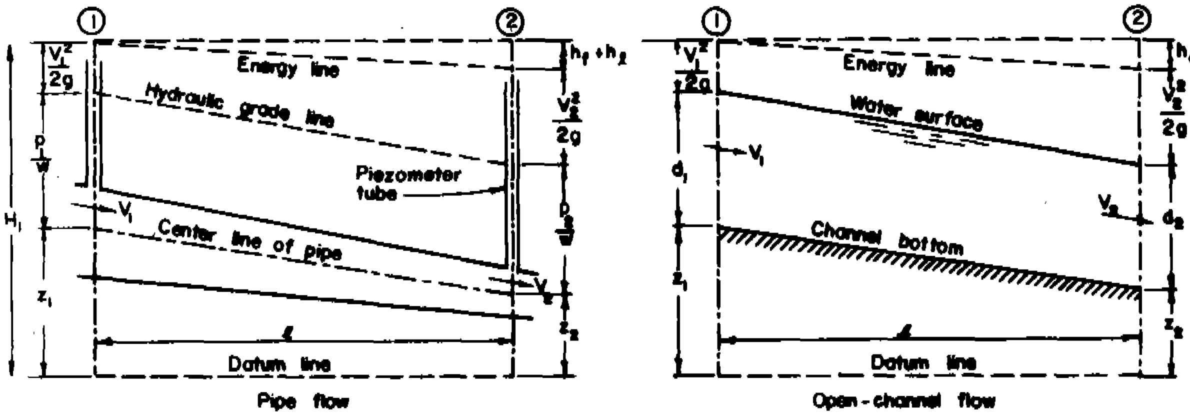



Figure 3-5 Relationship between energy forms in pipe and open channel flow

The total head, $H_1$, is a vertical distance and represents the value of the total energy in the system at Section 1. This is made up of the velocity head which is equivalent to the kinetic energy, the pressure head which is equivalent to the energy due to pressure, and the elevation head which is equivalent to the energy due to position.

In the case of channel flow, the velocity head is the difference in elevation between the energy line and the water surface.

In the case of pipe flow, the velocity head is the difference between the elevation of the energy line and the elevation to which water would rise in a piezometer tube. In pipe flow the pipe may be lowered or raised within the zone of the elevation and the pressure heads without changing the conditions of flow. 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 is increased and the pressure head is decreased. If the quantity of flow and diameter of pipe did not change, then the velocity head will remain the same.

## Bernoulli Principle

Bernoulli's principle is the application of the law of conservation of energy to fluid flow. It may be stated as follows: In frictionless flow, the sum of the kinetic energy, pressure energy, and elevation energy is equal at all sections along the stream. This means that if we measured the velocity head, the pressure head, and the elevation head at one station in a pipe or open channel carrying flowing water without friction, we would find that the total would be equal to the total of the velocity head, the pressure head, and the elevation head at a second station downstream in the same pipe or open channel, Figure 3-5. This is theoretical, but the principle is used to work out practical solutions. In practice, friction and all other energy losses must be considered and the energy equation becomes:

$$\frac{v_1^2}{2g} + \frac{p_1}{w} + z_1 = \frac{v_2^2}{2g} + \frac{p_2}{w} + z_2 + h_f + h_\ell \qquad \text{(Eq. 3-5)}$$

where $v$ = mean velocity of flow
$p$ = unit pressure
$w$ = unit weight of water
$g$ = acceleration of gravity
$z$ = elevation head
$h_\ell$ = all losses of head other than by friction between Stations 1 and 2 such as bends
$h_f$ = head loss by friction between Stations 1 and 2
sub 1,2 denotes upstream and downstream stations, respectively

The energy equation and the equation of continuity ($Q = a_1 v_1 = a_2 v_2$) are the two basic, simultaneous equations used in solving problems in water flow.

## Hydraulic and Energy Gradients

The hydraulic gradient in open-channel flow is the water surface, and in pipe flow it connects the elevations to which the water would rise in piezometer tubes along the pipe. The energy gradient is above the hydraulic gradient, a distance equal to the velocity head. In both open-channel and pipe flow, the fall (or slope) of the energy gradient for a given length of channel or pipe represents the loss of energy by friction. When considered together, the hydraulic gradient and the energy gradient reflect not only the loss of energy by friction, but also the conversions between the three forms of energy. See Figure 3-5.

## 5. PIPE FLOW

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

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.

The concept of flow continuity and the Bernoulli principle has been discussed in the preceding section. This section defines laminar and turbulent flow, discusses the commonly used discharge equations and outlines the hydraulics of pipelines and culverts.

### LAMINAR AND TURBULENT FLOW

Water flows with two different types of motion, laminar and turbulent.

Laminar flow occurs when the individual particles of water move in parallel layers. The velocities of these layers are not necessarily the same. However, the mean velocity of flow varies directly with the slope of the hydraulic gradient.

Turbulent flow is an irregular type of flow in which the particles follow unpredictable paths. In addition to the main velocity in the direction of flow, there are transverse components of velocity. The mean velocity of flow varies with the square root of the slope of the hydraulic gradient.

Laminar flow seldom occurs in pipe flow. It is the type of flow that water has through soils. For pipe flow the motion is turbulent.

### Friction Loss

The loss of energy or head resulting from turbulence created at the boundary between the sides of the conduit and the flowing water is called friction loss.

In a straight length of conduit, flowing full, with constant cross section and uniform roughness, the rate of loss of head by friction is constant and the energy gradient has a slope in the direction of flow equal to the friction head loss per foot of conduit.

Of the many equations that have been developed to express this loss of head, the following two are the most widely used:

<u>Manning's Equation</u>

The general form of Manning's equation is:

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

with the following nomenclature:

- $a$ = cross-sectional area of flow in ft.$^2$
- $d$ = diameter of pipe in feet
- $d_i$ = diameter of pipe in inches
- $g$ = acceleration of gravity = 32.2 ft. per sec.$^2$
- $H_1$ = loss of head in feet due to friction in length, L
- $K_c$ = head loss coefficient for any conduit
- $K_p$ = head loss coefficient for circular pipe
- $L$ = length of conduit in feet
- $n$ = Manning's roughness coefficient
- $p$ = wetted perimeter in feet
- $r$ = hydraulic radius in feet = $\frac{a}{p} = \frac{d}{4}$ for round pipe
- $s$ = loss of head in feet per foot of conduit = slope of energy grade and hydraulic gradelines in straight conduits of uniform cross section = $(H_1 + L)$
- $v$ = mean velocity of flow in ft. per sec.
- $Q$ = discharge or capacity in cu.ft. per sec.

Starting with Equation 3-6 solve for s, multiply numerator and denominator of right side of equation by 2g and substitute $(H_1 + L)$ for s. The result is:

$$H_1 = \frac{29.164\ n^2\ L\ v^2}{2g\ r^{4/3}}$$

the equation can be simplified by substituting

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

then the equation takes the form

$$H_1 = K_c\ L\ \frac{v^2}{2g} \qquad \text{(Eq. 3-8)}$$

Adaption of Equation 3-7 to circular pipes involves the substitution of (d + 4) for r and the change from d to $d_i$.

$$K_p = \frac{5087\ n^2}{d_i^{\ 4/3}} \qquad \text{(Eq. 3-9)}$$

Tables for values of $K_p$ and $K_c$ for the usual ranges of variables are given in Exhibit 3-4.

King's Handbook[(1)] gives a number of convenient working forms of Manning's formula and references to tables that will facilitate their use. Four of these are:

$$H_1 = 2.87\ n^2 \frac{Lv^2}{d^{\ 4/3}}$$

$$H_1 = 4.66\ n^2 \frac{LQ^2}{d^{\ 16/3}}$$

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

$$d_i = \left(\frac{1630\ Qn}{s^{\ 1/2}}\right)^{3/8}$$

Exhibit 3-5, which is based on the last of these equations, may be used to determine $d_i$, s, or Q when two of these quantities and n are known. Values of Manning's n are given in Table 3-1.

Hazen-Williams Equation

As generally used, this equation is:

$$v = 1.318\ C\ r^{0.63}\ s^{0.54} \qquad \text{(Eq. 3-10)}$$

Notation is the same as given for Manning's equation with the addition of C, the coefficient of roughness in Hazen-Williams formula.

Since Q = av, Equation 3-10 may be converted to the following formula for discharge in any conduit:

$$Q = 1.318\ a\ C\ r^{0.63}\ s^{0.54}$$

Substitution of a and r in terms of inside diameter of pipe in inches in this equation gives the following general formula for discharge in circular pipes:

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

Graphical solutions of Equation 3-11 for standard pipe ranging from 1 to 12 inches in diameter and a wide range in slope may be made by using Exhibit 3-6. Exhibit 3-7 gives losses for semi-rigid plastic pipe.

Values for C for different types of pipe are given in Table 3-2.

Regardless of the designer's preference of equations, the results should be checked against the application of State design criteria.

Table 3-1 Values of Manning's, n

| Description of pipe | Values of n | | |
|---|---|---|---|
| | Min. | Design | Max. |
| Cast-iron, coated | 0.010 | 0.012 - 0.014 | 0.014 |
| Cast-iron, uncoated | 0.011 | 0.013 - 0.015 | 0.015 |
| Wrought iron, galvanized | 0.013 | 0.015 - 0.017 | 0.017 |
| Wrought iron, black | 0.012 | | 0.015 |
| Steel, riveted and spiral | 0.013 | 0.015 - 0.017 | 0.017 |
| Annular corrugated metal | 0.021 | 0.021 - 0.025 | 0.0255 |
| Helical corrugated metal | 0.013 | 0.015 - 0.020 | 0.021 |
| Wood stave | 0.010 | 0.012 - 0.013 | 0.014 |
| Neat cement surface | 0.010 | | 0.013 |
| Concrete | 0.010 | 0.012 - 0.017 | 0.017 |
| Vitrified sewer pipe | 0.010 | 0.013 - 0.015 | 0.017 |
| Clay, common drainage tile | 0.011 | 0.012 - 0.014 | 0.017 |
| Corrugated plastic | 0.014 | 0.015 - 0.016 | 0.017 |

Table 3-2 Values of Hazen-Williams C

| Description of pipe | C |
|---|---|
| Very smooth pipe; straight alignment | 140 |
| Very smooth pipe; slight curvature | 130 |
| Cast iron, uncoated - new | 130 |
| 5 years old | 120 |
| 10 years old | 110 |
| 15 years old | 100 |
| 20 years old | 90 |
| 30 years old | 80 |
| coated - all ages | 130 |
| Steel pipe, welded, new | 130 |
| (Same deterioration with age as cast iron, uncoated) | |
| For permanent installation use | 100 |
| Wrought iron or standard galvanized steel - dia. 12-in. up | 110 |
| 4 to 12 in. | 100 |
| 4 in. down | 80 |
| Brass or lead, new | 140 |
| Concrete, very smooth, excellent joints | 140 |
| smooth, good joints | 120 |
| rough | 110 |
| Vitrified | 110 |
| Smooth wooden or wood stave | 120 |
| Asbestos cement | 140 |
| Corrugated pipe | 60 |
| Note: Pipes of small diameter, old age, and very rough inside surface, may give values as low as C = 40 | |

## Other Losses

In addition to the friction head losses, there are other losses of energy which occur as the result of turbulence created by changes in velocity and direction of flow. To facilitate their inclusion in Bernoulli's energy equation, such losses are commonly expressed in terms of the mean velocity head at some specific cross section of the pipe.

These losses are sometimes called minor losses, which may be a serious misnomer. In long pipelines, the entrance loss, bend losses, etc., may be only a small part of the total loss and in such cases can be ignored. Such is not the case in many structures such as culverts, drop inlets, and siphons which are relatively short. Safe design practice requires an estimate of such losses. If the estimate indicates that minor losses amount to 5 percent or more of the total head loss, they should be carefully evaluated and included in the flow calculations.

As velocities increase, careful determination of such minor losses becomes more important. With a mean velocity of 30 feet per second, the neglect of an entrance loss of 0.5 $v^2/2g$ results in an error in head loss of 7 feet. Whereas, if the mean velocity is 3 feet per second, neglect of such an entrance loss results in an error of only 0.07 foot.

Data on minor losses most commonly required are contained in Exhibit 3-8 of this chapter, and Section 5, Hydraulics,[6] and Section 15, Irrigation, of the National Engineering Handbook.

## HYDRAULICS OF PIPELINES

The pipe flow condition often found in SCS work is that of free flow discharge. See Figure 3-6.

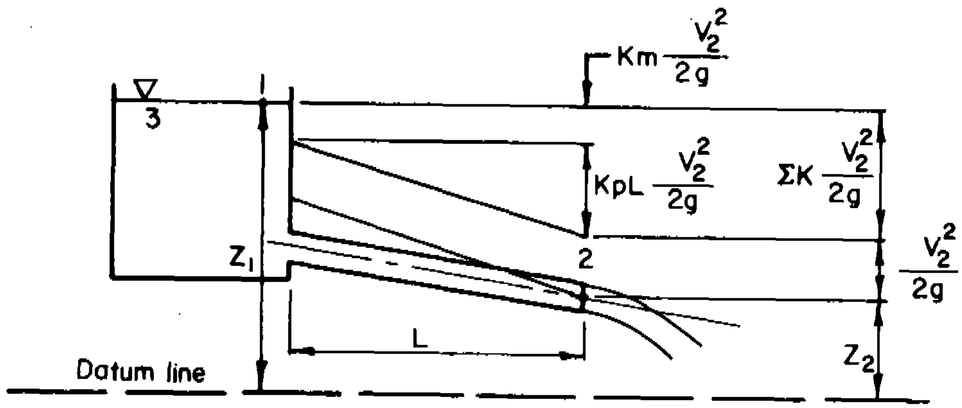



Figure 3-6 Pipe flow energy relationships

The general pipe flow equation is derived through use of the Bernoulli and continuity principles.

Equating the energy in Figure 3-6, using Equation 3-5:

$$0 + 0 + z_1 = \frac{v_2^2}{2g} + 0 + z_2 + \Sigma K\frac{v_2^2}{2g}$$

where $z_1$ = elevation head at station 1

$z_2$ = elevation head at station 2

$v_2$ = velocity head at station 2

$\Sigma K\frac{v_2^2}{2g}$ = sum of the minor head losses and pipe friction losses

Let $H = z_1 - z_2$

$$H = z_1 - z_2 = \frac{v_2^2}{2g}(1 + \Sigma K)$$

or

$$v_2 = \sqrt{\frac{2gH}{1+K_m+K_pL}}$$

and from the continuity principle

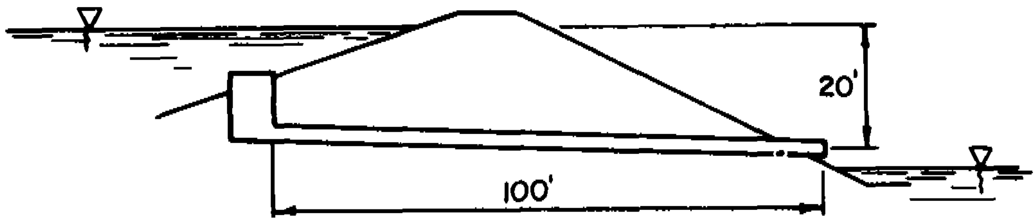
$$Q = a\sqrt{\frac{2gH}{1+K_m+K_pL}} \qquad \text{(Eq. 3-12)}$$

where Q = discharge-cfs
a = pipe area-sq.ft.
g = acceleration of gravity-ft/sec.$^2$
H = elevation head differential-ft.
$K_m$ = coefficient of minor losses
$K_p$ = pipe friction coefficient
L = pipe length-ft.

The following examples are applications of Equation 3-12.

Example 3-12

Determine the discharge of a drop inlet spillway with cantilevered outlet for a head H of 20 feet. The spillway is 24-inch diameter reinforced concrete pipe with Manning's n of 0.013, Table 3-1. $K_m$ is 1.0 for bend and entrance losses. See reference, sheet 2 of Exhibit 3-8.




Solution

Using equation (3-12)

$$Q = a\sqrt{\frac{2gH}{1+K_m+K_pL}}$$

1. Area
   Reference: Exhibit 3-4
   $a = 3.14$ sq ft

2. Friction loss coefficient
   Reference: Exhibit 3-4
   $K_p = 0.0124$

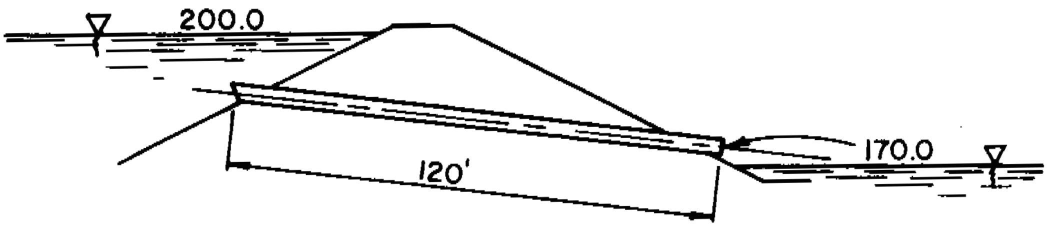
3. Discharge

$$Q = 3.14\sqrt{\frac{(2)\ (32.2)\ (20)}{1+1+(0.0124)\ (100)}}$$

$$Q = 62.5 \text{ cfs}$$

<u>Example 3-13</u>

A corrugated metal pipe with a hooded inlet and cantilevered outlet is to discharge 130 cfs when the reservoir water surface is at elevation 200.0 and the centerline of the outlet is at elevation 170.0. Determine the diameter of pipe required. Use Manning's n = 0.024, Table 3-1.




<u>Solution</u>

Select a diameter and determine the discharge using Equation 3-12.

$$Q = a \sqrt{\frac{2gH}{1+K_m+K_pL}}$$

<u>Trial 1</u>

1. Select d = 36 inches

2. Area
   Reference: Exhibit 3-4
   a = 7.07 sq.ft.

3. Friction loss coefficient
   Reference: Exhibit 3-4
   $K_p = 0.0246$

4. Minor loss coefficient
   Reference: Exhibit 3-8 ($K_m$=$K_e$+$K_b$=$K_e$+0=$K_e$)
   entrance $K_m = 1.00$

5. Discharge

$$Q = 7.07 \sqrt{\frac{(2)\ (32.2)\ (200-170)}{1+1.00+(0.0246)\ \ (120)}}$$

Q = 138.0 cfs

Trial 2

1. Select d = 30 inches

2. Area a = 4.91 sq.ft.

3. Friction loss coefficient $K_p$ = 0.0314

4. Minor loss coefficient $K_m$ = 1.00

5. Discharge

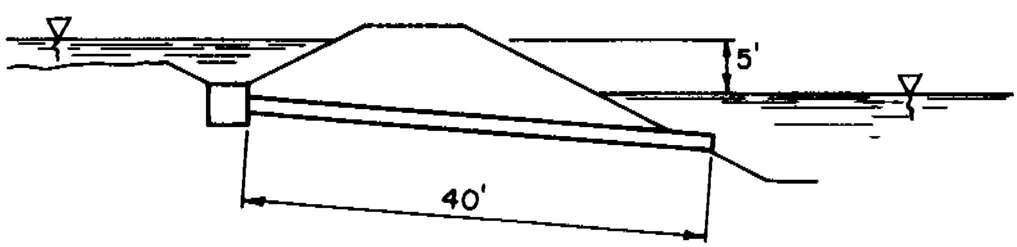
$$Q = 4.91 \sqrt{\frac{(2)\ (32.2)\ (200-170)}{1+1.00+(0.0314)\ (120)}}$$

$$Q = 90.0 \text{ cfs}$$

It can be seen from the foregoing two trials that the 36-inch pipe more nearly satisfies the required Q of 130 cfs and is the one to be installed.

Example 3-14

An 8-inch diameter concrete side drain inlet discharges below the water surface of a channel. The pipe is 40 feet long and flowing full with a head of 5 feet. Manning's n = 0.012, Table 3-1. Determine the discharge. Assume $K_m$ = entrance coefficient plus bend coefficient = 1.




The discharge equation for exit conditions other than free flow is the same as Equation 3-12.

$$Q = a \sqrt{\frac{2gH}{K_x+K_m+K_pL}}$$

$K_x$ = exit coefficient = 1.0

therefore

$$Q = a\sqrt{\frac{2gH}{1+K_m+K_pL}}$$

1. Area
   Reference: Exhibit 3-4
   $a = 0.349$ sq.ft.

2. Friction loss coefficient
   Reference: Exhibit 3-4
   $K_p = 0.0458$

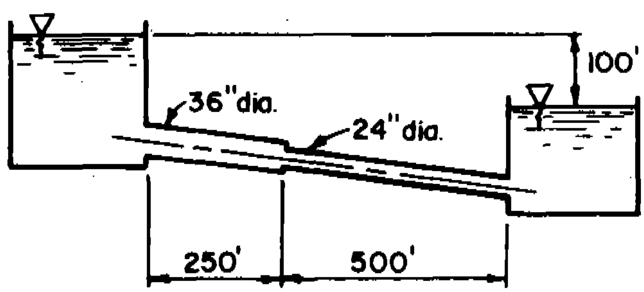
3. Discharge

$$Q = 0.349\sqrt{\frac{(2)\ (32.2)\ (5)}{1+1+(0.0458)\ (40)}}$$

$$Q = 3.2 \text{ cfs}$$

Example 3-15

A pipeline of 250 feet of 36-inch and 500 feet of 24-inch steel pipe connects two reservoirs. Determine the discharge if: the head is 100 feet, the entrance coefficient is 1, the contraction coefficient is 0.25, and Manning's n is 0.011.




To use Equation 3-12 the loss coefficients must be expressed in terms of a single-sized pipe.

In terms of the 24-inch pipe the coefficients in the example must be multiplied by the following ratio, C, which is based on the square of the ratio of areas.

$$C_{36} = \left(\frac{\text{area of 24" dia. pipe}}{\text{area of 36" dia. pipe}}\right)^2$$

$$C_{24} = \left(\frac{\text{area of 24" dia. pipe}}{\text{area of 24" dia.pipe}}\right)^2$$

Discharge in terms of 24-inch diameter pipe

1. Areas
   Reference: Exhibit 3-4
   24-inch dia. a = 3.14 sq.ft.
   36-inch dia. a = 7.07 sq.ft.

2. Friction loss coefficients
   Reference: Exhibit 3-4
   24-inch dia. $K_p$ = 0.00889
   36-inch dia. $K_p$ = 0.00518

3. Square of the ratio of areas

$$C_{24} = \left(\frac{3.14}{3.14}\right)^2 = 1.0$$

$$C_{36} = \left(\frac{3.14}{7.07}\right)^2 = 0.196$$

4. Sum of the loss coefficients

| Item | $K_p$ | L | K | C | Adj. loss coeff. |
|---|---|---|---|---|---|
| Entrance | - | - | 1.0 | 0.196 | 0.196 |
| 36" pipe | 0.00518 | 250 | 1.296 | 0.196 | 0.254 |
| Contraction | - | - | 0.25 | 0.196 | 0.049 |
| 24" pipe | 0.00889 | 500 | 4.45 | 1.0 | 4.45 |
| Exit | - | - | 1.0 | 1.0 | 1.0 |
| | | | | | 5.949 |

5. Discharge

$$Q = 3.14\sqrt{\frac{(2)(32.2)(100)}{5.949}}$$

$$= 103 \text{ cfs}$$

## HYDRAULICS OF CULVERTS (2)

There are two major types of culvert flow: 1) Flow with inlet control, and 2) flow with outlet control. For each type, different factors and formulas are used to compute the hydraulic capacity of the culvert. Under inlet control, the slope, 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 involves the additional consideration of the elevation of the tailwater in the outlet channel and the length of the culvert.

The need for making involved computations to determine the probable type of flow under which a culvert will operate may be avoided by computing headwater depths from Exhibits 3-9 through 3-12 for both inlet control and outlet control and then using the higher value for design.

Both inlet control and outlet control types of flow are discussed briefly in the following paragraphs.

### Culverts Flowing With Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the shape of the entrance. Figure 3-7 shows inlet control flow for three types of culvert entrances.

In inlet control the length of the culvert barrel and outlet conditions are not factors in determining culvert capacity.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth + velocity head). Because of the low velocities in most entrance pools, the water surface and the energy line at the entrance are assumed to coincide.

Headwater-discharge relationships for various types of circular culverts flowing with inlet control are based on laboratory research with models and verified in some instances by full-scale tests. Exhibits 3-9 and 3-10 give headwater-discharge relationships for round concrete and CM pipe culverts flowing with inlet control.

### Example 3-16

It is desired to determine the maximum discharge of an existing 42-inch concrete culvert. The allowable headwater depth (HW) upstream is 8.0 feet and the slope of the culvert is 0.02 ft/ft. The culvert has a projecting entrance condition and there will be no backwater from downstream flow. Assume inlet control.

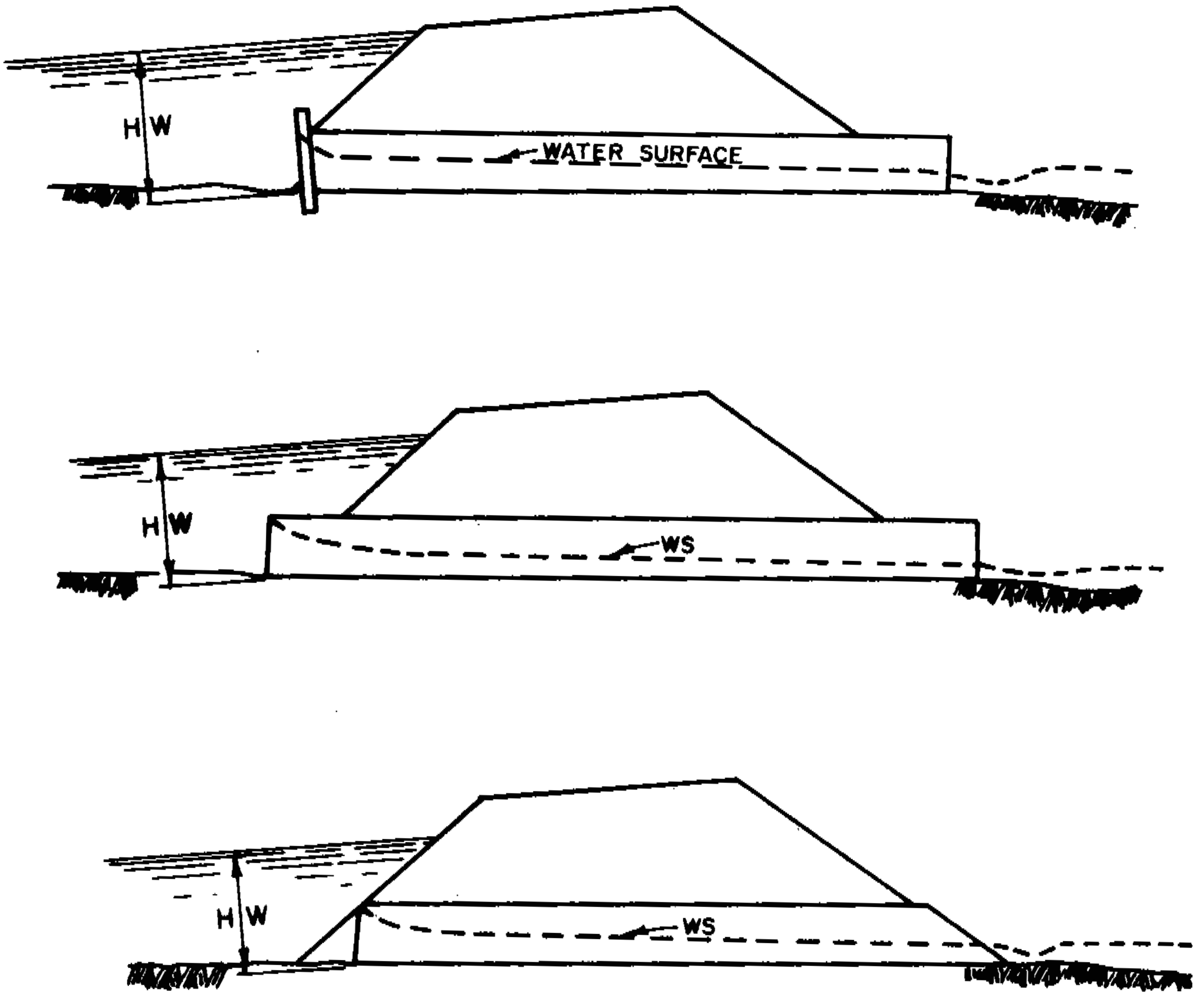



Figure 3-7 Culverts with inlet control

Using Exhibit 3-9, compute $\frac{HW}{D}$

$$\frac{HW}{D} = \frac{8(12)}{42} = 2.29$$

At 2.29 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 scale 4. On scale 5 read Q = 128 cfs.

Check for inlet control

$$s_o > s_n$$

where

$>$ = symbol for "is greater than"

$s_o$ = installed slope of culvert

$s_n$ = neutral slope - that slope of which the loss of head due to friction is equal to the gain in head due to elevation.

from Table 3-1, n(design) for concrete pipe = 0.012

from Exhibit 3-5, sheet 3 of 6

for Q = 128 cfs and D = 42 inches

$s_n$ = 0.013

therefore, the culvert is in inlet control, 0.02 >0.013

<u>Example 3-17</u>

Determine the required diameter of a corrugated metal culvert pipe to be installed in an existing channel. Q = 100 cfs, HW max. = 7.0 feet and $s_o$ = 0.03. There will be no backwater from downstream flow. Entrance to be mitered to conform to the slope of the embankment.

The solution of this problem must be made by trial pipe diameters and solution of HW by use of Exhibit 3-10.

Try D = 36"

draw a line through 36 inch on scale 4 and 100 cfs on scale 5 to an intersection with scale 1, then a horizontal line from scale 1 to scale 2, mitered inlet. On scale 2 read $\frac{HW}{D}$ = 3.8,

then HW = 3.8(3) = 11.4 feet too high

Try D = 48"

read from scale 2, $\frac{HW}{D} = 1.45$

HW = 1.45(4) = 5.80 feet        low

Try D = 42"

read from scale 2, $\frac{HW}{D} = 2.23$

HW = 2.23(3.5) = 7.8 feet        high

From the foregoing trials, it will be necessary to install the 48-inch diameter pipe if the HW is to be 7.0 feet maximum.

Check for inlet control

$s_o > s_n$

from Exhibit 3-5, sheet 6 of 6

$s_n = 0.017$

0.03 > 0.017    therefore, inlet control

Culverts Flowing With Outlet Control

Culverts flowing with outlet control can flow with the culvert barrel full for all or part of the barrel length. See Figure 3-8. If the entire cross section of the barrel is filled with water for the total length of the barrel, the culvert is said to be in full flow, Figure 3-8(a) and (b). One other type of outlet control is shown in Figure 3-8(c). For this condition, the elevation of the energy gradeline at the exit of the culvert is assumed at 3/4D. This is not an exact figure but it will give reasonable results.

The head, H, Figure 3-8(a), or energy required to pass a given quantity of water through a culvert with outlet control is made up of three parts. The parts are expressed in feet of water and include a velocity head, $H_v$, an entrance loss, $H_e$, and a friction loss, $H_f$. This energy is obtained from ponding of water at the entrance and is expressed by the equation

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

This equation in similar form has been derived in the section on Hydraulics of pipelines.

The entrance loss, $H_e$, depends upon the shape of the inlet edge. This loss is expressed as a coefficient, $K_e$, times the barrel velocity head. That is, $H_e = K_e \frac{v^2}{2g}$. Entrance loss coefficients, $K_e$, for various types of entrances when flow is in outlet control are given in Table 3-3.

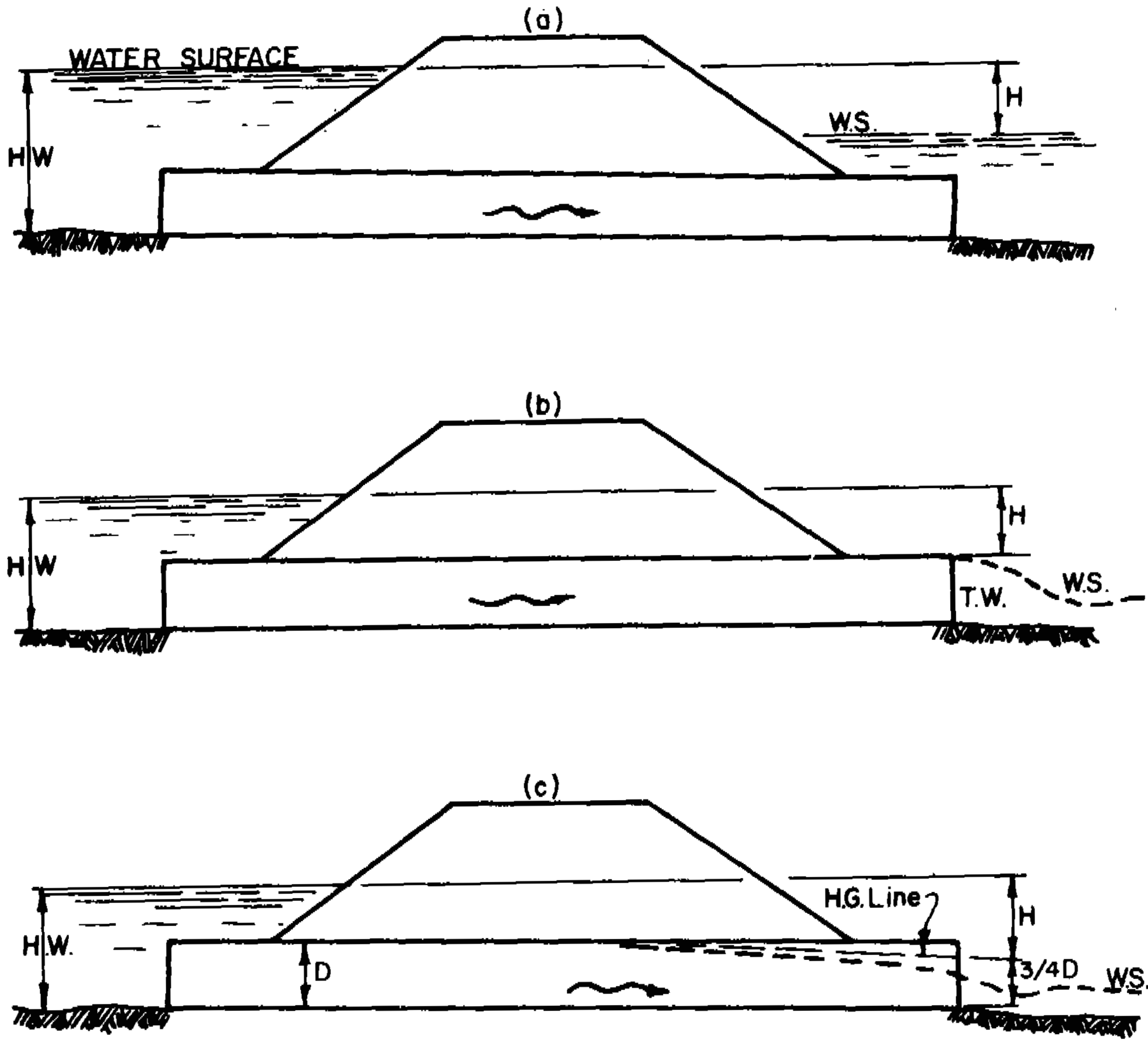



Figure 3-8 Culverts with outlet control

Table 3-3 Entrance Loss Coefficients

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

Note: *"End-section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control.

The friction loss, $H_f$, is the energy required to overcome the roughness of the culvert barrel and is expressed by the equation

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

$K_p$ values can be taken from Exhibit 3-4.

Headwater depth can be expressed as an equation for all outlet control conditions, including all depths of tailwater, TW. This is done by designating the vertical distance from the culvert invert at the outlet to the elevation from which H is measured as $h_o$.

$$HW = H + h_o - s_o \ell \qquad \text{(Eq. 3-14)}$$

where $\ell$ = length of culvert
$s_o$ = slope of culvert in feet per foot
H = head loss in feet as determined from the appropriate exhibit

When the elevation of the water surface in the outlet channel is equal to or above the top of the culvert opening at the outlet, Figure 3-9(a), $h_o$ is equal to the tailwater depth. If the tailwater elevation is below the top of the culvert opening at the outlet, Figure 3-9(b), $h_o$, is then by definition 3/4D.

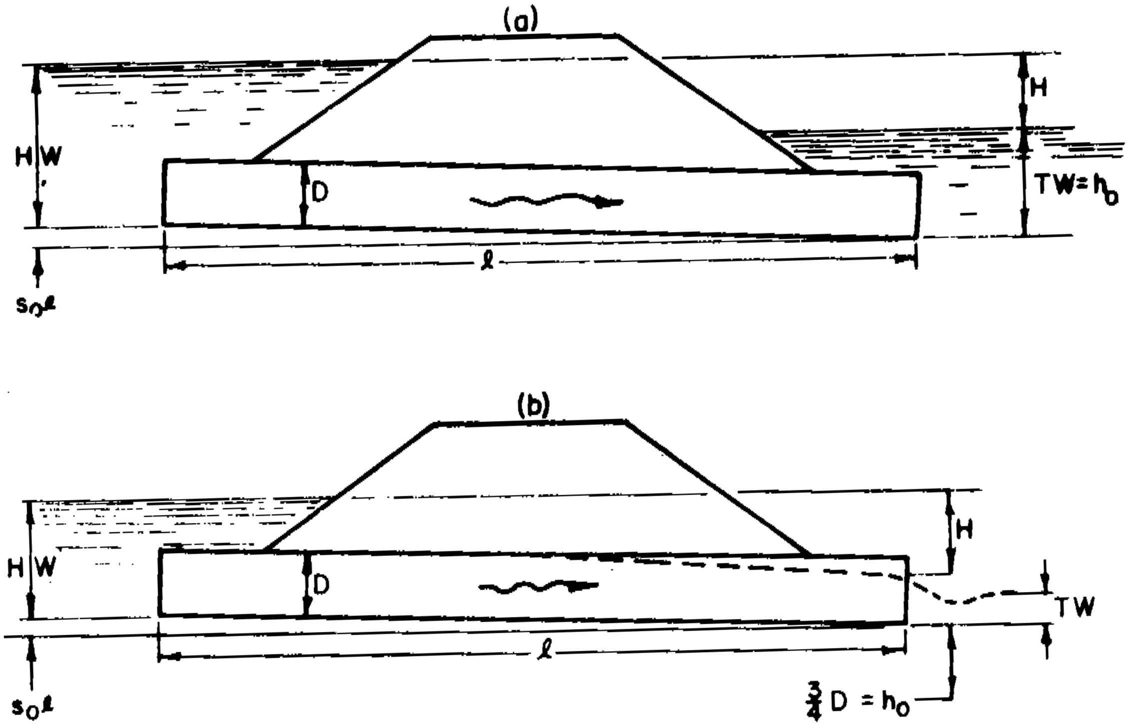



**Figure 3-9 Culvert water depth relationships**

Headwater-discharge relationships for various types of circular culverts flowing with outlet control may be solved by the use of Exhibits 3-11 and 3-12. For a different roughness coefficient $n_1$ than that of the exhibit n, use the length scales shown with an adjusted length, $\ell_1$, calculated by the formula

$$\ell_1 = \ell\left(\frac{n_1}{n}\right)^2$$

Example 3-18

It is desired to install 50 feet of concrete culvert pipe, $n = 0.012$, in a drainage channel for a road crossing. Design Q is 80 cfs with a tailwater depth of 3.0 feet. Slope of the culvert will be 0.002 foot per foot. Maximum headwater depth (HW) is 5 feet.

from Equation 3-14

$HW = H + h_o - s_o\ell$ or, $H = HW - h_o + s_o\ell$

$H = 5.0 - 3.0 + .002(50)$

$H = 2.1$ feet

and from Table 3-3

for a concrete pipe projecting from the fill with socket end upstream

$K_e = 0.2$

entering Exhibit 3-11

draw a line between $H = 2.1$ feet on the head scale and $Q = 80$ cfs on the discharge scale. Then on the length scale for $K_e = 0.2$, draw a second line from the 50-feet mark through the intersection of the first line with the "turning line" and on to the pipe diameter scale. The diameter scale intersection is at approximately 39 inches, therefore, use a 42-inch pipe.

Erosive Culvert Exit Velocities

A culvert, because of its hydraulic characteristics, increases the velocity of flow over that in the adjacent channel. High velocities may be damaging just downstream from the culvert outlet and the erosion potential at this point should be considered in culvert design. In many cases it is necessary to riprap the channel for a short distance downstream of the culvert exit.

## 6. OPEN CHANNEL FLOW

The flow of water in an open channel differs from pipe flow in one important respect. See Figure 3-5. Open channel flow must have a free water surface, whereas pipe flow has none since water must fill the whole conduit.

Flow calculations for open channels are complicated by the fact that the position of the water surface is likely to change with respect to time and the cross-sectional area. Also the depth of flow, discharge, and slopes of the channel bottom and water surface are interdependent. Channel cross sections can 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 that of rough, irregular riverbeds. Moreover, the roughness in an open channel varies with the position of the free water surface. Therefore, the proper selection of friction coefficients is more uncertain for open channels than for pipes. In general, the treatment of open channel flow is somewhat more empirical than that of pipe flow, but the empirical method is the best available. If cautiously applied, it results in practical values.

## TYPES OF CHANNEL FLOW

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.

1. Steady flow
    a. Uniform flow
    b. Nonuniform flow
        (1) Gradually varied flow
        (2) Rapidly varied flow
2. Unsteady flow
    a. Unsteady uniform flow (rare)
    b. Unsteady varied flow
        (1) Gradually varied unsteady flow
        (2) Rapidly varied unsteady flow

### Steady Flow and Unsteady Flow: Based on Time Interval

Flow in an open channel is steady if the depth of flow at a given cross section does not change, or if it can be assumed to be constant, during the time interval being considered. The flow is unsteady if the depth of flow at a given cross section changes with time.

In most open-channel problems it is necessary to study flow behavior only 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.

### Uniform Flow and Nonuniform Flow: Based on Channel Space Used

Open-channel flow is uniform if the depth of flow is the same at every section of the channel. A uniform flow may be steady or unsteady, depending on whether or not 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 if 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.

Various types of flow are shown in Figure 3-10.

## CHANNEL CROSS-SECTION ELEMENTS

The elements of cross sections of an open channel required for hydraulic computations are:

- a, the cross-sectional area of flow;
- p, the wetted perimeter, that is, the length of the boundary of the cross section in contact with the water;
- $r = \frac{a}{p}$, the hydraulic radius, which is the cross-sectional area of the stream divided by the wetted perimeter.

General formulas for determining area, wetted perimeter, hydraulic radius, and top widths of trapezoidal, rectangular, triangular, circular, and parabolic sections are given in Exhibit 3-13. Many tables are available showing hydraulic elements for various sizes and shapes of channels. The USDI Bureau of Reclamation "Hydraulic and Excavation Tables"(3) and Corps of Engineers "Excavation Tables"(4) are good books if there is much of this work to be done.

## MANNING'S EQUATION

The most widely used 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. They are equations in which the values of constants and exponents have been derived from experimental data. Manning's equation is one of the most widely accepted and commonly used of the open channel formulas:

$$v = \frac{1.486}{n} r^{2/3} s^{1/2} \qquad \text{(Eq. 3-15)}$$

- $v$ = mean velocity of flow in feet per second
- $r$ = hydraulic radius in feet
- $s$ = slope of the energy gradient
- $s_o$ = slope of channel bottom
- $n$ = coefficient of roughness

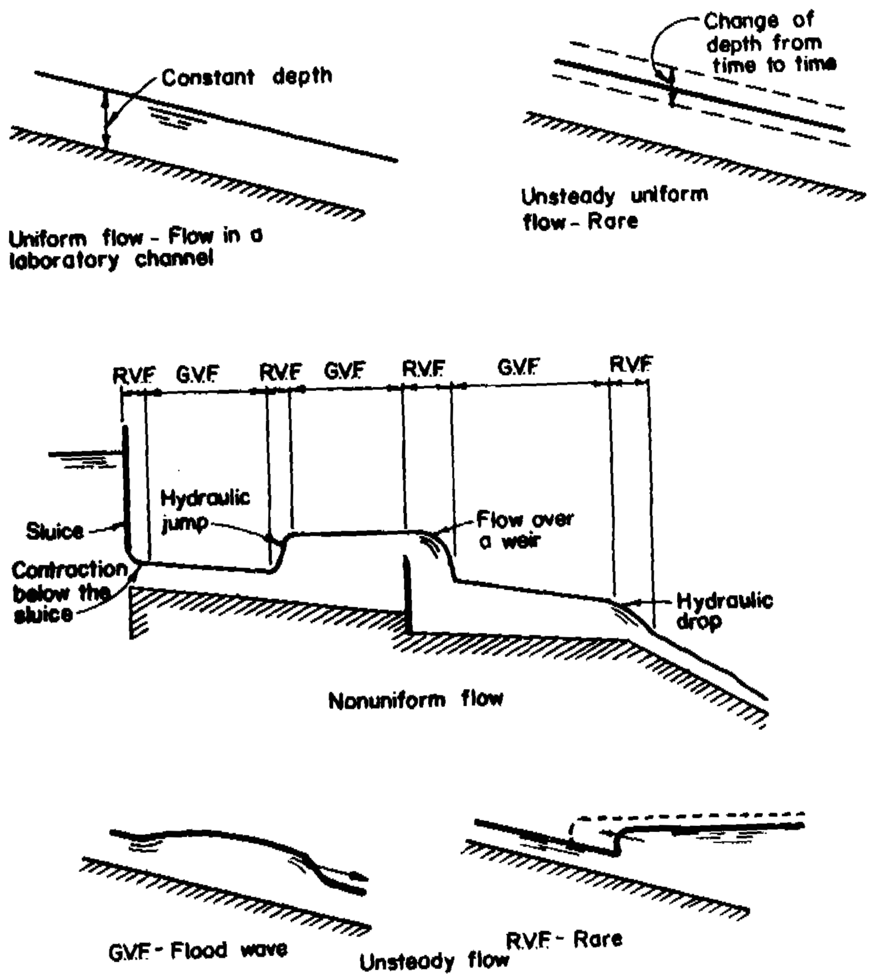



Figure 3-10 Various types of open-channel flow. G.V.F. = gradually varied flow; R.V.F. = rapidly varied flow.

Manning's equation has the advantage of simplicity and gives values of velocity consistent with experimental data. Exhibit 3-14, sheets 1 through 4, may be used to solve for v, r, s, and n when any three are known.

Since Q = av, Manning's equation may also be written:

$$Q = \frac{1.486}{n} a\ r^{2/3}\ s^{1/2} \qquad \text{(Eq. 3-16)}$$

where a = cross-sectional area in square feet.

There are many other forms of Manning's equation which are developed by algebraic changes[(1)] to solve for various elements when the other elements are known. These forms should be studied carefully. Having mastered the use of the formula, the tables, nomographs and charts can be used with confidence.

Coefficient of Roughness, n

The computed discharge for any given channel or pipe will be only as reliable as the estimated value of n used in making the computation. This estimate affects the design discharge capacity and the cost, and therefore, requires careful consideration.

In the case of pipes and lined channels, this estimate is easier to make but it should be made with care. A given situation will afford specific information on such factors as size and shape of cross section, alignment of the pipe or channel, and the type and condition of the material forming the wetted perimeter.

Knowledge of these factors, along with the results of experimental investigations and experience, makes possible selections of n values within reasonably well-defined limits.

Natural channels and excavated channels, subject to various types and degrees of change, present a more difficult problem. The selection of appropriate values for design of drainage, irrigation, and other excavated channels is covered by manual data relating to those subjects.

The value of n is influenced by several factors; those having the greatest influence are:

Physical Roughness

The types of natural material forming the bottom and sides and the degree of surface irregularity are the guides to evaluation. Soils made up of fine particles on smooth, uniform surfaces result in relatively low values of n. Coarse materials, such as gravel or boulders, and pronounced surface irregularity cause higher values of n.

Vegetation

The value of n should be an expression of the retardance to flow as it will be affected by height, density, and type of vegetation. Consideration should be given to density and distribution of the vegetation along the reach and the wetted perimeter; the degree to which the vegetation occupies or blocks the cross-sectional area of flow at different depths; and the degree to which the vegetation may be bent or the channel "shingled" by flows of different depths.

Cross Section

Gradual and uniform increases or decreases in cross-section size will not significantly affect n, but abrupt changes in size or the alternating of small and large sections call for the use of a somewhat larger n. Uniformity of cross-sectional shape will cause relatively little resistance to flow; whereas variation, particularly if it causes meandering of the major part of the flow from side to side of the channel, will increase n.

Channel Alignment

Curves with a relatively large radius and without frequent changes in direction of curvature will offer comparatively low resistance to flow. Severe meandering with the curves having relatively small radii will significantly increase n.

Silting or Scouring

Whether either or both of these processes are active, and whether they are likely to continue or develop in the future, is important. Active silting or scouring, since they result in channel variation of one form or another, will tend to increase n.

Obstructions

Log jams and deposits of any type of debris will increase the value of n; the degree of effect is dependent on the number, type, and size of obstructions.

The value of n, in a natural or constructed channel in earth, varies with the season and from year to year; it is not a fixed value. Each year n increases in the spring and summer as vegetation grows and foliage develops, and diminishes in the fall as the dormant season develops. The annual growth of vegetation, uneven accumulation of sediment in the channel, lodgment of debris, erosion and sloughing of banks, and other factors all tend to increase the value of n from year to year until the hydraulic efficiency of the channel is improved by clearing or clean-out.

All of these factors should be studied and evaluated with respect to 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. As a general guide to judgment, it can be accepted that conditions tending to induce turbulence will increase retardance. Refer to Chapters 7 and 14 of this manual for guidance in selecting retardance and n values.

## SPECIFIC ENERGY IN CHANNELS

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.

Figure 3-11 shows a section of channel in uniform flow. Here, for a given slope, roughness, cross section and rate of flow, the depth may be calculated from the Manning equation.

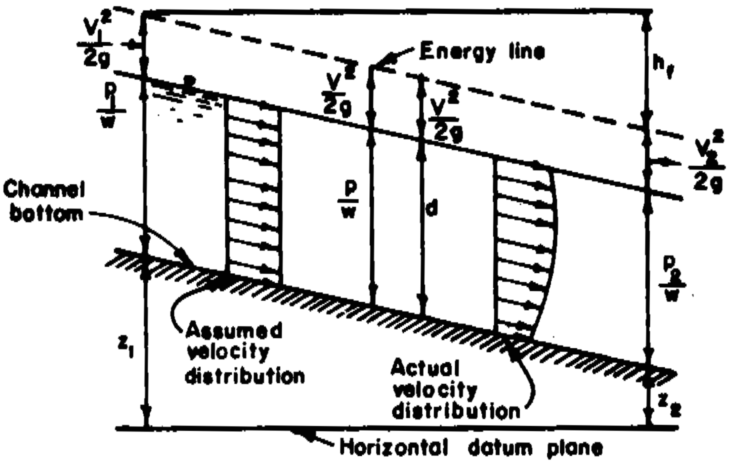



**Figure 3-11 Channel energy relationships**

Assuming a uniform velocity distribution, the Bernoulli equation may be written for a typical reach of channel as:

$$\frac{P_1}{w} + \frac{v_1^2}{2g} + z_1 = \frac{P_2}{w} + \frac{v_2^2}{2g} + z_2 + hf$$

which shows that energy is lost as flow occurs. However, the distance from channel bottom to energy line remains constant and is given by

$$H_e = \frac{P}{w} + \frac{v^2}{2g} = d + \frac{v^2}{2g} \qquad \text{(Eq. 3-17)}$$

in which $H_e$ is known as the specific energy. Obviously the specific energy in an open channel is the sum of the water depth and the velocity head.

The following section on critical flow illustrates another application of the specific energy equation to solve channel flow problems.

## CRITICAL FLOW CONDITIONS

Critical flow is the term used in open channel flow to define a dividing point between subcritical (tranquil) and supercritical (rapid) flow. At this point there exists certain relationships between specific energy and discharge and specific energy and depth of flow. As shown previously, specific energy is the total energy head at a cross section measured from the bottom of the channel. There are two conditions which describe critical flow:

1. The discharge is maximum for a given specific energy head.

2. The specific energy head is minimum for a given discharge.

Stated simply, the foregoing says that 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 the addition of specific energy.

### General Equation for Critical Flow

The general equation for critical flow in any channel is

$$\frac{Q^2}{g} = \frac{a^3}{T} \qquad \text{(Eq. 3-18)}$$

From Equation 3-18, $\frac{Q^2}{a^2} = \frac{ag}{T}$: and since $\frac{Q^2}{a^2} = v^2$ and

$a = d_m T$, the specific energy equation when flow is critical is

$$H_e = d + \frac{a}{2T} = d + \frac{d_m}{2} \qquad \text{(Eq. 3-19)}$$

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

A study of the specific energy diagram, Figure 3-12, will give a more thorough understanding of the relationships between discharge, energy, and depth when flow is critical.

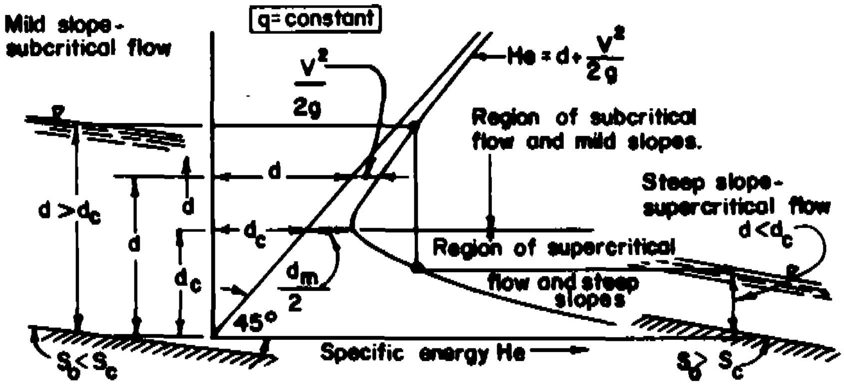



Figure 3-12 The Specific Energy Diagram

While studying this diagram, consider the following critical flow terms and their definitions:

<u>Critical Discharge</u>
The maximum discharge for a given specific energy, or a discharge which occurs with minimum specific energy.

<u>Critical Depth</u>
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.

<u>Critical Velocity</u>
The mean velocity when the discharge is critical.

<u>Critical Slope</u>
That slope which will sustain a given discharge at uniform, critical depth in a given channel.

<u>Subcritical Flow</u>
Those conditions of flow for which the depth is greater than critical and the velocity is less than critical.

<u>Supercritical Flow</u>
Those conditions of flow for which the depth is less than critical and the velocity is greater than critical.

The curve shows the variation of specific energy with depth of flow 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 3-17. Certain points, as illustrated by this curve, should be noted:

1. In a specific energy diagram the pressure head and velocity head are shown graphically. 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 $H_e = d$, to the curve of constant Q.

2. 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 stages, 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.

3. At depths of flow near the critical for any discharge, a minor change in specific energy will cause a much greater change in depths.

4. Through the major portion of the subcritical range the velocity head for any discharge is relatively small when compared to specific energy, and changes in depth are approximately equal to changes in specific energy.

5. 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.

## Instability of Critical Flow

The instability of uniform flow at or near critical depth is usually defined in terms of critical slope, $s_c$.

$s_c$ = critical slope - that slope which will sustain a given discharge in a given channel at uniform, critical depth.

The critical slope, $s_c$, is:

$$s_c = 14.56 \frac{n^2 d_m}{r^{4/3}} \qquad \text{(Eq. 3-20)}$$

Uniform flow at or near critical depth is unstable. This results from the fact that the unique relationship between energy head and depth of flow which must exist in critical flow is readily disturbed by minor changes in energy. Those who have seen uniform flow at or near critical depth have observed the unstable wavy surface that is caused by appreciable changes in depth resulting from minor changes in energy. This unstable range is defined as follows:

Unstable zone in the range $0.7s_c < s_o < 1.3s_c$

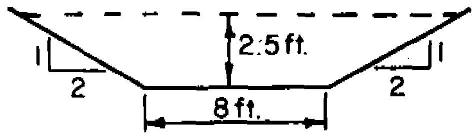
where $<$ = the symbol for "is less than"

Because of the unstable flow, channels carrying uniform flow at or near critical depth should not be used unless the situation allows no alternative. In this case allowance must be made in design for the height of the wave generated. Often when topography restricts the channel slope the flow can be forced into subcritical stable or supercritical stable by varying the width of the channel.

Open Channel Problems

Example 3-19

Given: Trapezoidal section




n = 0.02
s = 0.006

To determine: Q in cfs, and v in fps

Solution: from Exhibit 3-13

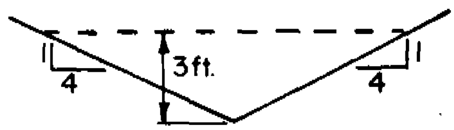
$$r = \frac{a}{p} = \frac{bd + zd^2}{b + 2d\sqrt{z^2 + 1}} = \frac{8(2.5) + 2(2.5)^2}{8 + 2(2.5)\sqrt{(2)^2 + 1}} = 1.695$$

enter Exhibit 3-14 with r = 1.695, s = 0.006, n = 0.02, and read v = 8.19 fps

then Q = av = 32.5(8.19) = 266 cfs

Example 3-20

Given: Triangular section




n = 0.025
s = 0.006

To determine: Q in cfs and v in fps

Solution: from Exhibit 3-13

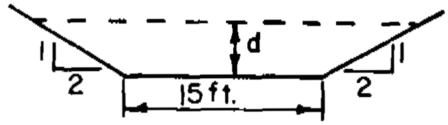
$$r = \frac{a}{p} = \frac{zd}{2\sqrt{z^2 + 1}} = \frac{4(3)}{2\sqrt{(4)^2 + 1}} = \frac{12}{8.246} = 1.455$$

enter Exhibit 3-14 with r = 1.455, s = 0.006, n = 0.025 and read v = 5.91 fps

then Q = av = 36(5.91) = 213 fps

Example 3-21

Given: Trapezoidal section



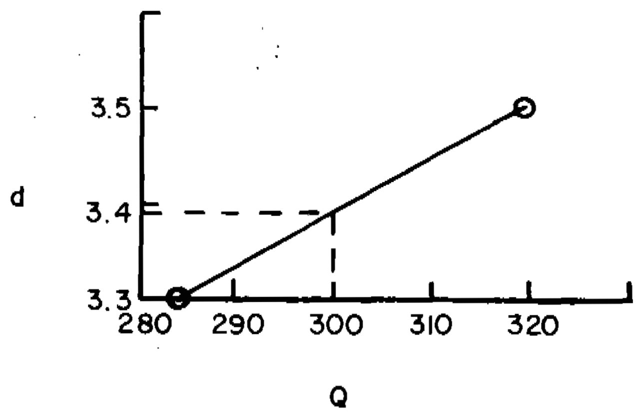

Q = 300 cfs
n = 0.02
s = 0.0009

To determine: d in ft. and v in ft/sec.

Solution: This can be solved by trial. First, assume a value for d and compute the values of a, p, r; then from Exhibit 3-14 find v and compute Q.

| Trial | d | a | p | r | v | Q |
|---|---|---|---|---|---|---|
| 1 | 3.0 | 63.0 | 28.42 | 2.21 | 3.80 | 239.4 |
| 2 | 3.5 | 77.0 | 30.65 | 2.51 | 4.14 | 318.8 |
| 3 | 3.3 | 71.3 | 29.76 | 2.39 | 3.98 | 283.8 |

Plot d against Q for trial 2 and 3 and read d = 3.39 ft., for Q = 300 cfs



For those having much of this work to do, the use of tables in King's Handbook, based on the equation $K' = \dfrac{Qn}{b^{8/3}\ s^{1/2}}$, will provide rapid direct solutions, i.e.,

$$b = 15^{8/3} = 1370 \text{ and } s^{1/2} = .009^{1/2} = 0.03$$

$$\text{then } K' = \frac{(300)(.02)}{(1370)(.03)} = 0.146$$

From the table of K' values for 2:1 sides and K' = 0.146

$$\frac{D}{b} = 0.226$$

$$D = (15)(0.226) = 3.39 \text{ feet}$$

The same procedures can be followed in solving for triangular sections.

## 7. WEIR FLOW

A weir is a notch of regular form through which water flows. The structure containing the notch is also called a weir. The edge over which the water flows is the crest of the weir. Two types of weir crests are common in soil conservation work, sharp-crested weirs and broad-crested weirs.

The sharp-crested weir is used only to measure the discharge of a channel or stream. The sharp edge causes the water to spring clear of the crest.

Most hydraulic structures in soil conservation work have broad-crested weirs. The crest is horizontal and long in the direction of flow so that the water lays on the crest rather than springing clear. The primary use of the broad-crested weir is for the control of flood flows, although water measurement can be incorporated as a secondary function. Chapter 6, Structures, of this manual describes in detail its many applications.

Examples of the two types of weir crest are shown in Figure 3-13 and 3-14.

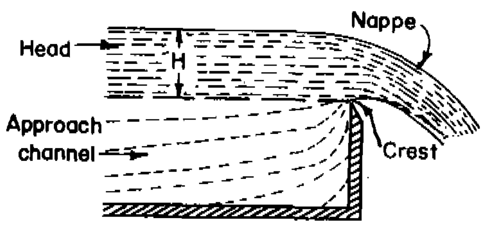



**Figure 3-13 Sharp-crested weir**

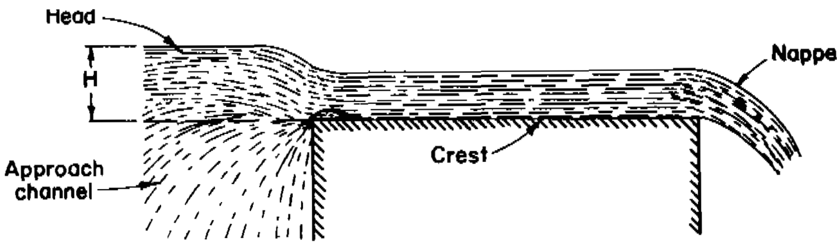



Figure 3-14 Broad-crested weir

If the overflowing sheet of water (nappe) discharges into the air, as above, the weir has free discharge. If the discharge is partially under water, as shown in Figure 3-15, the weir is submerged or drowned.

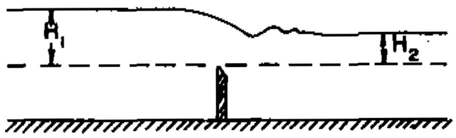



Figure 3-15 Submerged weir

BASIC EQUATION

The basic equation for all weirs is:

$$Q = CLH^{3/2} \qquad \text{(Eq. 3-21)}$$

where:

Q = discharge
H = measured head
L = length of weir
C = weir coefficient

Corrections are required to include effects of end contractions, velocity of approach and submergence.

CONTRACTIONS

The weir is contracted when the respective distances from the sides and the bottom of the channel of approach to the sides and crest of the weir are great enough to allow the water free lateral approach to the crest. If the weir conforms to the sides of the approach channel above the crest and the channel sides extend downstream beyond the crest, thereby preventing lateral expansion of the nappe, the weir has end contractions suppressed.

End contractions reduce the effective length of a weir. To allow for end contractions, the length of weir in the basic equation is adjusted as follows:

$$L = L' - 0.1NH \qquad \text{(Eq. 3-22)}$$

where:

L = effective length of weir
L' = measured length of weir
N = number of contractions

The above equation is generally applied only for sharp-crested weirs in soil conservation work. For most broad-crested weirs the end contractions will be either fully or partially suppressed. For drop spillways the basic formula can be used without modifying for contraction effect.

VELOCITY OF APPROACH

The velocity of approach is the average velocity in the approach channel. It is measured at a distance of about 3H upstream from the weir. The velocity head is added to the measured head to determine the discharge. Therefore:

$$Q = CL\left(H + \frac{v^2}{2g}\right)^{3/2} \qquad \text{(Eq. 3-23)}$$

WEIR COEFFICIENTS

Values of the weir coefficient, C, varies with the type of crest used. Weir coefficients for sharp-crested weirs normally used in soil conservation work are given under the following section on water measurement. The weir coefficient for broad-crested weirs commonly used in soil conservation work is C = 3.1.

SUBMERGED FLOW

When a weir is submerged, Figure 3-15, the discharge will be less than for the head in free flow. The reduction in flow can be expressed in terms of the ratio of the upstream head, $H_1$, to the downstream head, $H_2$.

When the ratio of $H_2/H_1$ for sharp-crested weirs reaches 0.3 to 0.4, the discharge may be reduced by 5 to 10 percent. For $H_2/H_1$ ratios of 0.6 to 0.7, reductions of 20 to 40 percent may be expected.

There is no great reduction in discharge for broad-crested weirs until the ratio of $H_2/H_1$ reaches 0.67. Then the discharge reduces rapidly as the submergence increases. For values of $H_2/H_1$ from 0.75 to 0.85, reductions from 10 to 30 percent occur.

When appreciable submergence is to be encountered, accurate discharge can be computed only through the use of refined procedures.

## 8. WATER MEASURING

The purpose of this section is to outline the most commonly used water measuring methods. The hydraulic principle and equation of flow is given for most methods. For convenience, the subject is divided into open channel flow and pipe flow.

### OPEN CHANNELS

The following methods of measurement apply to flows in open channels.

### Orifices

An orifice is a hole of regular form through which water flows.

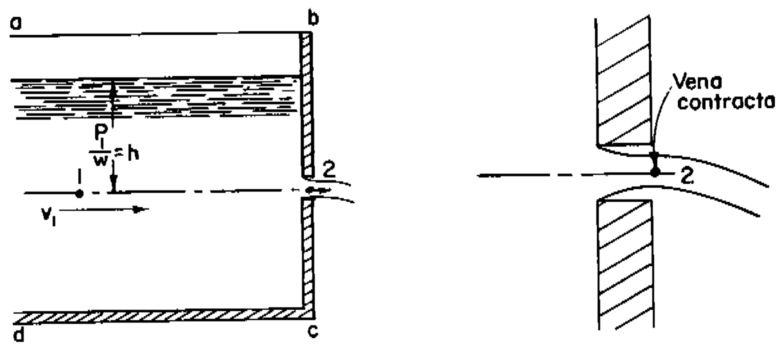



Figure 3-16 Flow through an orifice

Flow through an orifice is illustrated in Figure 3-16. The orifice shown is sharp-edged; i.e., it has a sharp upstream edge so that the water in passing touches only a line. If the orifice discharges into the air, it is said to have free discharge; and if it discharges under water, it is said to be submerged. Orifices may be circular, square, rectangular, or of any other regular form.

The Bernoulli equation written from point 1 to point 2, Figure 3-16, is

$$\frac{v_1^2}{2g} + \frac{P_1}{w} = \frac{v_2^2}{2g} + \frac{P_2}{w} + h_\ell$$

and

$$v2 = \sqrt{2g\left(\frac{P_1}{w} - \frac{P_2}{w} + \frac{v_1^2}{2g} - h_\ell\right)}$$

Point 2, located where the jet has ceased to contract is known as the vena contracta. Its pressure is that of the surrounding fluid. For discharge into the atmosphere $P_2$ is therefore zero on the gage scale. For large tanks, $v_1$ is so small that it may be neglected. Replacing $P_1/w$ with h and dropping the subscript of $v_2$, the equation may now be written

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

Neglecting energy losses, the equation for the theoretical velocity, $v_t$, becomes

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

The energy loss may be taken care of by applying a coefficient of velocity $C_v$ to the theoretical velocity as follows:

$$v = C_v \sqrt{2gh} \qquad \text{(Eq. 3-24)}$$

The discharge through an orifice is obtained from the product of the velocity and the area at the vena contracta. The area at the vena contracta, $a_2$, is less than the area of the orifice, a. The ratio between the two areas is called the coefficient of contraction $C_c$. Therefore,

$$a_2 = C_c a$$

and

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

The product of $C_c$ and $C_v$ is called the coefficient of discharge C. Equation for discharge may therefore be written

$$Q = Ca \sqrt{2gh} \qquad \text{(Eq. 3-25)}$$

The submerged orifice in Figure 3-17 is the type most often used in soil conservation work.

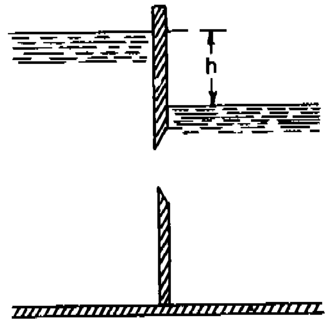



Figure 3-17 Submerged orifice

As in the free discharge orifice, Equations 3-24 and 3-25 also apply to the submerged orifice.

The coefficient of discharge for submerged orifices is approximately the same as that for free discharge orifices. The wide range in types of orifices and gates, a special form of orifice, makes it impractical to include tables of coefficients covering an adequate range of conditions. Manufacturers' publications are normally the best source of reliable coefficients for various types of gates and other appurtenances involving orifice flow.

Weirs

Sharp-crested weirs are used extensively for measuring the flow of water. The most common types--rectangular, Cipolletti, and 90° V-notch weirs--are shown in Figure 3-18.

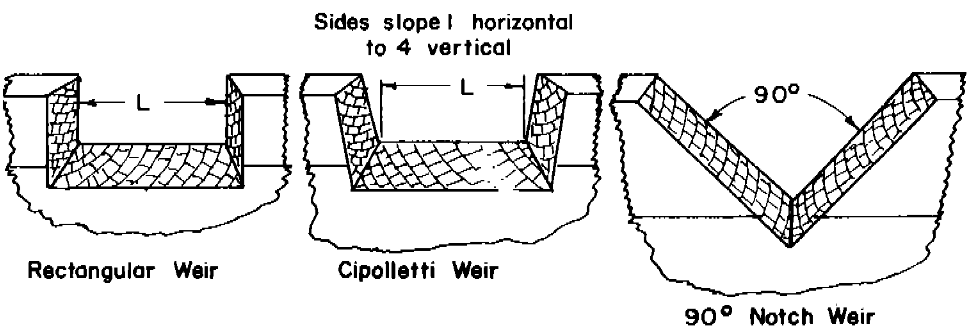



Figure 3-18 Types of weirs

As given in the preceding section, Weir Flow, the basic equation is

$$Q = CLH^{3/2}$$

Measurements made by means of a weir are accurate only when the weir is properly set and the head read at a point some distance above the crest so that the reading will not be affected by the downward curve of the water. See Figure 3-19. The weir should be at right angles to the stream at a point where the channel is straight, free from eddies and of sufficient width to produce full end contractions. The crest of the weir must be exactly level for the rectangular and Cipolletti types. The bottom of the notch must be set above the bottom of the channel a height equal to at least twice the maximum head, preferably more. If a weir is to continue to give reliable results, it must be maintained in such a way as to preserve the above-mentioned conditions.

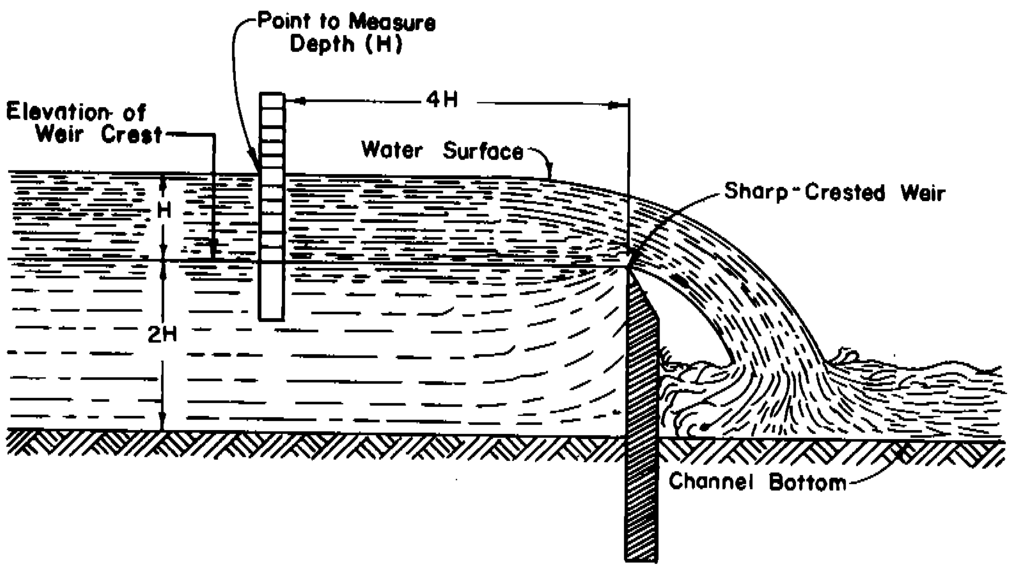



Figure 3-19 Profile of a sharp-crested weir

Rectangular Contracted Weir

A rectangular contracted weir has its crest and sides so far removed, respectively, from the bottom and sides of the weir box or channel in which it is set, that full contraction, or reduced area of flow, is developed. See Figure 3-20.

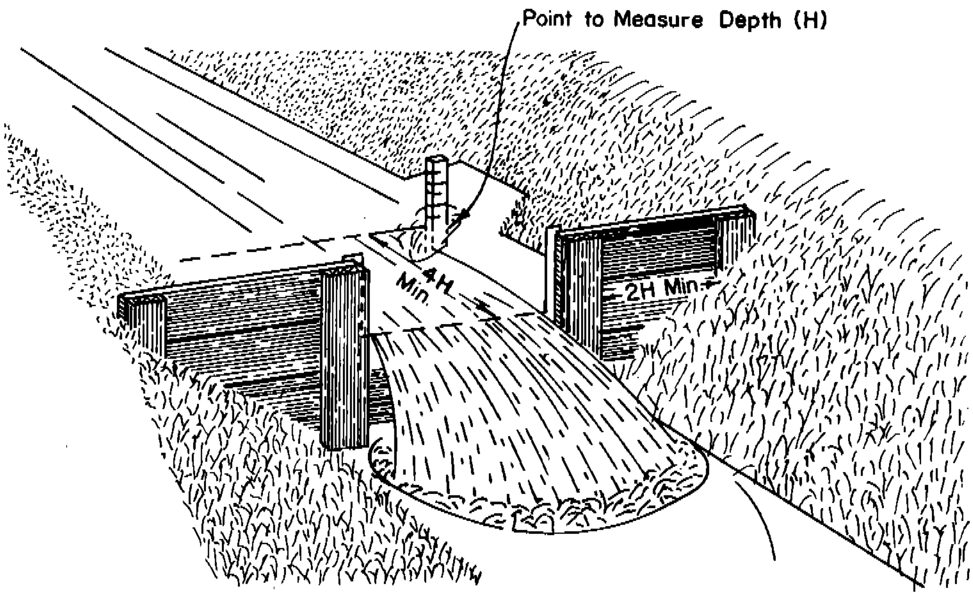



Figure 3-20 Rectangular contracted weir

For the rectangular weir the coefficient, C, is 3.33. Allowing for the contractions

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

where:

Q = discharge in cubic feet per second neglecting velocity of approach
L = the length of weir, in feet
H = head on the weir in feet measured at a point no less than 4 H upstream from the weir.

Discharges may be taken from Exhibit 3-15.

Rectangular Suppressed Weir

A rectangular suppressed weir has its crest so far removed from the bottom of the approach channel that full crest contraction is developed. The sides of the weir coincide with the sides of the approach channel which extend downstream beyond the crest and prevent lateral expansion of the nappe. A suppressed weir in a flume drop is illustrated in Figure 3-21.

Special care should be taken with this type of weir to provide aeration beneath the overflowing sheet at the crest. This is usually done by venting the underside of the nappe to the atmosphere at both sides of the weir box.

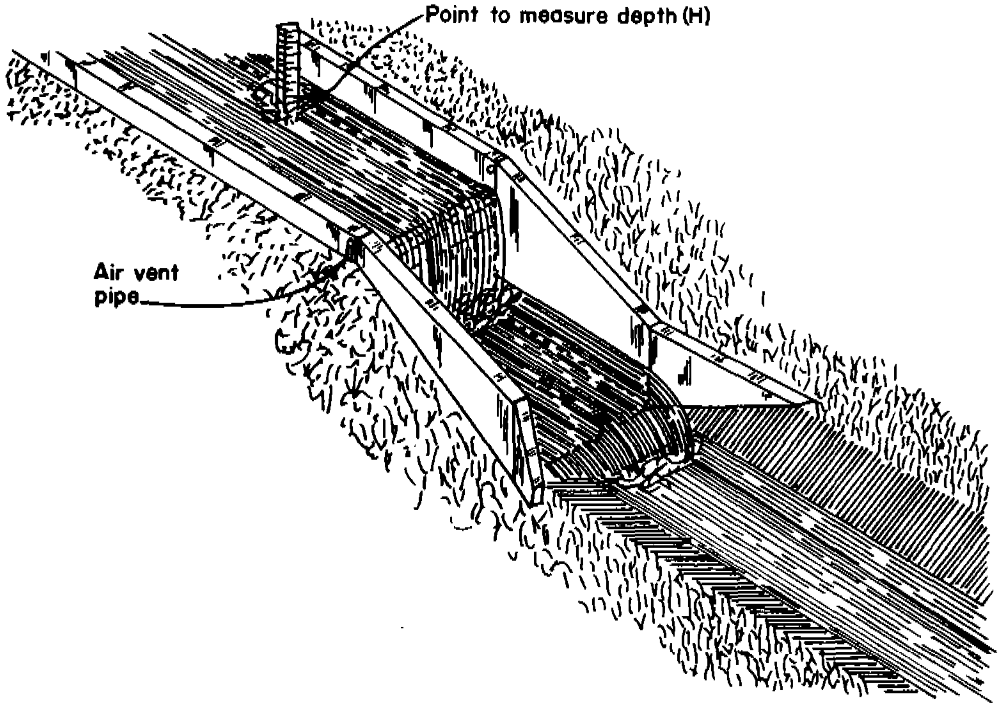



**Figure 3-21** Suppressed weir in a flume drop

The discharge equation for the rectangular suppressed weir is

$$Q = 3.33\ L\ H^{3/2}$$

and including the velocity of approach

$$Q' = 3.33\ L \left( (H + h_v)^{3/2} - h_v{}^{3/2} \right)$$

$$\text{where } h_v = \frac{(\text{velocity of approach})^2}{2g}$$

Discharge tables are available in the Bureau of Reclamation Water Measurement Manual, Table 8, pages 177 to 179.

## Cipolletti Weir

A Cipolletti weir, Figure 3-22, is trapezoidal in shape. Its crest and sides, which are of thin plate, are so far removed 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.

Since the Cipolletti weir is a contracted weir, it should be installed accordingly. However, its discharge is essentially as though its end contractions were suppressed. The effect of end contractions in reducing discharge has been overcome by sloping the sides of the weir.

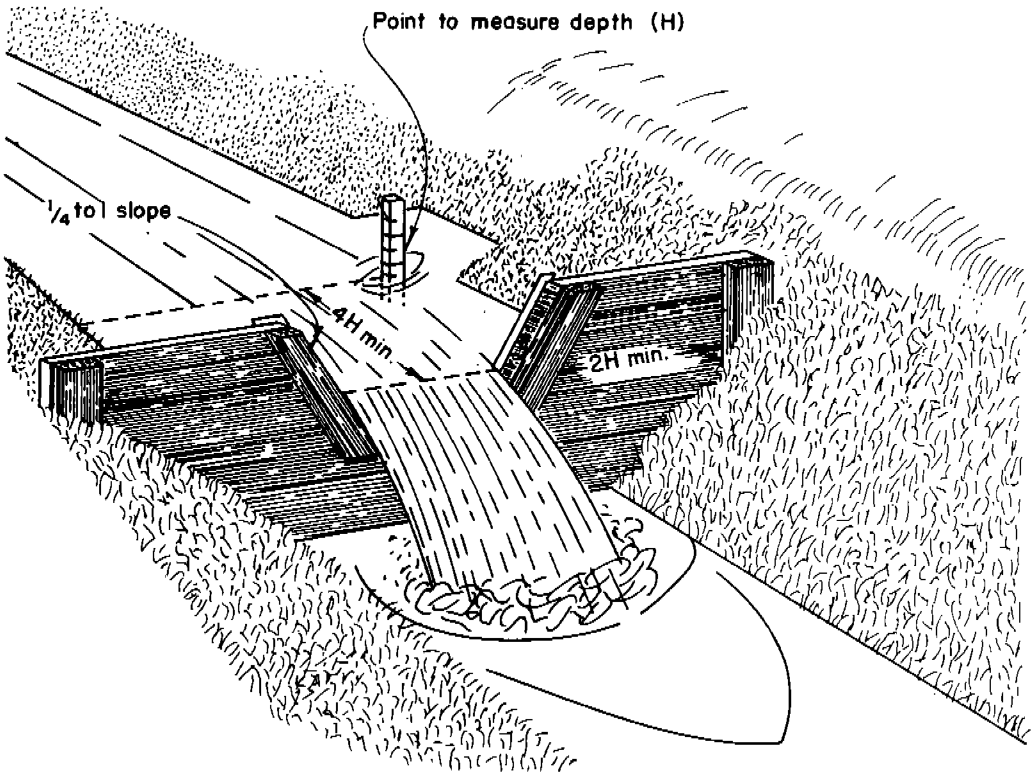



Figure 3-22 Cipolletti weir

The equation generally accepted for computing the discharge through Cipolletti weirs with complete contractions is:

$$Q = 3.367\ L\ H^{3/2}$$

The selected length of notch (L) should be at least 3H and preferably 4H or longer.

Discharges may be taken from Exhibit 3-16.

90° V-Notch Weir

The crest of the 90° V-notch weir consists of a thin plate, the sides of the notch being inclined 45° from the vertical. 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 notch and the channel floor.

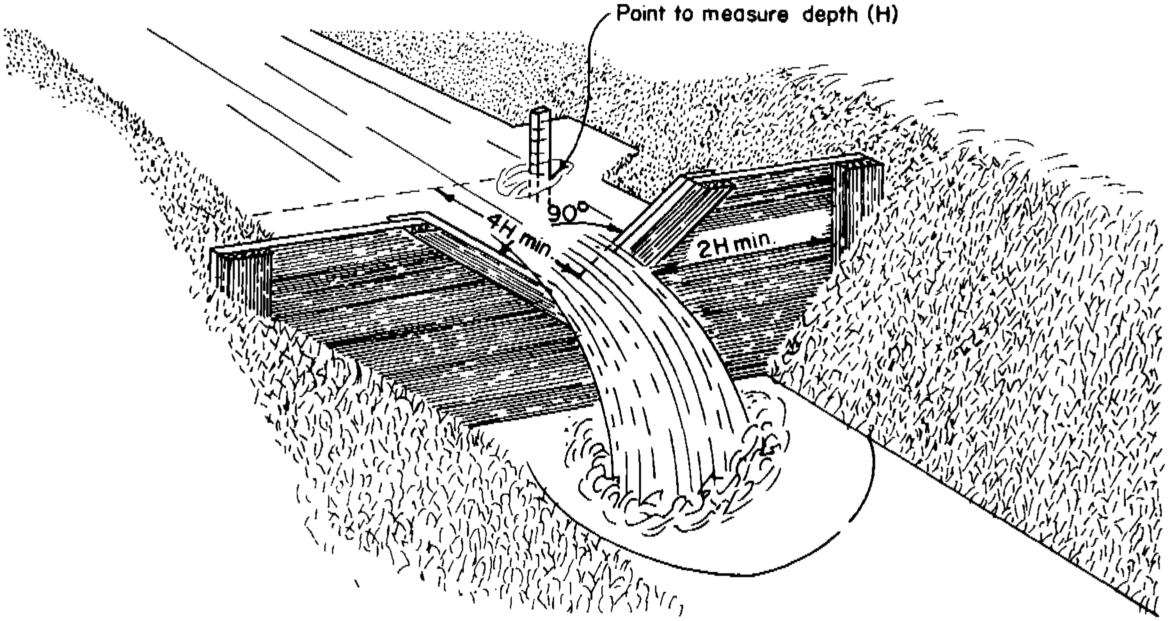



Figure 3-23 90° V-notch weir

The V-notch weir, Figure 3-23, is especially useful for measuring small discharges. The discharge equation used is

$$Q = 2.52H^{2.47}$$

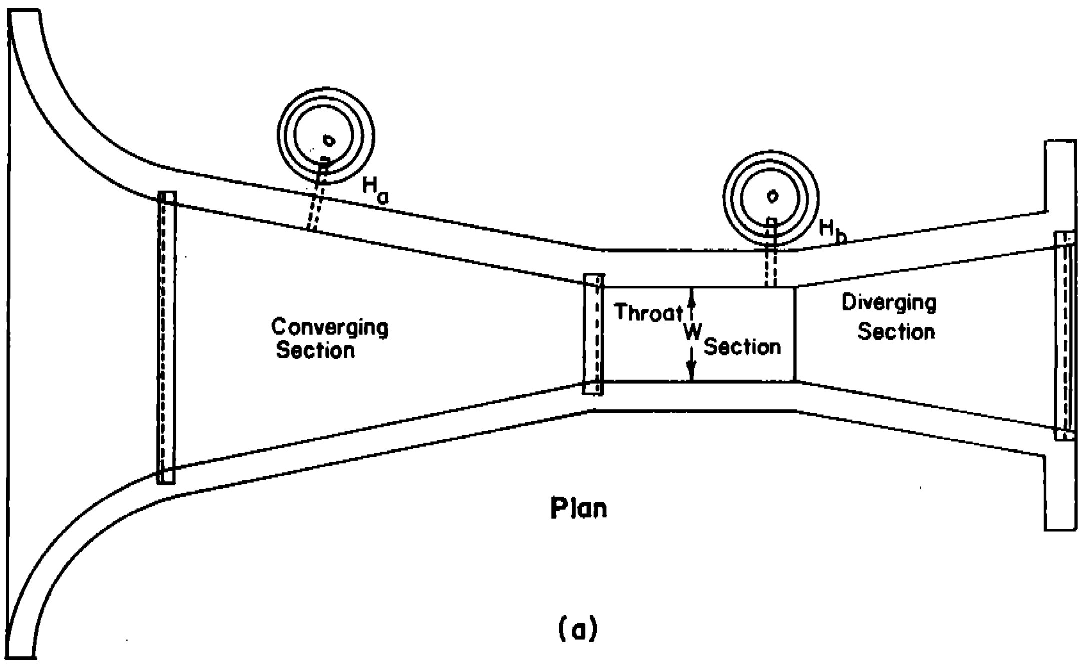
where

H = vertical distance in feet between the elevation of the vortex or lowest part of the notch and the elevation of the weir pond.

Exhibit 3-17 may be used for determining the discharge.

Parshall Flume

With a Parshall flume, Figure 3-24, 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.



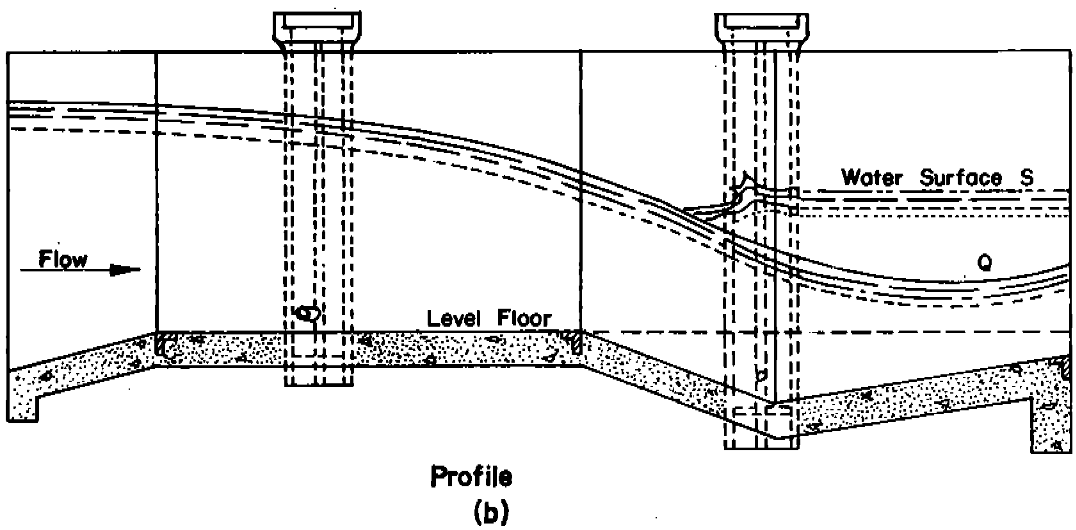




Figure 3-24 Parshall flume

There is no need of a pond above the Parshall flume as the velocity of approach has little effect on the accuracy of the measurement. It uses a small amount of head and can have a high degree of submergence without having to make corrections in the free-flow formula. It does not clog readily with floating trash and keeps itself clean of sand and silt. It requires but one reading of head ($H_a$) for determining the discharge except in cases of extreme submergence when both $H_a$ and $H_b$ should be read.

Table 9-14, Chapter 9, Section 15, of the National Engineering Manual gives free-flow discharge values for the Parshall flume.

## Trapezoidal Flume

The trapezoidal flume, Figure 3-25, obtains the discharge by measuring the loss in head caused by forcing a stream of water through the throat of a flume with a level bottom. See Figure 3-25.

An advantage of the trapezoidal flume is that the cross section corresponds to the shape of the common irrigation channel. It is particularly suited for use in concrete lined irrigation channels.

The equation for discharge derived by Robinson and Chamberlain (Transactions of the ASAE, 1960) is

$$Q = C'a_1\sqrt{2g(h_1-h_2)} \qquad \text{(Eq. 3-29)}$$

where:

$Q$ = discharge, cfs
$a_1$ = area at the flume entrance, sq.ft.
$h_1$ = head at the entrance, ft.
$h_2$ = head at the throat, ft.
$C'$ = discharge coefficient that includes the geometry of the structure

Further information on the trapezoidal flume can be found in USDA Agricultural Research Service Bulletin 41-140, dated March 1968.

## Current Meter

Basically, the current meter is a wheel having several cups or vanes. This wheel is rotated by the action of the current and the speed of the rotating wheel indicates the velocity of the current, based on a rating table furnished by the manufacturer.

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

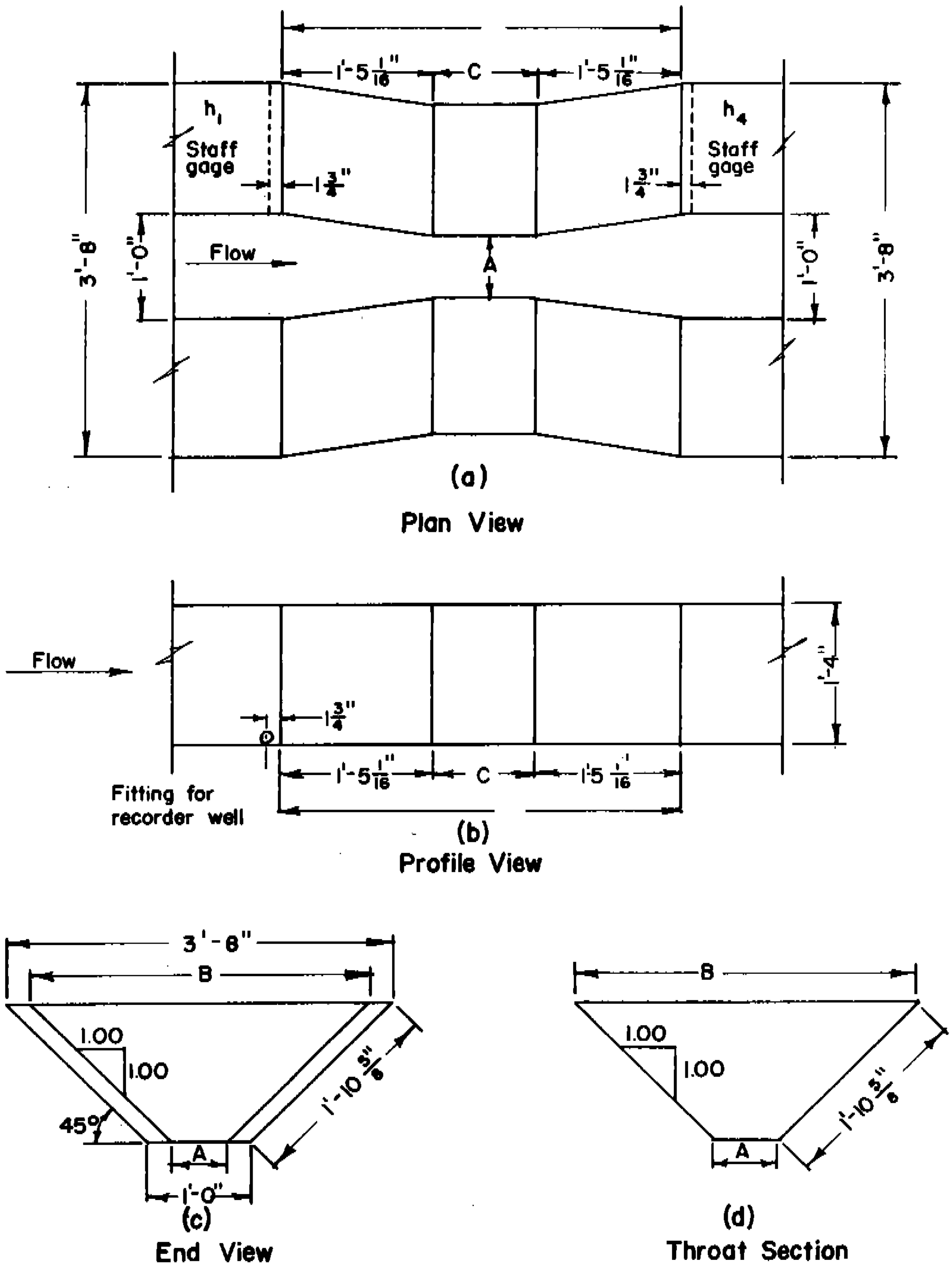



Figure 3-25 Trapezoidal flume

A common method used to determine mean velocity requires that readings be taken at only two points in each vertical; namely, 0.2 and 0.8 of the sounded depth measured from the water surface. The average of these two readings is the mean velocity in the vertical. Where the depth is too shallow to obtain two readings, one reading taken at 0.6 depth will represent the mean velocity.

The discharge of each segment of stream between adjacent verticals is the product of the area of the segment and the mean velocity in the segment. If $d_1$ and $d_2$ represent the depths of flow 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 as follows:

$$Q = W\left(\frac{d_1 + d_2}{2}\right)\left(\frac{v_1 + v_2}{2}\right)$$

The total discharge of the stream is the sum of such computations for the entire cross section.

See Reference (1) for more detailed information.

## Water-Stage Recorder

A water-stage recorder combines a clock and an instrument that draws a graph representing the rise and fall of a water surface with respect to time. Water-stage recorders are in common use at permanent gaging stations.

Water-stage recorders are desirable under the following conditions:

1. The flow in the stream or channel fluctuates rapidly, and occasional staff-gage readings would not give a satisfactory estimate of discharge.

2. The gaging station is hard to get to, or the available observers are not reliable.

3. There is a need for continuous records of flow for legal or technical purposes.

By the combined use of the stage-discharge curve (Figure 3-26) and the water-stage recorder, a hydrograph of the flow in a stream or channel may be plotted. A hydrograph is a curve 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.

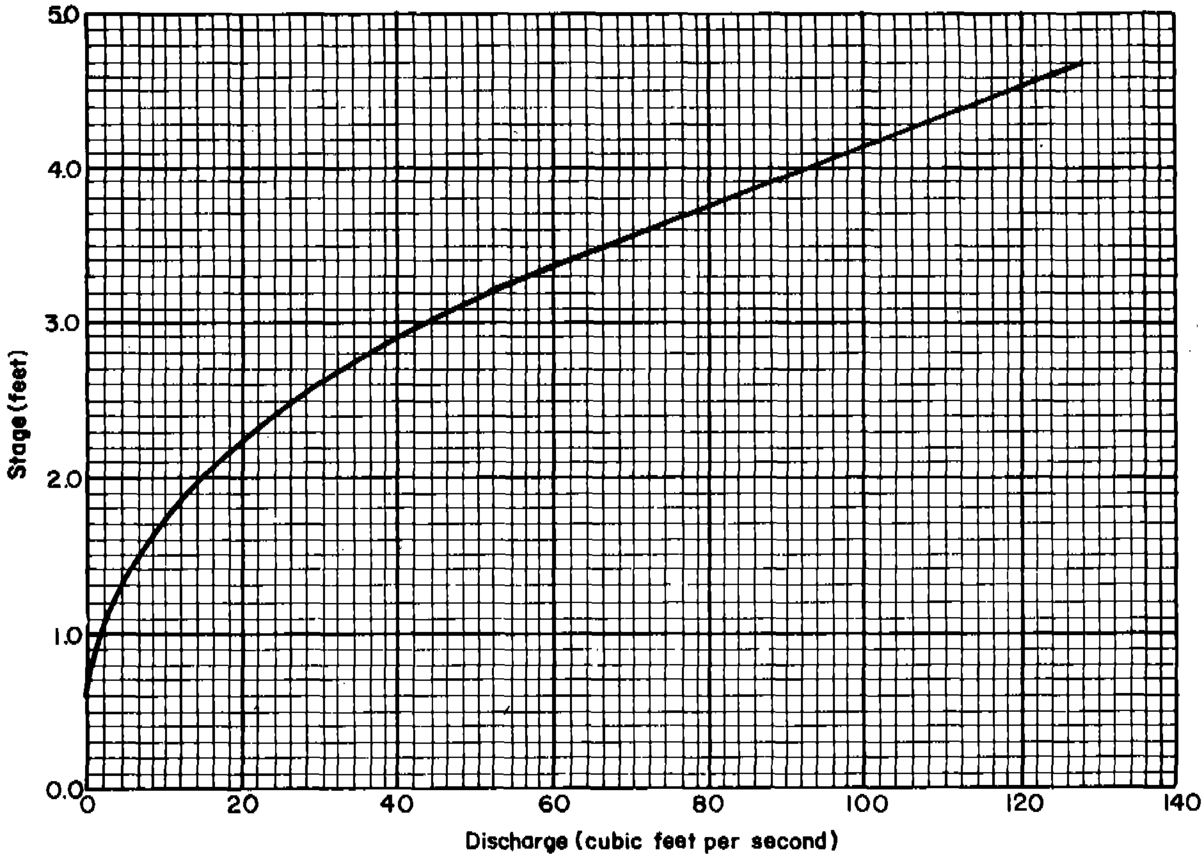



Figure 3-26 Stage-discharge curve for unlined irrigation canals

Measurements by Floats

The velocity of a canal or stream, and hence its discharge, may be determined approximately by the use of surface floats and channel cross sections.

A stretch of the canal, straight and uniform in cross section and grade, with a minimum of surface waves, should be chosen for this method. Surface velocity measurements should be made on a windless day, for even under the best conditions the floats often are diverted from a direct course between measuring stations.

The width of the canal should be divided into segments, and the average depth determined for each segment. The segments should be narrower in the outer thirds of the canal than in the central third. Float courses should be laid out in the middle of the strips defined by the segments. For regular-shaped channels flowing in a straight course under favorable conditions, the mean velocity of a strip in the channel is approximately 0.85 times its surface velocity. This value is an average of many observations. For any particular channel it may be as low as 0.80 or as high as 0.95

The velocity of the float in each strip, after being adjusted to mean velocity, multiplied by the cross-sectional area of the strip, will give 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 of these used for the surface velocity of the stream. The float method is an approximate method and should be used only with its limitations in mind.

## Slope-Area Method

The slope-area method consists of using the slope of the water surface in a uniform reach of channel, and the average cross-sectional area of that reach, to give a rate of discharge. The discharge may be computed from Manning's Equation 3-16:

$$Q = \frac{1.486}{n} ar^{2/3}s^{1/2}$$

A straight course of the channel should be chosen, at least 200 feet and preferably 1,000 feet in length. The course should be free of rapids, abrupt falls, sudden contractions, or expansions.

The slope of the water surface may be calculated by dividing the difference in the water surface elevations at the two ends of the course by the length of the course. If it is desired to develop a stage-discharge curve for the channel, gage points, carefully referenced to a common datum level, should be placed one on each bank of the channel and one in the center of the stream, in stilling-wells if possible.

In irregular channels, the area and the wetted perimeter at several cross sections is required and a mean value should be used in computing hydraulic radius.

Inasmuch as the proper selection of the roughness factor n for many streams is difficult, the discharge determined by the slope-area method is only approximate. Care must be taken to determine the slope and areas simultaneously when the water levels are changing. Various hydraulic textbooks and handbooks provide tables to assist in the computation of discharges from the above field data.

## Velocity-Head Rod

The velocity-head rod is a simple inexpensive rod that can be used to measure the approximate velocity in open channels, if depths and velocities are not too great. This infrequently-used method is discussed in detail in Chapter 9, Section 15, SCS National Engineering Handbook.

## PIPE FLOW

The following methods may be used in determining pipe flow:

### Orifice Flow

Pipe orifices, Figure 3-27, usually are circular orifices placed in or at the end of a horizontal pipe. The head on the orifice is measured with a manometer.

Where the orifice is placed in the pipe, the discharge will not be free and the head must be measured at points both upstream and downstream from the orifice. For a further discussion of this type of orifice, refer to King's Handbook of Hydraulics.

The pipe orifice commonly used in measuring irrigation water and the discharge from wells within a range of 50 to 2,000 gallons per minute has the circular orifice located at the end of the pipe.

The pipe must be level. A glass-tube manometer is placed about 24 inches upstream from the orifice. No elbows, valves, or other fittings should be closer than 4 feet upstream from the manometer. The ratio of the orifice diameter to the pipe diameter should be no less than 0.50 nor greater than 0.83. The ratio to be selected, however, must cause the pipe to flow full. The head in the manometer is measured with an ordinary carpenter's rule.

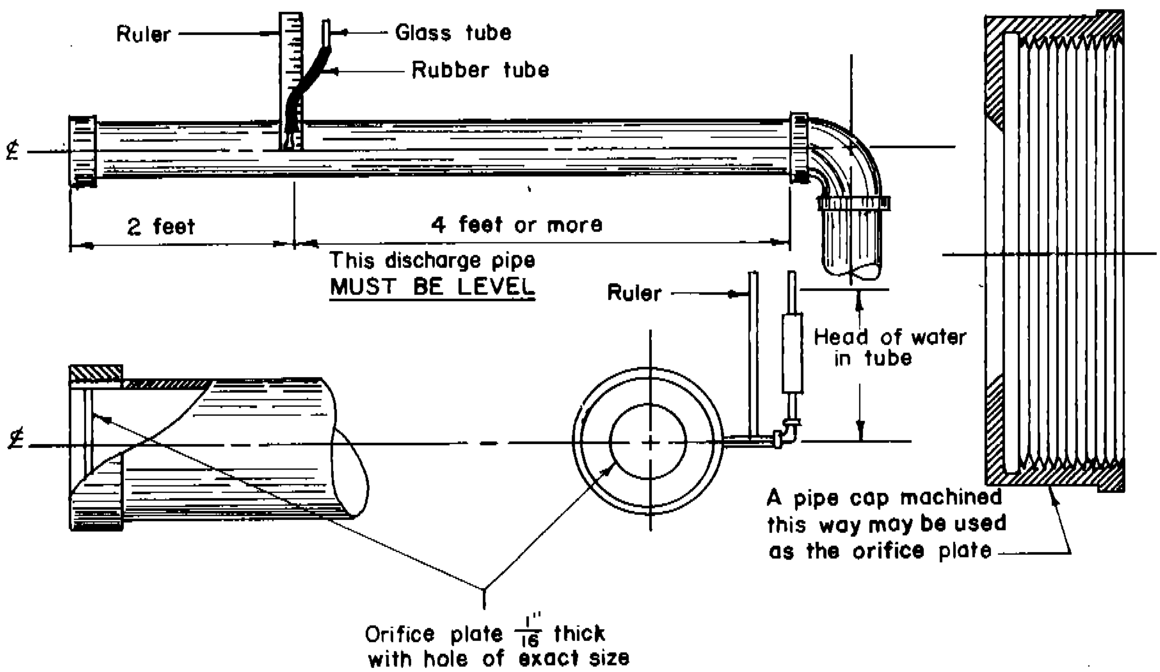



Figure 3-27 Pipe orifice

Discharge through the orifice is computed by Equation 3-25, or it can be read directly from Exhibit 3-18.

$$Q = Ca\sqrt{2gh}$$

where:

Q = orifice discharge in gallons per minute
C = coefficient which varies with the ratio of the orifice diameter to the pipe diameter as well as with all the other factors affecting flow in orifices. The value of the coefficient (C) may be taken from Figure 3-28
a = cross-sectional area of the orifice in square inches
g = acceleration due to gravity - 32.2 feet per second per second
h = head on the orifice in inches measured above its center

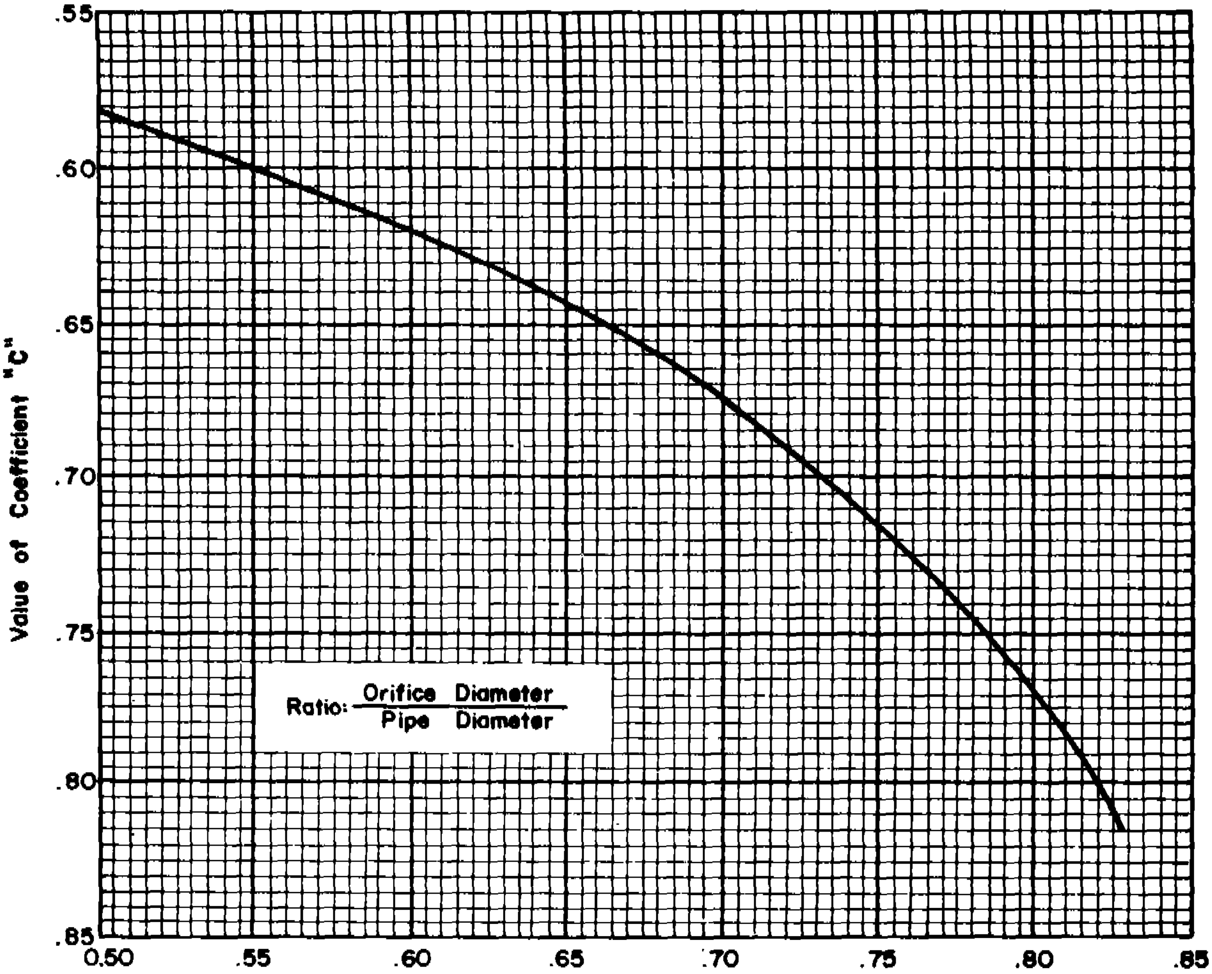



Figure 3-28 Orifice coefficients

California Pipe Method

B. R. Vanleer developed a method for measuring 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.

The method has four requirements for accurate results: 1) The discharge pipe must be level; 2) it must discharge partially full; 3) it must discharge freely into air; and 4) the velocity of approach must be a minimum. Figure 3-29 illustrates one method of meeting these requirements. 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 the equation:

$$Q = 8.69 \left(1 - \frac{a}{d}\right)^{1.88} d^{2.48} \qquad \text{(Eq. 3-30)}$$

where Q = discharge in cubic feet per second
a = distance, in feet, measured at the end of the pipe, from the top of the inside surface of the pipe to the water surface
d = internal diameter of the pipe in feet.

This equation was developed from experimental data for pipes 3 to 10 inches in diameter. In tests made by the Soil Conservation Service it was discovered that for depths greater than one-half the diameter of the pipe, or a/d less than 0.5, the discharges did not follow the equation. Extreme care should therefore be taken in using the equation and tables for conditions where a/d is less than 0.5.

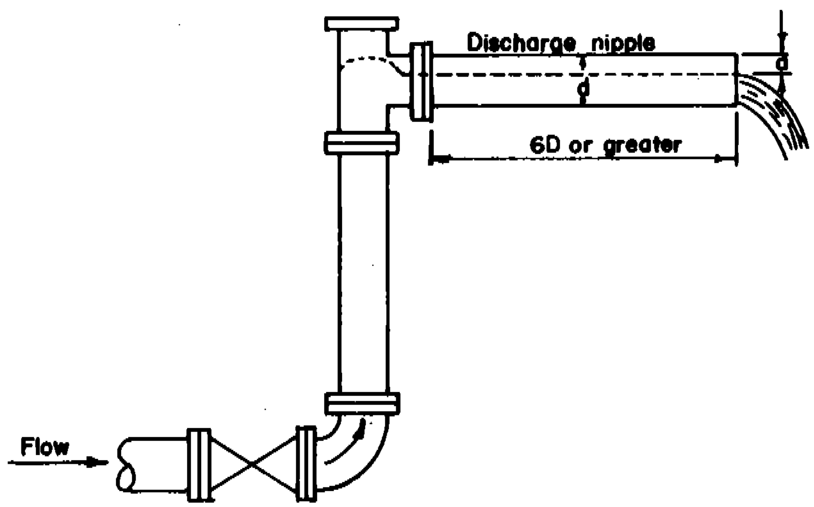



Figure 3-29 Measuring flow by the California pipe method

Tables 48 and 49 in the Water Measurement Manual, Bureau of Reclamation, provide assistance for solving the equation for discharge.

Coordinate Method

In this method, coordinates of the jet issuing from the end of a pipe are measured. The flow from pipes may be measured whether the pipe is discharging vertically upward, horizontally, or at some angle with the horizontal. Since the discharge pipe can be set in a horizontal position for measurement purposes, there is no need here for a discussion of flow from pipe in an angular position.

Coordinate methods are used to measure the flow from flowing wells (discharging vertically) and from small pumping plants (discharging horizontally). These methods have limited accuracy owing to the difficulty in making accurate measurements of the coordinates of the jet. They should be used only where facilities for making more accurate measurements by other methods are not available and where an error of up to 10 percent is permissible.

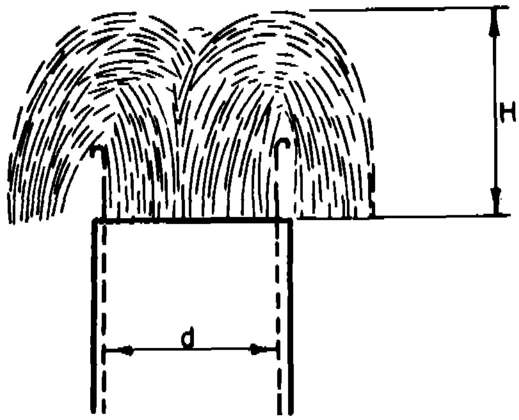



Figure 3-30 Required measurements to obtain flow from vertical pipes

To measure the flow from pipes discharging vertically upward, it is necessary to measure only the inside diameter of the pipe and the height of the jet above the pipe outlet (H), Figure 3-30. Exhibit 3-19 gives discharge values for pipe diameters up to 12 inches and jet heights up to 40 inches.

To measure the flow from pipes discharging horizontally, it is necessary to measure both a horizontal and a vertical distance from the top of the inside of the pipe to a point on the top of the jet. See Figure 3-31. These horizontal and vertical distances are called X and Y ordinates, respectively.

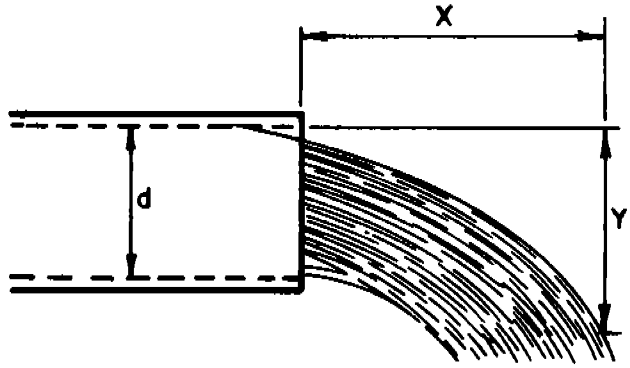



Figure 3-31 Required measurements to obtain flow from horizontal pipes

For reasonably accurate results, the discharge pipe must be level and long enough to permit the water to flow smoothly as it issues from the pipe. Exhibit 3-20 gives discharge values for pipe diameters up to 6 inches where the ordinate X is selected to be 0, 6, 12, or 18 inches. For pipes flowing less than 0.8 full at the end, the vertical distance Y can be measured at the end of the pipe where X = 0. Exhibit 3-20 is used to obtain the discharge. Exhibit 3-20 is also applicable either to conditions of full flow or partial flow.

| THIS | TIMES THIS | GIVES YOU THIS |
|---|---|---|
| **VOLUME** | | |
| 1 gallon (gal) | 231 | cubic inches (cu in.) |
| | .1337 | cubic feet (cu ft) |
| 1 million gallon (mg) | 3.0689 | acre feet (acre-ft) |
| 1 cubic foot | 1,728 | cubic inches |
| | 7.48 | gallons |
| 1 acre foot (amount of water required to cover one acre one foot deep) | 43,560 | cubic feet |
| | 325,850 | gallons |
| | 12 | acre inches |
| **WEIGHT** | | |
| 1 gallon | 8.33 | pounds (lb) |
| 1 cubic foot | 62.4 | pounds |
| **FLOW** | | |
| 1 gallon per minute (gpm) | 0.00223 | cubic feet per second (cfs) |
| | 1,440 | gallons per day (24 hours) |
| 1 million gallon per 24 hours (mgd) | 1.547 | cubic feet per second |
| | 695 | gallons per minute |
| 1 cubic foot per second | 7.48 | gallons per second (gps) |
| | 448.8 | gallons per minute |
| | 646,272 | gallons per day (24 hours) |
| | .992 | acre inch per hour |
| | 1.983 | acre feet per day (24 hours) |
| | 40 | miner's inches (legal value 1/) |
| | 50 | miner's inches (legal value 2/) |
| | 38.4 | miner's inches (in Colorado) |
| | 35.7 | miner's inches (in British Columbia) |
| 1 miner's inch | 11.25 | gpm when equivalent to 1/40 second foot |
| | 9 | gpm when equivalent to 1/50 second foot |

1/ In Arizona, California, Montana, Nevada and Oregon
2/ In Idaho, Kansas, Nebraska, New Mexico, North Dakota, South Dakota, and Utah

Exhibit 3-1 Water volume, weight and flow equivalents

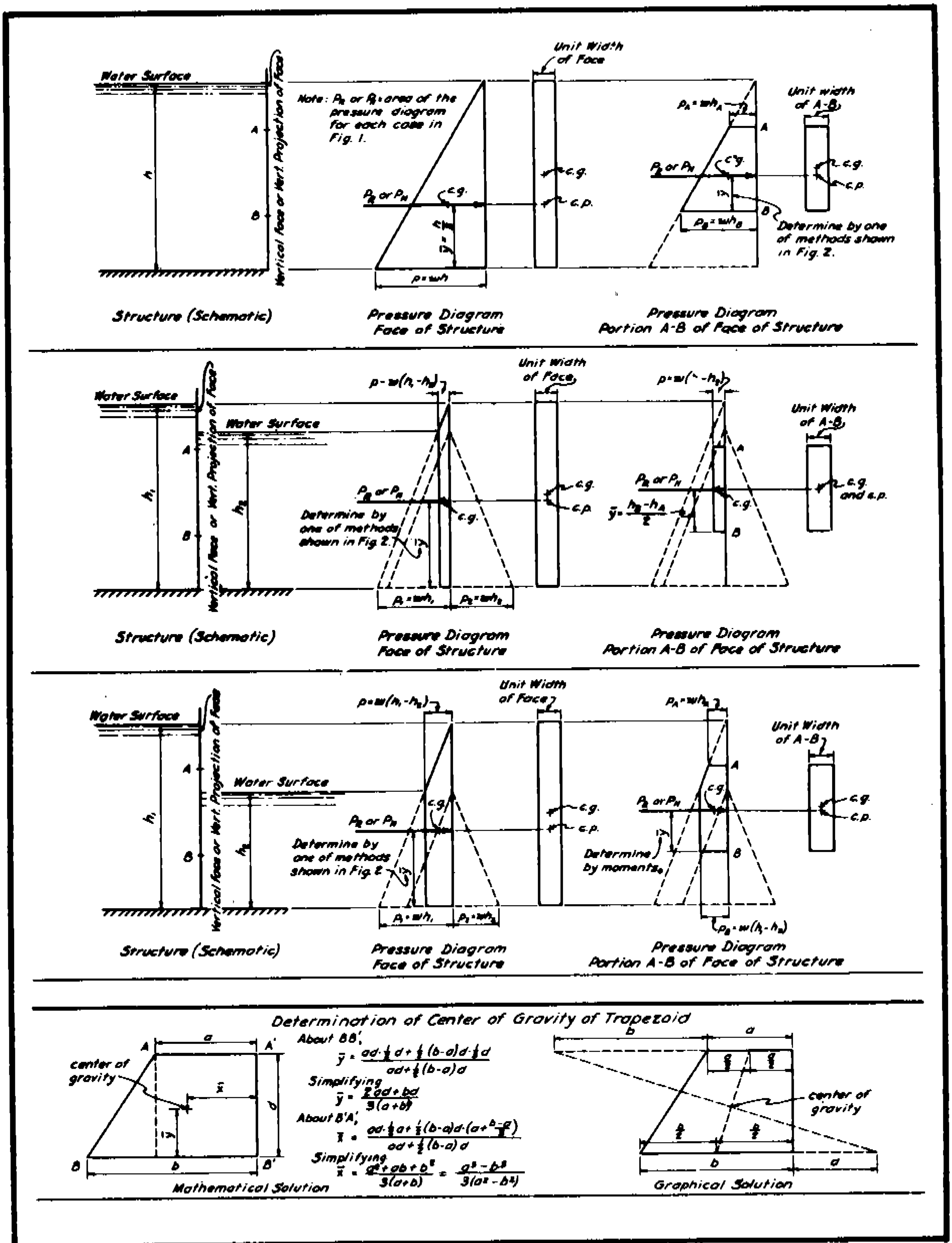
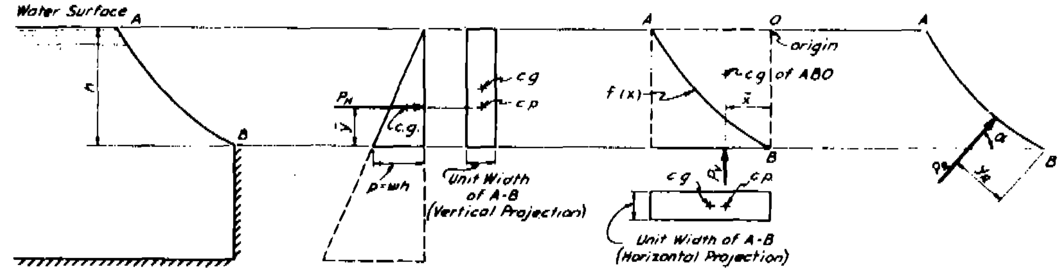



Exhibit 3-2 Pressure diagrams and methods of computing hydrostatic loads (Ref. NEH Section 5, ES-31)

(Sheet 1 of 2)




Note: The following solution is general and is valid for, (1),any shape of portion A-B and,(2), water on either side of A-B. However, when water is on the right of A-B, the pressure forces $P_H$, $P_V$, and $P_R$ act in directions opposite to those shown.

Structure (Schematic)

$P_H = \frac{1}{2} wh^2$

$\bar{y} = \frac{h}{3}$

Pressure Diagram of Horizontal Pressure Portion A-B

$P_V = wA_{ABO}$

$A_{ABO}$ = Area bounded by ABO $= \int_0^A f(x)\,dx$

$\bar{x} = \frac{\int_0^A x f(x)\,dx}{A_{ABO}}$

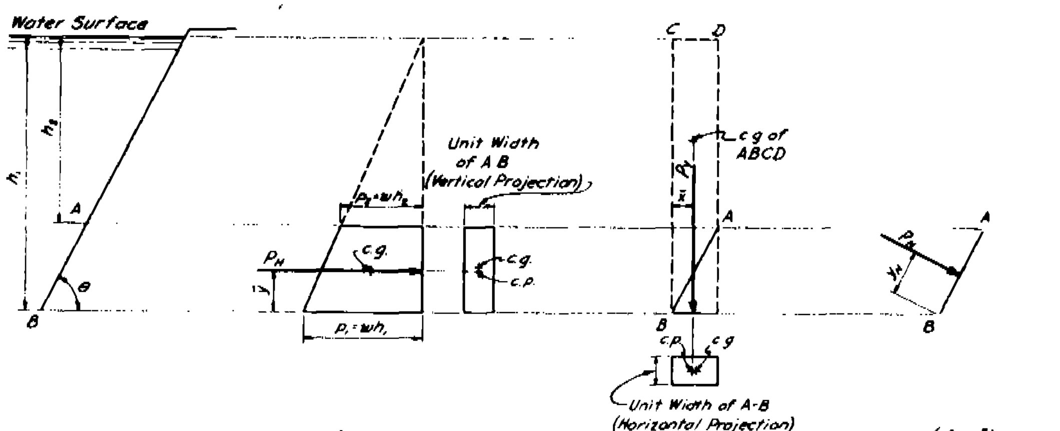
Vertical Pressure Force Portion A-B

$P_R = \sqrt{P_H^2 + P_V^2}$

$\sin\alpha = \frac{P_V}{P_R}$

$y_R = \frac{P_H \cdot \frac{h}{3} + P_V \cdot \bar{x}}{P_R}$

Resultant Pressure Force Portion A-B




Structure (Schematic)

$P_H = \frac{w}{2}(h_1^2 - h_2^2)$

$\bar{y} = \frac{(h_1-h_2)(h_1+2h_2)}{3(h_1+h_2)}$

Pressure Diagram of Horizontal Pressure Portion A-B

$P_V = \frac{w(h_1^2 - h_2^2)}{2\tan\theta}$

$\bar{x} = \frac{(h_1-h_2)(h_1+2h_2)}{3(h_1+h_2)\tan\theta}$

Vertical Pressure Force Portion A-B

$P_N = \frac{w(h_1^2 - h_2^2)}{2\sin\theta}$

$y_N = \frac{(h_1-h_2)(h_1+2h_2)}{3(h_1+h_2)\sin\theta}$

Normal Pressure Force Portion A-B

## 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 force, on the face, or a projection of the face, of a structure.
- h – height of water above a point, as indicated in "Structure (Schematic)", or as indicated by subscript.
- p. – intensity of pressure at a point indicated by subscript, or at 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 from given point perpendicular to line of action of $P_N$.
- $y_R$ – distance from given point perpendicular to line of action of $P_R$.

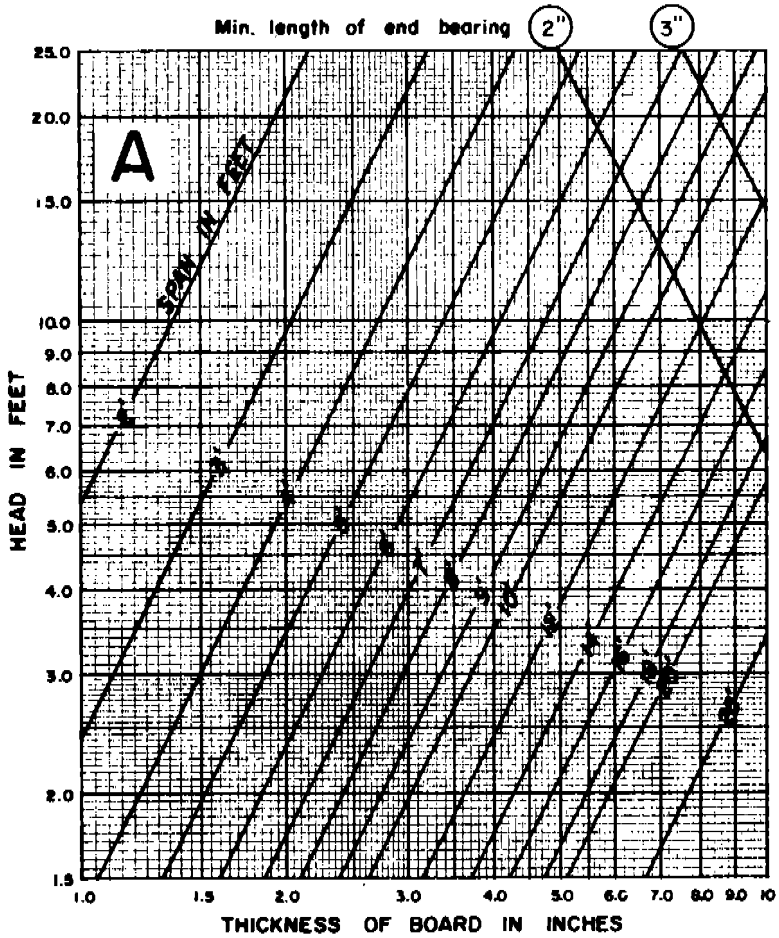
Exhibit 3-2 Pressure diagrams and methods of computing hydrostatic loads (Ref. NEH Section 5, ES-31)

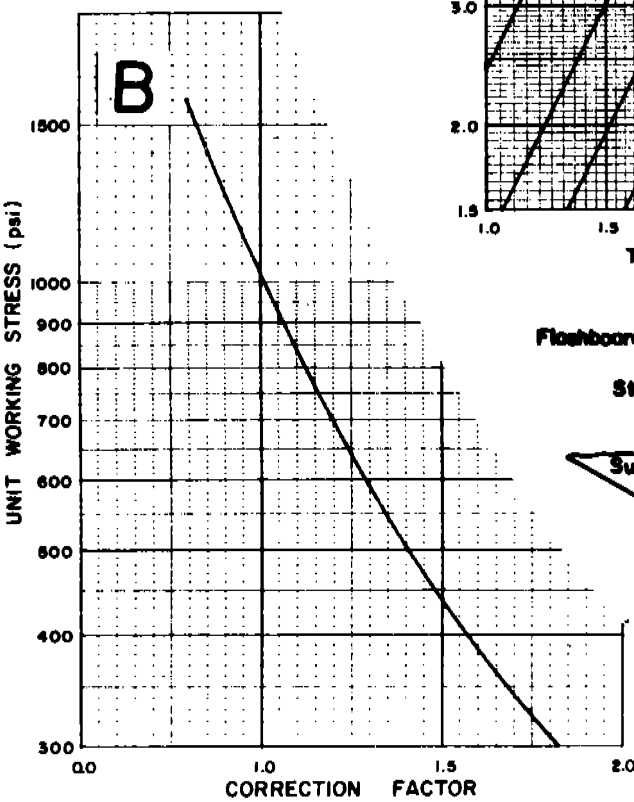
(Sheet 2 of 2)

Unit Working Stresses (psi) - Bending
Common or Construction Grade Timber
Wet and Weathered Conditions

| | |
|---|---|
| Douglas Fir, Coast Region | 750 |
| " " " " , Dense | 880 |
| " " , Rocky Mountain | 530 |
| Hemlock, Western | 680 |
| " " | 600 |
| Larch, Western | 680 |
| Pine, Southern Yellow | 750 |
| " " " , Dense | 880 |
| Pine, White, Yellow, Sugar, Ponderosa | 570 |
| Redwood | 600 |
| Tamarack | 680 |

NOTES:
(1) Chart A is based on a unit fiber stress of 1000 psi.
(2) See Std. Drwg. 88-26 for unit working stresses of other species of timber and moisture conditions.



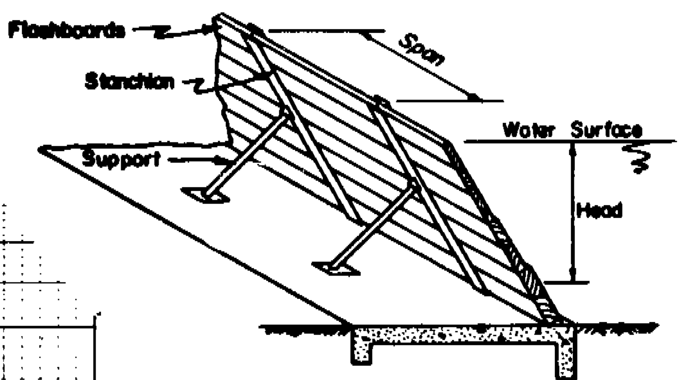




Exhibit 3-3 Required thickness of flashboards (ref. Western States Engineering Handbook Section 6)

**HEAD LOSS COEFFICIENT, $K_p$, FOR CIRCULAR PIPE FLOWING FULL** $K_p = \frac{5087\, n^2}{d_i^{4/3}}$

| Pipe diam. inches | Flow area sq. ft. | MANNING'S COEFFICIENT OF ROUGHNESS "n" 0.010 | 0.011 | 0.012 | 0.013 | 0.014 | 0.015 | 0.016 | 0.017 | 0.018 | 0.019 | 0.020 | 0.021 | 0.022 | 0.023 | 0.024 | 0.025 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6 | 0.196 | 0.0467 | 0.0565 | 0.0672 | 0.0789 | 0.0914 | 0.1050 | 0.1194 | 0.1348 | 0.151 | 0.168 | 0.187 | 0.206 | 0.226 | 0.247 | 0.269 | 0.292 |
| 8 | 0.349 | .0318 | .0385 | .0458 | .0537 | .0623 | .0715 | .0814 | .0919 | .1030 | .1148 | .1272 | .140 | .154 | .168 | .183 | .199 |
| 10 | 0.545 | .0236 | .0286 | .0340 | .0399 | .0463 | .0531 | .0604 | .0682 | .0765 | .0852 | .0944 | .1041 | .1143 | .1249 | .136 | .148 |
| 12 | 0.785 | .0185 | .0224 | .0267 | .0313 | .0363 | .0417 | .0474 | .0535 | .0600 | .0668 | .0741 | .0817 | .0896 | .0980 | .1067 | .1157 |
| | | | | | | | | | | | | | | | | | |
| 14 | 1.069 | .0151 | .0182 | .0217 | .0255 | .0295 | .0339 | .0386 | .0436 | .0488 | .0544 | .0603 | .0665 | .0730 | .0798 | .0868 | .0942 |
| 15 | 1.23 | .0138 | .0166 | .0198 | .0232 | .0270 | .0309 | .0352 | .0397 | .0446 | .0496 | .0550 | .0606 | .0666 | .0727 | .0792 | .0859 |
| 16 | 1.40 | .0126 | .0153 | .0182 | .0213 | .0247 | .0284 | .0323 | .0365 | .0409 | .0455 | .0505 | .0556 | .0611 | .0667 | .0727 | .0789 |
| | | | | | | | | | | | | | | | | | |
| 18 | 1.77 | .01078 | .0130 | .0155 | .0182 | .0211 | .0243 | .0276 | .0312 | .0349 | .0389 | .0431 | .0476 | .0522 | .0570 | .0621 | .0674 |
| 21 | 2.41 | .00878 | .01062 | .0126 | .0148 | .0172 | .0198 | .0225 | .0254 | .0284 | .0317 | .0351 | .0387 | .0425 | .0464 | .0506 | .0549 |
| 24 | 3.14 | .00735 | .00889 | .01058 | .0124 | .0144 | .0165 | .0188 | .0212 | .0238 | .0265 | .0294 | .0324 | .0356 | .0389 | .0423 | .0459 |
| 27 | 3.98 | .00628 | .00760 | .00904 | .01061 | .0123 | .0141 | .0161 | .0181 | .0203 | .0227 | .0251 | .0277 | .0304 | .0332 | .0362 | .0393 |
| | | | | | | | | | | | | | | | | | |
| 30 | 4.91 | .00546 | .00660 | .00786 | .00922 | .01070 | .01228 | .0140 | .0158 | .0177 | .0197 | .0218 | .0241 | .0264 | .0289 | .0314 | .0341 |
| 36 | 7.07 | .00428 | .00518 | .00616 | .00723 | .00839 | .00963 | .01096 | .0124 | .0139 | .0154 | .0171 | .0189 | .0207 | .0226 | .0246 | .0267 |
| 42 | 9.62 | .00348 | .00422 | .00502 | .00589 | .00683 | .00784 | .00892 | .01007 | .01129 | .0126 | .0139 | .0154 | .0169 | .0184 | .0201 | .0218 |
| 48 | 12.57 | .00292 | .00353 | .00420 | .00493 | .00572 | .00656 | .00747 | .00843 | .00945 | .01053 | .01166 | .0129 | .0141 | .0154 | .0168 | .0182 |
| 54 | 15.90 | .00249 | .00302 | .00359 | .00421 | .00488 | .00561 | .00638 | .00720 | .00808 | .00900 | .00997 | .01099 | .0121 | .0132 | .0144 | .0156 |
| 60 | 19.63 | .00217 | .00262 | .00312 | .00366 | .00424 | .00487 | .00554 | .00626 | .00702 | .00782 | .00866 | .00955 | .01048 | .0115 | .0125 | .0135 |

**HEAD LOSS COEFFICIENT, $K_c$, FOR SQUARE CONDUIT FLOWING FULL** $K_c = \frac{29.16\, n^2}{r^{4/3}}$

| Conduit Size feet | Flow area sq. ft. | MANNING'S COEFFICIENT OF ROUGHNESS "n" 0.012 | 0.013 | 0.014 | 0.015 | 0.016 |
|---|---|---|---|---|---|---|
| 2×2 | 4.00 | 0.01058 | 0.01242 | 0.01440 | 0.01653 | 0.01880 |
| 2½×2½ | 6.25 | 0.00786 | 0.00922 | 0.01070 | 0.01228 | 0.01397 |
| | | | | | | |
| 3×3 | 9.00 | .00616 | .00723 | .00839 | .00963 | .01096 |
| 3½×3½ | 12.25 | .00502 | .00589 | .00683 | .00784 | .00892 |
| 4×4 | 16.00 | .00420 | .00493 | .00572 | .00656 | .00746 |
| 4½×4½ | 20.25 | .00359 | .00421 | .00488 | .00561 | .00638 |
| | | | | | | |
| 5×5 | 25.00 | .00312 | .00366 | .00425 | .00487 | .00554 |
| 5½×5½ | 30.25 | .00275 | .00322 | .00374 | .00429 | .00488 |
| 6×6 | 36.00 | .00245 | .00287 | .00333 | .00382 | .00435 |
| 6½×6½ | 42.25 | .00220 | .00258 | .00299 | .00343 | .00391 |
| | | | | | | |
| 7×7 | 49.00 | .00199 | .00234 | .00271 | .00311 | .00354 |
| 7½×7½ | 56.25 | .00182 | .00213 | .00247 | .00284 | .00323 |
| 8×8 | 64.00 | .00167 | .00196 | .00227 | .00260 | .00296 |
| 8½×8½ | 72.25 | .00154 | .00180 | .00209 | .00240 | .00273 |
| | | | | | | |
| 9×9 | 81.00 | .00142 | .00167 | .00194 | .00223 | .00253 |
| 9½×9½ | 90.25 | .00133 | .00156 | .00180 | .00207 | .00236 |
| 10×10 | 100.00 | .00124 | .00145 | .00168 | .00193 | .00220 |

$$H_l = (K_p \text{ or } K_c)\, L\, \frac{v^2}{2g}$$

<u>Nomenclature</u>:

- $a$ = Cross-sectional area of flow in sq. ft.
- $d_i$ = Inside diameter of pipe in inches.
- $g$ = Acceleration of gravity = 32.2 ft. per sec.
- $H_l$ = Loss of head in feet due to friction in length L.
- $K_c$ = Head loss coefficient for square conduit flowing full.
- $K_p$ = Head loss coefficient for circular pipe flowing full.
- $L$ = Length of conduit in feet.
- $n$ = Manning's coefficient of roughness.
- $Q$ = Discharge or capacity in cu. ft. per sec.
- $r$ = Hydraulic radius in feet.
- $v$ = Mean velocity in ft. per sec.

<u>Example 1</u>: Compute the head loss in 300 ft. of 24 in. diam. concrete pipe flowing full and discharging 30 c.f.s. Assume $n = 0.015$

$$v = \frac{Q}{a} = \frac{30}{3.14} = 9.55 \text{ f.p.s.}; \quad \frac{v^2}{2g} = \frac{(9.55)^2}{64.4} = 1.42 \text{ ft.}$$

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

<u>Example 2</u>: Compute the discharge of a 250 ft., 3×3 square conduit flowing full if the loss of head is determined to be 2.25 ft. Assume $n = 0.014$.

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

$$v = \sqrt{64.4 \times 1.073} = 8.31; \quad Q = 9 \times 8.31 = 74.8 \text{ c.f.s.}$$

Exhibit 3-4 Head loss coefficients for circular and square conduits flowing full (Ref. NEH Section 5, ES-42)

93½%

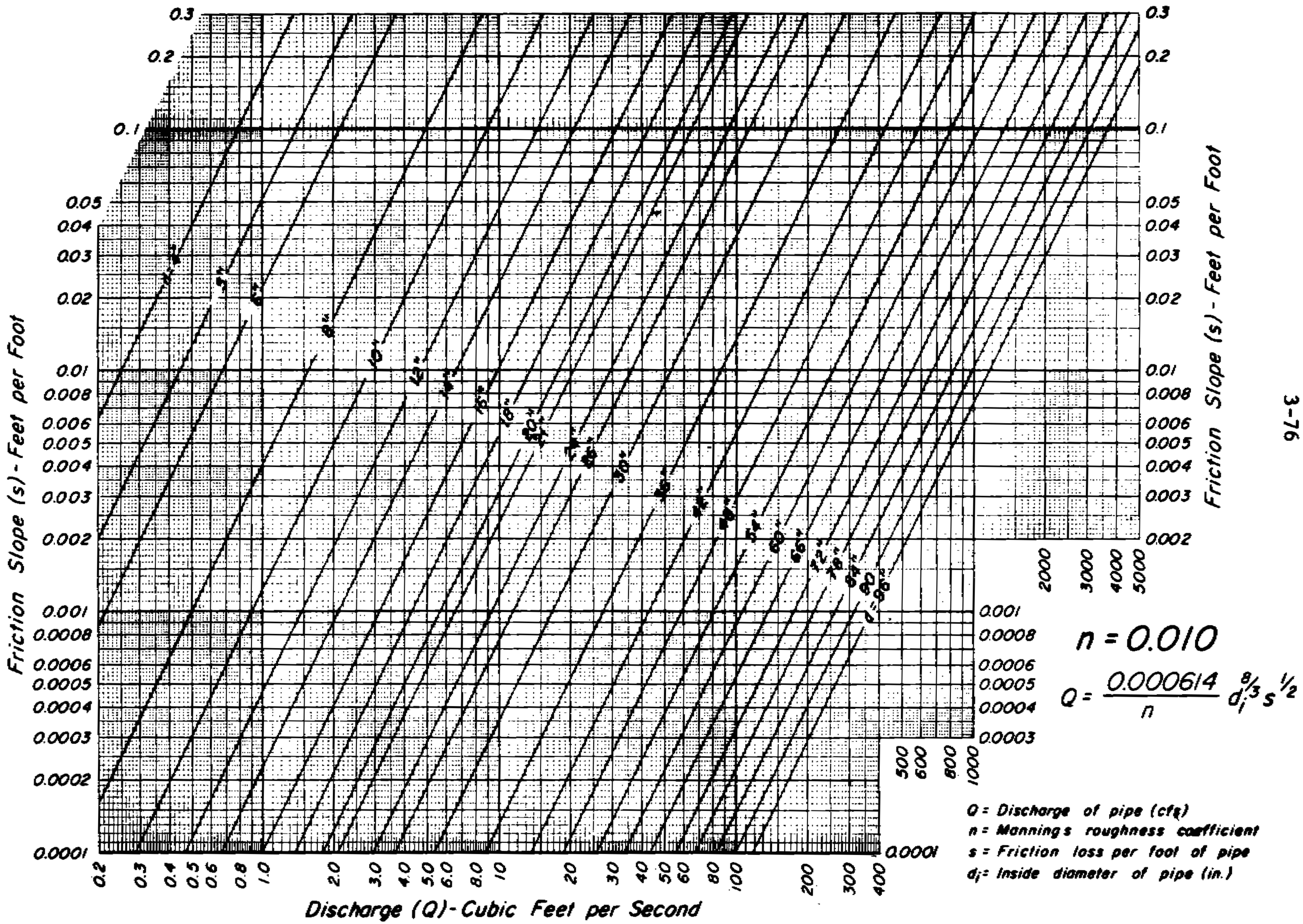



Exhibit 3-5 Discharge of circular pipes flowing full.
Manning's n = 0.010

(Sheet 1 of 6)

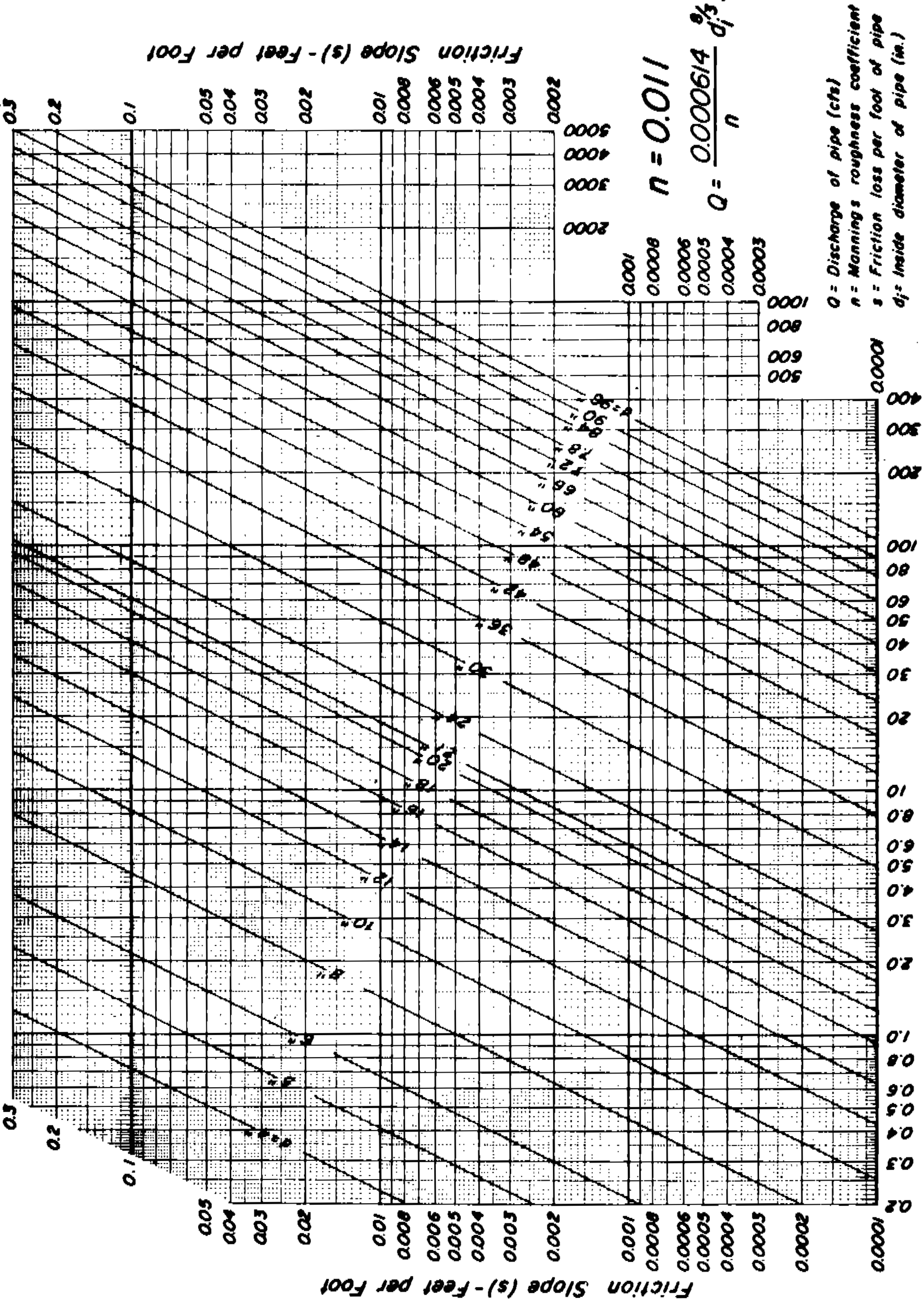



Exhibit 3-5 Discharge of circular pipes flowing full.
Manning's n = 0.011 (Ref. NEH Section 5, ES-54)
(Sheet 2 of 6)

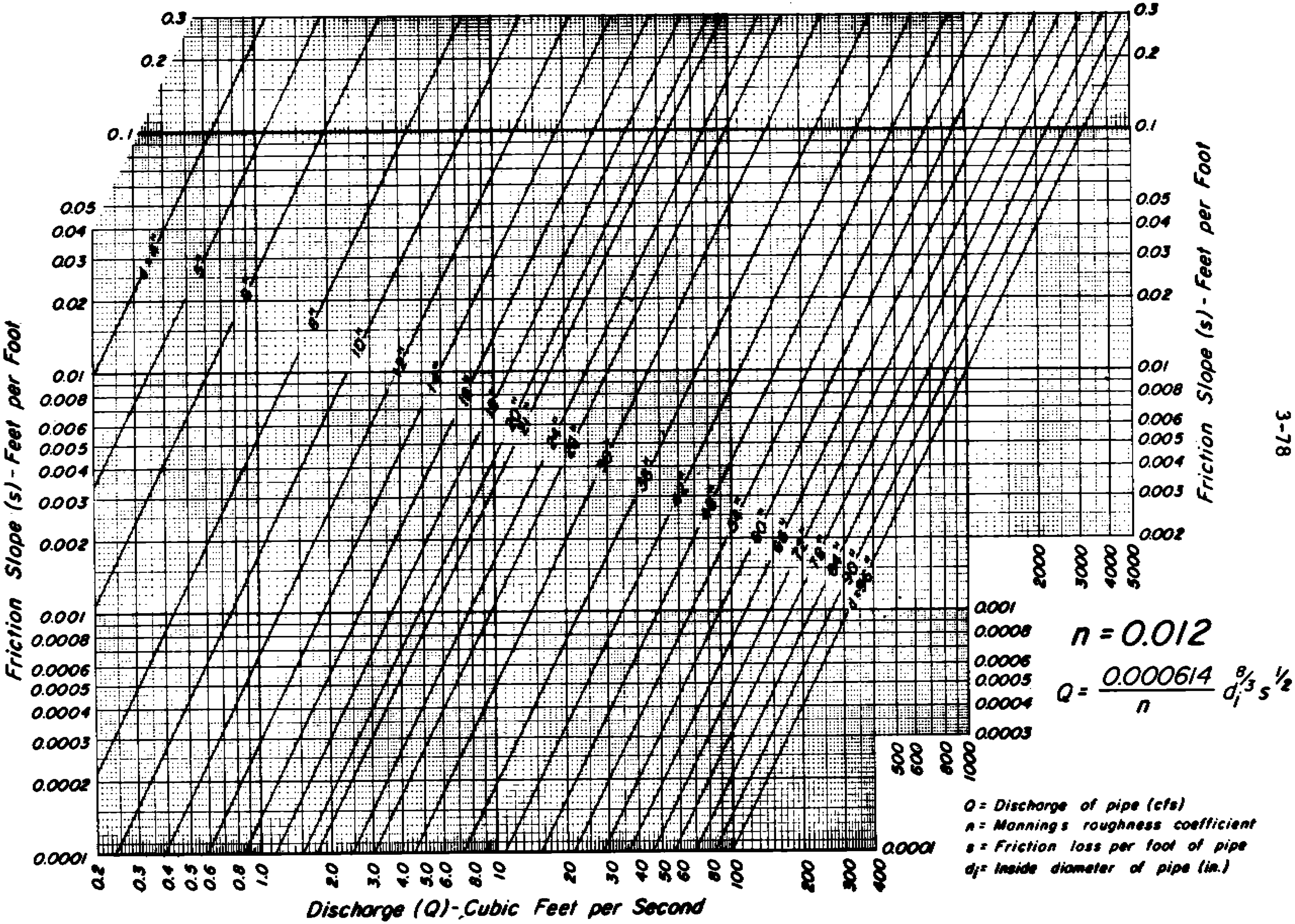



Exhibit 3-5 Discharge of circular pipes flowing full. Manning's n = 0.012

(Sheet 3 of 6)

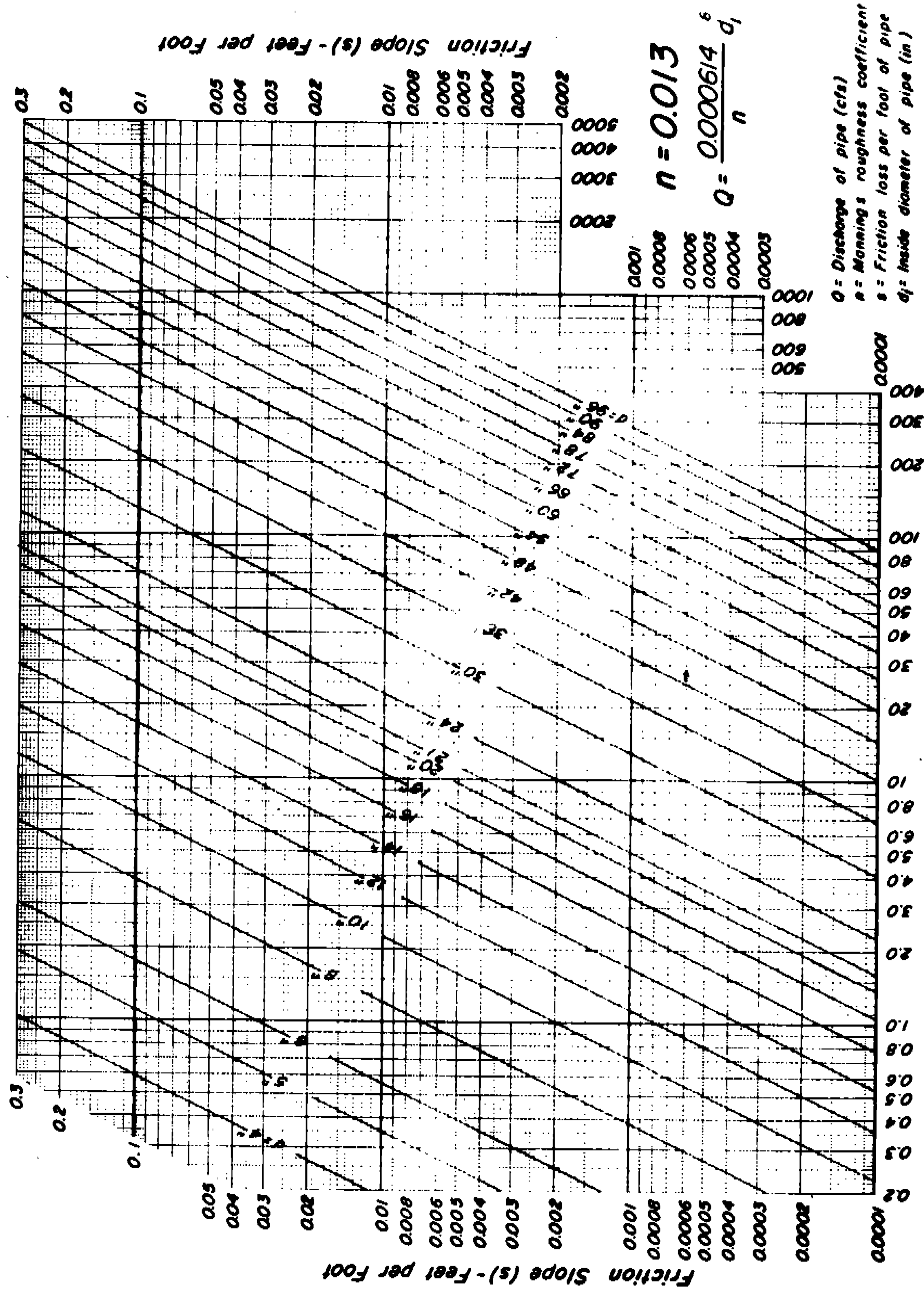



Exhibit 3-5 Discharge of circular pipes flowing full.
Manning's n = 0.013 (Ref. NEH Section 5, ES-54) (Sheet 4 of 6)

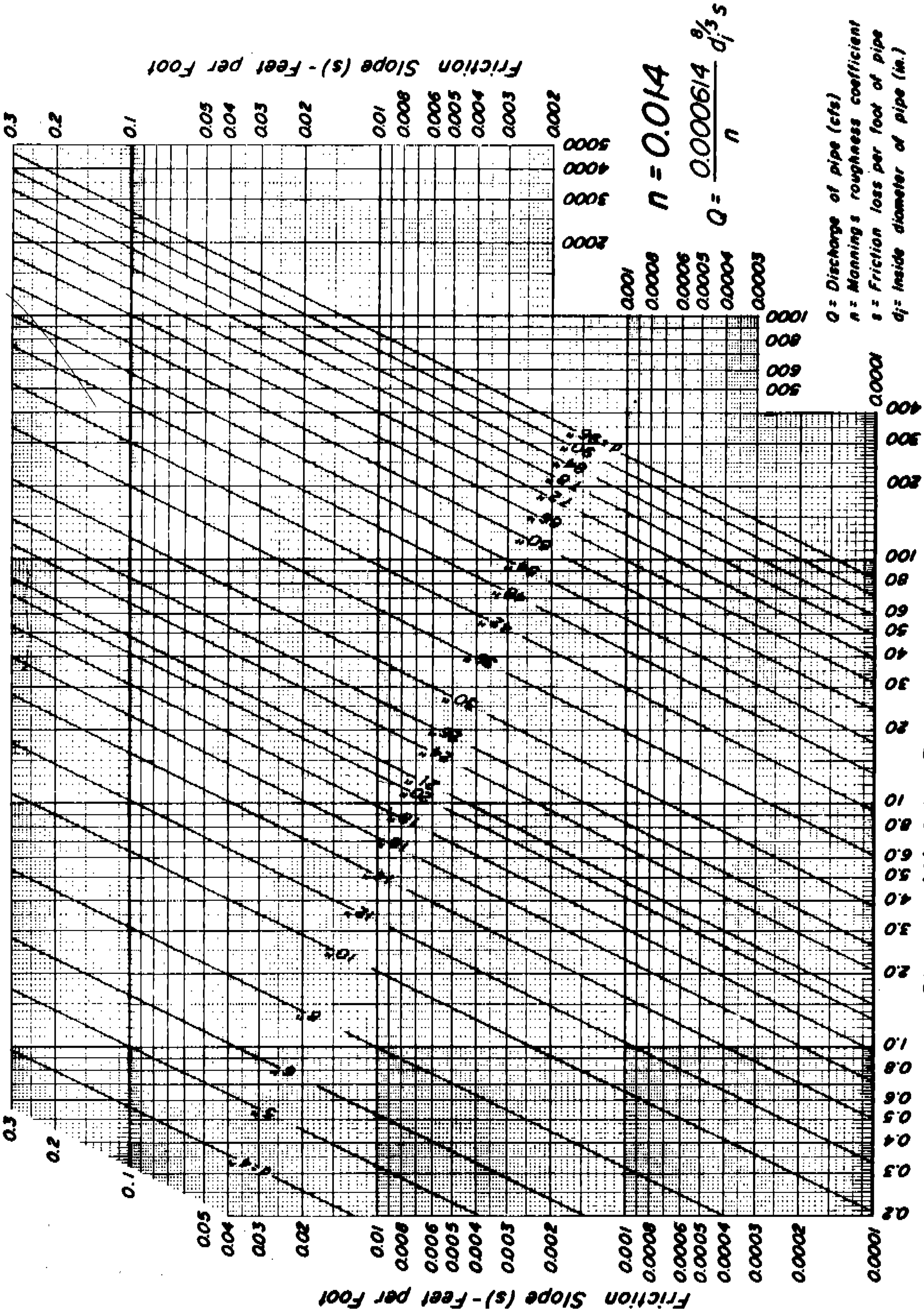



Exhibit 3-5 Discharge of circular pipes flowing full.
Manning's n = 0.014 (Sheet 5 of 6)

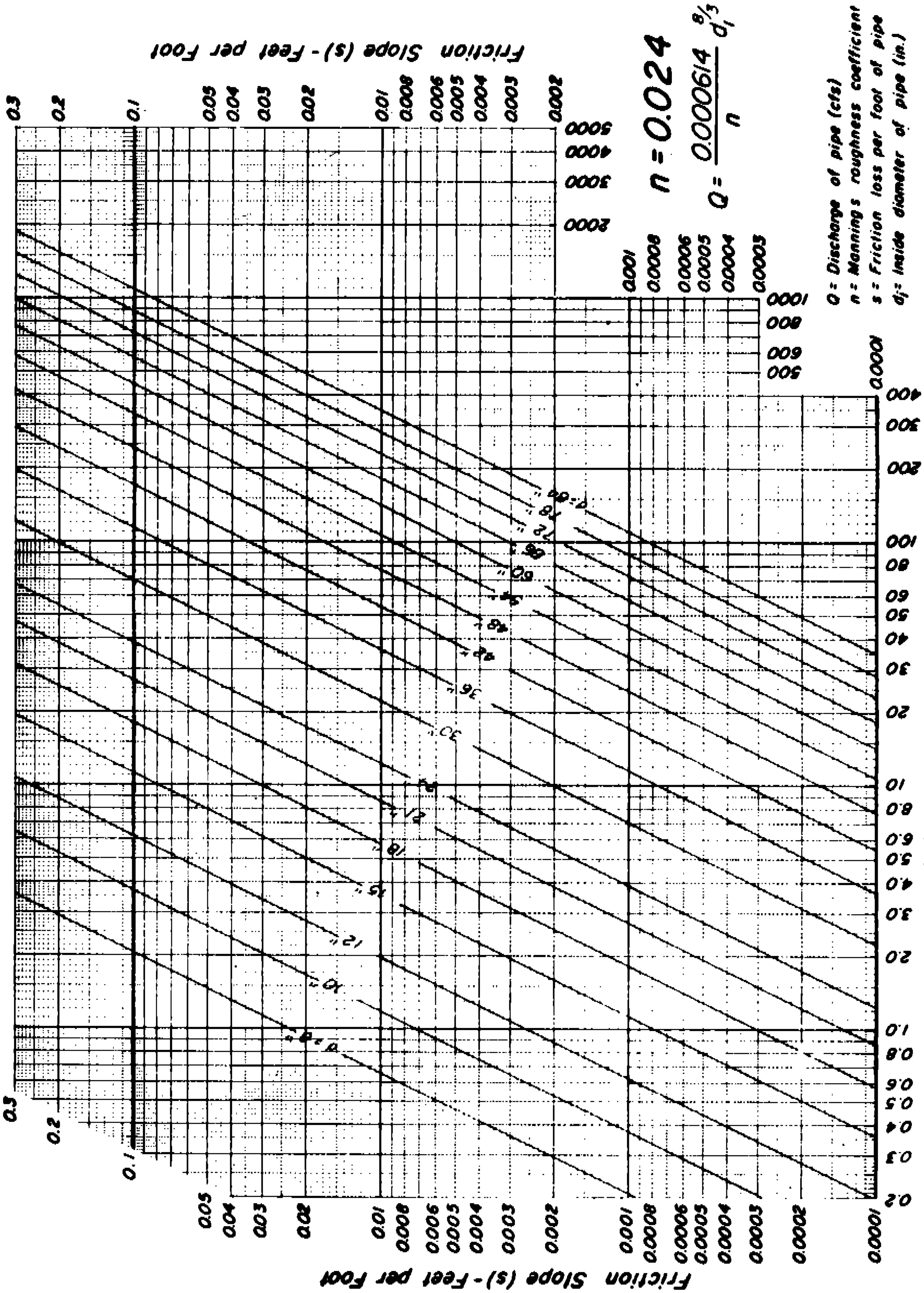
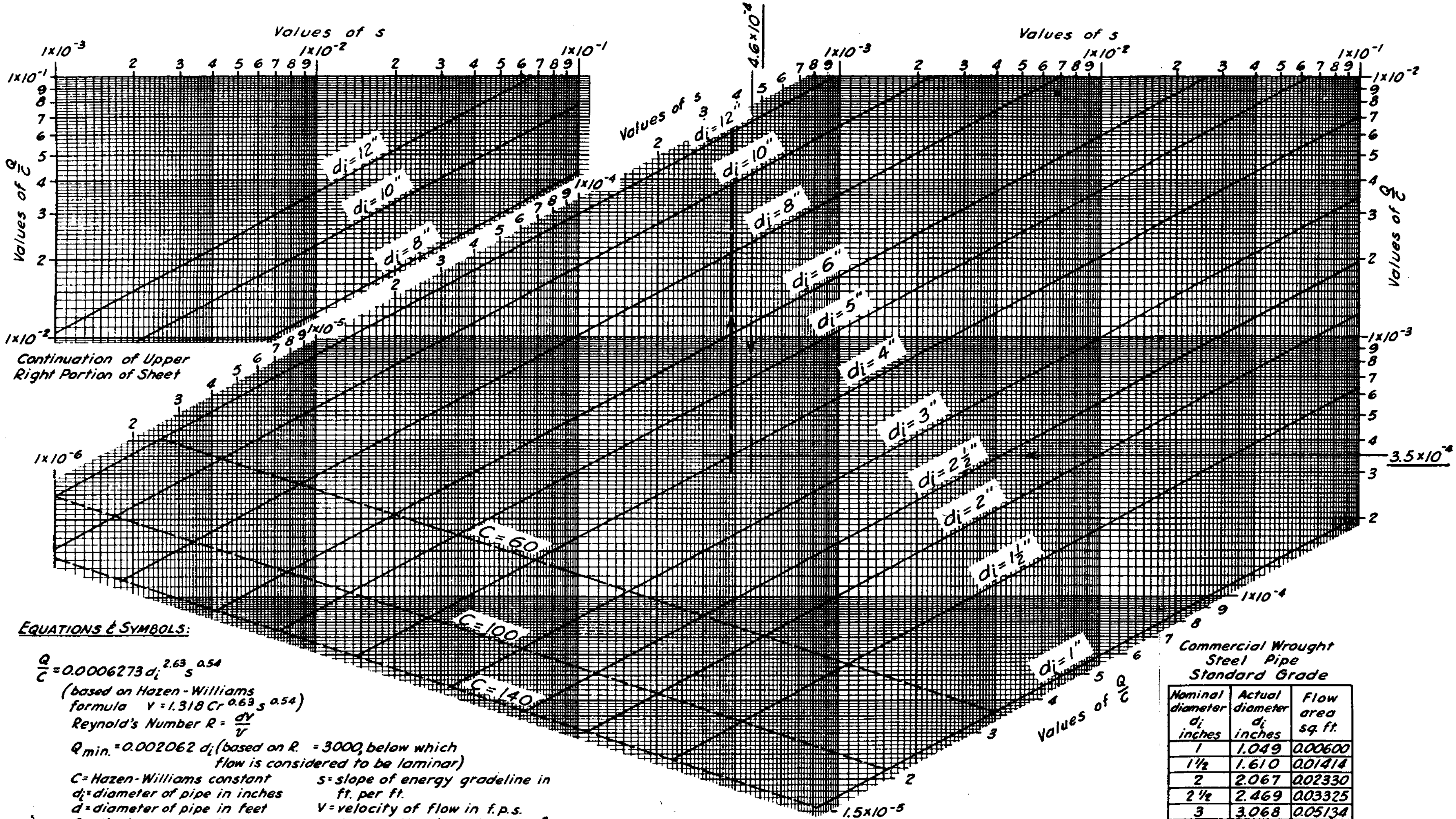



Exhibit 3-5 Discharge of circular pipes flowing full.
Manning's n = 0.024 (Sheet 6 of 6)




**EQUATIONS & SYMBOLS:**

$\frac{Q}{C} = 0.0006273\, d_i^{2.63}\, s^{0.54}$

(based on Hazen-Williams formula $V = 1.318\, C r^{0.63} s^{0.54}$)

Reynold's Number $R = \frac{dV}{\nu}$

$Q_{min.} = 0.002062\, d_i$ (based on $R = 3000$, below which flow is considered to be laminar)

C = Hazen-Williams constant
$d_i$ = diameter of pipe in inches
d = diameter of pipe in feet
Q = discharge in c.f.s.
r = hydraulic radius in feet
s = slope of energy gradeline in ft. per ft.
V = velocity of flow in f.p.s.
$\nu$ = kinematic viscosity in ft.$^2$ per sec. (assumed to be $1.05 \times 10^{-5}$ ft.$^2$ per sec. for water at 70°F.)

Note: Dashed lines pass through minimum values of $Q/C$ for the given C. For lesser values of $Q/C$, according to the assumptions of $V$ and $R$, there can be no assurance of turbulent flow.

Example: Find size pipe required to carry $Q = 0.042$ cfs with a maximum energy gradient $S = 0.00046$ if $C = 120$. Then $Q/C = \frac{0.042}{120} = 3.5 \times 10^{-4}$. Enter chart, with above values, and find $d_i = 4$ in.. Actual head loss in 4 in. pipe $= 3.85 \times 10^{-4}$ ft. per ft.

Commercial Wrought Steel Pipe
Standard Grade

| Nominal diameter $d_i$ inches | Actual diameter $d_i$ inches | Flow area sq. ft. |
|---|---|---|
| 1 | 1.049 | 0.00600 |
| 1½ | 1.610 | 0.01414 |
| 2 | 2.067 | 0.02330 |
| 2½ | 2.469 | 0.03325 |
| 3 | 3.068 | 0.05134 |
| 4 | 4.026 | 0.08840 |
| 5 | 5.047 | 0.1389 |
| 6 | 6.065 | 0.2006 |
| 8 | 7.981 | 0.3474 |
| 10 | 10.020 | 0.5475 |
| 12 | 12.000 | 0.7854 |

Exhibit 3-6 Solution of Hazen-Williams formula for round pipes (Ref. NEH Section 5, ES-40)

For IPS Pipe

| Q Gallons per min. | 1-inch 1.189 ID | 1¼-inch 1.502 ID | 1½-inch 1.720 ID | 2-inch 2.149 ID | 2½-inch 2.601 ID | 3-inch 3.166 ID | 3½-inch 3.620 ID | Q Gallons per min. |
|---|---|---|---|---|---|---|---|---|
| | Friction Head Loss in Feet per Hundred Feet | | | | | | | |
| 2 | .15 | .04 | .02 | | | | | 2 |
| 4 | .54 | .17 | .09 | .03 | .01 | | | 4 |
| 6 | 1.15 | .37 | .19 | .06 | .02 | | | 6 |
| 8 | 1.97 | .63 | .32 | .11 | .04 | .01 | | 8 |
| 10 | 2.98 | .95 | .49 | .16 | .06 | .02 | .01 | 10 |
| 15 | 6.32 | 2.03 | 1.04 | .35 | .14 | .05 | .02 | 15 |
| 20 | 10.79 | 3.46 | 1.78 | .60 | .23 | .09 | .04 | 20 |
| 25 | 16.30 | 5.22 | 2.70 | .91 | .36 | .13 | .07 | 25 |
| 30 | 22.86 | 7.32 | 3.78 | 1.27 | .50 | .19 | .10 | 30 |
| 35 | | 9.75 | 5.03 | 1.70 | .67 | .25 | .13 | 35 |
| 40 | | 12.46 | 6.46 | 2.18 | .86 | .32 | .17 | 40 |
| 45 | | 15.51 | 8.02 | 2.71 | 1.07 | .40 | .21 | 45 |
| 50 | | 18.87 | 9.75 | 3.30 | 1.30 | .49 | .25 | 50 |
| 55 | | 22.48 | 11.64 | 3.94 | 1.54 | .59 | .30 | 55 |
| 60 | | | 13.64 | 4.62 | 1.81 | .69 | .36 | 60 |
| 65 | | | 15.85 | 5.36 | 2.10 | .80 | .41 | 65 |
| 70 | | | 18.19 | 6.14 | 2.42 | .92 | .47 | 70 |
| 75 | | | 20.65 | 6.99 | 2.75 | 1.06 | .55 | 75 |
| 80 | | | 23.28 | 7.86 | 3.10 | 1.19 | .62 | 80 |
| 85 | | | | 8.81 | 3.47 | 1.33 | .69 | 85 |
| 90 | | | | 9.79 | 3.85 | 1.48 | .77 | 90 |
| 95 | | | | 10.82 | 4.25 | 1.64 | .85 | 95 |
| 100 | | | | 11.89 | 4.69 | 1.80 | .93 | 100 |
| 110 | | | | 14.21 | 5.59 | 2.14 | 1.11 | 110 |
| 120 | | | | 16.69 | 6.56 | 2.52 | 1.31 | 120 |
| 130 | | | | 19.35 | 7.63 | 2.92 | 1.53 | 130 |
| 140 | | | | 22.21 | 8.73 | 3.36 | 1.75 | 140 |
| 150 | | | | | 9.94 | 3.82 | 1.99 | 150 |
| 160 | | | | | 11.20 | 4.29 | 2.24 | 160 |
| 170 | | | | | 12.51 | 4.80 | 2.50 | 170 |
| 180 | | | | | 13.90 | 5.35 | 2.79 | 180 |
| 190 | | | | | 15.39 | 5.92 | 3.08 | 190 |
| 200 | | | | | 16.91 | 6.50 | 3.38 | 200 |
| 220 | | | | | 20.19 | 7.77 | 4.04 | 220 |
| 240 | | | | | 23.73 | 9.12 | 4.76 | 240 |
| 260 | | | | | | 10.57 | 5.51 | 260 |
| 280 | | | | | | 12.11 | 6.32 | 280 |
| 300 | | | | | | 13.78 | 7.18 | 300 |
| 320 | | | | | | 15.52 | 8.10 | 320 |
| 340 | | | | | | 17.37 | 9.07 | 340 |
| 360 | | | | | | 19.27 | 10.08 | 360 |
| 380 | | | | | | 21.33 | 11.13 | 380 |
| 400 | | | | | | 23.45 | 12.22 | 400 |
| 420 | | | | | | | 13.40 | 420 |
| 440 | | | | | | | 14.59 | 440 |
| 460 | | | | | | | 15.86 | 460 |
| 480 | | | | | | | 17.15 | 480 |
| 500 | | | | | | | 18.50 | 500 |

Table based on Hazen-Williams equation - $C_1$ = 150

To find friction head loss in PVC or ABS pipe having a standard dimension ratio other than 21, the values in the table should be multiplied by the appropriate conversion factor shown below:

| SDR No. | Conversion Factor |
|---|---|
| 13.5 | 1.35 |
| 17 | 1.13 |
| 21 | 1.00 |
| 26 | .91 |
| 32.5 | .84 |
| 41 | .785 |
| 51 | .75 |

**Exhibit 3-7 Friction head loss in semirigid plastic irrigation pipelines manufactured of PVC or ABS compounds. Standard dimension ratio - SDR = 21. (Ref. SCS - Fort Worth, Texas, 1967)**

For IPS Pipe

| Q Gallons per min. | 4-inch 4.072 ID | 5-inch 5.033 ID | 6-inch 5.993 ID | 8-inch 7.805 ID | 10-inch 9.728 ID | 12-inch 11.538 ID | Q Gallons per min. |
|---|---|---|---|---|---|---|---|
| | Friction Head Loss in Feet per Hundred Feet | | | | | | |
| 15 | .01 | | | | | | 15 |
| 20 | .02 | | | | | | 20 |
| 25 | .04 | .01 | | | | | 25 |
| 30 | .05 | .02 | | | | | 30 |
| 35 | .07 | .02 | .01 | | | | 35 |
| 40 | .09 | .03 | .01 | | | | 40 |
| 45 | .12 | .04 | .01 | | | | 45 |
| 50 | .14 | .05 | .02 | | | | 50 |
| 55 | .17 | .06 | .02 | | | | 55 |
| 60 | .20 | .07 | .03 | | | | 60 |
| 65 | .23 | .08 | .03 | .01 | | | 65 |
| 70 | .27 | .09 | .04 | .01 | | | 70 |
| 75 | .31 | .11 | .04 | .01 | | | 75 |
| 80 | .35 | .12 | .05 | .01 | | | 80 |
| 85 | .39 | .14 | .05 | .01 | | | 85 |
| 90 | .43 | .15 | .06 | .01 | | | 90 |
| 95 | .48 | .17 | .07 | .02 | | | 95 |
| 100 | .52 | .19 | .07 | .02 | | | 100 |
| 110 | .63 | .22 | .09 | .02 | | | 110 |
| 120 | .74 | .26 | .10 | .03 | .01 | | 120 |
| 130 | .85 | .30 | .12 | .03 | .01 | | 130 |
| 140 | .98 | .35 | .14 | .04 | .01 | | 140 |
| 150 | 1.11 | .40 | .16 | .05 | .01 | | 150 |
| 160 | 1.26 | .44 | .19 | .05 | .01 | | 160 |
| 170 | 1.41 | .49 | .21 | .06 | .02 | | 170 |
| 180 | 1.57 | .55 | .24 | .07 | .02 | .01 | 180 |
| 190 | 1.73 | .61 | .26 | .07 | .02 | .01 | 190 |
| 200 | 1.90 | .67 | .29 | .08 | .02 | .01 | 200 |
| 220 | 2.28 | .81 | .34 | .09 | .03 | .01 | 220 |
| 240 | 2.67 | .95 | .40 | .10 | .03 | .01 | 240 |
| 260 | 3.10 | 1.10 | .46 | .12 | .04 | .02 | 260 |
| 280 | 3.56 | 1.26 | .54 | .14 | .05 | .02 | 280 |
| 300 | 4.04 | 1.43 | .61 | .17 | .05 | .02 | 300 |
| 320 | 4.56 | 1.62 | .69 | .19 | .06 | .03 | 320 |
| 340 | 5.10 | 1.82 | .77 | .21 | .07 | .03 | 340 |
| 360 | 5.67 | 2.02 | .86 | .24 | .08 | .03 | 360 |
| 380 | 6.26 | 2.22 | .95 | .26 | .09 | .04 | 380 |
| 400 | 6.90 | 2.45 | 1.04 | .28 | .10 | .04 | 400 |
| 420 | 7.55 | 2.69 | 1.14 | .31 | .10 | .05 | 420 |
| 440 | 8.23 | 2.92 | 1.25 | .34 | .11 | .05 | 440 |
| 460 | 8.94 | 3.18 | 1.35 | .37 | .12 | .06 | 460 |
| 480 | 9.67 | 3.44 | 1.46 | .41 | .14 | .06 | 480 |
| 500 | 10.42 | 3.70 | 1.58 | .43 | .15 | .06 | 500 |
| 550 | 12.44 | 4.42 | 1.89 | .52 | .18 | .07 | 550 |
| 600 | 14.61 | 5.21 | 2.22 | .61 | .21 | .09 | 600 |

| SDR No. | Factor |
|---|---|
| 13.5 | 1.35 |
| 17 | 1.13 |
| 21 | 1.00 |
| 26 | .91 |
| 32.5 | .84 |
| 41 | .785 |
| 51 | .75 |

**Exhibit 3-7 Friction head loss in semirigid plastic irrigation pipelines manufactured of PVC or ABS compounds. Standard dimension ratio - SDR = 21. (Ref. SCS - Fort Worth, Texas, 1967)**

For 125 Pipe

| Q Gallons per min. | 4-inch 4.072 ID | 5-inch 5.033 ID | 6-inch 5.993 ID | 8-inch 7.805 ID | 10-inch 9.728 ID | 12-inch 11.538 ID | Q Gallons per min. |
|---|---|---|---|---|---|---|---|
| | Friction Head Loss in Feet per Hundred Feet | | | | | | |
| 600 | 14.61 | 5.21 | 2.22 | .61 | .21 | .09 | 600 |
| 650 | 16.94 | 6.04 | 2.58 | .71 | .24 | .10 | 650 |
| 700 | 19.45 | 6.92 | 2.96 | .81 | .28 | .12 | 700 |
| 750 | 22.08 | 7.87 | 3.36 | .93 | .32 | .14 | 750 |
| 800 | | 8.88 | 3.78 | 1.04 | .36 | .16 | 800 |
| 850 | | 9.93 | 4.24 | 1.17 | .40 | .17 | 850 |
| 900 | | 11.05 | 4.71 | 1.30 | .44 | .19 | 900 |
| 950 | | 12.18 | 5.21 | 1.44 | .49 | .21 | 950 |
| 1000 | | 13.40 | 5.73 | 1.58 | .54 | .23 | 1000 |
| 1050 | | 14.67 | 6.27 | 1.73 | .59 | .26 | 1050 |
| 1100 | | 16.00 | 6.83 | 1.88 | .65 | .28 | 1100 |
| 1150 | | 17.39 | 7.41 | 2.05 | .70 | .30 | 1150 |
| 1200 | | 18.80 | 8.02 | 2.21 | .76 | .33 | 1200 |
| 1250 | | 20.27 | 8.66 | 2.39 | .82 | .35 | 1250 |
| 1300 | | 21.78 | 9.32 | 2.57 | .88 | .37 | 1300 |
| 1350 | | | 9.99 | 2.76 | .95 | .40 | 1350 |
| 1400 | | | 10.66 | 2.95 | 1.01 | .43 | 1400 |
| 1450 | | | 11.40 | 3.16 | 1.08 | .47 | 1450 |
| 1500 | | | 12.13 | 3.35 | 1.15 | .50 | 1500 |
| 1600 | | | 13.68 | 3.78 | 1.30 | .56 | 1600 |
| 1700 | | | 15.29 | 4.23 | 1.45 | .62 | 1700 |
| 1800 | | | 16.99 | 4.70 | 1.62 | .70 | 1800 |
| 1900 | | | 18.81 | 5.20 | 1.79 | .77 | 1900 |
| 2000 | | | 20.66 | 5.72 | 1.97 | .84 | 2000 |
| 2100 | | | 22.61 | 6.26 | 2.15 | .93 | 2100 |
| 2200 | | | 24.67 | 6.83 | 2.34 | 1.01 | 2200 |
| 2300 | | | | 7.42 | 2.55 | 1.10 | 2300 |
| 2400 | | | | 8.02 | 2.76 | 1.19 | 2400 |
| 2500 | | | | 8.67 | 2.97 | 1.29 | 2500 |
| 2600 | SDR No. | Factor | | 9.31 | 3.20 | 1.39 | 2600 |
| 2700 | 13.5 | 1.35 | | 9.98 | 3.43 | 1.49 | 2700 |
| 2800 | 17 | 1.13 | | 10.67 | 3.67 | 1.59 | 2800 |
| 2900 | 21 | 1.00 | | 11.39 | 3.92 | 1.69 | 2900 |
| 3000 | 26 | .91 | | 12.10 | 4.17 | 1.81 | 3000 |
| 3100 | 32.5 | .84 | | 12.89 | 4.43 | 1.92 | 3100 |
| 3200 | 41 | .785 | | 13.66 | 4.71 | 2.04 | 3200 |
| 3300 | 51 | .75 | | 14.46 | 4.97 | 2.15 | 3300 |
| 3400 | | | | 15.29 | 5.27 | 2.28 | 3400 |
| 3500 | | | | 16.11 | 5.56 | 2.41 | 3500 |
| 3600 | | | | 16.99 | 5.85 | 2.53 | 3600 |
| 3700 | | | | 17.89 | 6.17 | 2.67 | 3700 |
| 3800 | | | | 18.76 | 6.47 | 2.80 | 3800 |
| 3900 | | | | 19.69 | 6.79 | 2.94 | 3900 |
| 4000 | | | | 20.67 | 7.11 | 3.08 | 4000 |

**Exhibit 3-7** **Friction head loss in semirigid plastic irrigation pipelines manufactured of PVC or ABS compounds. Standard dimension ratio - SDR = 21. (Ref. SCS - Fort Worth, Texas, 1967)**

For PIP Pipe

| Q Gallons per min. | 4-inch 3.736 ID | 6-inch 5.556 ID | 8-inch 7.382 ID | 10-inch 9.228 ID | 12-inch 11.074 ID | Q Gallons per min. |
|---|---|---|---|---|---|---|
| | Friction Head Loss in Feet per Hundred Feet | | | | | |
| 15 | .02 | | | | | 15 |
| 20 | .04 | | | | | 20 |
| 25 | .06 | | | | | 25 |
| 30 | .09 | .01 | | | | 30 |
| 35 | .12 | .02 | | | | 35 |
| 40 | .15 | .02 | | | | 40 |
| 45 | .18 | .03 | | | | 45 |
| 50 | .22 | .03 | | | | 50 |
| 55 | .27 | .04 | | | | 55 |
| 60 | .31 | .05 | | | | 60 |
| 65 | .36 | .05 | .01 | | | 65 |
| 70 | .42 | .06 | .02 | | | 70 |
| 75 | .47 | .07 | .02 | | | 75 |
| 80 | .53 | .08 | .02 | | | 80 |
| 85 | .60 | .09 | .02 | | | 85 |
| 90 | .66 | .10 | .02 | | | 90 |
| 95 | .73 | .11 | .03 | | | 95 |
| 100 | .80 | .12 | .03 | | | 100 |
| 110 | .96 | .14 | .03 | | | 110 |
| 120 | 1.13 | .16 | .04 | .01 | | 120 |
| 130 | 1.31 | .19 | .05 | .02 | | 130 |
| 140 | 1.50 | .22 | .05 | .02 | | 140 |
| 150 | 1.70 | .25 | .06 | .02 | | 150 |
| 160 | 1.92 | .28 | .07 | .02 | | 160 |
| 170 | 2.15 | .31 | .08 | .03 | | 170 |
| 180 | 2.39 | .35 | .09 | .03 | | 180 |
| 190 | 2.64 | .38 | .10 | .03 | | 190 |
| 200 | 2.90 | .42 | .11 | .04 | .01 | 200 |
| 220 | 3.46 | .50 | .13 | .04 | .02 | 220 |
| 240 | 4.07 | .59 | .15 | .05 | .02 | 240 |
| 260 | 4.72 | .68 | .17 | .06 | .02 | 260 |
| 280 | 5.41 | .78 | .20 | .07 | .03 | 280 |
| 300 | 6.15 | .89 | .22 | .08 | .03 | 300 |
| 320 | 6.93 | 1.00 | .25 | .08 | .03 | 320 |
| 340 | 7.76 | 1.12 | .28 | .09 | .04 | 340 |
| 360 | 8.62 | 1.25 | .31 | .11 | .04 | 360 |
| 380 | 9.53 | 1.38 | .35 | .12 | .05 | 380 |
| 400 | 10.48 | 1.52 | .38 | .13 | .05 | 400 |
| 420 | 11.47 | 1.66 | .42 | .14 | .06 | 420 |
| 440 | 12.50 | 1.81 | .45 | .15 | .06 | 440 |
| 460 | 13.58 | 1.96 | .49 | .17 | .07 | 460 |
| 480 | 14.69 | 2.13 | .53 | .18 | .07 | 480 |
| 500 | 15.84 | 2.29 | .57 | .19 | .08 | 500 |
| 550 | 18.90 | 2.74 | .69 | .23 | .10 | 550 |
| 600 | 22.21 | 3.21 | .81 | .27 | .11 | 600 |

Table based on Hazen-Williams equation - $C_1$ = 150.

To find friction head loss in PVC or ABS pipe having a standard dimension ratio other than 21, the values in the table should be multiplied by the appropriate conversion factor shown below:

| SDR No. | Conversion Factor |
|---|---|
| 13.5 | 1.34 |
| 17 | 1.13 |
| 21 | 1.00 |
| 26 | .91 |
| 32.5 | .84 |
| 41 | .785 |
| 51 | .75 |

Exhibit 3-7 Friction head loss in semirigid plastic irrigation pipelines manufactured of PVC or ABS compounds. Standard dimension ratio - SDR = 21. (Ref. SCS - Fort Worth, Texas, 1967)

For PIP Pipe

| Q Gallons per min. | 4-inch 3.736 ID | 6-inch 5.556 ID | 8-inch 7.382 ID | 10-inch 9.228 ID | 12-inch 11.074 ID | Q Gallons per min. |
|---|---|---|---|---|---|---|
| | Friction Head Loss in Feet per Hundred Feet | | | | | |
| 650 | | 3.73 | .93 | .31 | .13 | 650 |
| 700 | | 4.28 | 1.07 | .36 | .15 | 700 |
| 750 | | 4.86 | 1.22 | .41 | .17 | 750 |
| 800 | | 5.47 | 1.37 | .46 | .19 | 800 |
| 850 | | 6.13 | 1.53 | .52 | .21 | 850 |
| 900 | | 6.81 | 1.71 | .58 | .24 | 900 |
| 950 | | 7.53 | 1.89 | .64 | .26 | 950 |
| 1000 | | 8.28 | 2.07 | .70 | .29 | 1000 |
| 1050 | | 9.06 | 2.27 | .77 | .31 | 1050 |
| 1100 | | 9.87 | 2.47 | .83 | .34 | 1100 |
| 1150 | | 10.72 | 2.69 | .91 | .37 | 1150 |
| 1200 | | 11.60 | 2.91 | .98 | .40 | 1200 |
| 1250 | | 12.51 | 3.13 | 1.06 | .43 | 1250 |
| 1300 | | 13.45 | 3.37 | 1.14 | .47 | 1300 |
| 1350 | | 14.43 | 3.61 | 1.22 | .50 | 1350 |
| 1400 | | 15.43 | 3.87 | 1.30 | .54 | 1400 |
| 1450 | | 16.47 | 4.13 | 1.39 | .57 | 1450 |
| 1500 | | 17.54 | 4.39 | 1.48 | .61 | 1500 |
| 1600 | | 19.76 | 4.95 | 1.67 | .69 | 1600 |
| 1700 | | 22.11 | 5.54 | 1.87 | .77 | 1700 |
| 1800 | | 24.58 | 6.16 | 2.08 | .85 | 1800 |
| 1900 | | | 6.81 | 2.29 | .94 | 1900 |
| 2000 | | | 7.49 | 2.52 | 1.04 | 2000 |
| 2100 | | | 8.19 | 2.76 | 1.14 | 2100 |
| 2200 | | | 8.93 | 3.01 | 1.24 | 2200 |
| 2300 | | | 9.70 | 3.27 | 1.35 | 2300 |
| 2400 | | | 10.49 | 3.54 | 1.46 | 2400 |
| 2500 | | | 11.32 | 3.82 | 1.57 | 2500 |
| 2600 | | | 12.17 | 4.10 | 1.69 | 2600 |
| 2700 | | | 13.05 | 4.40 | 1.81 | 2700 |
| 2800 | | | 13.96 | 4.71 | 1.94 | 2800 |
| 2900 | | | 14.90 | 5.02 | 2.07 | 2900 |
| 3000 | | | 15.86 | 5.35 | 2.20 | 3000 |
| 3100 | | | 16.85 | 5.68 | 2.34 | 3100 |
| 3200 | | | 17.88 | 6.03 | 2.48 | 3200 |
| 3300 | | | 18.92 | 6.38 | 2.62 | 3300 |
| 3400 | | | 20.00 | 6.74 | 2.77 | 3400 |
| 3500 | | | 21.10 | 7.11 | 2.93 | 3500 |
| 3600 | | | 22.23 | 7.50 | 3.08 | 3600 |
| 3700 | | | 23.39 | 7.89 | 3.24 | 3700 |
| 3800 | | | 24.57 | 8.28 | 3.41 | 3800 |
| 3900 | | | | 8.69 | 3.58 | 3900 |
| 4000 | | | | 9.11 | 3.75 | 4000 |

| SDR No. | Factor |
|---|---|
| 13.5 | 1.34 |
| 17 | 1.13 |
| 21 | 1.00 |
| 26 | .91 |
| 32.5 | .84 |
| 41 | .785 |
| 51 | .75 |

**Exhibit 3-7 Friction head loss in semirigid plastic irrigation pipelines manufactured of PVC or ABS compounds. Standard dimension ratio - SDR = 21. (Ref. SCS - Fort Worth, Texas, 1967)**

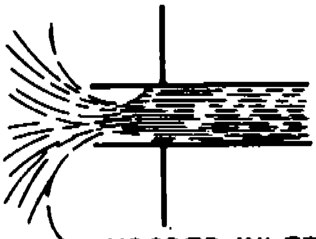
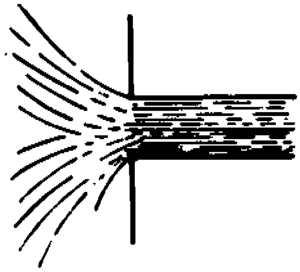
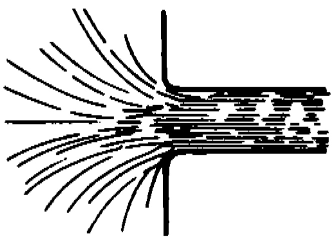
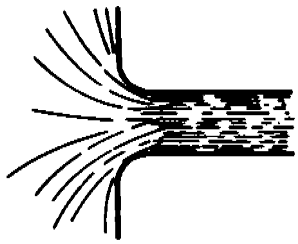
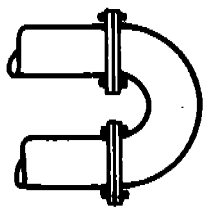
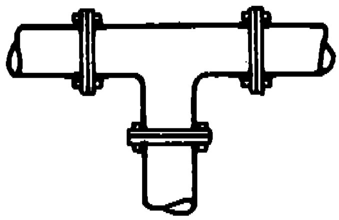
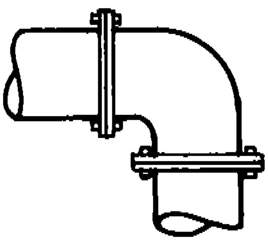
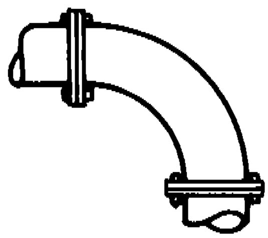
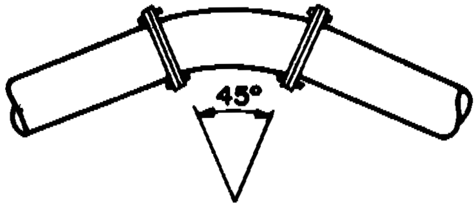
| PIPE ENTRANCES | | | |
|---|---|---|---|
| INWARD PROJECTING PIPE | $K_e$ | SHARP-CORNERED | $K_e$ |
|  | 0.78 |  | 0.50 |
| HOODED INLET | 1.00 | | |
| SLIGHTLY ROUNDED | $K_e$ | BELL MOUTH | $K_e$ |
|  | 0.23 |  | 0.04 |
| **PIPE BENDS** | | | |
| RETURN BEND | $K_{RB}$ | STANDARD-TEE | $K_{ST}$ |
|  | 2.20 |  | 1.80 |
| STANDARD 90°-ELBOW | $K_{90}$ | LONG RADIUS ELBOW | $K_{LR}$ |
|  | 0.90 |  | 0.60 |

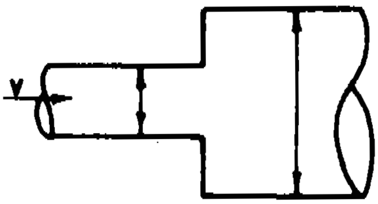
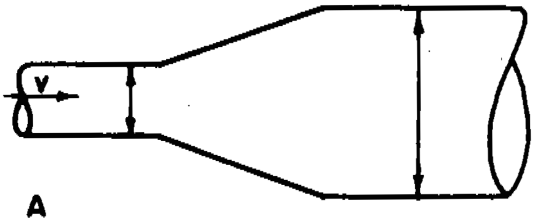
Exhibit 3-8 Head loss coefficients for pipe entrances and bends

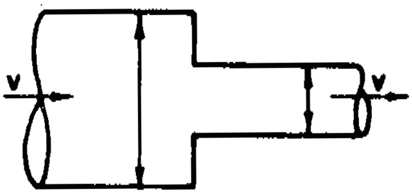
(Sheet 1 of 2)

## PIPE BENDS

| 45° ELBOW | $K_{45}$ |
|---|---|
|   | 0.42 |

## ENLARGEMENTS AND CONTRACTIONS

| SUDDEN ENLARGEMENT | $K_{SE}$ | GRADUAL ENLARGEMENT |
|---|---|---|
|   | $\left[1-\left(\frac{d_1}{d_2}\right)^2\right]^2$ |   |

| SUDDEN CONTRACTION | K FACTORS FOR A & B SEE: |
|---|---|
|   | REFERENCE:<br>HANDBOOK OF HYDRAULICS<br>BY H.W. KING<br>PAGES 6 - 18<br>(FOURTH EDITION) |

## VALVES

| WIDE OPEN | K | PARTIALLY CLOSED |
|---|---|---|
| FOOT VALVE | 0.40 | FOR K FACTORS SEE:<br>REFERENCE: |
| GLOBE VALVE | 10.00 | BY H.W. KING |
| ANGLE VALVE | 5.00 | PAGES 6 - 18 |
| GATE VALVE | 0.20 | (FOURTH EDITION) |

Exhibit 3-8 Head loss coefficients for pipe entrances and bends
(Sheet 2 of 2)

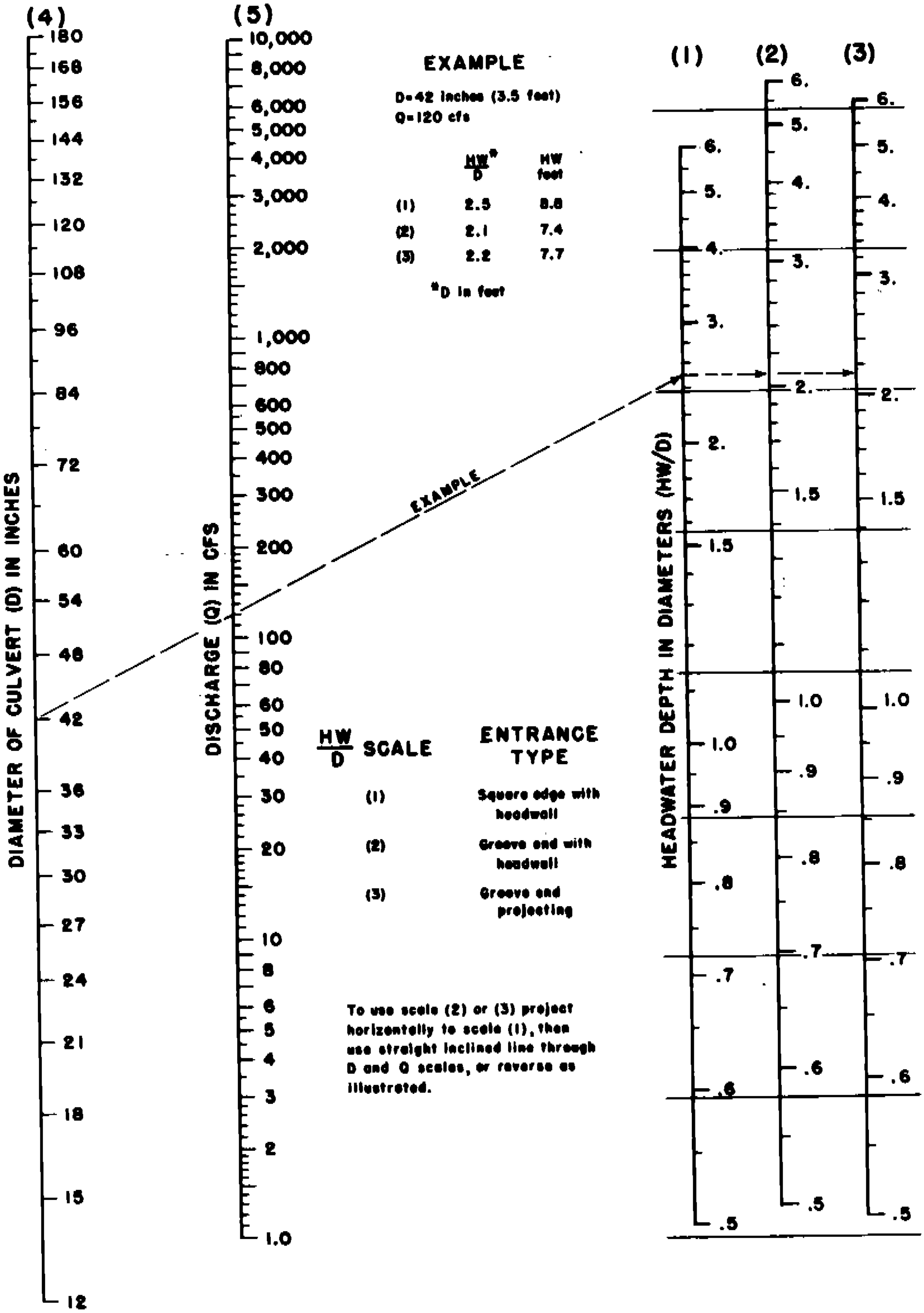



Exhibit 3-9 Headwater depth for concrete pipe culverts with inlet control (Ref. Hyd. Eng. Cir. No. 5, USBPR, 1965)

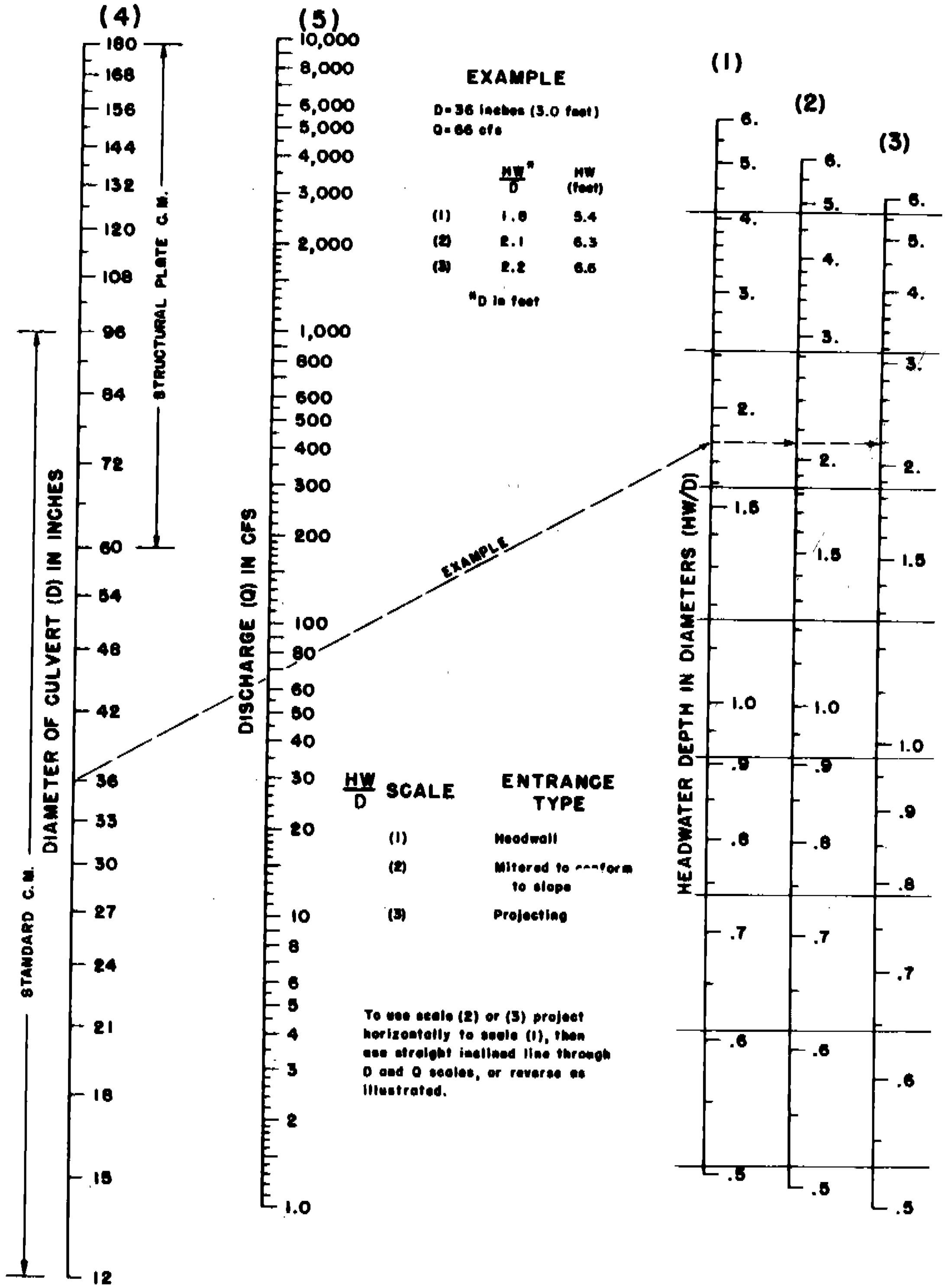



Exhibit 3-10 Headwater depth for CM pipe culverts with inlet control (Ref. Hyd. Eng. Cir. No. 5, USBPR, 1965)

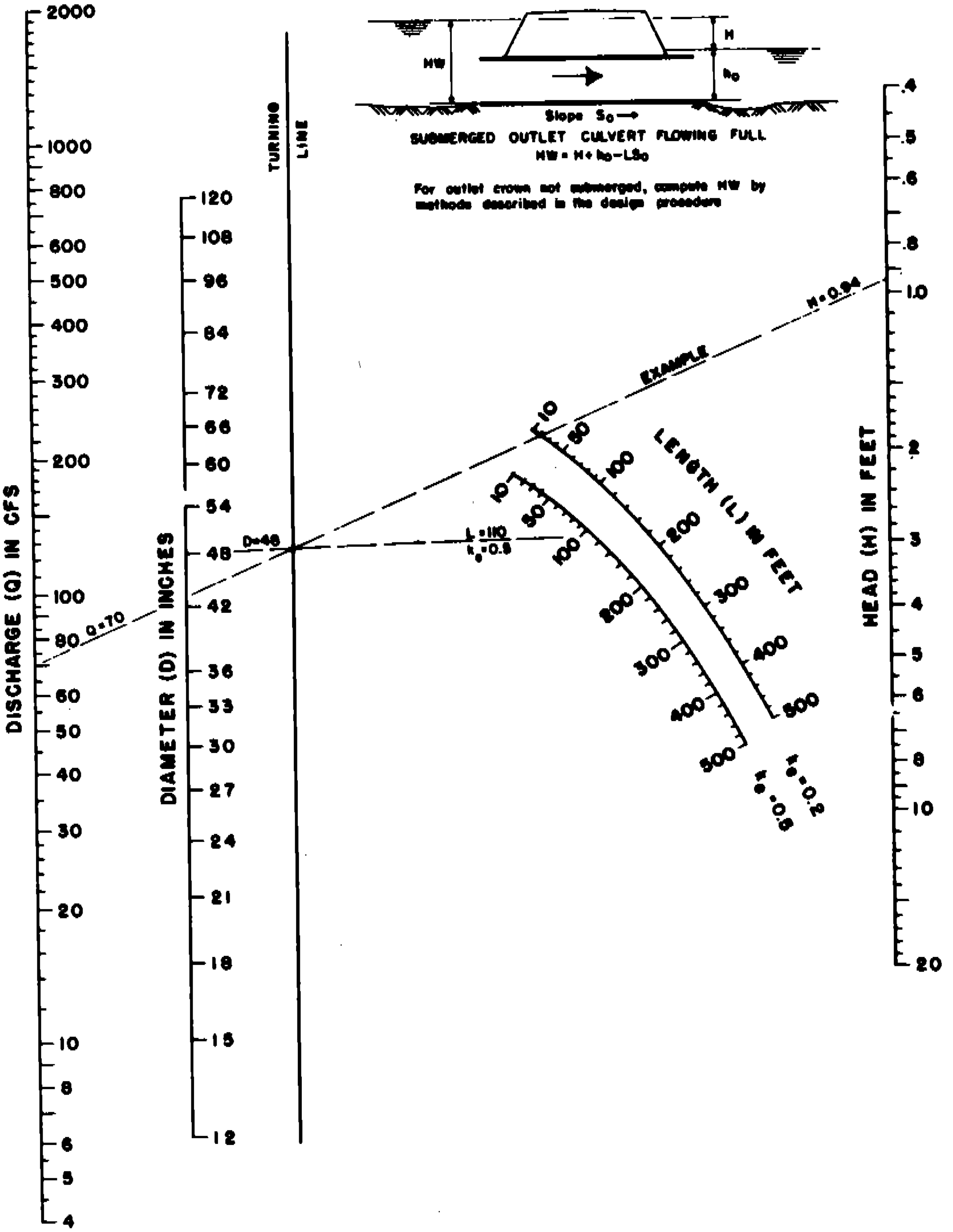



Exhibit 3-11 Head for concrete pipe culverts flowing full with outlet control n = 0.012 (Ref. Hyd. Eng. Cir. No. 5, USBPR, 1965)

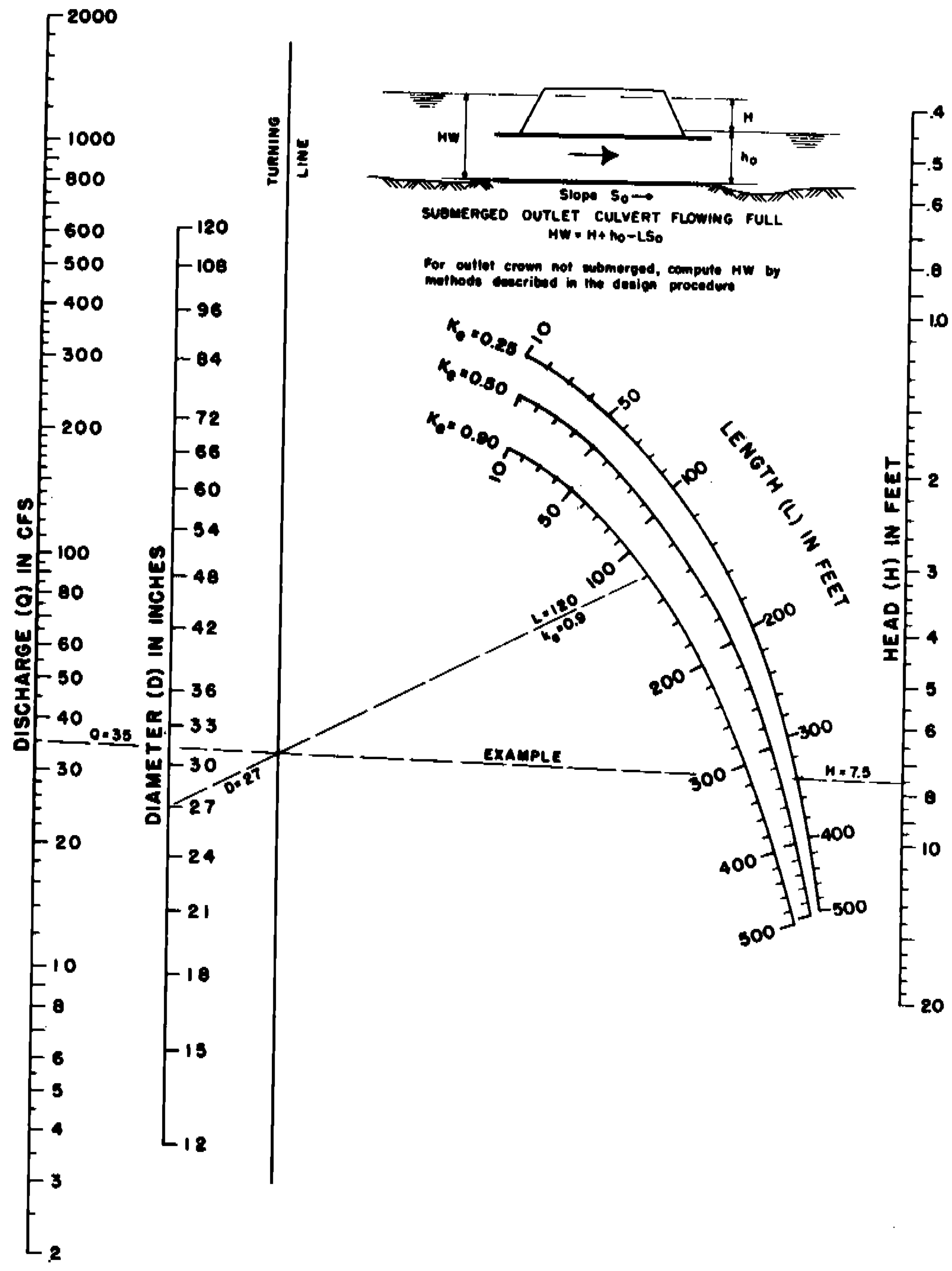
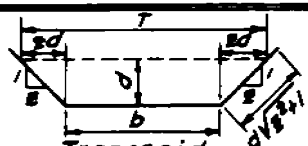
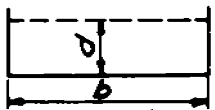
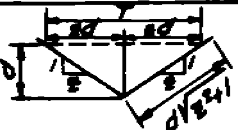
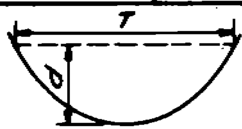
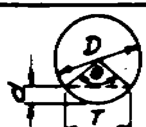
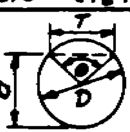



Exhibit 3-12 Head for CM pipe culverts flowing full with outlet control n = 0.024 (Ref. Hyd. Eng. Cir. No. 5, USBPR, 1965)

| Section | Area $a$ | Wetted Perimeter $p$ | Hydraulic Radius $r$ | Top Width $T$ |
|---|---|---|---|---|
|  Trapezoid | $bd+zd^2$ | $b+2d\sqrt{z^2+1}$ | $\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$ | $b+2zd$ |
|  Rectangle | $bd$ | $b+2d$ | $\frac{bd}{b+2d}$ | $b$ |
|  Triangle | $zd^2$ | $2d\sqrt{z^2+1}$ | $\frac{zd}{2\sqrt{z^2+1}}$ | $2zd$ |
|  Parabola | $\frac{2}{3}dT$ | $T+\frac{8d^2}{3T}$ [1] | $\frac{2dT^2}{3T^2+8d^2}$ [1] | $\frac{3a}{2d}$ |
|  Circle - < ½ 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 - > ½ 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 $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2}+\frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
} Insert $\theta$ in degrees in above equations

Exhibit 3-13 Elements of channel sections (Ref. NEH Section 5, ES-33)

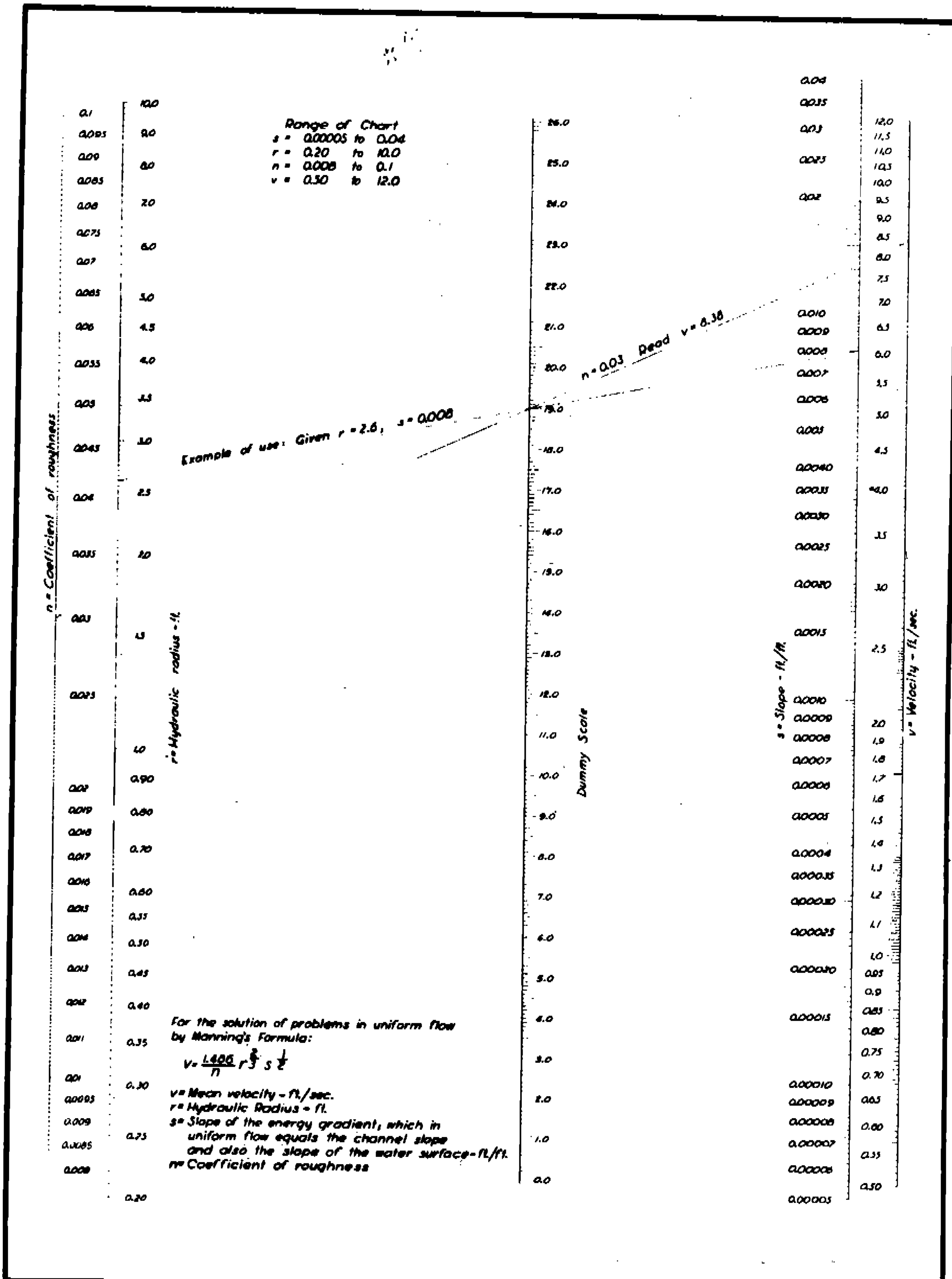



Exhibit 3-14 Solution of Manning's formula for uniform flow (Ref. NEH Section 5, ES-54)

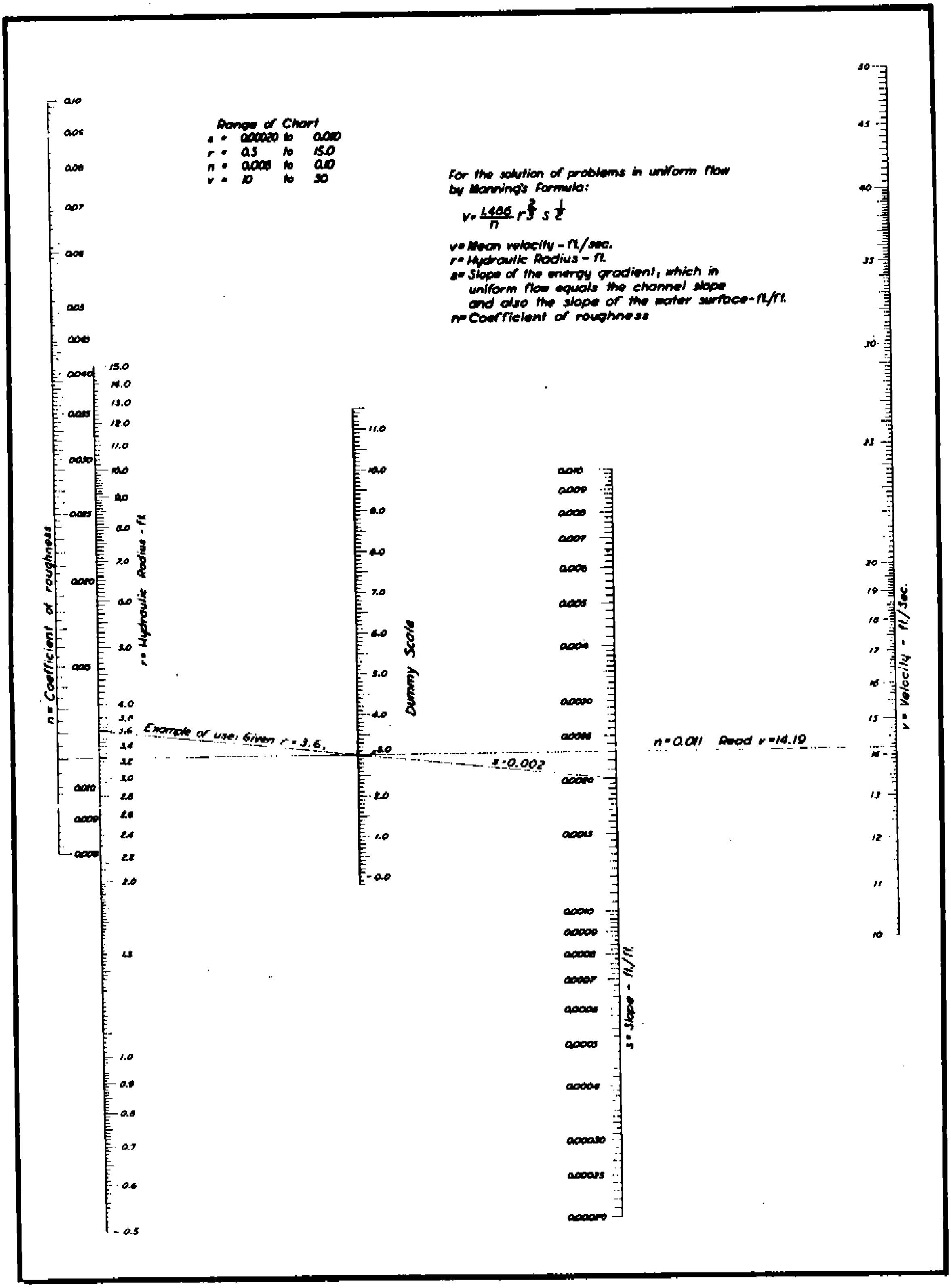



Exhibit 3-14 Solution of Manning's formula for uniform flow (Ref. NEH Section 5, ES-54) Sheet 2 of 4

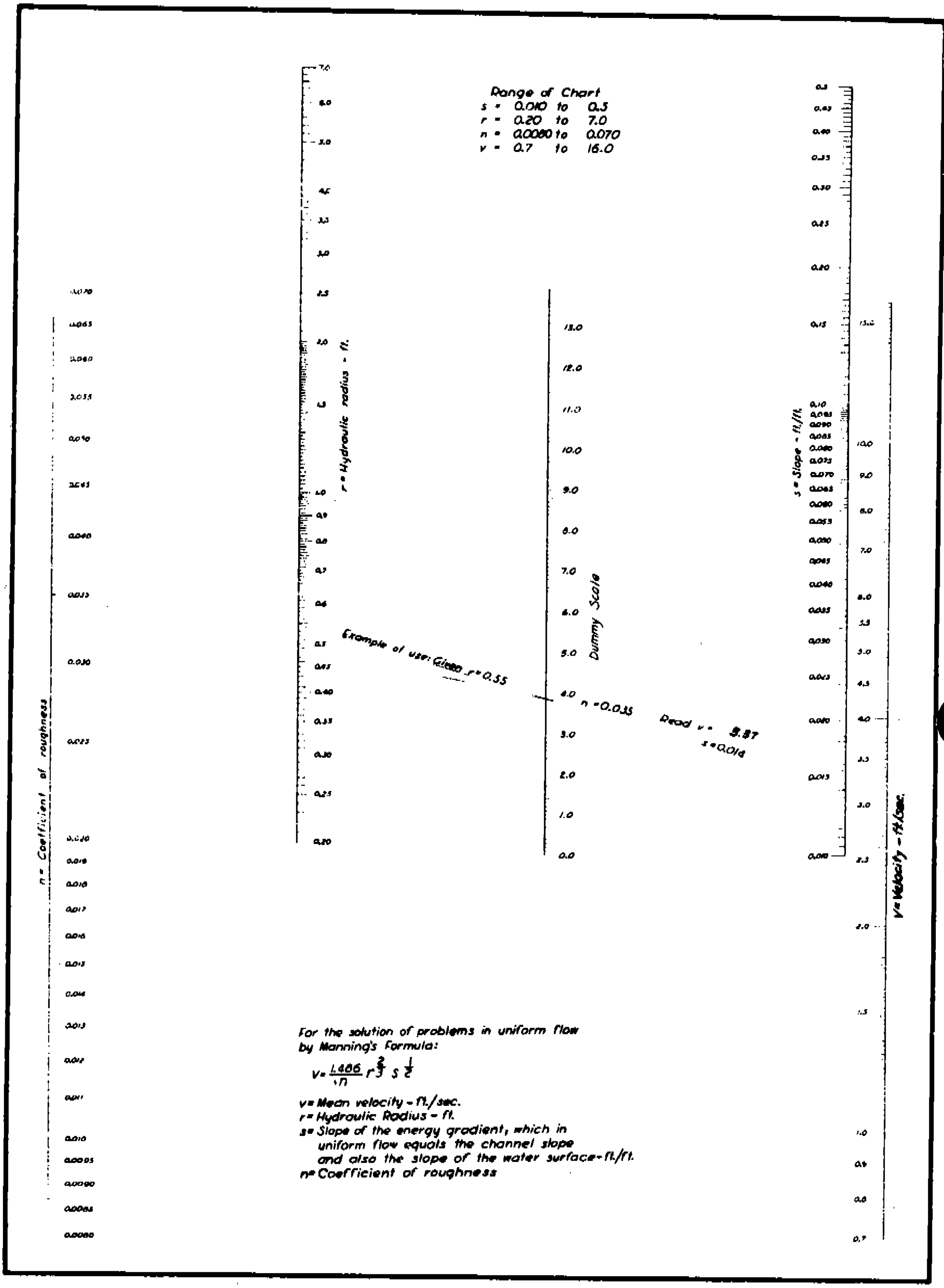



Exhibit 3-14 Solution of Manning's formula for uniform flow (Ref. NEH Section 5, ES-54)

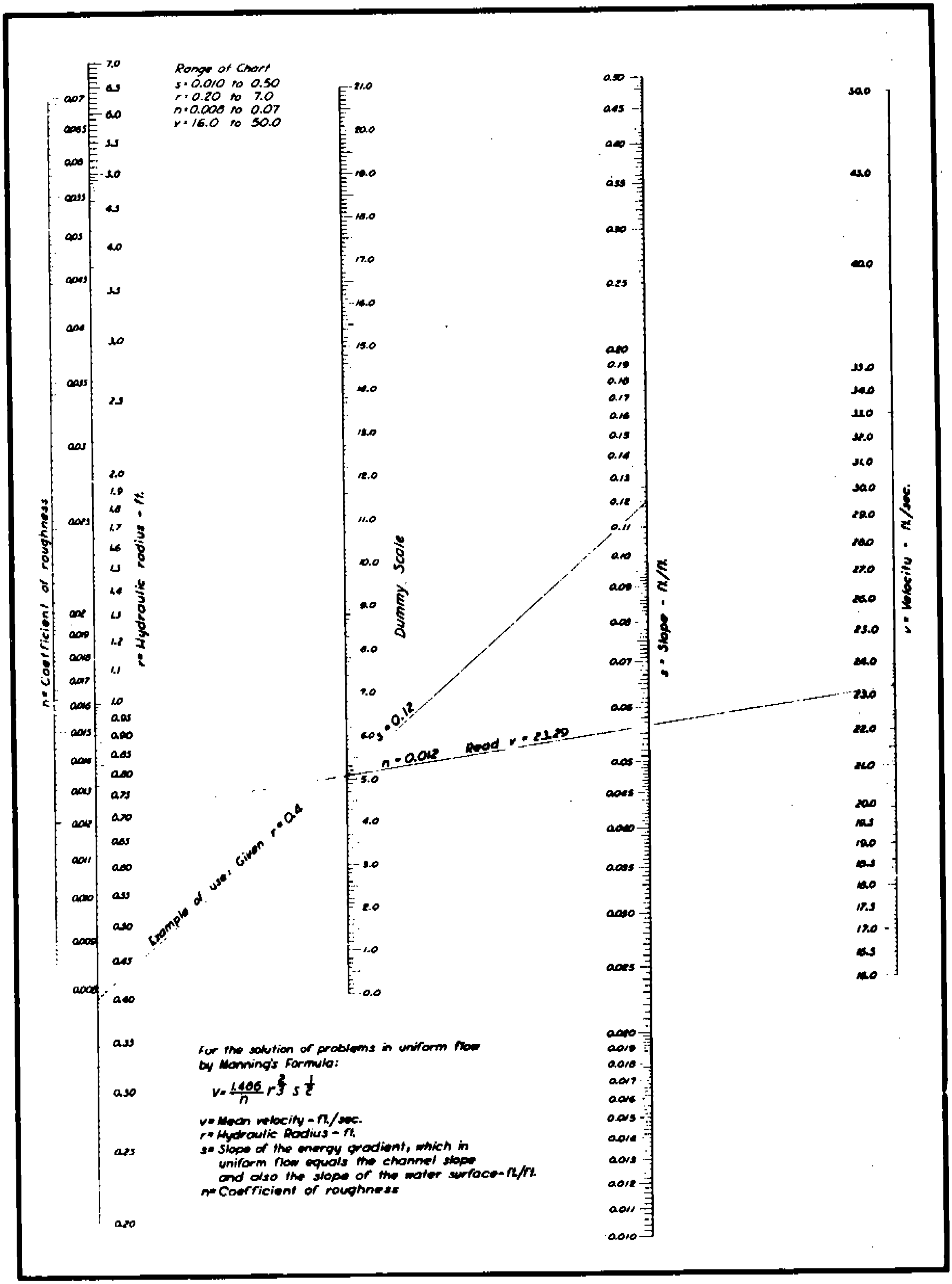



Exhibit 3-14 Solution of Manning's formula for uniform flow (Ref. NEH Section 5, ES-54)

Sheet 4 of 4

| Head, H[1] | | Discharge, Q, for crest length, L, of— | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 foot | 1.5 feet | 2 feet | 3 feet | 4 feet | 6 feet | 8 feet | 10 feet |
| *Feet* | *Inches* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* |
| 0.10 | 1 3/16 | 0.11 | 0.16 | 0.22 | 0.33 | 0.44 | 0.62 | 0.84 | 1.05 |
| .11 | 1 5/16 | .12 | .18 | .25 | .37 | .50 | .73 | .97 | 1.21 |
| .12 | 1 7/16 | .14 | .20 | .28 | .42 | .57 | .83 | 1.10 | 1.38 |
| .13 | 1 9/16 | .15 | .22 | .32 | .47 | .64 | .93 | 1.24 | 1.56 |
| .14 | 1 11/16 | .17 | .25 | .35 | .53 | .71 | 1.04 | 1.39 | 1.74 |
| .15 | 1 13/16 | .19 | .28 | .39 | .58 | .79 | 1.15 | 1.54 | 1.93 |
| .16 | 1 15/16 | .21 | .31 | .43 | .64 | .86 | 1.27 | 1.70 | 2.12 |
| .17 | 2 1/16 | .23 | .34 | .47 | .70 | .95 | 1.39 | 1.86 | 2.33 |
| .18 | 2 3/16 | .25 | .37 | .51 | .76 | 1.03 | 1.52 | 2.03 | 2.53 |
| .19 | 2 1/4 | .27 | .40 | .55 | .83 | 1.11 | 1.64 | 2.20 | 2.75 |
| .20 | 2 3/8 | .29 | .44 | .59 | .89 | 1.19 | 1.78 | 2.37 | 2.97 |
| .21 | 2 1/2 | .31 | .47 | .63 | .95 | 1.28 | 1.91 | 2.55 | 3.19 |
| .22 | 2 5/8 | .34 | .50 | .68 | 1.02 | 1.37 | 2.05 | 2.73 | 3.42 |
| .23 | 2 3/4 | .36 | .54 | .72 | 1.09 | 1.46 | 2.19 | 2.92 | 3.66 |
| .24 | 2 7/8 | .38 | .57 | .77 | 1.16 | 1.55 | 2.33 | 3.11 | 3.90 |
| .25 | 3 | .40 | .61 | .82 | 1.23 | 1.65 | 2.48 | 3.31 | 4.14 |
| .26 | 3 1/8 | .43 | .65 | .86 | 1.31 | 1.75 | 2.63 | 3.51 | 4.39 |
| .27 | 3 1/4 | .45 | .68 | .91 | 1.38 | 1.85 | 2.78 | 3.71 | 4.65 |
| .28 | 3 3/8 | .48 | .72 | .96 | 1.46 | 1.95 | 2.93 | 3.92 | 4.91 |
| .29 | 3 1/2 | .50 | .76 | 1.02 | 1.53 | 2.05 | 3.09 | 4.13 | 5.17 |
| .30 | 3 5/8 | .53 | .80 | 1.07 | 1.61 | 2.16 | 3.25 | 4.34 | 5.44 |
| .31 | 3 3/4 | .55 | .84 | 1.12 | 1.69 | 2.26 | 3.41 | 4.56 | 5.71 |
| .32 | 3 13/16 | .58 | .88 | 1.18 | 1.77 | 2.37 | 3.58 | 4.78 | 5.99 |
| .33 | 3 15/16 | .61 | .92 | 1.23 | 1.86 | 2.48 | 3.75 | 5.01 | 6.27 |
| .34 | 4 1/16 | .63 | .96 | 1.28 | 1.94 | 2.60 | 3.92 | 5.24 | 6.56 |
| .35 | 4 3/16 | .66 | 1.00 | 1.34 | 2.02 | 2.71 | 4.09 | 5.47 | 6.85 |
| .36 | 4 5/16 | .69 | 1.04 | 1.40 | 2.11 | 2.82 | 4.26 | 5.70 | 7.14 |
| .37 | 4 7/16 | .72 | 1.08 | 1.45 | 2.20 | 2.94 | 4.44 | 5.94 | 7.44 |
| .38 | 4 9/16 | .74 | 1.13 | 1.51 | 2.28 | 3.06 | 4.62 | 6.18 | 7.74 |
| .39 | 4 11/16 | .77 | 1.17 | 1.57 | 2.37 | 3.18 | 4.80 | 6.43 | 8.05 |
| .40 | 4 13/16 | .80 | 1.21 | 1.63 | 2.46 | 3.30 | 4.99 | 6.67 | 8.36 |
| .41 | 4 15/16 | .83 | 1.26 | 1.69 | 2.55 | 3.42 | 5.17 | 6.92 | 8.67 |
| .42 | 5 1/16 | .86 | 1.30 | 1.75 | 2.65 | 3.54 | 5.36 | 7.18 | 8.99 |
| .43 | 5 3/16 | .89 | 1.35 | 1.81 | 2.74 | 3.67 | 5.55 | 7.43 | 9.31 |
| .44 | 5 1/4 | .92 | 1.40 | 1.88 | 2.83 | 3.80 | 5.75 | 7.69 | 9.63 |
| .45 | 5 3/8 | .96 | 1.44 | 1.94 | 2.93 | 3.93 | 5.94 | 7.95 | 9.96 |
| .46 | 5 1/2 | .99 | 1.49 | 2.00 | 3.03 | 4.05 | 6.14 | 8.22 | 10.3 |
| .47 | 5 5/8 | 1.02 | 1.54 | 2.07 | 3.12 | 4.18 | 6.34 | 8.48 | 10.6 |
| .48 | 5 3/4 | 1.05 | 1.59 | 2.13 | 3.22 | 4.32 | 6.54 | 8.75 | 11.0 |
| .49 | 5 7/8 | 1.08 | 1.64 | 2.20 | 3.32 | 4.45 | 6.74 | 9.03 | 11.3 |
| .50 | 6 | 1.11 | 1.68 | 2.26 | 3.42 | 4.58 | 6.95 | 9.30 | 11.7 |
| .51 | 6 1/8 | 1.15 | 1.73 | 2.33 | 3.52 | 4.72 | 7.15 | 9.58 | 12.0 |
| .52 | 6 1/4 | 1.18 | 1.78 | 2.40 | 3.62 | 4.86 | 7.36 | 9.86 | 12.4 |
| .53 | 6 3/8 | 1.21 | 1.84 | 2.46 | 3.73 | 4.99 | 7.57 | 10.1 | 12.7 |
| .54 | 6 1/2 | 1.25 | 1.89 | 2.53 | 3.83 | 5.13 | 7.79 | 10.4 | 13.1 |
| .55 | 6 5/8 | 1.28 | 1.94 | 2.60 | 3.94 | 5.27 | 8.00 | 10.7 | 13.4 |
| .56 | 6 3/4 | 1.31 | 1.99 | 2.67 | 4.04 | 5.42 | 8.22 | 11.0 | 13.8 |
| .57 | 6 13/16 | 1.35 | 2.04 | 2.74 | 4.15 | 5.56 | 8.43 | 11.3 | 14.2 |
| .58 | 6 15/16 | 1.38 | 2.09 | 2.81 | 4.26 | 5.70 | 8.65 | 11.6 | 14.5 |
| .59 | 7 1/16 | 1.42 | 2.15 | 2.88 | 4.36 | 5.85 | 8.88 | 11.9 | 14.9 |
| .60 | 7 3/16 | 1.45 | 2.20 | 2.96 | 4.47 | 6.00 | 9.10 | 12.2 | 15.3 |
| .61 | 7 5/16 | 1.49 | 2.25 | 3.03 | 4.59 | 6.14 | 9.33 | 12.5 | 15.7 |
| .62 | 7 7/16 | 1.52 | 2.31 | 3.10 | 4.69 | 6.29 | 9.55 | 12.8 | 16.1 |
| .63 | 7 9/16 | 1.56 | 2.36 | 3.17 | 4.81 | 6.44 | 9.78 | 13.1 | 16.4 |
| .64 | 7 11/16 | 1.60 | 2.42 | 3.25 | 4.92 | 6.59 | 10.0 | 13.4 | 16.8 |

Exhibit 3-15 Discharge for contracted rectangular weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950)

(Sheet 1 of 3)

| Head, H[1] | | Discharge, Q, for crest length, L, of— | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 foot | 1.5 feet | 2 feet | 3 feet | 4 feet | 6 feet | 8 feet | 10 feet |
| *Feet* | *Inches* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* |
| 0.65 | 7 13/16 | 1.63 | 2.47 | 3.32 | 5.03 | 6.75 | 10.2 | 13.7 | 17.2 |
| .66 | 7 15/16 | 1.67 | 2.53 | 3.40 | 5.15 | 6.90 | 10.5 | 14.0 | 17.6 |
| .67 | 8 1/16 | 1.71 | 2.59 | 3.47 | 5.26 | 7.05 | 10.7 | 14.4 | 18.0 |
| .68 | 8 3/16 | 1.74 | 2.64 | 3.56 | 5.38 | 7.21 | 10.9 | 14.7 | 18.4 |
| .69 | 8 1/4 | 1.78 | 2.70 | 3.63 | 5.49 | 7.36 | 11.2 | 15.0 | 18.8 |
| .70 | 8 3/8 | 1.82 | 2.76 | 3.71 | 5.61 | 7.52 | 11.4 | 15.3 | 19.2 |
| .71 | 8 1/2 | 1.86 | 2.81 | 3.78 | 5.73 | 7.68 | 11.7 | 15.7 | 19.6 |
| .72 | 8 5/8 | 1.90 | 2.87 | 3.86 | 5.85 | 7.84 | 11.9 | 16.0 | 20.1 |
| .73 | 8 3/4 | 1.93 | 2.93 | 3.94 | 5.97 | 8.00 | 12.2 | 16.3 | 20.5 |
| .74 | 8 7/8 | 1.97 | 2.99 | 4.02 | 6.09 | 8.17 | 12.4 | 16.6 | 20.9 |
| .75 | 9 | 2.01 | 3.05 | 4.10 | 6.21 | 8.33 | 12.7 | 17.0 | 21.3 |
| .76 | 9 1/8 | 2.05 | 3.11 | 4.18 | 6.33 | 8.49 | 12.9 | 17.3 | 21.7 |
| .77 | 9 1/4 | 2.09 | 3.17 | 4.26 | 6.45 | 8.66 | 13.2 | 17.7 | 22.2 |
| .78 | 9 3/8 | 2.13 | 3.23 | 4.34 | 6.58 | 8.82 | 13.4 | 18.0 | 22.6 |
| .79 | 9 1/2 | 2.17 | 3.29 | 4.42 | 6.70 | 8.99 | 13.7 | 18.3 | 23.0 |
| .80 | 9 5/8 | 2.21 | 3.35 | 4.51 | 6.83 | 9.16 | 13.9 | 18.7 | 23.4 |
| .81 | 9 3/4 | 2.25 | 3.41 | 4.59 | 6.95 | 9.33 | 14.2 | 19.0 | 23.9 |
| .82 | 9 13/16 | 2.29 | 3.47 | 4.67 | 7.08 | 9.50 | 14.4 | 19.4 | 24.3 |
| .83 | 9 15/16 | 2.33 | 3.54 | 4.75 | 7.21 | 9.67 | 14.7 | 19.7 | 24.8 |
| .84 | 10 1/16 | 2.37 | 3.60 | 4.84 | 7.33 | 9.84 | 15.0 | 20.1 | 25.2 |
| .85 | 10 3/16 | 2.41 | 3.66 | 4.92 | 7.46 | 10.0 | 15.2 | 20.4 | 25.7 |
| .86 | 10 5/16 | 2.46 | 3.72 | 5.01 | 7.59 | 10.2 | 15.5 | 20.8 | 26.1 |
| .87 | 10 7/16 | 2.50 | 3.79 | 5.10 | 7.72 | 10.4 | 15.7 | 21.1 | 26.6 |
| .88 | 10 9/16 | 2.54 | 3.85 | 5.18 | 7.85 | 10.5 | 16.0 | 21.5 | 27.0 |
| .89 | 10 11/16 | 2.58 | 3.92 | 5.27 | 7.99 | 10.7 | 16.3 | 21.9 | 27.5 |
| .90 | 10 13/16 | 2.62 | 3.98 | 5.35 | 8.12 | 10.9 | 16.5 | 22.2 | 27.9 |
| .91 | 10 15/16 | 2.67 | 4.05 | 5.44 | 8.25 | 11.1 | 16.8 | 22.6 | 28.4 |
| .92 | 11 1/16 | 2.71 | 4.11 | 5.53 | 8.38 | 11.2 | 17.1 | 23.0 | 28.8 |
| .93 | 11 3/16 | 2.75 | 4.18 | 5.62 | 8.52 | 11.4 | 17.4 | 23.3 | 29.3 |
| .94 | 11 1/4 | 2.79 | 4.24 | 5.71 | 8.65 | 11.6 | 17.6 | 23.7 | 29.8 |
| .95 | 11 3/8 | 2.84 | 4.31 | 5.80 | 8.79 | 11.8 | 17.9 | 24.1 | 30.2 |
| .96 | 11 1/2 | 2.88 | 4.37 | 5.89 | 8.93 | 12.0 | 18.2 | 24.5 | 30.7 |
| .97 | 11 5/8 | 2.93 | 4.44 | 5.98 | 9.06 | 12.2 | 18.5 | 24.8 | 31.2 |
| .98 | 11 3/4 | 2.97 | 4.51 | 6.07 | 9.20 | 12.3 | 18.8 | 25.2 | 31.7 |
| .99 | 11 7/8 | 3.01 | 4.57 | 6.15 | 9.34 | 12.5 | 19.0 | 25.6 | 32.2 |
| 1.00 | 12 | 3.06 | 4.64 | 6.25 | 9.48 | 12.7 | 19.3 | 26.0 | 32.6 |
| 1.01 | 12 1/8 | ------ | 4.71 | 6.34 | 9.62 | 12.9 | 19.6 | 26.4 | 33.1 |
| 1.02 | 12 1/4 | ------ | 4.78 | 6.43 | 9.76 | 13.1 | 19.9 | 26.7 | 33.6 |
| 1.03 | 12 3/8 | ------ | 4.85 | 6.52 | 9.90 | 13.3 | 20.2 | 27.1 | 34.1 |
| 1.04 | 12 1/2 | ------ | 4.92 | 6.62 | 10.0 | 13.5 | 20.5 | 27.5 | 34.6 |
| 1.05 | 12 5/8 | ------ | 4.98 | 6.71 | 10.2 | 13.7 | 20.7 | 27.9 | 35.1 |
| 1.06 | 12 3/4 | ------ | 5.05 | 6.80 | 10.3 | 13.8 | 21.0 | 28.3 | 35.6 |
| 1.07 | 12 13/16 | ------ | 5.12 | 6.90 | 10.5 | 14.0 | 21.3 | 28.7 | 36.1 |
| 1.08 | 12 15/16 | ------ | 5.20 | 6.99 | 10.6 | 14.2 | 21.6 | 29.1 | 36.6 |
| 1.09 | 13 1/16 | ------ | 5.26 | 7.09 | 10.8 | 14.4 | 21.9 | 29.5 | 37.1 |
| 1.10 | 13 3/16 | ------ | 5.34 | 7.19 | 10.9 | 14.6 | 22.2 | 29.9 | 37.6 |
| 1.11 | 13 5/16 | ------ | 5.41 | 7.28 | 11.0 | 14.8 | 22.5 | 30.3 | 38.1 |
| 1.12 | 13 7/16 | ------ | 5.48 | 7.38 | 11.2 | 15.0 | 22.8 | 30.7 | 38.6 |
| 1.13 | 13 9/16 | ------ | 5.55 | 7.47 | 11.3 | 15.2 | 23.1 | 31.1 | 39.1 |
| 1.14 | 13 11/16 | ------ | 5.62 | 7.57 | 11.5 | 15.4 | 23.4 | 31.5 | 39.6 |
| 1.15 | 13 13/16 | ------ | 5.69 | 7.66 | 11.6 | 15.6 | 23.7 | 31.9 | 40.1 |
| 1.16 | 13 15/16 | ------ | 5.77 | 7.76 | 11.8 | 15.8 | 24.0 | 32.3 | 40.6 |
| 1.17 | 14 1/16 | ------ | 5.84 | 7.86 | 11.9 | 16.0 | 24.3 | 32.7 | 41.2 |
| 1.18 | 14 3/16 | ------ | 5.91 | 7.96 | 12.1 | 16.2 | 24.6 | 33.1 | 41.7 |
| 1.19 | 14 1/4 | ------ | 5.98 | 8.06 | 12.2 | 16.4 | 24.9 | 33.6 | 42.2 |

Exhibit 3-15 Discharge for contracted rectangular weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950)

| Head, H[1] | | Discharge, Q, for crest length, L, of— | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 foot | 1.5 feet | 2 feet | 3 feet | 4 feet | 6 feet | 8 feet | 10 feet |
| *Feet* | *Inches* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* |
| 1.20 | 14⅜ | ------ | 6.06 | 8.16 | 12.4 | 16.6 | 25.2 | 34.0 | 42.7 |
| 1.21 | 14½ | ------ | 6.13 | 8.26 | 12.5 | 16.8 | 25.5 | 34.4 | 43.2 |
| 1.22 | 14⅝ | ------ | 6.20 | 8.35 | 12.7 | 17.0 | 25.8 | 34.8 | 43.8 |
| 1.23 | 14¾ | ------ | 6.28 | 8.46 | 12.8 | 17.2 | 26.2 | 35.2 | 44.3 |
| 1.24 | 14⅞ | ------ | 6.35 | 8.56 | 13.0 | 17.4 | 26.5 | 35.6 | 44.8 |
| 1.25 | 15 | ------ | 6.43 | 8.66 | 13.1 | 17.6 | 26.8 | 36.1 | 45.4 |
| 1.26 | 15⅛ | ------ | -------- | -------- | 13.3 | 17.9 | 27.1 | 36.5 | 45.9 |
| 1.27 | 15¼ | ------ | -------- | -------- | 13.4 | 18.1 | 27.4 | 36.9 | 46.4 |
| 1.28 | 15⅜ | ------ | -------- | -------- | 13.6 | 18.3 | 27.7 | 37.3 | 47.0 |
| 1.29 | 15½ | ------ | -------- | -------- | 13.8 | 18.5 | 28.0 | 37.8 | 47.5 |
| 1.30 | 15⅝ | ------ | -------- | -------- | 13.9 | 18.7 | 28.3 | 38.2 | 48.1 |
| 1.31 | 15¾ | ------ | -------- | -------- | 14.1 | 18.9 | 28.7 | 38.6 | 48.6 |
| 1.32 | 15 13/16 | ------ | -------- | -------- | 14.2 | 19.1 | 29.0 | 39.1 | 49.2 |
| 1.33 | 15 15/16 | ------ | -------- | -------- | 14.4 | 19.3 | 29.3 | 39.5 | 49.7 |
| 1.34 | 16 1/16 | ------ | -------- | -------- | 14.6 | 19.6 | 29.6 | 39.9 | 50.3 |
| 1.35 | 16 3/16 | ------ | -------- | -------- | 14.7 | 19.8 | 30.0 | 40.4 | 50.8 |
| 1.36 | 16 5/16 | ------ | -------- | -------- | 14.9 | 20.0 | 30.3 | 40.8 | 51.4 |
| 1.37 | 16 7/16 | ------ | -------- | -------- | 15.0 | 20.2 | 30.6 | 41.3 | 51.9 |
| 1.38 | 16 9/16 | ------ | -------- | -------- | 15.2 | 20.4 | 30.9 | 41.7 | 52.5 |
| 1.39 | 16 11/16 | ------ | -------- | -------- | 15.4 | 20.6 | 31.3 | 42.1 | 53.1 |
| 1.40 | 16 13/16 | ------ | -------- | -------- | 15.5 | 20.9 | 31.6 | 42.6 | 53.6 |
| 1.41 | 16 15/16 | ------ | -------- | -------- | 15.7 | 21.1 | 31.9 | 43.0 | 54.2 |
| 1.42 | 17 1/16 | ------ | -------- | -------- | 15.9 | 21.3 | 32.2 | 43.5 | 54.7 |
| 1.43 | 17 3/16 | ------ | -------- | -------- | 16.0 | 21.5 | 32.6 | 43.9 | 55.3 |
| 1.44 | 17¼ | ------ | -------- | -------- | 16.2 | 21.7 | 32.9 | 44.4 | 55.9 |
| 1.45 | 17⅜ | ------ | -------- | -------- | 16.3 | 22.0 | 33.2 | 44.8 | 56.5 |
| 1.46 | 17½ | ------ | -------- | -------- | 16.5 | 22.2 | 33.6 | 45.3 | 57.0 |
| 1.47 | 17⅝ | ------ | -------- | -------- | 16.7 | 22.4 | 33.9 | 45.7 | 57.6 |
| 1.48 | 17¾ | ------ | -------- | -------- | 16.9 | 22.6 | 34.2 | 46.2 | 58.2 |
| 1.49 | 17⅞ | ------ | -------- | -------- | 17.0 | 22.9 | 34.6 | 46.6 | 58.8 |
| 1.50 | 18 | ------ | -------- | -------- | 17.2 | 23.1 | 34.9 | 47.1 | 59.3 |

[1] Values of discharge for heads up to 0.20 foot (crest lengths 1, 1.5, 2, 3, and 4 feet) do not follow the formula, but are taken directly from the calibration curve. The discharge for heads 0.10 to 1.5 feet for the 6-, 8-, and 10-foot weirs are as computed by the formula $Q=3.33\ (L-0.2H)\ H^{3/2}$.

Exhibit 3-15 Discharge for contracted rectangular weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950)

(Sheet 3 of 3)

| Head, H[1] | | Discharge, *Q*, for crest length, *L*, of— | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 foot | 1.5 feet | 2 feet | 3 feet | 4 feet | 6 feet | 8 feet | 10 feet |
| *Feet* | *Inches* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* |
| 0. 10 | 1 3/16 | 0. 11 | 0. 16 | 0. 23 | 0. 33 | 0. 44 | 0. 64 | 0. 85 | 1. 06 |
| . 11 | 1 5/16 | . 12 | . 18 | . 26 | . 38 | . 50 | . 74 | . 98 | 1. 23 |
| . 12 | 1 7/16 | . 14 | . 21 | . 29 | . 43 | . 57 | . 84 | 1. 12 | 1. 40 |
| . 13 | 1 9/16 | . 16 | . 24 | . 32 | . 48 | . 64 | . 95 | 1. 26 | 1. 58 |
| . 14 | 1 11/16 | . 17 | . 26 | . 36 | . 54 | . 71 | 1. 06 | 1. 41 | 1. 73 |
| . 15 | 1 13/16 | . 19 | . 29 | . 39 | . 59 | . 79 | 1. 17 | 1. 56 | 1. 96 |
| . 16 | 1 15/16 | . 21 | . 32 | . 43 | . 65 | . 87 | 1. 29 | 1. 72 | 2. 15 |
| . 17 | 2 1/16 | . 23 | . 36 | . 47 | . 71 | . 96 | 1. 41 | 1. 89 | 2. 36 |
| . 18 | 2 3/16 | . 25 | . 39 | . 51 | . 77 | 1. 04 | 1. 54 | 2. 06 | 2. 57 |
| . 19 | 2 1/4 | . 28 | . 42 | . 56 | . 83 | 1. 12 | 1. 67 | 2. 23 | 2. 79 |
| . 20 | 2 3/8 | . 30 | . 45 | . 60 | . 90 | 1. 20 | 1. 81 | 2. 41 | 3. 01 |
| . 21 | 2 1/2 | . 32 | . 48 | . 64 | . 97 | 1. 29 | 1. 94 | 2. 59 | 3. 24 |
| . 22 | 2 5/8 | . 35 | . 52 | . 69 | 1. 04 | 1. 38 | 2. 08 | 2. 78 | 3. 47 |
| . 23 | 2 3/4 | . 37 | . 55 | . 74 | 1. 11 | 1. 47 | 2. 23 | 2. 97 | 3. 71 |
| . 24 | 2 7/8 | . 39 | . 59 | . 79 | 1. 18 | 1. 57 | 2. 38 | 3. 17 | 3. 96 |
| . 25 | 3 | . 42 | . 63 | . 84 | 1. 25 | 1. 67 | 2. 53 | 3. 37 | 4. 21 |
| . 26 | 3 1/8 | . 45 | . 67 | . 89 | 1. 33 | 1. 77 | 2. 68 | 3. 57 | 4. 46 |
| . 27 | 3 1/4 | . 47 | . 70 | . 94 | 1. 40 | 1. 87 | 2. 83 | 3. 78 | 4. 72 |
| . 28 | 3 3/8 | . 50 | . 74 | . 99 | 1. 48 | 1. 97 | 2. 99 | 3. 99 | 4. 99 |
| . 29 | 3 1/2 | . 53 | . 79 | 1. 04 | 1. 56 | 2. 08 | 3. 15 | 4. 21 | 5. 26 |
| . 30 | 3 5/8 | . 56 | . 83 | 1. 10 | 1. 64 | 2. 19 | 3. 32 | 4. 43 | 5. 53 |
| . 31 | 3 3/4 | . 59 | . 87 | 1. 15 | 1. 73 | 2. 30 | 3. 49 | 4. 65 | 5. 81 |
| . 32 | 3 13/16 | . 61 | . 91 | 1. 21 | 1. 80 | 2. 41 | 3. 66 | 4. 88 | 6. 09 |
| . 33 | 3 15/16 | . 64 | . 95 | 1. 27 | 1. 89 | 2. 52 | 3. 83 | 5. 11 | 6. 38 |
| . 34 | 4 1/16 | . 67 | 1. 00 | 1. 32 | 1. 98 | 2. 64 | 4. 00 | 5. 34 | 6. 67 |
| . 35 | 4 3/16 | . 70 | 1. 04 | 1. 38 | 2. 07 | 2. 75 | 4. 18 | 5. 58 | 6. 97 |
| . 36 | 4 5/16 | . 73 | 1. 09 | 1. 44 | 2. 16 | 2. 87 | 4. 36 | 5. 82 | 7. 27 |
| . 37 | 4 7/16 | . 77 | 1. 13 | 1. 50 | 2. 25 | 2. 99 | 4. 55 | 6. 06 | 7. 58 |
| . 38 | 4 9/16 | . 80 | 1. 18 | 1. 57 | 2. 34 | 3. 11 | 4. 73 | 6. 31 | 7. 89 |
| . 39 | 4 11/16 | . 83 | 1. 23 | 1. 63 | 2. 43 | 3. 24 | 4. 92 | 6. 56 | 8. 20 |
| . 40 | 4 13/16 | . 87 | 1. 28 | 1. 69 | 2. 53 | 3. 36 | 5. 11 | 6. 81 | 8. 52 |
| . 41 | 4 15/16 | . 90 | 1. 32 | 1. 76 | 2. 62 | 3. 49 | 5. 30 | 7. 07 | 8. 84 |
| . 42 | 5 1/16 | . 93 | 1. 37 | 1. 82 | 2. 72 | 3. 61 | 5. 50 | 7. 33 | 9. 16 |
| . 43 | 5 3/16 | . 97 | 1. 42 | 1. 89 | 2. 81 | 3. 74 | 5. 70 | 7. 59 | 9. 49 |
| . 44 | 5 1/4 | 1. 00 | 1. 47 | 1. 95 | 2. 91 | 3. 87 | 5. 90 | 7. 86 | 9. 83 |
| . 45 | 5 3/8 | 1. 04 | 1. 53 | 2. 02 | 3. 01 | 4. 01 | 6. 10 | 8. 13 | 10. 2 |
| . 46 | 5 1/2 | 1. 07 | 1. 58 | 2. 09 | 3. 11 | 4. 14 | 6. 30 | 8. 40 | 10. 5 |
| . 47 | 5 5/8 | 1. 11 | 1. 63 | 2. 16 | 3. 21 | 4. 28 | 6. 51 | 8. 68 | 10. 8 |
| . 48 | 5 3/4 | 1. 15 | 1. 68 | 2. 23 | 3. 32 | 4. 41 | 6. 72 | 8. 96 | 11. 2 |
| . 49 | 5 7/8 | 1. 18 | 1. 74 | 2. 30 | 3. 42 | 4. 55 | 6. 93 | 9. 24 | 11. 5 |
| . 50 | 6 | 1. 22 | 1. 79 | 2. 37 | 3. 53 | 4. 69 | 7. 14 | 9. 52 | 11. 9 |
| . 51 | 6 1/8 | 1. 26 | 1. 85 | 2. 44 | 3. 64 | 4. 83 | 7. 36 | 9. 81 | 12. 3 |
| . 52 | 6 1/4 | 1. 30 | 1. 90 | 2. 51 | 3. 74 | 4. 97 | 7. 57 | 10. 1 | 12. 6 |
| . 53 | 6 3/8 | 1. 34 | 1. 96 | 2. 59 | 3. 85 | 5. 12 | 7. 79 | 10. 4 | 13. 0 |
| . 54 | 6 1/2 | 1. 38 | 2. 02 | 2. 66 | 3. 96 | 5. 26 | 8. 02 | 10. 7 | 13. 4 |
| . 55 | 6 5/8 | 1. 42 | 2. 07 | 2. 74 | 4. 07 | 5. 41 | 8. 24 | 11. 0 | 13. 7 |
| . 56 | 6 3/4 | 1. 46 | 2. 13 | 2. 81 | 4. 18 | 5. 56 | 8. 47 | 11. 3 | 14. 1 |
| . 57 | 6 13/16 | 1. 50 | 2. 19 | 2. 89 | 4. 30 | 5. 71 | 8. 69 | 11. 6 | 14. 5 |
| . 58 | 6 15/16 | 1. 54 | 2. 25 | 2. 97 | 4. 41 | 5. 86 | 8. 92 | 11. 9 | 14. 9 |
| . 59 | 7 1/16 | 1. 58 | 2. 31 | 3. 05 | 4. 53 | 6. 01 | 9. 15 | 12. 2 | 15. 3 |
| . 60 | 7 3/16 | 1. 62 | 2. 37 | 3. 13 | 4. 64 | 6. 17 | 9. 39 | 12. 5 | 15. 6 |
| . 61 | 7 5/16 | 1. 67 | 2. 43 | 3. 20 | 4. 76 | 6. 32 | 9. 62 | 12. 8 | 16. 0 |
| . 62 | 7 7/16 | 1. 71 | 2. 49 | 3. 28 | 4. 88 | 6. 47 | 9. 86 | 13. 1 | 16. 4 |
| . 63 | 7 9/16 | 1. 75 | 2. 55 | 3. 37 | 5. 00 | 6. 63 | 10. 1 | 13. 5 | 16. 8 |
| . 64 | 7 11/16 | 1. 80 | 2. 62 | 3. 45 | 5. 12 | 6. 79 | 10. 3 | 13. 8 | 17. 2 |

Exhibit 3-16 Discharge for Cipolletti weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950) (Sheet 1 of 3)

| Head, H[1] | | Discharge, *Q*, for crest length, *L*, of— | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 foot | 1.5 feet | 2 feet | 3 feet | 4 feet | 6 feet | 8 feet | 10 feet |
| *Feet* | *Inches* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* |
| 0.65 | 7 13/16 | 1.84 | 2.68 | 3.53 | 5.24 | 6.95 | 10.6 | 14.1 | 17.6 |
| .66 | 7 15/16 | 1.89 | 2.75 | 3.61 | 5.36 | 7.11 | 10.8 | 14.4 | 18.1 |
| .67 | 8 1/16 | 1.93 | 2.81 | 3.70 | 5.48 | 7.28 | 11.1 | 14.8 | 18.5 |
| .68 | 8 3/16 | 1.98 | 2.87 | 3.79 | 5.61 | 7.44 | 11.3 | 15.1 | 18.9 |
| .69 | 8 1/4 | 2.02 | 2.94 | 3.87 | 5.73 | 7.61 | 11.6 | 15.4 | 19.3 |
| .70 | 8 3/8 | 2.07 | 3.01 | 3.95 | 5.86 | 7.77 | 11.8 | 15.8 | 19.7 |
| .71 | 8 1/2 | 2.12 | 3.07 | 4.04 | 5.99 | 7.94 | 12.1 | 16.1 | 20.1 |
| .72 | 8 5/8 | 2.16 | 3.14 | 4.13 | 6.12 | 8.11 | 12.3 | 16.5 | 20.6 |
| .73 | 8 3/4 | 2.21 | 3.21 | 4.22 | 6.24 | 8.28 | 12.6 | 16.8 | 21.0 |
| .74 | 8 7/8 | 2.26 | 3.28 | 4.31 | 6.38 | 8.45 | 12.9 | 17.1 | 21.4 |
| .75 | 9 | 2.31 | 3.35 | 4.40 | 6.51 | 8.62 | 13.1 | 17.5 | 21.9 |
| .76 | 9 1/8 | 2.36 | 3.42 | 4.49 | 6.64 | 8.80 | 13.4 | 17.8 | 22.3 |
| .77 | 9 1/4 | 2.41 | 3.49 | 4.58 | 6.77 | 8.97 | 13.6 | 18.2 | 22.7 |
| .78 | 9 3/8 | 2.46 | 3.56 | 4.67 | 6.90 | 9.15 | 13.9 | 18.6 | 23.2 |
| .79 | 9 1/2 | 2.51 | 3.63 | 4.76 | 7.04 | 9.33 | 14.2 | 18.9 | 23.6 |
| .80 | 9 5/8 | 2.56 | 3.70 | 4.85 | 7.18 | 9.51 | 14.5 | 19.3 | 24.1 |
| .81 | 9 3/4 | 2.61 | 3.77 | 4.95 | 7.31 | 9.69 | 14.7 | 19.6 | 24.5 |
| .82 | 9 13/16 | 2.66 | 3.84 | 5.04 | 7.45 | 9.87 | 15.0 | 20.0 | 25.0 |
| .83 | 9 15/16 | 2.71 | 3.92 | 5.14 | 7.59 | 10.0 | 15.3 | 20.4 | 25.5 |
| .84 | 10 1/16 | 2.77 | 3.99 | 5.23 | 7.73 | 10.2 | 15.6 | 20.7 | 25.9 |
| .85 | 10 3/16 | 2.82 | 4.07 | 5.33 | 7.87 | 10.4 | 15.8 | 21.1 | 26.4 |
| .86 | 10 5/16 | 2.87 | 4.14 | 5.43 | 8.01 | 10.6 | 16.1 | 21.5 | 26.9 |
| .87 | 10 7/16 | 2.93 | 4.22 | 5.52 | 8.15 | 10.8 | 16.4 | 21.9 | 27.3 |
| .88 | 10 9/16 | 2.98 | 4.29 | 5.62 | 8.30 | 11.0 | 16.7 | 22.2 | 27.8 |
| .89 | 10 11/16 | 3.04 | 4.37 | 5.72 | 8.44 | 11.2 | 17.0 | 22.6 | 28.3 |
| .90 | 10 13/16 | 3.09 | 4.45 | 5.82 | 8.59 | 11.4 | 17.2 | 23.0 | 28.7 |
| .91 | 10 15/16 | 3.15 | 4.53 | 5.92 | 8.73 | 11.6 | 17.5 | 23.4 | 29.2 |
| .92 | 11 1/16 | 3.20 | 4.60 | 6.02 | 8.88 | 11.7 | 17.8 | 23.8 | 29.7 |
| .93 | 11 3/16 | 3.26 | 4.68 | 6.13 | 9.03 | 11.9 | 18.1 | 24.2 | 30.2 |
| .94 | 11 1/4 | 3.32 | 4.76 | 6.23 | 9.17 | 12.1 | 18.4 | 24.5 | 30.7 |
| .95 | 11 3/8 | 3.37 | 4.84 | 6.33 | 9.32 | 12.3 | 18.7 | 24.9 | 31.2 |
| .96 | 11 1/2 | 3.43 | 4.92 | 6.44 | 9.48 | 12.5 | 19.0 | 25.3 | 31.7 |
| .97 | 11 5/8 | 3.49 | 5.00 | 6.55 | 9.62 | 12.7 | 19.3 | 25.7 | 32.2 |
| .98 | 11 3/4 | 3.55 | 5.09 | 6.64 | 9.78 | 12.9 | 19.6 | 26.1 | 32.7 |
| .99 | 11 7/8 | 3.61 | 5.17 | 6.75 | 9.93 | 13.1 | 19.9 | 26.5 | 33.2 |
| 1.00 | 12 | 3.67 | 5.25 | 6.86 | 10.1 | 13.3 | 20.2 | 26.9 | 33.7 |
| 1.01 | 12 1/8 | ------ | 5.33 | 6.96 | 10.2 | 13.5 | 20.5 | 27.3 | 34.2 |
| 1.02 | 12 1/4 | ------ | 5.42 | 7.07 | 10.4 | 13.7 | 20.8 | 27.7 | 34.7 |
| 1.03 | 12 3/8 | ------ | 5.50 | 7.18 | 10.6 | 13.9 | 21.1 | 28.2 | 35.2 |
| 1.04 | 12 1/2 | ------ | 5.59 | 7.29 | 10.7 | 14.2 | 21.4 | 28.6 | 35.7 |
| 1.05 | 12 5/8 | ------ | 5.67 | 7.40 | 10.9 | 14.4 | 21.7 | 29.0 | 36.2 |
| 1.06 | 12 3/4 | ------ | 5.76 | 7.51 | 11.0 | 14.6 | 22.0 | 29.4 | 36.7 |
| 1.07 | 12 13/16 | ------ | 5.84 | 7.62 | 11.2 | 14.8 | 22.4 | 29.8 | 37.3 |
| 1.08 | 12 15/16 | ------ | 5.93 | 7.73 | 11.4 | 15.0 | 22.7 | 30.2 | 37.8 |
| 1.09 | 13 1/16 | ------ | 6.02 | 7.84 | 11.5 | 15.2 | 23.0 | 30.6 | 38.3 |
| 1.10 | 13 3/16 | ------ | 6.11 | 7.96 | 11.7 | 15.4 | 23.3 | 31.1 | 38.8 |
| 1.11 | 13 5/16 | ------ | 6.20 | 8.07 | 11.8 | 15.6 | 23.6 | 31.5 | 39.4 |
| 1.12 | 13 7/16 | ------ | 6.29 | 8.18 | 12.0 | 15.8 | 23.9 | 31.9 | 39.9 |
| 1.13 | 13 9/16 | ------ | 6.38 | 8.29 | 12.2 | 16.0 | 24.3 | 32.4 | 40.4 |
| 1.14 | 13 11/16 | ------ | 6.47 | 8.41 | 12.3 | 16.3 | 24.6 | 32.8 | 41.0 |
| 1.15 | 13 13/16 | ------ | 6.56 | 8.53 | 12.5 | 16.5 | 24.9 | 33.2 | 41.5 |
| 1.16 | 13 15/16 | ------ | 6.65 | 8.65 | 12.7 | 16.7 | 25.2 | 33.6 | 42.1 |
| 1.17 | 14 1/16 | ------ | 6.74 | 8.76 | 12.8 | 16.9 | 25.6 | 34.1 | 42.6 |
| 1.18 | 14 3/16 | ------ | 6.83 | 8.88 | 13.0 | 17.2 | 25.9 | 34.5 | 43.2 |
| 1.19 | 14 1/4 | ------ | 6.93 | 9.00 | 13.2 | 17.4 | 26.2 | 35.0 | 43.7 |

Exhibit 3-16 Discharge for Cipolletti weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950)

(Sheet 2 of 3)

| Head, H[1] | | Discharge, Q, for crest length, L, of— | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 foot | 1.5 feet | 2 feet | 3 feet | 4 feet | 6 feet | 8 feet | 10 feet |
| *Feet* | *Inches* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* | *Second-feet* |
| 1.20 | 14 3/8 | ------ | 7.02 | 9.12 | 13.4 | 17.6 | 26.6 | 35.4 | 44.3 |
| 1.21 | 14 1/2 | ------ | 7.11 | 9.24 | 13.5 | 17.8 | 26.9 | 35.8 | 44.8 |
| 1.22 | 14 5/8 | ------ | 7.20 | 9.36 | 13.7 | 18.0 | 27.2 | 36.3 | 45.4 |
| 1.23 | 14 3/4 | ------ | 7.30 | 9.48 | 13.9 | 18.3 | 27.6 | 36.7 | 45.9 |
| 1.24 | 14 7/8 | ------ | 7.40 | 9.60 | 14.0 | 18.5 | 27.9 | 37.2 | 46.5 |
| 1.25 | 15 | ------ | 7.49 | 9.72 | 14.2 | 18.7 | 28.2 | 37.6 | 47.1 |
| 1.26 | 15 1/8 | ------ | -------- | -------- | 14.4 | 19.0 | 28.6 | 38.1 | 47.6 |
| 1.27 | 15 1/4 | ------ | -------- | -------- | 14.6 | 19.2 | 28.9 | 38.5 | 48.2 |
| 1.28 | 15 3/8 | ------ | -------- | -------- | 14.7 | 19.4 | 29.3 | 39.0 | 48.8 |
| 1.29 | 15 1/2 | ------ | -------- | -------- | 14.9 | 19.6 | 29.6 | 39.5 | 49.3 |
| 1.30 | 15 5/8 | ------ | -------- | -------- | 15.1 | 19.9 | 29.9 | 39.9 | 49.9 |
| 1.31 | 15 3/4 | ------ | -------- | -------- | 15.3 | 20.1 | 30.3 | 40.4 | 50.5 |
| 1.32 | 15 13/16 | ------ | -------- | -------- | 15.5 | 20.3 | 30.6 | 40.8 | 51.1 |
| 1.33 | 15 15/16 | ------ | -------- | -------- | 15.6 | 20.6 | 31.0 | 41.3 | 51.6 |
| 1.34 | 16 1/16 | ------ | -------- | -------- | 15.8 | 20.8 | 31.3 | 41.8 | 52.2 |
| 1.35 | 16 3/16 | ------ | -------- | -------- | 16.0 | 21.1 | 31.7 | 42.2 | 52.8 |
| 1.36 | 16 5/16 | ------ | -------- | -------- | 16.2 | 21.3 | 32.0 | 42.7 | 53.4 |
| 1.37 | 16 7/16 | ------ | -------- | -------- | 16.4 | 21.5 | 32.4 | 43.2 | 54.0 |
| 1.38 | 16 9/16 | ------ | -------- | -------- | 16.6 | 21.8 | 32.7 | 43.7 | 54.6 |
| 1.39 | 16 11/16 | ------ | -------- | -------- | 16.8 | 22.0 | 33.1 | 44.1 | 55.2 |
| 1.40 | 16 13/16 | ------ | -------- | -------- | 16.9 | 22.3 | 33.5 | 44.6 | 55.8 |
| 1.41 | 16 15/16 | ------ | -------- | -------- | 17.1 | 22.5 | 33.8 | 45.1 | 56.4 |
| 1.42 | 17 1/16 | ------ | -------- | -------- | 17.3 | 22.8 | 34.2 | 45.6 | 57.0 |
| 1.43 | 17 3/16 | ------ | -------- | -------- | 17.5 | 23.0 | 34.6 | 46.1 | 57.6 |
| 1.44 | 17 1/4 | ------ | -------- | -------- | 17.7 | 23.3 | 34.9 | 46.5 | 58.2 |
| 1.45 | 17 3/8 | ------ | -------- | -------- | 17.9 | 23.5 | 35.3 | 47.0 | 58.8 |
| 1.46 | 17 1/2 | ------ | -------- | -------- | 18.1 | 23.8 | 35.6 | 47.5 | 59.4 |
| 1.47 | 17 5/8 | ------ | -------- | -------- | 18.3 | 24.0 | 36.0 | 48.0 | 60.0 |
| 1.48 | 17 3/4 | ------ | -------- | -------- | 18.5 | 24.3 | 36.4 | 48.5 | 60.6 |
| 1.49 | 17 7/8 | ------ | -------- | -------- | 18.7 | 24.5 | 36.7 | 49.0 | 61.2 |
| 1.50 | 18 | ------ | -------- | -------- | 18.9 | 24.8 | 37.1 | 49.5 | 61.8 |

[1] Values of discharge for heads up to 0.20 foot (crest lengths 1, 1.5, 2, 3, and 4 feet) do not follow the formula but are taken directly from the calibration curve. The discharge for heads 0.10 to 1.5 feet for the 6-, 8,- and 10-foot weirs are as computed by the formula $Q=3.367LH^{3/2}$.

Exhibit 3-16 Discharge for Cipolletti weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950)

(Sheet 3 of 3)

| Head, H | | Discharge, Q | Head, H | | Discharge, Q | Head, H | | Discharge, Q |
|---|---|---|---|---|---|---|---|---|
| *Feet* | *Inches* | *Second-feet* | *Feet* | *Inches* | *Second-feet* | *Feet* | *Inches* | *Second-feet* |
| 0.15 | 1 13/16 | 0.022 | 0.55 | 6 5/8 | 0.564 | 0.95 | 11 3/8 | 2.19 |
| .16 | 1 15/16 | .025 | .56 | 6 3/4 | .590 | .96 | 11 1/2 | 2.25 |
| .17 | 2 1/16 | .029 | .57 | 6 13/16 | .617 | .97 | 11 5/8 | 2.31 |
| .18 | 2 3/16 | .034 | .58 | 6 15/16 | .644 | .98 | 11 3/4 | 2.37 |
| .19 | 2 1/4 | .040 | .59 | 7 1/16 | .672 | .99 | 11 7/8 | 2.43 |
| .20 | 2 3/8 | .046 | .60 | 7 3/16 | .700 | 1.00 | 12 | 2.49 |
| .21 | 2 1/2 | .052 | .61 | 7 5/16 | .730 | 1.01 | 12 1/8 | 2.55 |
| .22 | 2 5/8 | .058 | .62 | 7 7/16 | .760 | 1.02 | 12 1/4 | 2.61 |
| .23 | 2 3/4 | .065 | .63 | 7 9/16 | .790 | 1.03 | 12 3/8 | 2.68 |
| .24 | 2 7/8 | .072 | .64 | 7 11/16 | .822 | 1.04 | 12 1/2 | 2.74 |
| .25 | 3 | .080 | .65 | 7 13/16 | .854 | 1.05 | 12 5/8 | 2.81 |
| .26 | 3 1/8 | .088 | .66 | 7 15/16 | .887 | 1.06 | 12 3/4 | 2.87 |
| .27 | 3 1/4 | .096 | .67 | 8 1/16 | .921 | 1.07 | 12 13/16 | 2.94 |
| .28 | 3 3/8 | .105 | .68 | 8 3/16 | .955 | 1.08 | 12 15/16 | 3.01 |
| .29 | 3 1/2 | .115 | .69 | 8 1/4 | .991 | 1.09 | 13 1/16 | 3.08 |
| .30 | 3 5/8 | .125 | .70 | 8 3/8 | 1.03 | 1.10 | 13 3/16 | 3.15 |
| .31 | 3 3/4 | .136 | .71 | 8 1/2 | 1.06 | 1.11 | 13 5/16 | 3.22 |
| .32 | 3 13/16 | .147 | .72 | 8 5/8 | 1.10 | 1.12 | 13 7/16 | 3.30 |
| .33 | 3 15/16 | .159 | .73 | 8 3/4 | 1.14 | 1.13 | 13 9/16 | 3.37 |
| .34 | 4 1/16 | .171 | .74 | 8 7/8 | 1.18 | 1.14 | 13 11/16 | 3.44 |
| .35 | 4 3/16 | .184 | .75 | 9 | 1.22 | 1.15 | 13 13/16 | 3.52 |
| .36 | 4 5/16 | .197 | .76 | 9 1/8 | 1.26 | 1.16 | 13 15/16 | 3.59 |
| .37 | 4 7/16 | .211 | .77 | 9 1/4 | 1.30 | 1.17 | 14 1/16 | 3.67 |
| .38 | 4 9/16 | .225 | .78 | 9 3/8 | 1.34 | 1.18 | 14 3/16 | 3.75 |
| .39 | 4 11/16 | .240 | .79 | 9 1/2 | 1.39 | 1.19 | 14 1/4 | 3.83 |
| .40 | 4 13/16 | .256 | .80 | 9 5/8 | 1.43 | 1.20 | 14 3/8 | 3.91 |
| .41 | 4 15/16 | .272 | .81 | 9 3/4 | 1.48 | 1.21 | 14 1/2 | 3.99 |
| .42 | 5 1/16 | .289 | .82 | 9 13/16 | 1.52 | 1.22 | 14 5/8 | 4.07 |
| .43 | 5 3/16 | .306 | .83 | 9 15/16 | 1.57 | 1.23 | 14 3/4 | 4.16 |
| .44 | 5 1/4 | .324 | .84 | 10 1/16 | 1.61 | 1.24 | 14 7/8 | 4.24 |
| .45 | 5 3/8 | .343 | .85 | 10 3/16 | 1.66 | 1.25 | 15 | 4.33 |
| .46 | 5 1/2 | .362 | .86 | 10 5/16 | 1.71 | 1.26 | 15 1/8 | 4.42 |
| .47 | 5 5/8 | .382 | .87 | 10 7/16 | 1.76 | 1.27 | 15 1/4 | 4.51 |
| .48 | 5 3/4 | .403 | .88 | 10 9/16 | 1.81 | 1.28 | 15 3/8 | 4.60 |
| .49 | 5 7/8 | .424 | .89 | 10 11/16 | 1.86 | 1.29 | 15 1/2 | 4.69 |
| .50 | 6 | .445 | .90 | 10 13/16 | 1.92 | 1.30 | 15 5/8 | 4.78 |
| .51 | 6 1/8 | .468 | .91 | 10 15/16 | 1.97 | 1.31 | 15 3/4 | 4.87 |
| .52 | 6 1/4 | .491 | .92 | 11 1/16 | 2.02 | 1.32 | 15 13/16 | 4.96 |
| .53 | 6 3/8 | .515 | .93 | 11 3/16 | 2.08 | 1.33 | 15 15/16 | 5.06 |
| .54 | 6 1/2 | .539 | .94 | 11 1/4 | 2.13 | 1.34 | 16 1/16 | 5.15 |

Exhibit 3-17 Discharge for 90° v-notch weirs (Ref. Parshall, R.L., Measuring Water in Irrigation Channels, U.S. Dept. Agr. Cir. 843, 1950)

| Head (inches) | 3-in. orifice | | 4-in. orifice | | 5-in. orifice | | 6-in. orifice | | 7-in. orifice | 8-in. orifice |
|---|---|---|---|---|---|---|---|---|---|---|
| | 4-in. Pipe | 6-in. Pipe | 6-in. Pipe | 8-in. Pipe | 6-in. Pipe | 8-in. Pipe | 8-in. Pipe | 10-in. Pipe | 10-in. Pipe | 10-in. Pipe |
| | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. |
| 6 | 108 | 82 | 160 | 150 | 305 | 240 | 408 | 345 | | |
| 8 | 122 | 94 | 185 | 170 | 350 | 280 | 458 | 395 | 600 | 935 |
| 10 | 133 | 104 | 205 | 190 | 393 | 316 | 508 | 445 | 666 | 1040 |
| 12 | 146 | 114 | 225 | 208 | 430 | 346 | 556 | 490 | 728 | 1120 |
| 14 | 157 | 123 | 243 | 224 | 465 | 376 | 599 | 530 | 785 | 1194 |
| 16 | 167 | 132 | 257 | 238 | 495 | 402 | 636 | 568 | 838 | 1266 |
| 18 | 178 | 140 | 271 | 252 | 524 | 426 | 672 | 604 | 887 | 1336 |
| 20 | 187 | 148 | 285 | 266 | 548 | 449 | 708 | 636 | 933 | 1404 |
| 22 | 197 | 156 | 299 | 279 | 572 | 470 | 744 | 664 | 979 | 1471 |
| 24 | 205 | 164 | 310 | 291 | 596 | 488 | 776 | 692 | 1022 | 1529 |
| 26 | 214 | 171 | 323 | 303 | 620 | 504 | 805 | 720 | 1064 | 1585 |
| 28 | 222 | 177 | 335 | 314 | 644 | 520 | 831 | 747 | 1104 | 1641 |
| 30 | 230 | 183 | 346 | 325 | 668 | 536 | 857 | 773 | 1143 | 1697 |
| 32 | 239 | 189 | 357 | 335 | 692 | 552 | 882 | 799 | 1181 | 1753 |
| 34 | 246 | 195 | 369 | 345 | 715 | 568 | 907 | 824 | 1218 | 1809 |
| 36 | 254 | 200 | 380 | 354 | 737 | 584 | 931 | 847 | 1251 | 1865 |
| 38 | 260 | 205 | 390 | 363 | 759 | 600 | 955 | 867 | 1281 | |
| 40 | 266 | 210 | 401 | 371 | 781 | 616 | 979 | 887 | 1311 | |
| 42 | 272 | 214 | 411 | 380 | 800 | 631 | 1001 | 906 | 1341 | |
| 44 | 278 | 219 | 420 | 388 | 820 | 645 | 1023 | 925 | 1371 | |
| 46 | 284 | 224 | 429 | 396 | 837 | 659 | 1045 | 944 | 1401 | |
| 48 | 290 | 229 | 440 | 405 | 855 | 672 | 1067 | 963 | 1431 | |
| 50 | 296 | 234 | 448 | 413 | 872 | 686 | 1089 | 982 | 1461 | |
| 52 | 302 | 238 | 457 | 421 | 888 | 700 | 1110 | 1000 | 1491 | |
| 54 | 307 | 243 | 465 | 429 | 904 | 714 | 1130 | 1018 | 1520 | |
| 56 | 313 | 248 | 472 | 437 | 919 | 727 | 1150 | 1036 | 1548 | |
| 58 | 317 | 252 | 480 | 445 | 934 | 739 | 1170 | 1052 | 1574 | |
| 60 | 323 | 257 | 489 | 453 | 948 | 751 | 1190 | 1068 | 1598 | |
| 62 | 328 | 262 | 496 | 461 | 961 | 763 | 1209 | 1084 | | |
| 64 | 333 | 266 | 504 | 469 | 974 | 775 | 1227 | 1099 | | |
| 66 | 338 | 271 | 513 | 475 | 988 | 787 | 1245 | 1113 | | |
| 68 | 343 | 275 | 520 | 483 | 1002 | 799 | 1263 | 1127 | | |
| 70 | 349 | 280 | 525 | 491 | 1016 | 811 | 1280 | 1140 | | |

Exhibit 3-18 Discharge from circular pipe orifices with free discharge (Ref. Layne Well Water Systems, Layne and Bowler, Inc., Memphis, Tenn., 1951)

| Jet height (inches) | Diameter of pipe (inches) | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2 Std.[2] | 3 Std. | 4 O.D.[3] | 4 Std. | 5 O.D. | 5 Std. | 6 O.D. | 6 Std. | 8 O.D. | 8 Std. | 10 O.D. | 10 Std. | 12 O.D. | 12 Std. |
| | G.p.m. | G.p.m. | G.p.m. | | G.p.m. | | G.p.m. | | G.p.m. | | G.p.m. | | G.p.m. | |
| 2......... | 28 | 57 | 75 | 86 | 103 | 115 | 137 | 150 | 200 | 215 | 265 | 285 | 330 | 355 |
| 2 1/2..... | 31 | 69 | 95 | 108 | 132 | 150 | 182 | 205 | 275 | 290 | 357 | 385 | 450 | 480 |
| 3......... | 34 | 78 | 112 | 128 | 160 | 183 | 225 | 250 | 340 | 367 | 450 | 490 | 570 | 610 |
| 3 1/2..... | 37 | 86 | 124 | 145 | 183 | 210 | 262 | 293 | 405 | 440 | 555 | 610 | 705 | 755 |
| 4......... | 40 | 92 | 135 | 160 | 205 | 235 | 295 | 330 | 465 | 510 | 660 | 725 | 845 | 910 |
| 4 1/2 .... | 42 | 98 | 144 | 173 | 225 | 257 | 320 | 365 | 520 | 570 | 760 | 845 | 980 | 1060 |
| 5......... | 45 | 104 | 154 | 184 | 240 | 275 | 345 | 395 | 575 | 630 | 840 | 940 | 1120 | 1200 |
| 6......... | 50 | 115 | 169 | 205 | 266 | 306 | 385 | 445 | 670 | 730 | 1000 | 1125 | 1370 | 1500 |
| 7......... | 54 | 125 | 186 | 223 | 293 | 336 | 420 | 485 | 750 | 820 | 1150 | 1275 | 1600 | 1730 |
| 8......... | 58 | 134 | 202 | 239 | 315 | 360 | 450 | 520 | 810 | 890 | 1270 | 1420 | 1775 | 1950 |
| 9......... | 62 | 143 | 215 | 254 | 335 | 383 | 480 | 550 | 870 | 955 | 1360 | 1550 | 1930 | 2140 |
| 10........ | 66 | 152 | 227 | 268 | 356 | 405 | 510 | 585 | 925 | 1015 | 1450 | 1650 | 2070 | 2280 |
| 12........ | 72 | 167 | 255 | 295 | 390 | 450 | 565 | 650 | 1010 | 1120 | 1600 | 1830 | 2300 | 2550 |
| 14........ | 78 | 182 | 275 | 320 | 420 | 485 | 610 | 705 | 1100 | 1220 | 1730 | 2000 | 2530 | 2800 |
| 16........ | 83 | 195 | 295 | 345 | 455 | 520 | 655 | 755 | 1180 | 1300 | 1870 | 2140 | 2720 | 3000 |
| 18........ | 89 | 208 | 315 | 367 | 480 | 555 | 700 | 800 | 1265 | 1400 | 2000 | 2280 | 2900 | |
| 20........ | 94 | 220 | 333 | 388 | 510 | 590 | 740 | 850 | 1335 | 1480 | 2100 | 2420 | | |
| 25........ | 107 | 248 | 377 | 440 | 580 | 665 | 830 | 960 | 1520 | 1670 | 2380 | 2720 | | |
| 30........ | 117 | 275 | 420 | 485 | 640 | 740 | 925 | 1050 | 1690 | 1870 | 2650 | 3000 | | |
| 35........ | 127 | 300 | 455 | 525 | 695 | 800 | 1000 | 1150 | 1820 | 2020 | 2850 | | | |
| 40........ | 137 | 320 | 490 | 565 | 745 | 865 | 1075 | 1230 | 1970 | 2160 | | | | |

[2] Standard pipe.
[3] Outside diameter of well casing.

Exhibit 3-19 Flow of water from vertical pipes (Ref. Discharge Curves in Utah Eng. Expt. Sta. Bulletin 5, "Measurement of Irrigation Water," June 1955)

WHEN X = 0

| Y (inches) | Size of pipe (nominal diameter) | | | | |
|---|---|---|---|---|---|
| | 2-inch | 3-inch | 4-inch | 5-inch | 6-inch |
| | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. |
| 0.20 | | 67.7 | 180 | 308 | |
| .30 | | 66.5 | 175 | 303 | 530 |
| .40 | | 65.1 | 171 | 298 | 518 |
| .50 | | 63.6 | 166 | 293 | 506 |
| .60 | 18.3 | 62.0 | 161 | 287 | 494 |
| .70 | 17.6 | 60.4 | 156 | 282 | 482 |
| .80 | 16.7 | 58.4 | 150 | 277 | 470 |
| .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 |

Exhibit 3-20 Flow of water from horizontal pipes (Ref. Purdue Eng. Expt. Bulletin 32, "Measurement of pipe flow by the coordinate method," August 1928) (Sheet 1 of 3)

WHEN X = 6 INCHES

| Y (inches) | Size of pipe (nominal diameter) | | | | |
|---|---|---|---|---|---|
| | 2-inch | 3-inch | 4-inch | 5-inch | 6-inch |
| | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. |
| 0.24 | 177 | 310 | 548 | | |
| .36 | 146 | 274 | 503 | 969 | 1243 |
| .48 | 126 | 247 | 462 | 857 | 1113 |
| .60 | 111 | 229 | 435 | 772 | 1019 |
| .72 | 100 | 215 | 404 | 705 | 947 |
| .84 | 92 | 202 | 377 | 646 | 889 |
| .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 | | | | |

WHEN X = 12 INCHES

| Y (inches) | 2-inch | 3-inch | 4-inch | 5-inch | 6-inch |
|---|---|---|---|---|---|
| .96 | 157 | 319 | 570 | 1014 | |
| 1.08 | 148 | 305 | 548 | 974 | 1315 |
| 1.20 | 139 | 292 | 530 | 925 | 1257 |
| 1.80 | 114 | 247 | 444 | 763 | 1055 |
| 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 |

**Exhibit 3-20** **Flow of water from horizontal pipes (Ref. Purdue Eng. Expt. Bulletin 32, "Measurement of pipe flow by the coordinate method," August 1928)**

(Sheet 2 of 3)

WHEN X = 18 INCHES

| Y (inches) | Size of pipe (nominal diameter) | | | | |
|---|---|---|---|---|---|
| | 2-inch | 3-inch | 4-inch | 5-inch | 6-inch |
| | G.p.m. | G.p.m. | G.p.m. | G.p.m. | G.p.m. |
| 1.80 | 166 | 346 | 624 | 1014 | 1400 |
| 2.40 | 144 | 305 | 557 | 907 | 1261 |
| 3.00 | 129 | 274 | 503 | 826 | 1153 |
| 3.60 | 117 | 251 | 462 | 754 | 1068 |
| 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 |

Exhibit 3-20 **Flow of water from horizontal pipes (Ref. Purdue Eng. Expt. Bulletin 32, "Measurement of pipe flow by the coordinate method," August 1928)** (Sheet 3 of 3)

Chapter 3

## References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2. Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, Bureau of Public Roads, Department of Transportation

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4. Hydraulic Tables, U. S. Department of Defense, Corps of Engineers

5. Chow, VenTe; Open-Channel Hydraulics, McGraw-Hill Book Co., Inc., 1959

6. National Engineering Handbook, Section 5, Hydraulics


In addition to the above references, liberal use was made of material from the following references:


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