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Chapter A3

MEASUREMENT OF PEAK DISCHARGE AT CULVERTS BY INDIRECT METHODS

By G. L. Bodhaine

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PREFACE

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SYMBOLS AND UNITS

Symbol	Definition	Unit
A	Area.	ft^2
A_0	Area of culvert barrel.	ft^2
A_c	Area of section of flow at critical depth.	ft^2
b	Width of contracted flow section for box culvert.	ft
C	Coefficient of discharge; also, coefficient for computing various culvert properties; subscripts refer to specific items, as a for area, k for conveyance, m for mean depth, p for wetted perimeter, q for discharge, r for hydraulic radius, and t for top width.	
D	Maximum inside vertical dimension of culvert barrel, or the inside diameter of a circular section. (For corrugated pipes, D is measured as the minimum inside diameter.)	ft
D_m	Maximum inside diameter of pipe culvert at entrance.	ft
d	Depth of flow measured from the lowest point in the cross section for culverts.	ft
d_c	Maximum depth in critical-flow section.	ft
d_m	Mean depth.	ft
F	Froude number.	
g	Gravitational constant (acceleration).	ft/sec^2
H_0	Specific energy.	ft
h	Static or piezometric head above an arbitrary datum.	ft
h_c	d_c+z for type 1 culvert flow.	ft
h_e	Head loss due to entrance contraction.	ft
h_f	Head loss due to friction.	ft
h_v	Velocity head at a section.	ft
K	Conveyance of a section.	ft^3/sec
K_c	Conveyance of critical depth section.	ft^3/sec
K_0	Conveyance of full culvert barrel.	ft^3/sec
k	Adjustment factor; subscripts refer to specific items, as a for skewed abutments with dikes, L for length, r and R for radius, w for length of wingwalls, and θ for wingwall angle.	
L	Length of culvert barrel, bridge abutment, or broad-crested weir in direction of flow.	ft
L_p	Distance a culvert barrel projects beyond a headwall or embankment.	ft
L_w	Distance from approach section to entrance of culvert, upstream side of contraction, or crest of weir.	ft
m	Channel-contraction ratio.	
n	Manning roughness of coefficient.	$ft^{1/6}$
n_c	Composite value of roughness coefficient.	$ft^{1/6}$
P	Wetted perimeter of cross section of flow.	ft
P_p	Wetted perimeter of the paved invert of a culvert.	ft
Q	Total discharge.	ft^3/sec
R	Hydraulic radius.	ft
R_0	Hydraulic radius of a culvert barrel.	ft
r	Radius of entrance rounding.	ft
S	Friction slope.	
S_c	Bed slope of culvert for which the normal depth and the critical depth are equal.	
S_0	Bed slope of culvert barrel.	
T	Width of a section at the water surface.	ft
V	Mean velocity of flow in a section.	ft/sec
V_0	Full culvert velocity.	ft/sec
w	Measure of the length of a wingwall or chamfer.	ft
x	Length of part-full flow.	ft
z	Elevation of a point above a datum.	ft
$_{1,2}$	Subscripts which denote the location of cross sections or section properties in downstream order.	
α	Velocity-head coefficient.	
θ	Acute angle between a wingwall and plane of contraction or headwall; and the bevel angle.	
$<$	Less than.	
$\leqq$	Equal to or less than.	
$>$	Greater than.	
$\geqq$	Equal to or greater than.	

MEASUREMENT OF PEAK DISCHARGE AT CULVERTS BY INDIRECT METHODS

By G. L. Bodhaine

Abstract

This chapter classifies culvert flow into six types, gives discharge equations based on continuity and energy equations, and describes procedures for measuring peak discharges using culverts in the field. Discharge coefficients for a variety of geometries and flow types are given. Ten examples detail step-by-step computation procedures.

Introduction

The peak discharge through culverts can be determined from high-water marks that define the headwater and tailwater elevations. This indirect method is used extensively to measure flood discharges from small drainage areas.

The head-discharge characteristics of culverts have been studied in laboratory investigations by the U.S. Geological Survey, the Bureau of Public Roads, and many universities. The procedures given in this report are based on the information obtained in these studies and in field studies of the flow through culverts at sites where the discharge was known.

Description of Culvert Flow

The placement of a roadway fill and culvert in a stream channel causes an abrupt change in the character of flow. This channel transition results in rapidly varied flow in which acceleration rather than boundary friction plays the primary role. The flow in the approach channel to the culvert is usually tranquil and fairly uniform. Within the culvert, however, the flow may be tranquil, critical, or rapid if the culvert is partly filled, or the culvert may flow full under pressure.

The physical features associated with culvert flow are illustrated in figure 1. They are the approach channel cross section at a distance equivalent to one opening width upstream from

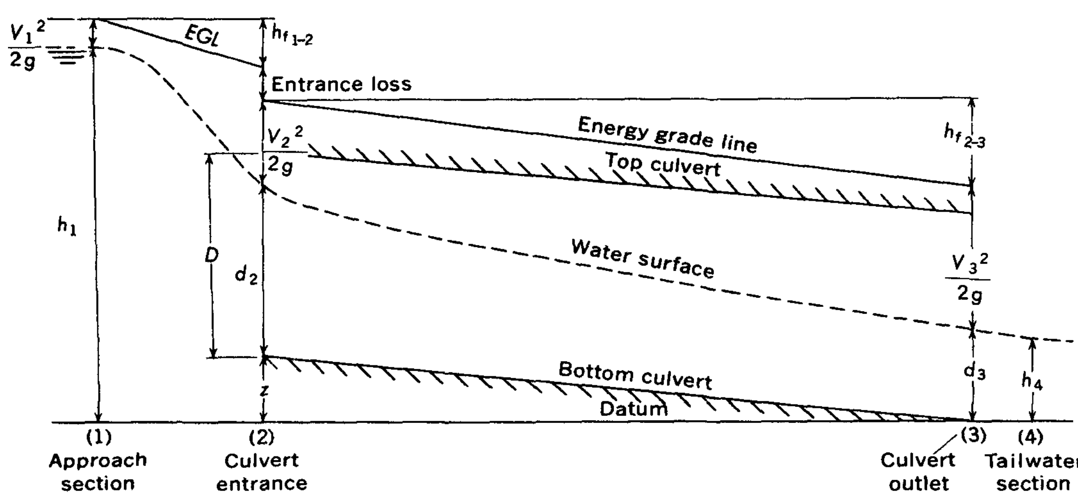


Figure 1.—Definition sketch of culvert flow. Note.—The loss of energy near the entrance is related to the sudden contraction and subsequent expansion of the live stream within the culvert barrel.

the entrance; the culvert entrance; the culvert barrel; the culvert outlet, the farthest downstream section of the barrel; and the tailwater representing the getaway channel.

The change in the water-surface profile in the approach channel reflects the effect of acceleration due to contraction of the cross-sectional area. Loss of energy near the entrance is related to the sudden contraction and subsequent expansion of the live stream within the barrel, and entrance geometry has an important influence on this loss. Loss of energy due to barrel friction is usually minor, except in long rough barrels on flat slopes. The important features that control the stage-discharge relationship at the approach section can be the occurrence of critical depth in the culvert, the elevation of the tailwater, the entrance or barrel geometry, or a combination of these.

The peak discharge through a culvert is determined by application of the continuity equation and the energy equation between the approach section and a section within the culvert barrel. The *location of the downstream* section depends on the state of flow in the culvert barrel. For example, if critical flow occurs at the culvert entrance, the headwater elevation is not a function of either the barrel friction loss or the tailwater elevation, and the terminal section is located at the upstream end of the culvert.

Information obtained in the field survey includes the peak elevation of the water surface upstream and downstream from the culvert and the geometry of the culvert and approach channel. Reliable high-water marks can rarely be found in the culvert barrel; therefore, the type of flow that occurred during the peak flow cannot always be determined directly from field data, and classification becomes a trial-and-error procedure.

General classification of flow

For convenience in computation, culvert flow has been classified into six types on the basis of the location of the control section and the relative heights of the headwater and tailwater elevations. The six types of flow are illustrated in figure 2, and pertinent characteristics of each type are given in table 1. From this information

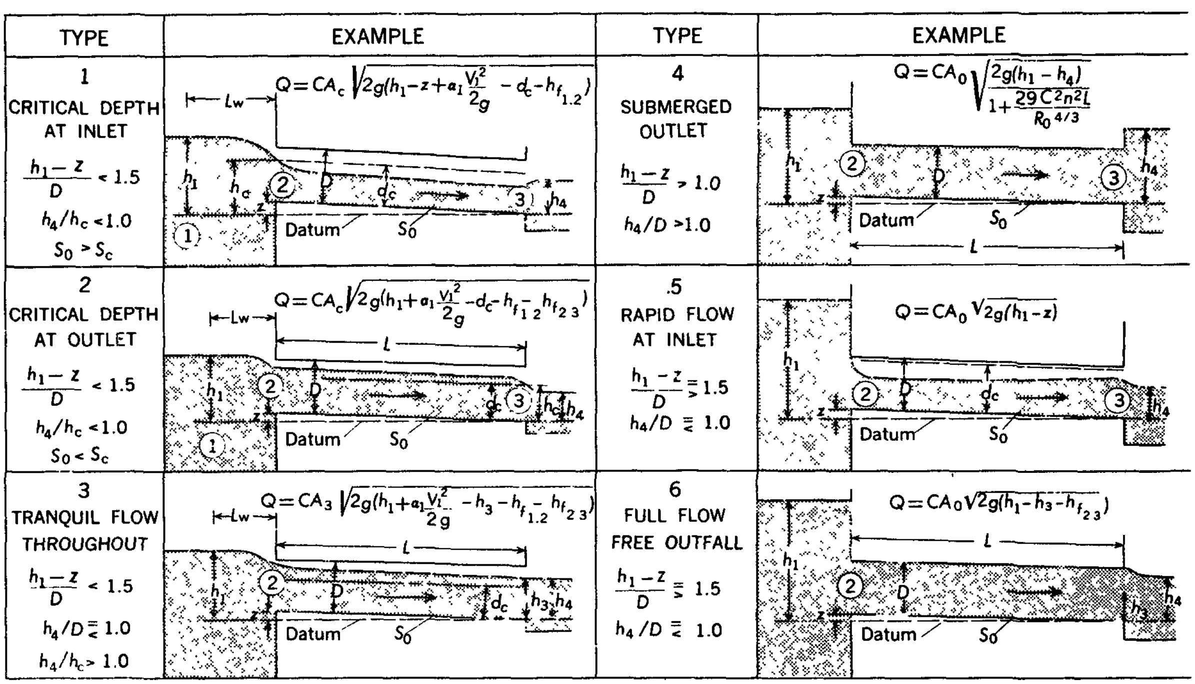


Figure 2.—Classification of culvert flow.

the following general classification of types of flow can be made:

1. If h_4/D is equal to or less than 1.0 and $(h_1-z)/D$ is less than 1.5, only types 1, 2, and 3 flow are possible.
2. If h_4/D is greater than 1.0, only type 4 flow is possible.
3. If h_4/D is equal to or less than 1.0 and $(h_1-z)/D$ is equal to or greater than 1.5, only types 5 and 6 flow are possible.

Further identification of the type of flow requires a trial-and-error procedure which is described in a subsequent section of this chapter.

Discharge Equations

Discharge equations have been developed for each type of flow by application of the continuity and energy equations between the approach section and the terminal section. The discharge may be computed directly from these equations after the type of flow has been identified. Discharge equations for critical depth at a section are used to identify flow types 1 and 2; thus, these equations are also included in the following sections.

Critical depth

Flow at critical depth may occur at either the upstream or the downstream end of a culvert, depending on the headwater elevation, the slope of the culvert, and the tailwater elevation. To obtain flow at critical depth, the headwater elevation above the upstream invert must be less than 1.5 times the diameter or height of the culvert. Type 1 flow will occur if the tailwater elevation is lower than the water-surface elevation at critical depth, and if the bed slope of the culvert is greater than the critical slope. Type 2 flow will occur if the bed slope is less than the critical slope.

Critical depth, d_c, is the depth at the point of minimum specific energy for a given discharge and cross section. The relation between specific energy and depth is illustrated in figure 3. The specific energy, H_0, is the height of the energy grade line above the lowest point in the cross section. Thus,

$$H_0=d+\frac{V^2}{2g},$$

where

H_0=specific energy,
d=maximum depth in the section,
V=mean velocity in the section, and
g=acceleration of gravity.

It can be shown that at the point of minimum specific energy and critical depth, d_c,

$$\frac{Q^2}{g}=\frac{A^3}{T},$$

and

$$\frac{V^2}{g}=d_m=\frac{A}{T},$$

where

Q=discharge,
A=area of cross section below the water surface,
T=width of the section at the water surface,
d_c=maximum depth of water in the critical-flow section, and
d_m=mean depth in section=A/T.

Table 1.—Characteristics of flow types

[D=maximum vertical height of barrel and diameter of circular culverts]

Flow type	Barrel flow	Location of terminal section	Kind of control	Culvert slope	$\frac{h_1-z}{D}$	$\frac{h_4}{h_c}$	$\frac{h_4}{D}$
1	Partly full	Inlet	Critical depth	Steep	<1.5	<1.0	$\leqq 1.0$
2	do	Outlet	do	Mild	<1.5	<1.0	$\leqq 1.0$
3	do	do	Backwater	do	<1.5	>1.0	$\leqq 1.0$
4	Full	do	do	Any	>1.0	------	>1.0
5	Partly full	Inlet	Entrance geometry	do	$\geqq 1.5$	------	$\leqq 1.0$
6	Full	Outlet	Entrance and barrel geometry.	do	$\geqq 1.5$	------	$\leqq 1.0$

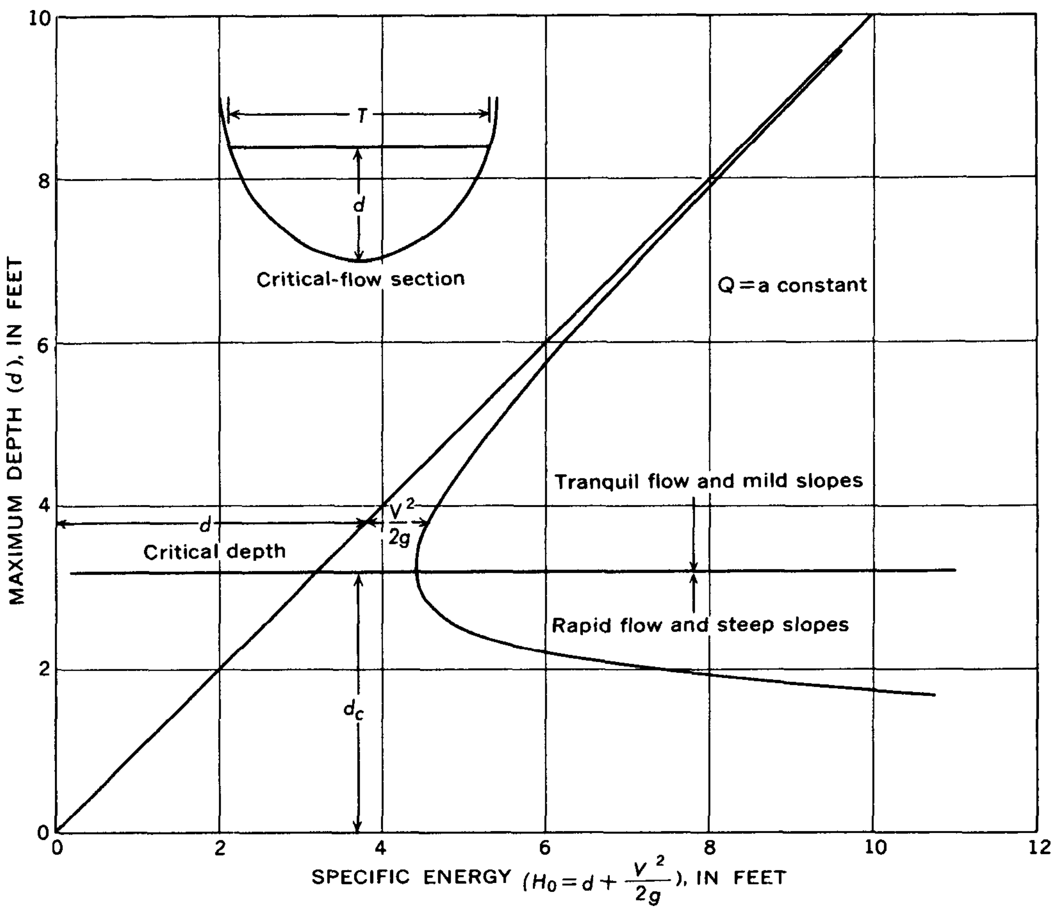


Figure 3.—Relation between specific energy and depth.

For the condition of minimum specific energy and critical depth, the discharge equation for a section of any shape can be written

$$Q=A_c^{3/2}\sqrt{\frac{g}{T}}, \quad (1)$$

or

$$Q=A_c\sqrt{gd_m}. \quad (2)$$

The discharge equation can be simplified according to the shape of the sections. Thus, for rectangular sections,

$$Q=5.67bd_c^{3/2}, \quad (3)$$

and for circular sections,

$$Q=C_qD^{5/2}, \quad (4)$$

where

b=width of section,
C_q=function of d_c/D,
d_c=maximum depth of water in the critical-flow section, and
D=inside diameter of circular section.

The two types of flow in this classification are 1 and 2.

Type 1 flow

In type 1 flow, as illustrated on figure 2, the water passes through critical depth near the culvert entrance. The headwater-diameter ratio, $(h_1-z)/D$, is limited to a maximum of 1.5 and the culvert barrel flows partly full. The slope of the culvert barrel, S_0, must be greater than the critical slope, S_c, and the tailwater elevation, h_4, must be less than the elevation of the water surface at the control section, h_2.

The discharge equation is

$$Q=CA_c\sqrt{2g\left(h_1-z+\frac{\alpha_1V_1^2}{2g}-d_c-h_{f_{1-2}}\right)}, \quad (5)$$

where

C = the discharge coefficient,
A_c = the flow area at the control section,
V_1 = the mean velocity in the approach section,
α_1 = the velocity-head coefficient at the approach section, and
$h_{f_{1-2}}$ = the head loss due to friction between the approach section and the inlet $= L_w(Q^2/K_1K_c)$, and
K = conveyance $= (1.486/n)R^{2/3}A$. (6)

Other notation is evident from figure 1 or it has been previously explained. The discharge coefficient, C, is discussed in detail in the section entitled "Coefficients of Discharge."

Type 2 flow

Type 2 flow, as shown on figure 2, passes through critical depth at the culvert outlet. The headwater-diameter ratio does not exceed 1.5, and the barrel flows partly full. The slope of the culvert is less than critical, and the tailwater elevation does not exceed the elevation of the water surface at the control section, h_3.

The discharge equation is

$$Q=CA_c\sqrt{2g\left(h_1+\frac{\alpha_1V_1^2}{2g}-d_c-h_{f_{1-2}}-h_{f_{2-3}}\right)}, \quad (7)$$

where

$h_{f_{2-3}}$ = the head loss due to friction in the culvert barrel $= L(Q^2/K_2K_3)$.

Flow with backwater

When backwater is the controlling factor in culvert flow, critical depth cannot occur and the upstream water-surface elevation for a given discharge is a function of the surface elevation of the tailwater. If the culvert flows partly full, the headwater-diameter ratio is less than 1.5; or if it flows full, both ends of the culvert are completely submerged and the headwater-diameter ratio may be any value greater than 1.0. The two types of flow in this classification are 3 and 4.

Type 3 flow

Type 3 flow is tranquil throughout the length of the culvert, as indicated on figure 2. The headwater-diameter ratio is less than 1.5, and the culvert barrel flows partly full. The tailwater elevation does not submerge the culvert outlet, but it does exceed the elevation of critical depth at the terminal section.

The lower limit of tailwater must be such that (1) if the culvert slope is steep enough that under free-fall conditions critical depth at the inlet would result from a given elevation of headwater, the tailwater must be at an elevation higher than the elevation of critical depth at the inlet; and (2) if the culvert slope is mild enough that under free-fall conditions critical depth at the outlet would result from a given elevation of headwater, then the tailwater must be at an elevation higher than the elevation of critical depth at the outlet.

The discharge equation for this type of flow is

$$Q=CA_3\sqrt{2g\left(h_1+\frac{\alpha_1V_1^2}{2g}-h_3-h_{f_{1-2}}-h_{f_{2-3}}\right)} \quad (8)$$

Type 4 flow

In this classification the culvert is submerged by both headwater and tailwater, as is shown in figure 2. The headwater-diameter ratio can be anything greater than 1.0. No differentiation is made between low-head and high-head flow on this basis for type 4 flow. The culvert flows full and the discharge may be computed directly from the energy equation between sections 1 and 4. Thus,

$$[h_1+h_{v_1}=h_4+h_{v_4}+h_{f_{1-2}}+h_e+h_{f_{2-3}}+h_{f_{3-4}}+(h_{v_3}-h_{v_4})],$$

where

h_e = head loss due to entrance contraction.

In the derivation of the discharge formula shown below, the velocity head at section 1 and the friction loss between sections 1 and 2 and between sections 3 and 4 have been neglected. Between sections 3 and 4 the energy loss due to sudden expansion is assumed to be $(h_{v_3}-h_{v_4})$. Thus,

$$h_1=h_4+h_e+h_{f_{2-3}}+h_{v_3},$$

or

$$Q=CA_0\sqrt{\frac{2g(h_1-h_4)}{1+\frac{29C^2n^2L}{R_0^{4/3}}}}. \quad (9)$$

Flow under high head

High-head flow will occur if the tailwater is below the crown at the outlet and the headwater-diameter ratio is equal to or greater than 1.5. This is an approximate criterion. The two types of flow under this category are 5 and 6.

As shown in figure 2, part-full flow under a high head is classified as type 5. The flow pattern is similar to that downstream from a sluice gate with rapid flow near the entrance. The occurrence of type 5 flow requires a relatively square entrance that will cause contraction of the area of live flow to less than the area of the culvert barrel. In addition, the combination of barrel length, roughness, and bed slope must be such that the contracted jet will not expand to the full area of the barrel. If the water surface of the expanding flow comes in contact with the top of the culvert, type 6 flow will occur, because the passage of air to the culvert will be sealed off causing the culvert to flow full throughout its length. Under these conditions, the headwater surface drops, indicating a more efficient use of the culvert barrel.

Within a certain range either type 5 or type 6 flow may occur, depending upon factors that are very difficult to evaluate. For example, the wave pattern superimposed on the water-surface profile through the culvert can be important in determining full or part-full flow. Within the range of geometries tested, however, the flow type generally can be predicted from a knowledge of entrance geometry and length, culvert slope, and roughness of the culvert barrel.

Type 5 flow

Type 5 flow is rapid at the inlet. The headwater-diameter ratio exceeds 1.5 as shown on figure 2, and the tailwater elevation is below the crown at the outlet. The top edge of the culvert entrance contracts the flow in a manner similar to a sluice gate. The culvert barrel flows partly full and at a depth less than critical. The discharge equation is

$$Q=CA_0\sqrt{2g(h_1-z)}. \qquad (10)$$

Type 6 flow

In type 6 flow the culvert is full under pressure with free outfall as shown in figure 2. The headwater-diameter ratio exceeds 1.5 and the tailwater does not submerge the culvert outlet. The discharge equation between sections 1 and 3, neglecting $V_1^2/2g$ and $h_{f_{1-2}}$, is

$$Q=CA_0\sqrt{2g(h_1-h_3-h_{f_{2-3}})}. \qquad (11)$$

A straightforward application of equation 11 is hampered by the necessity of determining h_3, which varies from a point below the center of the outlet to its top, even though the water surface is at the top of the culvert. This variation in piezometric head is a function of the Froude number. This difficulty has been circumvented by basing the data analysis upon dimensionless ratios of physical dimensions related to the Froude number. These functional relationships have been defined by laboratory experiment, and their use is explained on page 31.

Field Data

Make a transit survey of floodmarks and accurately measure the culvert geometry as soon after the flood as possible. Use the methods of surveying previously described; read elevations of highwater marks, hubs, reference marks, and culvert features to hundredths of a foot and ground elevations to tenths. Obtain high-water profiles as well as a complete description of the culvert geometry.

Describe entrance and getaway conditions if not evident from other data. Choose roughness coefficients (values of n) for the culvert as well as for the approach section, and obtain stereophotographs documenting pertinent features. Describe any unusual conditions at the site. Appraise the possibility of entrance or barrel obstruction at the time of the peak; document evidence obtained from observers.

Determine the elevation of the low point of the crown of the road over the culvert or make note that there is a high fill. If there is a possibility that water flowed across the roadway, define a profile along the crown or high point of the road.

High-water profiles

Obtain high-water profiles which adequately define the headwater and tailwater elevations in much the same manner as those for a slope-area or contracted-opening survey described in Benson and Dalrymple (1967). If high-water marks are available within a culvert, locate and level to them for use in checking computed elevations.

Headwater

Obtain the location and elevation of flood-marks along the embankment and upstream from the culvert. For a definite approach channel, obtain marks along the banks from the culvert entrance upstream for a distance of at least two culvert diameters. Where ponded conditions exist, determine the headwater elevation from marks along the banks or upstream from the opening where there is little or no velocity. In doubtful cases of ponding, conditions approximating ponding can be assumed if high-water marks along the embankments approach a level surface away from the culvert opening.

Tailwater

Obtain tailwater elevations along the embankment or the channel close to the outlet, but not to represent the elevation of the issuing jet, which may be higher than the tailwater pool. If marks cannot be found in the immediate vicinity of the culvert, extend the profile upstream to the outlet on the basis of the existing profiles.

Approach section

An approach section usually is necessary, but if the area of the approach channel is estimated as equal to or greater than five times the area of the culvert barrel, zero approach velocity in the approach section may be assumed, and an approach section is not required. To avoid the possibility of the approach section being within the drawdown region, locate it one culvert width upstream from the culvert entrance; or where wingwalls exist, a distance upstream from the end of the wingwalls equal to the width between wingwalls at their upstream end. If the wingwalls do not cause a significant contraction, the approach section may be closer than this, but not closer than one culvert width. Take the cross section at right angles to the channel. If high-water marks cannot be found at the location specified for the approach section, take the approach section where high-water marks can be found. Drive stakes at the ends of the cross section.

One culvert width at a multiple culvert installation may be considered as the sum of the widths of the individual culverts.

Select values of n and points of subdivision, if any, and record them clearly in the field notes.

Culvert geometry

Obtain complete details of culvert dimensions (measured by steel tape) including projections, wingwall angles, size of fillets and chamfers, degree of entrance rounding, size and shape of opening, type of entrance, and length. Describe the material of which the culvert is made (concrete, corrugated metal, iron, or other) as well as its condition (new, fair, poor, or other). Record the value of n for the culvert. (See section below on roughness coefficients.)

If dimensions of corrugated pipes, riveted pipe-arches, and multiplate pipe-arches differ significantly from design dimensions, the tabulated property coefficients listed in the tables may not be directly applicable. For this reason check in the field the pertinent dimensions of every culvert used.

In referring to culvert dimensions the usual practice of highway engineers of specifying the horizontal or width dimension first generally should be followed. For example, a 12- × 10-foot culvert has a barrel 12 feet wide and 10 feet high.

Normal culverts

Where the culvert opening is normal to the axis of the culvert, measure the elevation of the invert at the opening or headwall. For corrugated pipes, measure the invert elevation at the top of the first full corrugation and for concrete pipes, at the point of minimum diameter (not down in the bell). For rectangular shapes, obtain invert elevations usually at the center and edges, except in very wide sections where they should be obtained every 2 or 3 feet across the width. Elevations may be needed at closer intervals for irregular sections. The elevation of the downstream invert is determined similarly. Always obtain the elevation of the crown at both ends of the culvert. Also determine the elevations of the ends of the culvert aprons.

Locate the positions of the culvert, wingwalls, aprons, and other features. Measure the length of the culvert and length of aprons with a tape.

Skewed culverts

A skewed culvert is one in which the headwall is not normal to the centerline of the culvert or, in the event of no headwall, the end is not cut off squarely. Pipes and pipe-arches as well as box culverts are sometimes skewed. Where this occurs, measure the wingwall angle as for a normal culvert, as the acute angle at which the wingwall and headwall join.

Measure the invert elevation for a skewed culvert on a line normal to the axis of the culvert and at the point where the full section of the culvert begins (minimum section).

Measure the approach length, L_w, to the invert line described above. Measure the length of the culvert, L, between invert lines, the shortest length of the culvert.

Mitered culverts

Many pipes and pipe-arches are mitered to be flush with the highway embankment. Determine the invert elevations at the extreme ends of the pipe. Often, the first section of a mitered pipe is laid on a different slope from the rest of the culvert; therefore, obtain also the elevation of the invert at full pipe. Determine the elevation of the crown at both ends of the full pipe section. Measure the total length of the culvert to determine the slope (z is measured at ends of the invert). Also determine the short length (the full section of the culvert) and the length of miter.

For headwater-diameter ratios less than 1.0 measure the approach length from section 1 to the point where the headwater elevation intersects the miter, figure 4. For ratios greater than 1.0, L_w is the distance from section 1 to the beginning of the full section of the culvert. For outlet control, measure the length, L, to the point where d_c or h_3 intersects the downstream miter, or for a culvert flowing full, to the end of the full section.

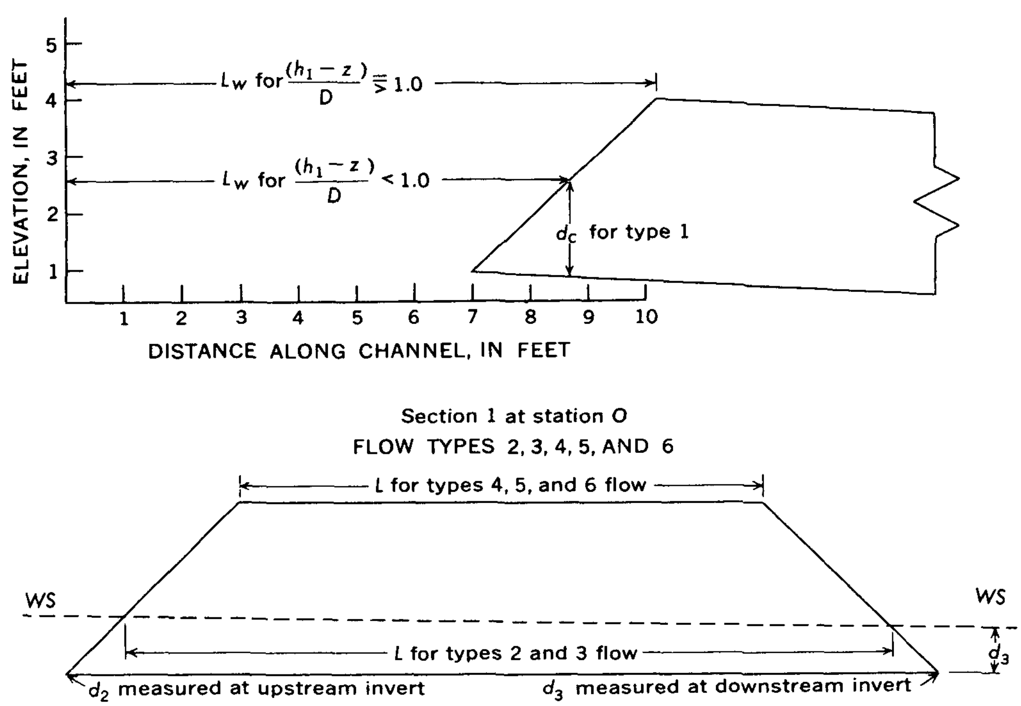


Figure 4.—Approach and culvert lengths for mitered pipe.

For type 1 flow, d_c is assumed to occur at the point where the headwater intersects the mitered entrance.

Photographs

Obtain stereophotographs showing culvert details and all pertinent conditions upstream and downstream from the culvert. These pictures are extremely important and may often avert a return trip to the site if certain data are unintentionally omitted during the field survey. Good photographs are a required part of the data necessary before computations can be completely reviewed.

General views of the relationship of the culvert to the approach channel, to crest-stage gages if they exist, and to the getaway conditions are useful. Take at least one closeup of the culvert entrance to show entrance detail. A level rod standing at the entrance furnishes a permanent record of culvert height and is a good reference for other details. Where road overflow occurs, include a view showing the entire overflow section.

Special conditions

Hydraulic characteristics of culverts in the field can be greatly different from closely controlled laboratory conditions. Before coefficients and methods derived in the laboratory can be applied to field installations, consider any features that tend to destroy model-prototype similarity.

Debris

Examine drift found lodged at the inlet of a culvert after a rise and evaluate its effect. It is not uncommon for material to float above a culvert at the peak without causing obstruction and then lodge at the bottom when the water subsides. However, if examination shows it to be well compacted in the culvert entrance and probably in the same position as during the peak, measure the obstructed area and deduct it from the total area. Sand and gravel found within a culvert barrel is often deposited after the extreme velocities of peak flow have passed; where this occurs, use the full area of the culvert. Careful judgment must be exercised because, in many places, levels before and after a peak show virtually the same invert elevations even though high velocities occurred.

Where discharge will be computed many times through a culvert that has a shifting bottom (natural bottom or deposited material), a cross section should be run after any severe flood, or at least once a year, and a record kept to evaluate the effect.

In certain areas ice and snow may present problems. Ice very often causes backwater partly blocking the culvert entrance. Snow frequently causes the deposition of misleading high-water marks as it melts.

Break in slope

Sometimes culverts are installed with a break in bottom slope. At other times a break in slope will occur as a result of uneven settling in soft fill material. Determine the elevation and location of the invert at the break.

A break in slope frequently occurs where a culvert is lengthened during road reconstruction. In rare cases the size and shape of the culvert may be changed at this time.

Natural bottom

Many culverts, especially small bridge-type structures and multiplate arches, have natural stream bottoms. The irregularity of the bottom may present difficulties in applying these data to the formulas for certain types of flow. Compute slope using average bottom elevations. The determination of depth to the minimum elevation (definition of d) in the cross section or to the average elevation has no effect in flow types 1, 2, and 3 so long as h_1, d_c, and h_3 are measured at the same points. For flow types 5 and 6, use the average bottom elevation to determine h_1.

Because natural bottoms in culverts usually cause nonuniformity in cross-sectional areas, special treatment must be given when the culvert is flowing full. An example is in type 4, where the standard formula is not applicable and the routing method of computing discharge should be used.

Roughness Coefficients

Roughness coefficients for use in the Manning equation should be selected in the field for both the approach section and the culvert at the time of the field survey.

Approach section

Select roughness coefficients for the approach section as outlined in the discussion of "Field Data." These coefficients will usually be in the range between 0.030 and 0.060 at culverts, because stream channels are usually kept cleared in the vicinity of the culvert entrance. At times the approach roughness coefficient may be lower than 0.030 when the culvert apron and wingwalls extend upstream to, or through, the approach section.

Select points of subdivision of the cross section in the field and assign values of n to the various parts. For crest-stage gages where various headwater elevations are used, n and the points of subdivision may change. For these sections, note the elevations at which the changes take place.

Culvert

Field inspection is always necessary before n values are assigned to any culvert. The condition of the material, the type of joint, and the kind of bottom, whether natural or constructed, all influence the selection of roughness coefficients.

Corrugated metal

A number of laboratory tests have been run to determine the roughness coefficient for corrugated-metal pipes of all sizes.

Standard riveted section

The corrugated metal used in the manufacture of standard pipes and pipe-arches has a 2⅔-inch pitch with a rise of ½ inch. According to laboratory tests (Neill, 1962), n values for full pipe flow vary from 0.0266 for a 1-foot-diameter pipe to 0.0224 for an 8-foot-diameter pipe for the velocities normally encountered in culverts. Tests indicate that n is slightly smaller for pipes flowing part full than for full pipe flow. The following are the results of tests by Neill (1962), and these values may be used:

Pipe diameter (feet)	n
1	0.027
2	.025
3–4	.024
5–7	.023
8	.022

A single value of 0.024 is considered satisfactory for both partly full and full pipe flow for most computations. This applies to all riveted pipes and pipe-arches of standard sizes.

Multiplate section

In multiplate construction the corrugations are much larger, having a 6-inch pitch with a 2-inch rise. Tests show n values to be somewhat higher than for riveted-pipe construction. Average n values from various experiments range from 0.034 for a 5-foot-diameter pipe to 0.027 for a 22-foot pipe. A straight line relationship of n values is assumed to exist for diameters between 5 and 22 feet. Use the following roughness coefficients:

Pipe diameter (feet)	n
5–6	0.034
7–8	.033
9–11	.032
12–13	.031
14–15	.030
16–18	.029
19–20	.028
21–22	.027

A corrugated pipe with corrugations half the size of those in multiplate construction, 3-inch pitch with a 1-inch rise, is being made in both standard and multiplate sections. Until actual tests are run to obtain n values, use average roughness coefficients between equal sizes of standard and multiplate sections—for example, use an n value of 0.028 for a 7-foot diameter pipe.

Paved inverts

In many instances the bottom parts of corrugated pipe and pipe-arch culverts are paved, usually with a bituminous material. This reduces the roughness coefficient to a value between that normally used and 0.012. The reduction is directly proportional to the amount of paved surface area in contact with the water, or wetted perimeter. The composite value of n for standard pipes and pipe-arches may be computed by the equation,

$$n_c = \frac{0.012P_p + 0.024(P - P_p)}{P}, \qquad (12)$$

where

P_p=length of wetted perimeter that is paved, and
P=total length of wetted perimeter.

For multiplate construction the value of 0.024 must be replaced with the correct value corresponding to the size of the pipe.

Concrete

The roughness coefficient of concrete is dependent upon the condition of the concrete and the irregularities of the surface resulting from construction. Suggested values of n for general use are:

Condition of concrete	n
Very smooth (spun pipe)	0.010
Smooth (cast or tamped pipe)	.011–0.015
Ordinary field construction	.012– .015
Badly spalled	.015– .020

At times, sections of concrete pipe become displaced either vertically or laterally, resulting in a much rougher interior surface than normal. Where this occurs, increase n commensurate with the degree of displacement of the culvert sections. Laboratory tests have shown that the displacement must be considerable before the roughness coefficient is very much affected.

Slight bends or changes in alinement of the culvert will not affect the roughness coefficient. However, the effects of fairly sharp bends or angles can be compensated for by raising the n value, as is done in slope-area measurements. Russell (1935) showed that for extremely sharp bends (90°) the head loss may vary from 0.2 to to 1.0 times the velocity head, depending on the radius of the bend and the velocity. The lower value applies to velocities of 2 or 3 feet per second and radii of 1–8 feet, and the higher value to velocities of 15–20 feet per second and radii of 40–60 feet. King (1954) stated that the losses in a 45° bend may be about ¾ as great, and for a 22½° bend about ½ as great as those of a 90° bend.

Other materials

Occasionally culverts will be constructed of some material other than concrete or corrugated metal. Manning's coefficients (King, 1954) for some of these materials are:

Material	n
Welded steel	0.012
Wood stave	.012
Cast iron	.013
Vitrified clay	.013
Riveted steel	.015

Culverts made from cement rubble or rock may have roughness coefficients ranging from 0.020 to 0.030, depending on the type of material and the care with which it is laid.

Natural bottom

Many culverts, especially the large arch type, are constructed with the natural channel as the bottom. The bottom roughness usually weights the composite roughness coefficient quite heavily, especially when the bottom is composed of cobbles and large angular rock. The formula used for paved inverts can be used here if the correct n values are substituted therein.

Computation of Discharge

The first step in the computation of discharge is to determine the type of flow. Under low heads, headwater-diameter ratios less than 1.5, type 3 flow will occur if the elevation of the downstream water surface is higher than the water-surface elevation at critical depth. If the tailwater elevation is lower than the water-surface elevation at critical depth, type 1 flow will occur with the bed slope of the culvert greater than the critical slope, or type 2 flow will occur with the bed slope less than the critical slope. Type 5 or 6 flow will occur with high heads, headwater-diameter ratios greater than 1.5, depending on the steepness of the culvert and the entrance conditions.

Discharge coefficients are a vital part of each culvert computation. These are discussed in detail on pages 37–45.

Tables 2–4 have information that applies to circular sections, riveted pipe-arches, and multiplate pipe-arches. Figures 5–8 are graphs

Table 2.—Properties of circular pipes, riveted pipe-arches, and multiplate pipe-arches

Circular pipes

Diam (in)	Diam D (ft)	Area A_0 (sq ft)	D^2	$D^{5/2}$	$D^{8/3}$	Diam (in)	Diam D (ft)	Area A_0 (sq ft)	D^2	$D^{5/2}$	$D^{8/3}$
6	0.500	0.196	0.250	0.177	0.157	60	5.00	19.6	25.0	55.9	73.0
8	.667	.349	.444	.363	.339	66	5.50	23.8	30.2	71.0	94.1
10	.833	.545	.694	.634	.614	72	6.00	28.3	36.0	88.2	119
12	1.00	.785	1.00	1.00	1.00	78	6.50	33.2	42.2	108	147
15	1.25	1.23	1.56	1.75	1.81	84	7.00	38.5	49.0	130	179
18	1.50	1.77	2.25	2.76	2.95	96	8.00	50.3	64.0	181	256
21	1.75	2.41	3.06	4.05	4.44	108	9.00	63.6	81.0	243	351
24	2.00	3.14	4.00	5.66	6.34	120	10	78.5	100	316	464
30	2.50	4.91	6.25	9.88	11.5	132	11	95.0	121	401	599
36	3.00	7.07	9.00	15.6	18.7	144	12	113	144	499	755
42	3.50	9.62	12.2	22.9	28.1	156	13	133	169	609	935
48	4.00	12.6	16.0	32.0	40.3	168	14	154	196	733	1,140
54	4.50	15.9	20.2	43.0	55.1	180	15	177	225	871	1,370

Pipe-arches

[Span (b), maximum width; Rise (D), maximum height]

Nominal dimensions		Span (b)	Rise (D)	Area (A)	D^2	$D^{5/2}$	$D^{8/3}$	Nominal dimensions		Span (b)	Rise (D)	Area (A)	D^2	$D^{5/2}$	$D^{8/3}$
Inches × Inches		Feet		Square feet				Inches × Inches		Feet		Square feet			
Riveted pipe-arches															
18	11	1.50	0.92	1.07	0.84	0.80	0.79	50	31	4.17	2.58	8.47	6.67	10.7	12.6
22	13	1.83	1.08	1.48	1.17	1.22	1.24	58	36	4.83	3.00	11.4	9.00	15.6	18.7
29	18	2.42	1.50	2.86	2.25	2.76	2.95	65	40	5.42	3.33	14.1	11.1	20.3	24.8
36	22	3.00	1.83	4.27	3.36	4.55	5.03	72	44	6.00	3.67	17.0	13.4	25.8	32.0
43	27	3.58	2.25	6.43	5.06	7.59	8.69								

Multiplate pipe-arches

Ft	in	× Ft	in							Ft	in	× Ft	in						
6*	1	4	7	6.08	4.58	22	20.97	44.89	57.9	11	7	7	5	11.62	7.42	67	55.06	150.0	209
6	4	4	9	6.34	4.76	24	22.66	49.43	64.1	11	10	7	7	11.82	7.61	71	57.91	159.8	224
6	9	4	11	6.76	4.91	26	24.11	53.42	69.6	12	4	7	9	12.32	7.75	74	60.06	167.2	235
7*	0	5	1	7.02	5.09	28	25.91	58.45	76.7	12	6	7	11	12.52	7.93	78	62.88	177.1	250
7	3	5	3	7.24	5.27	31	27.77	63.76	84.1	12	8	8	1	12.70	8.12	81	65.93	187.9	266
7	8	5	5	7.70	5.42	33	29.38	68.39	90.6	12*	10	8	4	12.86	8.31	85	69.06	199.1	283
7	11	5	7	7.94	5.60	35	31.36	74.21	98.9	13	5	8	5	13.40	8.44	89	71.23	206.9	295
8*	2	5	9	8.14	5.78	38	33.41	80.32	108	13	11	8	7	13.94	8.58	93	73.62	215.6	309
8	7	5	11	8.62	5.92	40	35.05	85.27	115	14	1	8	9	14.12	8.77	97	76.91	227.8	327
8	10	6	1	8.84	6.11	43	37.33	92.28	125	14	3	8	11	14.28	8.96	101	80.28	240.3	346
9	4	6	3	9.32	6.26	46	39.19	98.05	133	14	10	9	1	14.82	9.10	105	82.81	249.8	361
9	6	6	5	9.52	6.44	49	41.47	105.2	144	15*	4	9	3	15.34	9.23	109	85.19	258.8	375
9	9	6	7	9.72	6.62	52	43.82	112.8	155	15	6	9	5	15.54	9.42	113	88.74	272.3	396
10	3	6	9	10.22	6.77	55	45.83	119.3	164	15	8	9	7	15.70	9.61	118	92.35	286.3	417
10	8	6	11	10.70	6.91	58	47.75	125.5	173	15	10	9	10	15.86	9.80	122	96.04	300.7	440
10	11	7	1	10.92	7.09	61	50.27	133.8	186	16	5	9	11	16.42	9.93	126	98.61	310.7	456
11*	5	7	3	11.40	7.24	64	52.42	141.0	196	16*	7	10	1	16.58	10.12	130	102.41	325.8	479

*Size for which coefficients are provided in table 4.
Dimensions are from "Handbook of Drainage and Construction Products," Armco Drainage & Metal Products, Inc., 1955.

Table 3.—Coefficients for pipe of circular section flowing partly full

[Coefficients for (1) area, (2) wetted perimeter, (3) hydraulic radius, (4) conveyance, (5) discharge for critical-depth flow, and (6) top width]

d/D [1]	(1) $A=C_aD^2$ C_a	(2) $P=C_pD$ C_p	(3) $R=C_rD$ C_r	(4) $K=C_k\frac{D^{8/3}}{n}$ C_k	(5) $Q=C_qD^{5/2}$ C_q	(6) $T=C_tD$ C_t
0.01	0.0013	0.2003	0.0066	0.000068	0.0006	0.199
.02	.0037	.2838	.0132	.000307	.0025	.280
.03	.0069	.3482	.0197	.000747	.0055	.341
.04	.0105	.4027	.0262	.001376	.0098	.392
.05	.0147	.4510	.0325	.002228	.0153	.436
.06	.0192	.4949	.0389	.00328	.0220	.475
.07	.0242	.5355	.0451	.00457	.0298	.510
.08	.0294	.5735	.0513	.00601	.0389	.543
.09	.0350	.6094	.0575	.00775	.0491	.572
.10	.0409	.6435	.0635	.00966	.0605	.600
.11	.0470	.6761	.0695	.0118	.0731	.626
.12	.0534	.7075	.0755	.0142	.0868	.650
.13	.0600	.7377	.0813	.0168	.1016	.673
.14	.0668	.7670	.0871	.0195	.1176	.694
.15	.0739	.7954	.0929	.0225	.1347	.714
.16	.0811	.8230	.0985	.0257	.1530	.733
.17	.0885	.8500	.1042	.0291	.1724	.751
.18	.0961	.8763	.1097	.0327	.1928	.768
.19	.1039	.9020	.1152	.0366	.2144	.785
.20	.1118	.9273	.1206	.0405	.2371	.800
.21	.1199	.9521	.1259	.0446	.2609	.815
.22	.1281	.9764	.1312	.0491	.2857	.828
.23	.1365	1.0003	.1364	.0537	.3116	.842
.24	.1449	1.0239	.1416	.0586	.3386	.854
.25	.1535	1.0472	.1466	.0634	.3666	.866
.26	.1623	1.0701	.1516	.0685	.3957	.877
.27	.1711	1.0928	.1566	.0740	.4259	.888
.28	.1800	1.1152	.1614	.0792	.4571	.898
.29	.1890	1.1373	.1662	.0848	.4893	.908
.30	.1982	1.1593	.1709	.0907	.523	.917
.31	.2074	1.1810	.1756	.0968	.557	.925
.32	.2167	1.2025	.1802	.1027	.592	.933
.33	.2260	1.2239	.1847	.1088	.628	.940
.34	.2355	1.2451	.1891	.1155	.666	.947
.35	.2450	1.2661	.1935	.1220	.704	.954
.36	.2546	1.2870	.1978	.1283	.743	.960
.37	.2642	1.3078	.2020	.1350	.784	.966
.38	.2739	1.3284	.2062	.1421	.825	.971
.39	.2836	1.3490	.2102	.1488	.867	.975
.40	.2934	1.3694	.2142	.1561	.910	.980
.41	.3032	1.3898	.2182	.1631	.955	.984
.42	.3130	1.4101	.2220	.1702	1.000	.987
.43	.3229	1.4303	.2258	.1780	1.046	.990
.44	.3328	1.4505	.2295	.1854	1.093	.993
.45	.3428	1.4706	.2331	.1931	1.141	.995
.46	.3527	1.4907	.2366	.2002	1.190	.997
.47	.3627	1.5108	.2401	.2080	1.240	.998
.48	.3727	1.5308	.2435	.2160	1.291	.999
.49	.3827	1.5508	.2468	.2235	1.343	1.000
.50	.3927	1.5708	.2500	.2317	1.396	1.000
0.51	0.4027	1.5908	0.2531	0.2394	1.449	1.000
.52	.4127	1.6108	.2562	.2472	1.504	.999
.53	.4227	1.6308	.2592	.2556	1.560	.998
.54	.4327	1.6509	.2621	.2630	1.616	.997
.55	.4426	1.6710	.2649	.2710	1.674	.995
.56	.4526	1.6911	.2676	.2791	1.733	.993
.57	.4625	1.7113	.2703	.2873	1.792	.990
.58	.4724	1.7315	.2728	.2955	1.853	.987
.59	.4822	1.7518	.2753	.3031	1.915	.984
.60	.4920	1.7722	.2776	.3115	1.977	.980
.61	.5018	1.7926	.2799	.3192	2.041	.975
.62	.5115	1.8132	.2821	.3268	2.106	.971
.63	.5212	1.8338	.2842	.3346	2.172	.966
.64	.5308	1.8546	.2862	.3423	2.239	.960
.65	.5404	1.8755	.2882	.3501	2.307	.954
.66	.5499	1.8965	.2900	.3579	2.376	.947
.67	.5594	1.9177	.2917	.3658	2.446	.940
.68	.5687	1.9391	.2933	.3727	2.518	.933
.69	.5780	1.9606	.2948	.3805	2.591	.925
.70	.5872	1.9823	.2962	.3874	2.666	.917
.71	.5964	2.0042	.2975	.3953	2.741	.908
.72	.6054	2.0264	.2987	.4021	2.819	.898
.73	.6143	2.0488	.2998	.4090	2.898	.888
.74	.6231	2.0714	.3008	.4157	2.978	.877
.75	.6319	2.0944	.3017	.4226	3.061	.866
.76	.6405	2.1176	.3024	.4283	3.145	.854
.77	.6489	2.1412	.3031	.4349	3.231	.842
.78	.6573	2.1652	.3036	.4415	3.320	.828
.79	.6655	2.1895	.3039	.4470	3.411	.815
.80	.6736	2.2143	.3042	.4524	3.505	.800
.81	.6815	2.2395	.3043	.4578	3.602	.785
.82	.6893	2.2653	.3043	.4630	3.702	.768
.83	.6969	2.2916	.3041	.4681	3.806	.751
.84	.7043	2.3186	.3038	.4731	3.914	.733
.85	.7115	2.3462	.3033	.4768	4.028	.714
.86	.7186	2.3746	.3026	.4816	4.147	.694
.87	.7254	2.4038	.3018	.4851	4.272	.673
.88	.7320	2.4341	.3007	.4884	4.406	.650
.89	.7384	2.4655	.2995	.4916	4.549	.626
.90	.7445	2.4981	.2980	.4935	4.70	.600
.91	.7504	2.5322	.2963	.4951	4.87	.572
.92	.7560	2.5681	.2944	.4966	5.06	.543
.93	.7612	2.6061	.2921	.4977	5.27	.510
.94	.7662	2.6467	.2895	.4979	5.52	.475
.95	.7707	2.6906	.2865	.4970	5.81	.436
.96	.7749	2.7389	.2829	.4963	6.18	.392
.97	.7785	2.7934	.2787	.4940	6.67	.341
.98	.7817	2.8578	.2735	.4902	7.41	.280
.99	.7841	2.9412	.2666	.4824	8.83	.199
1.00	.7854	3.1416	.2500	.4633	-------	.000

[1] d=maximum depth of water in feet; D=diameter of pipe, in feet.

Table 4.—Coefficients for pipe-arches flowing partly full

[Coefficients for (1) area, (2) hydraulic radius, (3) conveyance, (4) mean depth, (5) discharge for critical-depth flow]

d/D[1]	(1)	(2)	(3)	(4)	(5)	d/D[1]	(1)	(2)	(3)	(4)	(5)
	$A=C_aD^2$	$R=C_rD$	$K=C_k\frac{D^{8/3}}{n}$	$d_m=C_mD$	$Q=C_qD^{5/2}$		$A=C_aD^2$	$R=C_rD$	$K=C_k\frac{D^{8/3}}{n}$	$d_m=C_mD$	$Q=C_qD^{5/2}$
	C_a	C_r	C_k	C_m	C_q		C_a	C_r	C_k	C_m	C_q
A. Nominal size 6 feet 1 inch × 4 feet 7 inches											
0.01	0.005	0.016	0.000	0.016	0.003	0.51	0.586	0.289	0.380	0.450	2.23
.02	.009	.019	.001	.019	.007	.52	.600	.292	.393	.463	2.32
.03	.014	.025	.002	.025	.012	.53	.614	.296	.406	.476	2.41
.04	.019	.029	.003	.029	.018	.54	.628	.300	.419	.490	2.50
.05	.026	.036	.004	.035	.028	.55	.643	.304	.431	.504	2.59
.06	.033	.042	.006	.041	.038	.56	.657	.307	.445	.519	2.69
.07	.042	.048	.008	.048	.052	.57	.671	.311	.458	.535	2.79
.08	.050	.054	.011	.054	.066	.58	.686	.314	.471	.551	2.89
.09	.059	.061	.014	.061	.082	.59	.700	.318	.484	.568	2.99
.10	.068	.067	.017	.067	.099	.60	.714	.321	.497	.584	3.10
.11	.078	.075	.021	.076	.122	.61	.724	.321	.505	.597	3.17
.12	.089	.082	.025	.084	.145	.62	.733	.322	.512	.609	3.25
.13	.099	.089	.029	.091	.170	.63	.743	.322	.519	.621	3.32
.14	.110	.096	.034	.099	.195	.64	.752	.323	.526	.634	3.40
.15	.120	.102	.039	.106	.221	.65	.762	.323	.533	.647	3.48
.16	.131	.109	.044	.114	.251	.66	.776	.327	.547	.666	3.59
.17	.142	.115	.050	.122	.282	.67	.790	.330	.561	.686	3.71
.18	.154	.121	.056	.129	.314	.68	.805	.334	.576	.706	3.84
.19	.165	.127	.062	.137	.346	.69	.819	.337	.590	.727	3.96
.20	.176	.132	.068	.144	.380	.70	.833	.341	.604	.749	4.09
.21	.189	.139	.075	.153	.418	.71	.843	.341	.611	.766	4.19
.22	.201	.145	.082	.162	.458	.72	.852	.341	.619	.784	4.28
.23	.213	.151	.090	.170	.499	.73	.862	.342	.626	.803	4.38
.24	.226	.157	.098	.178	.541	.74	.871	.342	.633	.822	4.48
.25	.238	.163	.105	.186	.583	.75	.881	.342	.640	.841	4.58
26	.252	.170	.115	.197	.635	.76	.890	.343	.649	.864	4.70
.27	.267	.177	.125	.207	.688	.77	.900	.345	.657	.889	4.81
.28	.281	.184	.135	.217	.742	.78	.909	.346	.666	.914	4.93
.29	.295	.191	.145	.227	.797	.79	.919	.347	.675	.940	5.06
.30	.309	.197	.156	.236	.854	.80	.928	.349	.684	.967	5.18
.31	.324	.203	.166	.246	.912	.81	.936	.347	.687	.998	5.31
.32	.338	.209	.177	.256	.972	.82	.944	.345	.690	1.03	5.43
.33	.352	.215	.188	.266	1.03	.83	.951	.344	.694	1.06	5.57
.34	.367	.221	.199	.276	1.09	.84	.959	.342	.697	1.10	5.70
.35	.381	.227	.210	.286	1.16	.85	.966	.341	.701	1.14	5.85
.36	.392	.231	.219	.295	1.21	.86	.973	.338	.702	1.18	6.01
.37	.404	.234	.228	.303	1.26	.87	.980	.336	.704	1.23	6.18
.38	.415	.238	.237	.312	1.32	.88	.986	.334	.706	1.29	6.35
.39	.427	.241	.246	.320	1.37	.89	.993	.332	.707	1.35	6.54
.40	.438	.245	.255	.329	1.42	.90	1.00	.330	.709	1.41	6.74
.41	.450	.249	.265	.338	1.49	.91	1.00	.326	.707	1.50	6.98
.42	.463	.252	.275	.347	1.55	.92	1.01	.323	.705	1.60	7.23
.43	.475	.256	.285	.357	1.61	.93	1.01	.319	.704	1.71	7.52
.44	.488	.260	.295	.366	1.67	.94	1.02	.316	.702	1.83	7.82
.45	.500	.263	.305	.376	1.74	.95	1.02	.313	.701	2.02	8.25
.46	.514	.268	.317	.388	1.82	.96	1.03	.304	.691	--------	--------
.47	.528	.272	.330	.400	1.90	.97	1.03	.296	.682	--------	--------
.48	.543	.276	.342	.412	1.98	.98	1.04	.288	.673	--------	--------
.49	.557	.281	.355	.424	2.06	.99	1.04	.281	.665	--------	--------
.50	.571	.285	.367	.436	2.14	1.00	1.05	.274	.658	--------	--------

[1] d=depth of water, in feet; D=rise, in feet.

Table 4.—Coefficients for pipe-arches flowing partly full—Continued

d/D[1]	(1) $A=C_aD^2$	(2) $R=C_rD$	(3) $K=C_k\frac{D^{8/3}}{n}$	(4) $d_m=C_mD$	(5) $Q=C_qD^{5/2}$	d/D	(1) $A=C_aD^2$	(2) $R=C_rD$	(3) $K=C_k\frac{D^{8/3}}{n}$	(4) $d_m=C_mD$	(5) $Q=C_qD^{5/2}$
	C_a	C_r	C_k	C_m	C_q		C_a	C_r	C_k	C_m	C_q
B. Nominal size 7 feet 0 inch × 5 feet 1 inch											
0.01	0.006	0.017	0.001	0.017	0.004	0.51	0.594	0.283	0.380	0.447	2.25
.02	.012	.022	.001	.022	.010	.52	.609	.287	.394	.462	2.35
.03	.016	.026	.002	.026	.015	.53	.625	.292	.409	.476	2.45
.04	.021	.029	.003	.029	.020	.54	.640	.296	.423	.491	2.55
.05	.028	.035	.004	.035	.029	.55	.656	.301	.437	.506	2.65
.06	.035	.040	.006	.039	.039	.56	.667	.303	.447	.518	2.72
.07	.044	.047	.009	.047	.055	.57	.679	.305	.457	.530	2.80
.08	.054	.054	.011	.054	.071	.58	.690	.307	.467	.542	2.89
.09	.064	.061	.015	.061	.089	.59	.702	.309	.476	.555	2.97
.10	.073	.067	.018	.067	.108	.60	.714	.311	.486	.568	3.05
.11	.085	.075	.022	.076	.133	.61	.725	.313	.497	.582	3.14
.12	.096	.083	.027	.084	.159	.62	.737	.315	.507	.597	3.23
.13	.108	.090	.032	.092	.186	.63	.748	.317	.517	.613	3.32
.14	.120	.096	.037	.100	.215	.64	.760	.320	.528	.628	3.42
.15	.131	.103	.043	.107	.244	.65	.771	.322	.538	.644	3.51
.16	.141	.108	.048	.114	.270	.66	.784	.324	.550	.662	3.62
.17	.151	.114	.053	.120	.298	.67	.798	.326	.562	.681	3.74
.18	.161	.119	.058	.127	.326	.68	.811	.329	.574	.701	3.85
.19	.171	.124	.063	.133	.354	.69	.824	.331	.586	.721	3.97
.20	.181	.128	.068	.139	.383	.70	.837	.333	.597	.741	4.09
.21	.194	.135	.076	.147	.422	.71	.848	.335	.608	.761	4.20
.22	.206	.141	.083	.156	.461	.72	.860	.337	.619	.781	4.31
.23	.218	.147	.090	.164	.502	.73	.872	.339	.629	.801	4.43
.24	.231	.153	.098	.172	.544	.74	.883	.341	.640	.822	4.54
.25	.243	.159	.106	.181	.586	.75	.895	.343	.651	.844	4.66
.26	.258	.166	.115	.191	.639	.76	.905	.343	.659	.868	4.78
.27	.272	.172	.125	.201	.693	.77	.915	.344	.667	.892	4.90
.28	.287	.179	.135	.211	.749	.78	.925	.345	.676	.918	5.03
.29	.302	.185	.146	.222	.806	.79	.935	.345	.684	.945	5.16
.30	.316	.192	.156	.232	.864	.80	.945	.346	.692	.972	5.29
.31	.330	.198	.167	.242	.921	.81	.956	.346	.700	1.01	5.45
.32	.344	.204	.177	.251	.979	.82	.968	.346	.709	1.04	5.61
.33	.358	.209	.188	.261	1.04	.83	.980	.345	.717	1.08	5.78
.34	.372	.215	.198	.271	1.10	.84	.991	.345	.725	1.12	5.95
.35	.386	.221	.209	.281	1.16	.85	1.003	.345	.733	1.16	6.13
.36	.401	.226	.221	.292	1.23	.86	1.009	.342	.734	1.21	6.30
.37	.417	.232	.234	.304	1.30	.87	1.015	.340	.735	1.26	6.47
.38	.432	.238	.246	.316	1.38	.88	1.021	.338	.736	1.32	6.65
.39	.447	.243	.259	.327	1.45	.89	1.027	.335	.737	1.38	6.84
.40	.463	.248	.272	.339	1.53	.90	1.034	.333	.738	1.44	7.04
.41	.474	.252	.281	.348	1.59	.91	1.039	.330	.737	1.52	7.28
.42	.486	.255	.290	.357	1.65	.92	1.044	.327	.736	1.61	7.52
.43	.498	.259	.300	.366	1.71	.93	1.050	.324	.735	1.72	7.82
.44	.509	.262	.310	.375	1.77	.94	1.055	.321	.734	1.85	8.15
.45	.521	.265	.319	.384	1.83	.95	1.061	.318	.734	2.04	8.59
.46	.532	.268	.329	.394	1.90	.96	1.064	.310	.724	--------	--------
.47	.544	.270	.338	.404	1.96	.97	1.068	.302	.715	--------	--------
.48	.555	.273	.347	.413	2.03	.98	1.072	.295	.706	--------	--------
.49	.567	.275	.357	.423	2.09	.99	1.076	.288	.698	--------	--------
.50	.579	.278	.366	.433	2.16	1.00	1.081	.282	.691	--------	--------

[1] d=depth of water, in feet; D=rise, in feet.

Table 4.—Coefficients for pipe-arches flowing partly full—Continued

d/D[1]	(1) $A=C_aD^2$ C_a	(2) $R=C_rD$ C_r	(3) $K=C_k\frac{D^{8/3}}{n}$ C_k	(4) $d_m=C_mD$ C_m	(5) $Q=C_qD^{5/2}$ C_q	d/D[1]	(1) $A=C_aD^2$ C_a	(2) $R=C_rD$ C_r	(3) $K=C_k\frac{D^{8/3}}{n}$ C_k	(4) $d_m=C_mD$ C_m	(5) $Q=C_qD^{5/2}$ C_q
C. Nominal size 8 feet 2 inches × 5 feet 9 inches											
0.01	0.007	0.015	0.001	0.015	0.005	0.51	0.658	0.304	0.442	0.497	2.63
.02	.013	.021	.002	.021	.011	.52	.673	.308	.456	.511	2.73
.03	.020	.026	.003	.026	.019	.53	.688	.312	.470	.525	2.83
.04	.027	.030	.004	.030	.026	.54	.703	.315	.484	.539	2.93
.05	.036	.037	.006	.036	.039	.55	.718	.319	.498	.553	3.03
.06	.045	.044	.008	.042	.052	.56	.730	.322	.509	.566	3.11
.07	.063	.057	.014	.056	.085	.57	.742	.324	.520	.578	3.20
.08	.081	.070	.020	.070	.121	.58	.754	.326	.531	.590	3.29
.09	.087	.073	.023	.073	.133	.59	.765	.328	.541	.603	3.37
.10	.093	.077	.025	.077	.146	.60	.777	.331	.552	.616	3.46
.11	.106	.085	.030	.086	.177	.61	.789	.333	.563	.631	3.56
.12	.119	.094	.036	.096	.209	.62	.801	.335	.574	.645	3.65
.13	.132	.101	.043	.105	.243	.63	.813	.337	.585	.661	3.75
.14	.145	.109	.049	.114	.279	.64	.825	.339	.596	.676	3.85
.15	.158	.116	.056	.123	.315	.65	.837	.341	.607	.692	3.95
.16	.172	.123	.063	.132	.354	.66	.849	.342	.618	.708	4.05
.17	.185	.131	.071	.141	.394	.67	.861	.344	.628	.724	4.16
.18	.198	.138	.078	.149	.434	.68	.873	.345	.639	.740	4.26
.19	.211	.144	.086	.158	.476	.69	.885	.347	.649	.757	4.37
.20	.224	.151	.094	.166	.519	.70	.897	.348	.660	.774	4.48
.21	.239	.159	.104	.176	.570	.71	.909	.350	.671	.796	4.60
.22	.254	.166	.114	.186	.622	.72	.921	.352	.682	.819	4.73
.23	.269	.174	.124	.196	.676	.73	.933	.354	.693	.843	4.86
.24	.284	.181	.135	.205	.731	.74	.945	.355	.704	.867	4.99
.25	.299	.188	.146	.215	.786	.75	.957	.357	.716	.892	5.13
.26	.314	.194	.157	.225	.845	.76	.966	.357	.722	.916	5.24
.27	.329	.201	.168	.235	.905	.77	.975	.357	.729	.940	5.36
.28	.344	.207	.179	.245	.966	.78	.984	.357	.736	.964	5.48
.29	.359	.213	.190	.255	1.03	.79	.993	.357	.743	.990	5.61
.30	.374	.218	.201	.265	1.09	.80	1.002	.358	.750	1.02	5.73
.31	.386	.223	.211	.273	1.14	.81	1.011	.357	.756	1.03	5.83
.32	.398	.227	.220	282	1.20	.82	1.020	.356	.761	1.05	5.93
.33	.410	.231	.229	.290	1.25	.83	1.029	.355	.767	1.07	6.03
.34	.422	.235	.239	.299	1.31	.84	1.038	.355	.772	1.08	6.13
.35	.434	.239	.248	.308	1.36	.85	1.047	.354	.778	1.10	6.23
.36	.449	.244	260	.319	1.44	.86	1.054	.352	.780	1.17	6.46
.37	.463	.250	.273	.330	1.51	.87	1.062	.349	.783	1.24	6.71
.38	.478	.255	.286	.342	1.59	.88	1.070	.347	.785	1.32	6.97
.39	.493	.260	.299	.354	1.66	.89	1.078	.345	.788	1.41	7.26
.40	.508	.265	.312	.365	1.74	.90	1.085	.343	.790	1.51	7.58
.41	.521	.268	.322	.376	1.81	.91	1.091	.339	.789	1.60	7.83
.42	.535	.272	.333	.387	1.89	.92	1.097	.336	.788	1.69	8.10
.43	.548	.275	.344	.398	1.96	.93	1.103	.332	.787	1.82	8.45
.44	.561	.278	.355	.409	2.04	.94	1.109	.329	.786	1.97	8.85
.45	.574	.281	.366	.420	2.11	.95	1.115	.326	.785	2.17	9.32
.46	.588	.285	.378	.433	2.19	.96	1.119	.318	.775	--------	--------
.47	.602	.289	.391	.445	2.28	.97	1.123	.311	.765	--------	--------
.48	.615	.293	.403	.457	2.36	.98	1.127	.304	.756	--------	--------
.49	.629	.296	.415	.470	2.45	.99	1.130	.297	.748	--------	--------
.50	.643	.300	.428	.483	2.54	1.00	1.137	.291	.739	--------	--------

[1] d=depth of water, in feet; D=rise, in feet.

Table 4.—Coefficients for pipe-arches flowing partly full—Continued

d/D[1]	(1) $A=C_aD^2$	(2) $R=C_rD$	(3) $K=C_k\frac{D^{8/3}}{n}$	(4) $d_m=C_mD$	(5) $Q=C_qD^{5/2}$	d/D[1]	(1) $A=C_aD^2$	(2) $R=C_rD$	(3) $K=C_k\frac{D^{8/3}}{n}$	(4) $d_m=C_mD$	(5) $Q=C_qD^{5/2}$
	C_a	C_r	C_k	C_m	C_q		C_a	C_r	C_k	C_m	C_q
D. Nominal sizes 11 feet 5 inches × 7 feet 3 inches and 12 feet 10 inches × 8 feet 4 inches											
0.01	0.007	0.014	0.001	0.014	0.004	0.51	0.710	.0311	0.484	0.488	2.81
.02	.013	.021	.002	.021	.011	.52	.724	.315	.498	.501	2.91
.03	.020	.027	.003	.027	.019	.53	.739	.318	.512	.514	3.01
.04	.027	.032	.004	.032	.028	.54	.754	.321	.526	.527	3.11
.05	.037	.038	.006	.038	.041	.55	.769	.324	.539	.541	3.21
.06	.046	.044	.009	.043	.054	.56	.783	.327	.553	.555	3.31
.07	.057	.050	.012	.050	.072	.57	.797	.330	.566	.569	3.41
.08	.068	.056	.015	.056	.091	.58	.811	.333	.580	.584	3.52
.09	.079	.063	.019	.063	.113	.59	.825	.336	.593	.599	3.63
.10	.091	.069	.023	.069	.136	.60	.840	.339	.607	.614	3.73
.11	.104	.078	.028	.078	.165	.61	.854	.342	.620	.630	3.85
.12	.117	.085	.034	.086	.196	.62	.867	.344	.633	.647	3.96
.13	.131	.093	.040	.094	.228	.63	.881	.347	.646	.664	4.08
.14	.144	.100	.046	.101	.261	.64	.895	.349	.659	.682	4.20
.15	.158	.107	.053	.109	.296	.65	.909	.351	.673	.700	4.32
.16	.173	.115	.061	.118	.337	.66	.921	.352	.683	.716	4.42
.17	.188	.123	.069	.127	.380	.67	.934	.354	.694	.733	4.53
.18	.203	.130	.077	.136	.424	.68	.946	.355	.704	.750	4.65
.19	.218	.137	.086	.144	.470	.69	.958	.356	.715	.767	4.76
.20	.233	.144	.095	.153	.517	.70	.970	.357	.725	.785	4.88
.21	.248	.151	.105	.162	.569	.71	.982	.358	.735	.806	5.00
.22	.264	.159	.115	.172	.622	.72	.994	.359	.746	.827	5.13
.23	.280	.166	.126	.181	.676	.73	1.005	.360	.756	.848	5.26
.24	.295	.173	.136	.191	.732	.74	1.017	.361	.766	.871	5.39
.25	.311	.180	.147	.200	.789	.75	1.029	.362	.776	.894	5.52
.26	.328	.188	.160	.211	.856	.76	1.039	.361	.783	.918	5.65
.27	.345	.195	.173	.222	.924	.77	1.048	.361	.790	.944	5.78
.28	.363	.203	.186	.233	.994	.78	1.058	.361	.796	.971	5.92
.29	.380	.210	.199	.244	1.07	.79	1.068	.360	.803	.999	6.05
.30	.397	.217	.213	.255	1.14	.80	1.077	.360	.810	1.03	6.20
.31	.411	.222	.224	.264	1.20	.81	1.086	.359	.815	1.06	6.35
.32	.425	.227	.235	.274	1.26	.82	1.096	.358	.821	1.10	6.51
.33	.440	.232	.247	.283	1.33	.83	1.105	.357	.827	1.13	6.68
.34	.454	.237	.258	.293	1.39	.84	1.114	.357	.832	1.17	6.85
.35	.468	.241	.269	.302	1.46	.85	1.123	.356	.838	1.22	7.03
.36	.484	.247	.283	.313	1.54	.86	1.132	.354	.842	1.27	7.24
.37	.499	.252	.296	.324	1.61	.87	1.140	.353	.846	1.33	7.46
.38	.515	.257	.309	.335	1.69	.88	1.149	.352	.851	1.39	7.69
.39	.530	.262	.323	.346	1.77	.89	1.158	.350	.855	1.46	7.93
.40	546	.267	.337	.357	1.85	.90	1.167	.349	.859	1.53	8.19
.41	.561	.272	.349	.369	1.93	.91	1.173	.346	.858	1.62	8.46
.42	.576	.276	.362	.380	2.01	.92	1.179	.342	.857	1.71	8.75
.43	.591	.280	.376	.391	2.10	.93	1.185	.339	.856	1.85	9.14
.44	.605	.284	.389	.403	2.18	.94	1.191	.336	.855	2.00	9.57
.45	.620	.288	.402	.415	2.27	.95	1.197	.332	.854	2.22	10.1
.46	.635	.292	.416	.427	2.35	.96	1.203	.324	.844	--------	--------
.47	.650	.296	429	.439	2.44	.97	1.209	.317	.835	--------	--------
.48	.665	.300	.443	.451	2.53	.98	1.215	.310	.826	--------	--------
.49	.680	.304	.457	.463	2.63	.99	1.220	.303	.818	--------	--------
.50	.695	.308	.471	.476	2.72	1.00	1.226	.297	.810	--------	--------

[1] d=depth of water, in feet; D=rise, in feet.

Table 4.—Coefficients for pipe-arches flowing partly full—Continued

d/D[1]	(1)	(2)	(3)	(4)	(5)	d/D[1]	(1)	(2)	(3)	(4)	(5)
	$A=C_aD^2$	$R=C_rD$	$K=C_k\frac{D^{8/3}}{n}$	$d_m=C_mD$	$Q=C_qD^{5/2}$		$A=C_aD^2$	$R=C_rD$	$K=C_k\frac{D^{8/3}}{n}$	$d_m=C_mD$	$Q=C_qD^{5/2}$
	C_a	C_r	C_k	C_m	C_q		C_a	C_r	C_k	C_m	C_q
E. Nominal sizes 16 feet 7 inches × 10 feet 1 inch and 15 feet 4 inches × 9 feet 3 inches, and all riveted pipe-arches											
0.01	0.013	0.031	0.002	0.031	0.013	0.51	0.747	0.318	0.517	0.494	2.98
.02	0.27	0.43	.005	0.43	.031	.52	.761	.321	.531	.506	3.07
.03	.040	.053	.008	.053	.052	.53	.775	.324	.544	.518	3.17
.04	.053	.060	.012	.060	.073	.54	.789	.327	.557	.531	3.26
.05	.066	.069	.017	.069	.098	.55	.804	.330	.571	.544	3.36
0.6	.080	.076	.021	.075	.124	.56	.819	.333	.585	.558	3.47
.07	.093	.082	.026	.082	.150	.57	.834	.336	.599	.573	3.58
.08	.106	.087	.031	.087	.177	.58	.849	.339	.613	.587	3.69
.09	.119	.093	.036	.093	.205	.59	.864	.342	.627	.602	3.80
.10	.131	.098	.042	.098	.234	.60	.879	.343	.642	.617	3.92
.11	.144	.105	.048	.105	.265	.61	.892	.343	.651	.632	4.03
.12	.157	.110	.054	.111	.296	.62	.905	.344	.659	.648	4.14
.13	.169	.116	.060	.117	.329	.63	.919	.344	.668	.663	4.25
.14	.182	.121	.066	.122	.362	.64	.932	.344	.677	.679	4.36
.15	.195	.126	.073	.128	.395	.65	.945	.345	.686	.696	4.47
.16	.209	.132	.081	.135	.437	.66	.959	.345	.701	.714	4.60
.17	.223	.139	.089	.143	.480	.67	.972	.349	.716	.732	4.72
.18	.238	.145	.097	.151	.523	.68	.985	.353	.731	.751	4.85
.19	.252	.150	.106	.158	.568	.69	.998	.357	.746	.771	4.97
.20	.266	.156	.114	.165	.614	.70	1.012	.361	.762	.791	5.11
.21	.280	.162	.124	.173	.661	.71	1.024	.362	.772	.811	5.23
.22	.294	.168	.133	.181	.709	.72	1.036	.362	.783	.832	5.37
.23	.308	.173	.142	.188	.758	.73	1.049	.363	.793	.854	5.50
.24	.322	.179	.152	.196	.808	.74	1.061	.364	.803	.876	5.64
.25	.335	.185	.162	.204	.859	.75	1.073	.365	.814	.899	5.77
.26	.351	.191	.173	.213	.919	.76	1.084	.364	.821	.925	5.91
.27	.366	.197	.184	.223	.981	.77	1.094	.364	.829	.952	6.06
.28	.382	.203	.196	.232	1.04	.78	1.105	.364	.837	.979	6.21
.29	.397	.209	.208	.242	1.11	.79	1.116	.364	.845	1.01	6.36
.30	.413	.215	.220	.251	1.17	.80	1.126	.363	.852	1.04	6.51
.31	.430	.222	.234	.262	1.25	.81	1.136	.362	.858	1.07	6.68
.32	.447	.229	.248	.273	1.33	.82	1.147	.361	.864	1.11	6.86
.33	.464	.235	.262	.284	1.40	.83	1.157	.361	.871	1.15	7.04
.34	.481	.241	.277	.296	1.48	.84	1.167	.360	.877	1.19	7.23
.35	.498	.247	.292	.307	1.57	.85	1.177	.359	.883	1.24	7.43
.36	.513	.252	.305	.318	1.64	.86	1.186	.357	.886	1.29	7.65
.37	.529	.257	.318	.328	1.72	.87	1.195	.355	.890	1.35	7.88
.38	.544	.262	.331	.339	1.80	.88	1.204	.353	.894	1.41	8.12
.39	.560	.267	.345	.350	1.88	.89	1.213	.352	.898	1.48	8.37
.40	.575	.272	.358	.361	1.96	.90	1.222	.350	.902	1.55	8.64
.41	.591	.277	.373	.372	2.05	.91	1.228	.346	.900	1.64	8.93
.42	.607	.282	.388	.384	2.14	.92	1.235	.343	.899	1.74	9.25
.43	.623	.286	.402	.396	2.23	.93	1.241	.340	.898	1.87	9.64
.44	.639	.291	.417	.408	2.32	.94	1.247	.336	.897	2.02	10.0
.45	.655	.296	.432	.420	2.41	.95	1.254	.333	.896	2.22	10.6
.46	.671	.300	.446	.432	2.50	.96	1.259	.326	.886	--------	--------
.47	.686	.304	.461	.444	2.59	.97	1.264	.319	.876	--------	--------
.48	.702	.308	.475	.456	2.69	.98	1.268	.312	.867	--------	--------
.49	.717	.311	.489	.469	2.79	.99	1.273	.306	.859	--------	--------
.50	.732	.315	.504	.481	2.88	1.00	1.280	.300	.848	--------	--------

[1] d=depth of water, in feet; D=rise, in feet.

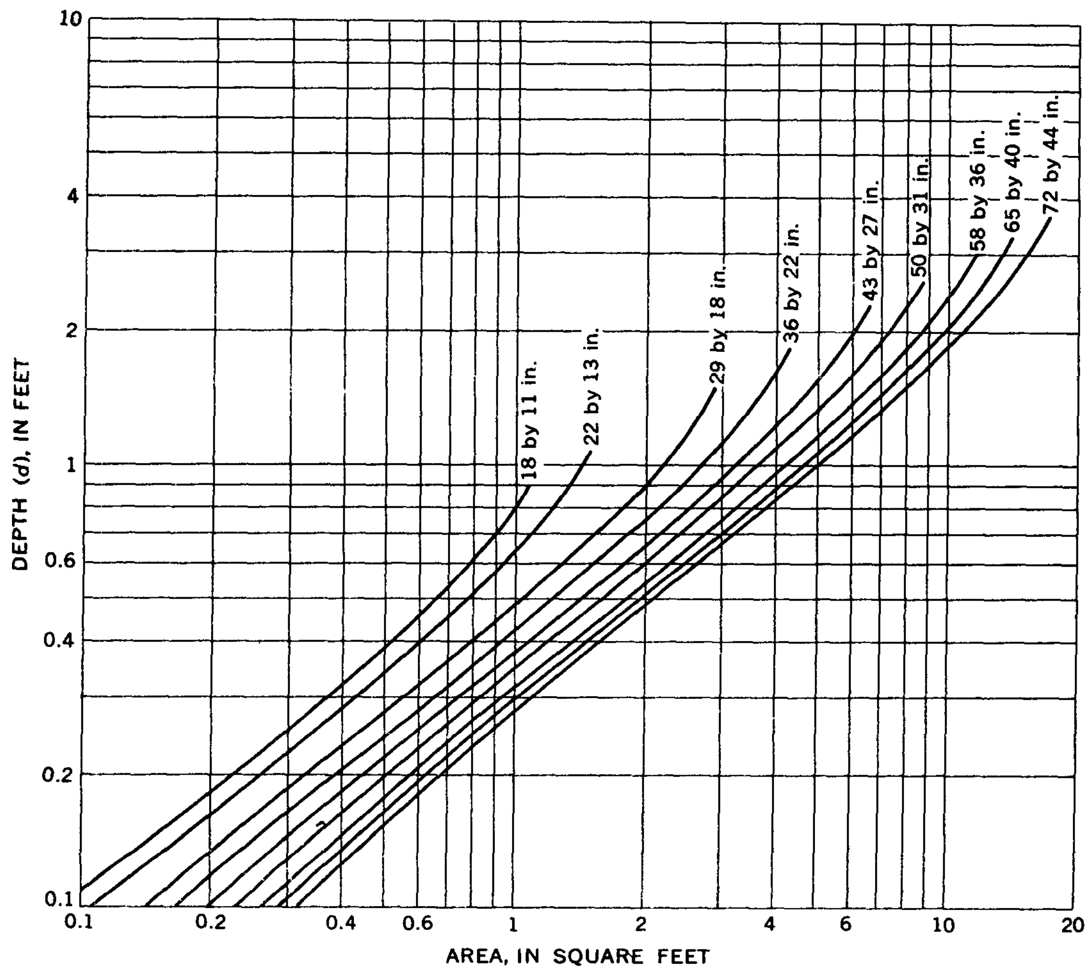


Figure 5.—Depth-area curves for riveted pipe-arches.

showing properties of certain size pipes, and pipe-arches. The purpose of these figures is to show the general shape these curves will follow as well as to show certain types of curves that may be of value for simplifying computations of odd-shaped culverts.

Flow at critical depth

At first glance, it may not be possible to tell whether type 1, 2, or 3 (a backwater condition) flow occurred. If the culvert is very steep and the getaway conditions are good, the flow will be type 1. For culverts set on zero grade with good getaway conditions and no backwater, type 2 flow is well assured. In both cases the type of flow must be proved. For fairly flat slopes and when backwater may be a factor, there is always the possibility of type 3 flow occurring. The following computational procedures will identify the type of flow.

Type 1 flow

The general procedure is to (1) assume that type 1 flow occurred, (2) compute the elevation of the water surface at critical depth and the critical slope, and (3) compare the critical slope with the bed slope, and the water-surface elevation at critical depth with the tailwater elevation. This will generally result in positive identification of types 1 or 3 flow or narrow the possible flow conditions to types 2 or 3.

If critical depth occurs at the inlet, the discharge may be computed with the applicable critical-depth equations, 1, 2, 3, 4, and the energy equation 5 as written between the approach section and the inlet.

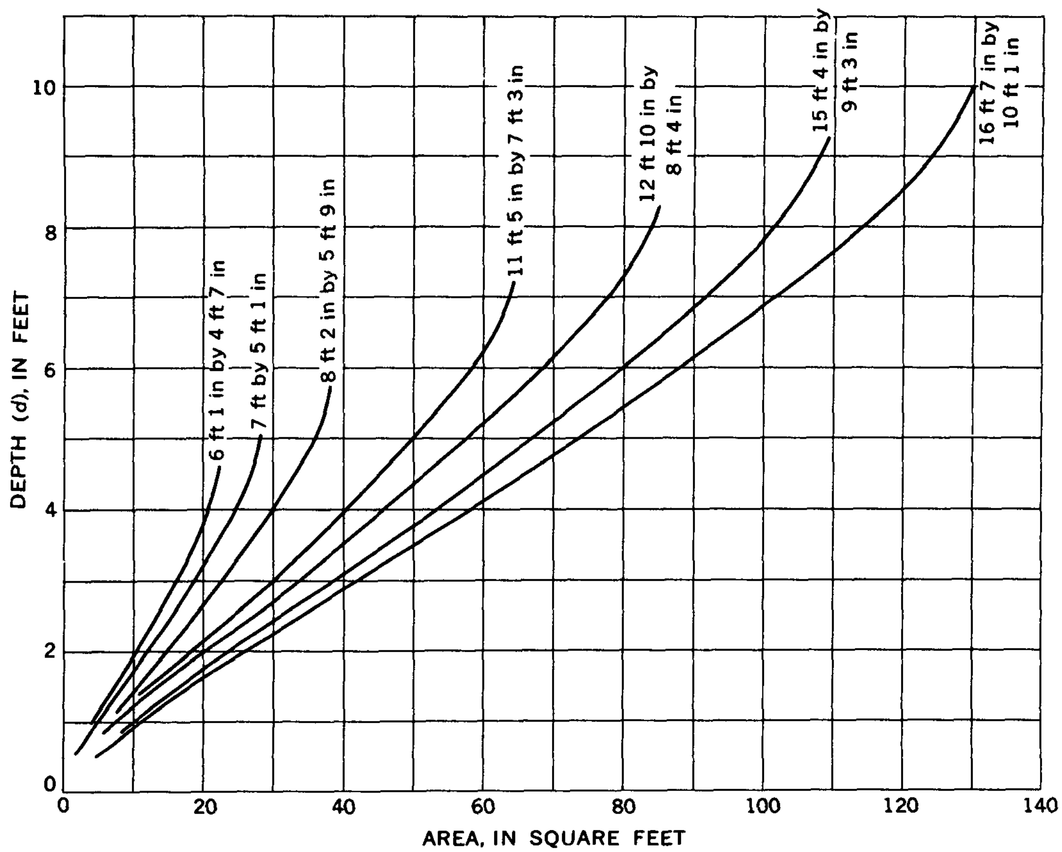


Figure 6.—Depth-area curves for multiplate pipe-arches.

In type 1 flow critical depth normally is assumed to occur at the inlet or upstream end of the culvert. However, a limited number of current-meter measurements have shown that, for mitered pipes on steep slopes, this assumption will show less water than is actually flowing through the culvert. For mitered pipes, assume that critical depth occurs at the point where the headwater elevation intersects the mitered entrance. For large culverts the difference in elevation between the inverts at each end of the miter may be several tenths of a foot. Computations of type 1 flow for these two invert elevations will show considerably different discharges.

Normally, a critical depth is assumed which fixes the value of the remaining unknown terms. A good first approximation is $d_c=0.66(h_1-z)$. Successive approximations of d_c will quickly converge toward the solution.

To check the assumption of type 1 flow, the critical slope for the culvert is computed as $S_c=(Q/K_c)^2$. Here S_c is the critical slope and K_c is the conveyance of the section of flow at critical depth at the inlet. If $S_c<S_0$ and $h_4<h_c$, the assumption of type 1 flow has been proved and the correct discharge has been computed. If $h_4>h_c$, type 3 flow has been identified and the discharge may be computed as outlined on page 30. If $h_4<h_c$ and $S_c>S_0$, type 2 flow is assumed and the analysis is continued as outlined on pages 25–30.

For circular, pipe-arch, and rectangular sections, the identification of flow type is simplified by figures 9, 10, and 11. The procedure is outlined below.

Circular sections

1. Compute $(h_1-z)/D$ and $(S_0D^{1/3})/n^2$ and plot the point on figure 9. A point to the right of the curve indicates type 1 flow and a point to the left, type 2 flow.
2. Determine d_c from figure 10.
3. Compare (d_c+z) with h_4.

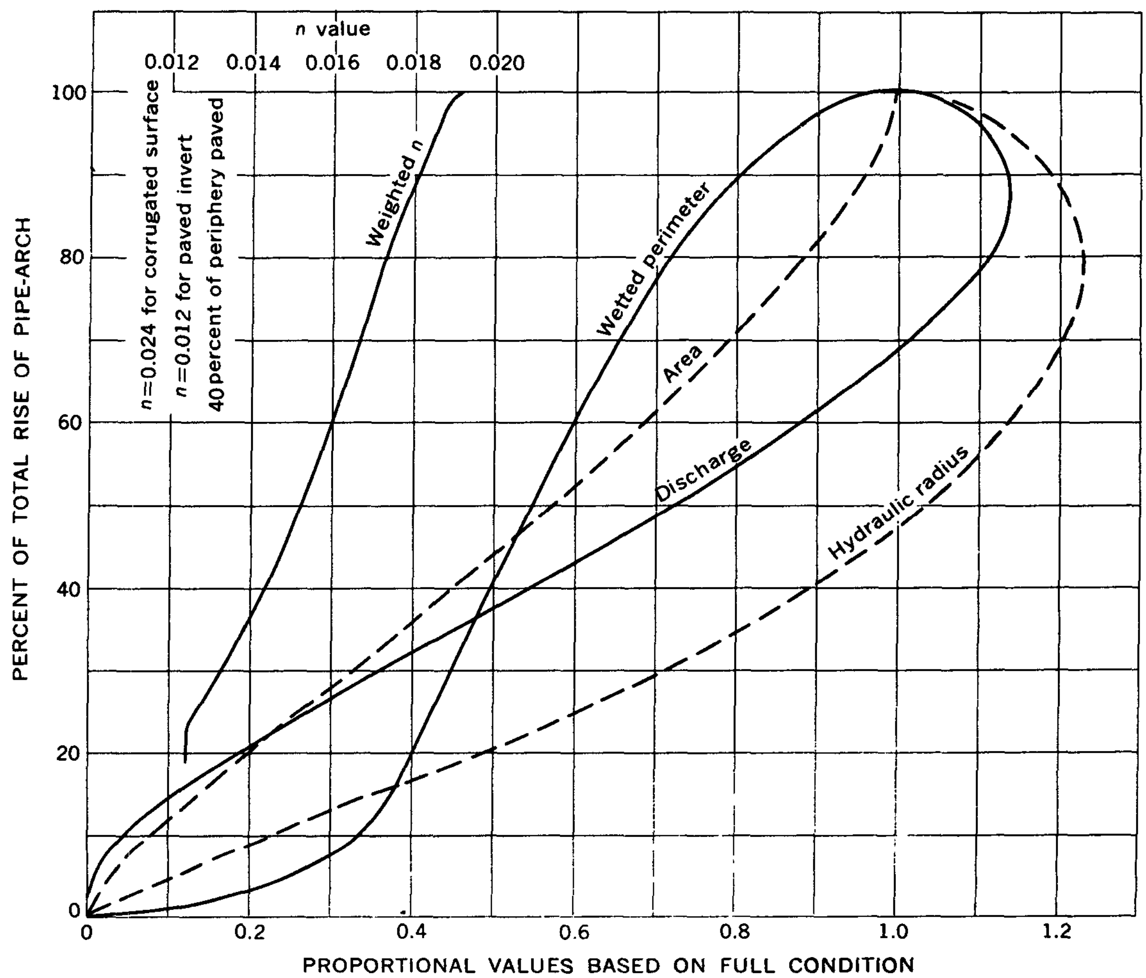


Figure 7.—Hydraulic properties of the paved-invert pipe-arch.

Pipe-arch data for full flow are as follows:

Span (in)	Rise (in)	Area (sq ft)	R (ft)
18	11	1.1	0.280
22	13	1.6	.340
25	16	2.2	.400
29	18	2.8	.446
36	22	4.4	.560
43	27	6.4	.679
50	31	8.7	.791
58	36	11.4	.891
65	40	14.3	1.01
72	44	17.6	1.12

4. Type 1 flow can occur only if the criterion of step 1 is met and $h_4 < (d_c + z)$. If type 1 flow is identified, compute the discharge with equations 4 and 5 as outlined below. Type of flow must always be proved after final computations are made.

Computation steps:

1. Compute C, the discharge coefficient.
2. Enter figure 10 with $(h_1 - z)/D$ and select value of d_c/D from the appropriate curve. Compute d_c.
3. With this value of d_c, enter equation 4 and compute Q.
4. Compute $\alpha_1 V_1^2/2g$, $h_{f_{1-2}}$, and A_c.

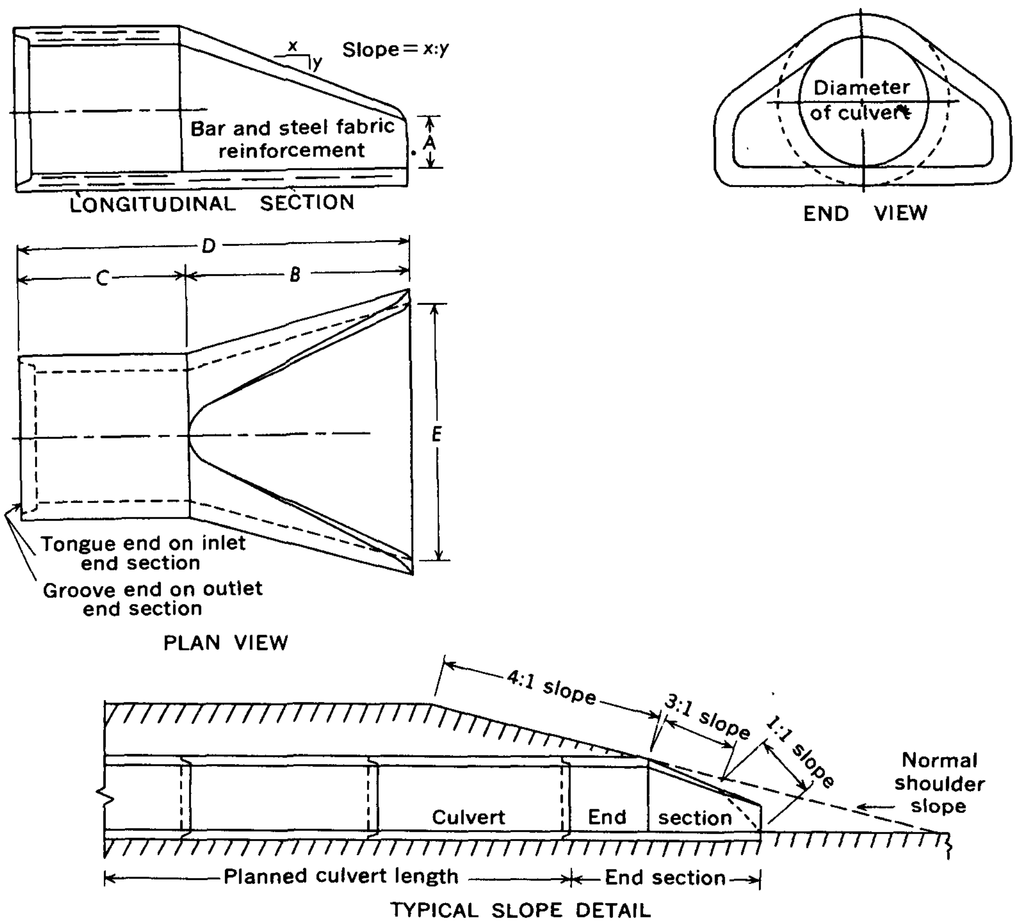


Figure 8.—Dimensions of flared end sections for reinforced concrete pipe.

Selected dimensions for various diameters of pipe

Diam (in)	A (ft	in)	B (ft	in)	C (ft	in)	D (ft	in)	E (ft	in)
12	0	4	2	0	4	0⅞	6	0⅞	2	0
15	0	6	2	3	3	10	6	1	2	6
18	0	9	2	3	3	10	6	1	3	0
21	0	9	3	0	3	1½	6	1½	3	6
24*	0	9½	3	7½	2	6	6	1½	4	0
27	0	10½	4	1½	2	0	6	1½	4	6
30*	1	0	4	6	1	7¾	6	1¾	5	0
36	1	3	5	3	2	10¾	8	1¾	6	0
42	1	9	5	3	2	11	8	2	6	6
48	2	0	6	0	2	2	8	2	7	0
54	2	3	5	5	2	9¼	8	2¼	7	6

*Overall length (D) of Iowa design is 8 ft 1½ in for 24 in and 8 ft 1¾ in for 30 in.

NOTE.—Slope 3:1 for all sizes except 54 in. which is 2.4:1.

5. Compute Q from equation 5. Generally this computed Q will closely check the assumed Q from step 3. If it does not, repeat steps 2–5, using $[h_1+\alpha_1 V_1^2/2g-h_{f_{1-2}}]$ for $[h_1]$ in step 2.

When the two discharges check within 1 percent, the final result may be considered satisfactory.

Pipe-arch sections

Type 1 flow in a riveted pipe-arch is computed in exactly the same steps as for a circular section by using the pipe-arch data in figures 9 and 10. Not all multiplate pipe-arches are geometrically similar to riveted pipe-arches or to each other. Therefore the curves of figures 9 and 10 will provide less accurate values.

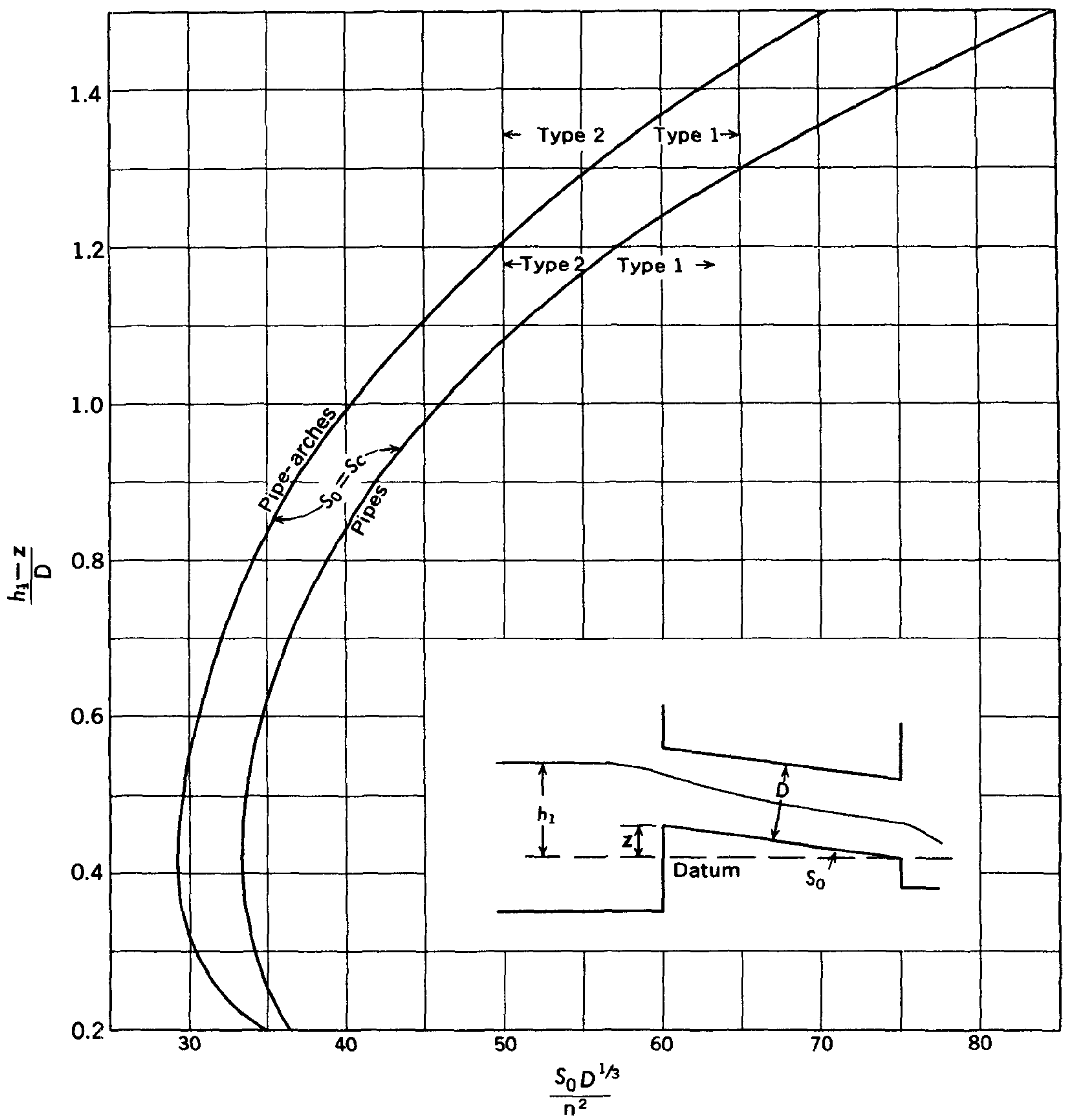


Figure 9.—Critical slope as a function of head for pipe and pipe-arch culverts, with free outfall.

However, because the values from figures 9 and 10 are only approximations, they are satisfactory to use for multiplate pipe-arch flow computation.

Rectangular sections

1. Compute the factors for the ordinate and abscissa of figure 11 for the culvert, assuming $d_c = 0.66(h_1 - z)$, and plot the point. A point to the right of the line indicates type 1 flow, and a point to the left indicates type 2 flow.
2. Compare $(d_c + z)$ with h_4.
3. Type 1 flow can occur only if the criterion of step 1 is met and $h_4 < (d_c + z)$. If type 1 flow is identified, compute the discharge with equations 3 and 5 as outlined below.

Computation steps:

1. Compute C.
2. Determine d_c factor from table on page 25.
3. Assume $d_c = d_c$ factor times $(h_1 - z)$.
4. With value of d_c from step 3, enter equation 3 and compute Q.
5. Compute $\alpha_1 V_1^2/2g$, $h_{f_{1-2}}$, and A_c.

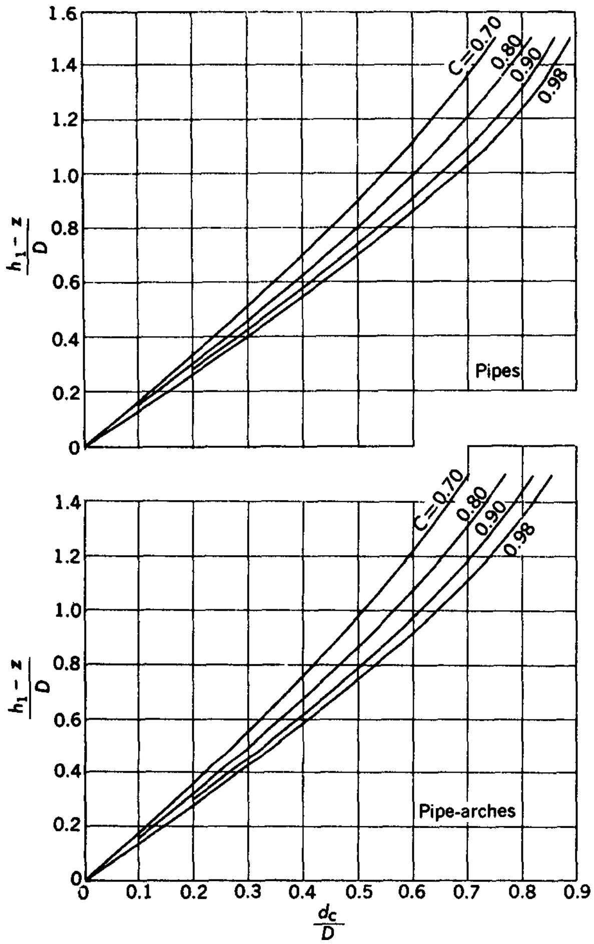


Figure 10.—Relation between head and critical depth in pipe and pipe-arch culverts.

6. Compute Q from equation 5. Generally this computed Q will closely check the assumed Q from step 4. If it does not, repeat steps 2–6, using

   $[h_1+\alpha_1 V_1^2/2g-h_{f_{1-2}}]$ for $[h_1]$ in step 3.

Computation of type 1 flow with ponded headwater for rectangular box culverts set flush in a vertical headwall is simplified by the fact that C is limited to values from 0.95 to 0.98. The factor 0.66 in the formula $d_c=0.66$ (h_1-z) can be refined to give a final result with one computation. The following table gives factors for various values of C:

C	d_c *factor*
0.98	0.658
.97	.653
.96	.648
.95	.643

Irregular sections

Arches and all other culverts that have irregularly shaped bottoms or tops (including rectangular shapes with fillets but excluding pipe-arches) are considered in this category. The same general procedure is used in computing discharge for irregular shapes, except that equation 1 or 2 must be used with equation 5 to obtain the unique solution. For rectangular culverts with fillets, a variation of equation 3, in the form of $Q=5.67\,Td_m^{3/2}$, may be used.

If a number of discharge computations will be made for a given irregularly shaped culvert, prepare graphs of area, wetted perimeter, and conveyance.

Type 2 flow

If type 2 flow is correctly assumed, the critical depth should occur at the outlet. The flow equations used for the computation of type 1 flow are also applicable here, with the further provision that the barrel friction loss must be accounted for in the energy equation since the control section has shifted to the outlet.

The discharge and the critical depth must be computed by solution of equation 7 and the applicable critical-depth equation 1, 2, 3, or 4. The solution is tedious, because to compute the barrel friction loss, $h_{f_{2-3}}$, the height of the water surface at the inlet must be established.

The complete equation for determining the depth d_2 at the inlet is

$$h_1-z=d_2+\frac{V_2^2}{2g}+\left(\frac{1}{C^2}-1\right)\frac{V_3^2}{2g}+h_{f_{1-2}}-\frac{\alpha_1 V_1^2}{2g}. \tag{13}$$

Even though entrance losses actually are a function of the terminal velocity rather than of the entrance velocity, the above equation may be simplified for most computations by assuming $V_3=V_2$. Under average conditions this is a fair approximation. Also, as a general rule, where ponded conditions exist above the culvert, or the approach velocity head and the approach friction loss are compensating, these factors may be neglected. Thus, the equation is simplified to

$$h_1-z=d_2+\frac{V_2^2}{2gC^2}. \tag{14}$$

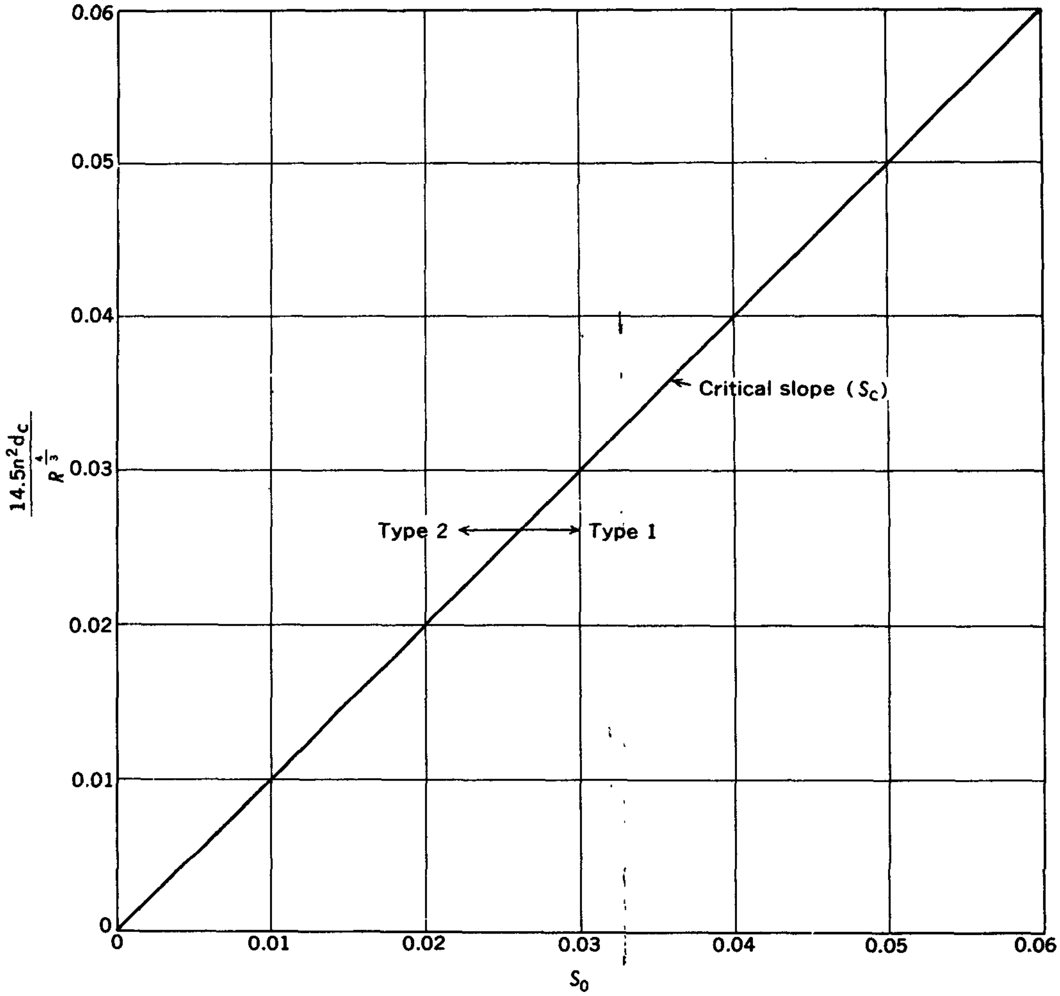


Figure 11.—*Critical slope for culverts of rectangular section, with free outfall.*

Figures 12 and 13 are graphical solutions of equation 14 for circular and riveted pipe-arch sections. Figure 14 is the solution for square or rectangular sections. These curves are based on ponded approach conditions and the assumption that $V_3=V_2$. Similar curves may be constructed for any given culvert shape, but the development is so tedious that it generally is not worthwhile.

In the event that approach velocity head and friction loss cannot be neglected, the (h_1-z) term should be increased by their algebraic sum.

In long rough culverts V_3 is much larger than V_2; therefore,

$$\frac{V_2^2}{2gC^2}<<\frac{V_2^2}{2g}+\left(\frac{1}{C^2}-1\right)\frac{V_3^2}{2g}. \qquad (15)$$

The smaller the discharge coefficient, the greater will be the difference in d_2. The d_2 computed from figures 12, 13, and 14 may be greatly in error when this condition exists. The proper way to determine d_2 in this situation is by routing. The routing method is described on page 31.

The procedure outlined below is recommended for computation of discharge.

Circular sections

Computation steps:

1. Compute C and enter figure 10 with $(h_1-z)/D$. Select the value of d_c/D from the appropriate curve.
2. Use 95 percent of d_c/D as a first approximation to compute d_c. The 95 percent is

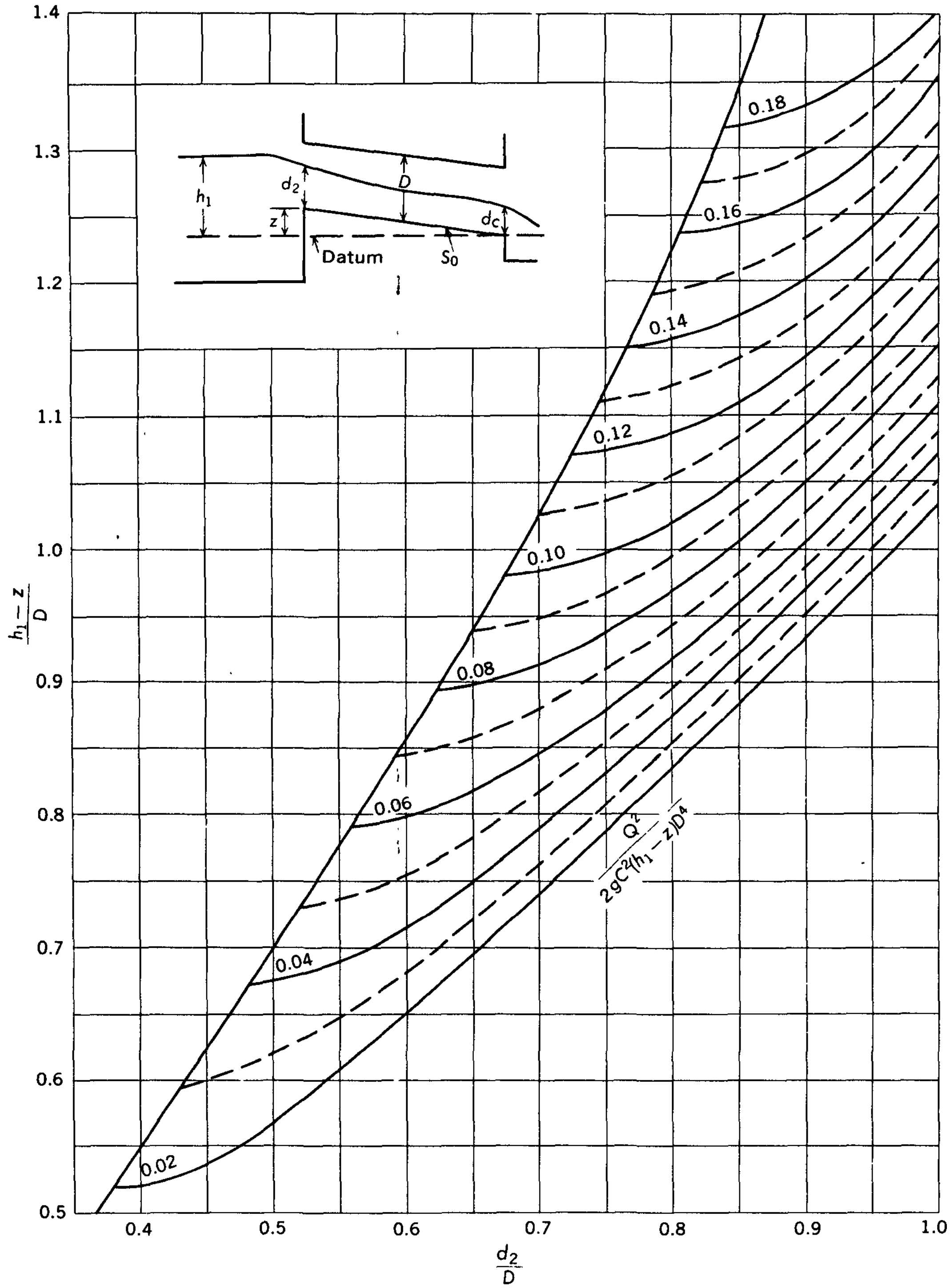


Figure 12.—Relation between head and depth of water at inlet with critical depth at outlet for culverts of circular section.

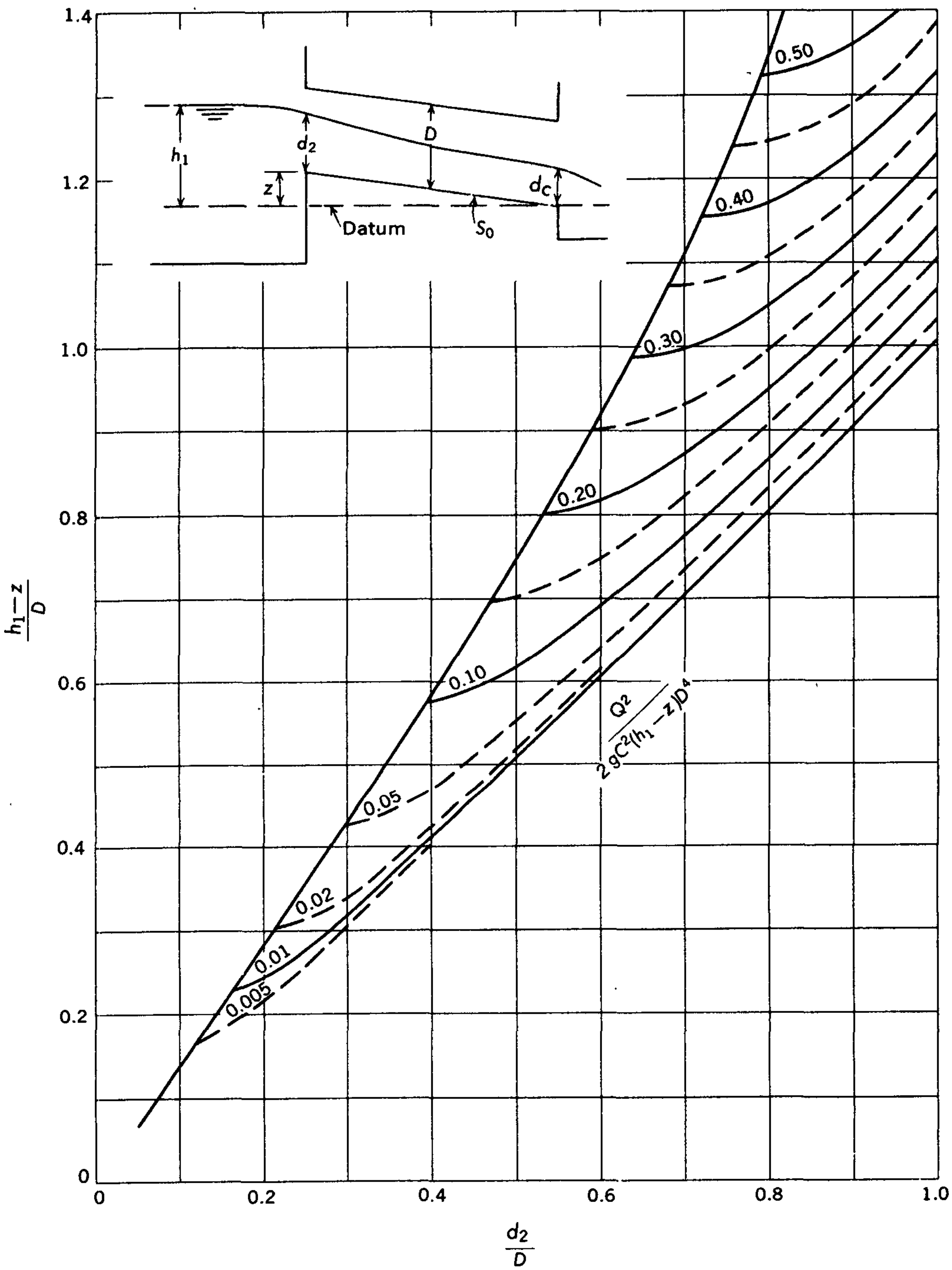


Figure 13.—Relation between head and depth of water at inlet with critical depth at outlet for pipe-arch culverts.

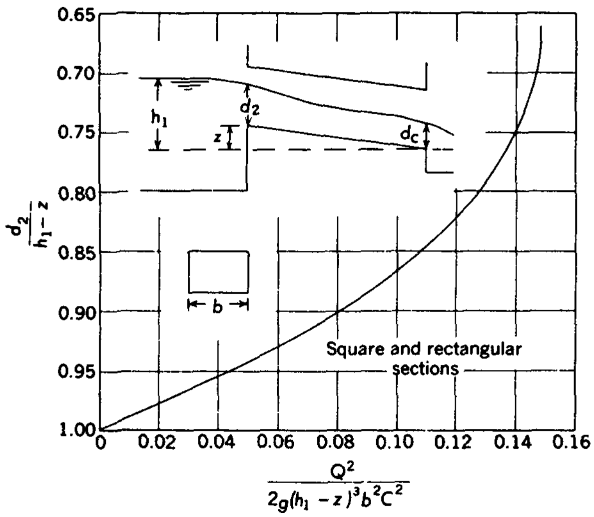


Figure 14.—Relation between head and depth of water at inlet with critical depth at outlet for culverts of rectangular section.

used to compensate for the friction loss in the culvert.

3. With this first approximation of d_c, use equation 4 to obtain a trial value of Q.
4. Compute $Q^2/2gC^2(h_1-z)(D^4)$ and enter figure 12 with $(h_1-z)/D$ to obtain d_2/D. Compute d_2.
5. Having trial values of Q, d_2, and d_3 (which is d_c), compute

   (a) $\alpha_1 V_1^2/2g$

   (b) $h_{f_{1-2}}$

   (c) $h_{f_{2-3}}$.
6. Compute $H=h_1+\alpha_1V_1^2/2g-h_{f_{1-2}}-h_{f_{2-3}}$.
7. Use value from step 6 as numerator in ratio H/D. Use this ratio as ordinate in figure 10 to read d_c/D.
8. From step 7, compute d_c.
9. Using value of d_c from step 8 in equation 4, compute Q.
10. Compute $Q^2/2gC^2(h_1-z)(D^4)$, using Q from step 9.
11. Use the value from step 10 in figure 12 to obtain d_2/D and compute d_2.
12. Compute the velocity head and friction with latest values of Q, d_c, and d_2. Also compute A_c and K_c.
13. Compute Q from equation 7. The computed Q should closely check the assumed Q of step 9.
14. If the discharge computed with equation 7 is not within 1 percent of the discharge computed in step 9, the assumed value of d_c is incorrect. The correct value of d_c must be determined by successive approximation, repeating the procedure outlined above.
15. After the discharge and the elevation of the water surface at critical depth are established, the assumption of type 2 flow is checked by comparing the elevation of critical depth with the tailwater elevation.

If $h_c>h_4$, type 2 flow occurred. If $h_c<h_4$, type 3 flow occurred, and the discharge may be computed as outlined in the next section.

Pipe-arch sections

Type 2 flow in a riveted pipe-arch is computed in exactly the same manner as for a circular section by using the pipe-arch data in figures 10 and 13. These curves will give approximate values applicable to multiplate pipe-arch computations.

Rectangular sections

Computation steps:

1. Compute C.
2. Determine d_c factor from the table on page 25.
3. Assume d_c (which is d_3)$=d_c$ factor times (h_1-z). Note.—The d_c factor from page 25 may be reduced 0.03 to approximate the friction loss in the culvert.
4. Compute Q from equation 3.
5. Compute $Q^2/2g(h_1-z)^3b^2C^2$ and enter figure 14. Obtain a value of $d_2/(h_1-z)$ and compute d_2.
6. Having trial values of Q, d_2, and d_3 (which is d_c) compute

   (a) $\alpha_1V_1^2/2g$

   (b) $h_{f_{1-2}}$

   (c) $h_{f_{2-3}}$.
7. Compute $H=h_1+\alpha_1V_1^2/2g-h_{f_{1-2}}-h_{f_{2-3}}$.
8. Assume $d_c=d_c$ factor times value from step 7.
9. Using value of d_c from step 8 in equation 3, compute Q.

10. Compute $Q^2/2g(h_1-z)^3b^2C^2$ and enter figure 14. Obtain a value of $d_2/(h_1-z)$ and compute d_2.
11. Compute $\alpha_1 V_1^2/2g$, $h_{f_{1-2}}$, $h_{f_{2-3}}$ from latest values of Q, d_2 and d_c.
12. Compute Q from equation 7. The computed Q should closely check the assumed Q of step 9.
13. If the discharge computed with equation 7 is not equal to the discharge computed in step 9, the procedure given above must be repeated until agreement within 1 percent is reached.
14. The assumption of type 2 flow is checked as for circular sections.

Irregular sections

The same general rules apply to type 2 flow through irregular sections as apply to type 1 flow. Special consideration should be given to roughness coefficients, slope of culvert, and change in shape of culvert cross section. However, the general equations 1 and 2 for critical-depth flow may be used with equation 7 for computing discharge.

Equation 13 should be used to determine depth of water d_2 at the inlet because many irregularly shaped culverts have rough barrels. In these computations, d_2 must be measured in a manner similar to the measurement of d_c, either as the depth to the average bottom or the depth to the lowest point in the section. The same criteria must be used in measuring h_1 and z.

For certain conditions it may be necessary to use the routing method to compute discharge.

Flow with backwater

In flow with backwater, types 3 and 4, critical depth does not occur in the culvert, and the upstream elevation of the water surface for a given discharge is a function of the surface elevation of the tailwater.

Type 3 flow

Water-surface elevations h_1 and h_4 can normally be established from highwater marks, and it is assumed that h_3 equals h_4. The following procedure is recommended in computing discharge:

1. Assume a discharge. A fair approximation is $0.95A_3\sqrt{2g(h_1-h_4)}$.
2. Determine the depth at the inlet d_2 by trial solution of equation 13 or directly from figures 12, 13, or 14 if the culvert has a circular, pipe-arch, or rectangular section.
3. Compute the conveyance of the sections at the approach, the inlet, and the outlet.
4. Compute the friction loss between the approach and the inlet,

$$h_{f_{1-2}}=L_w(Q^2/K_1K_2),$$

and between the inlet and the outlet

$$h_{f_{2-3}}=L(Q^2/K_2K_3).$$

5. Compute the approach velocity head, $\alpha_1 V_1^2/2g$.
6. Compute the discharge with equation 8.
7. If the discharge computed with equation 8 is not equal to the assumed discharge, then another discharge should be assumed and the procedure outlined above repeated.

Type 4 flow

Generally for type 4 flow, ponded conditions exist. If water is not ponded, h_1 should be adjusted for velocity head in the approach section and friction loss between the approach section and the inlet.

Discharge is computed directly from equation 9, where A_0 and R_0 are the area and hydraulic radius, respectively, for a full culvert.

A constant can be determined for any given culvert, so the discharge can be computed simply by multiplying the constant, which equals

$$CA_0\sqrt{\frac{2g}{1+\frac{29C^2n^2L}{R_0^{4/3}}}}$$

by the square root of the difference between headwater and tailwater elevations, $\sqrt{h_1-h_4}$. Note that type 4 flow is independent of the culvert slope.

Flow under high head

Type 5 or 6 flow will occur if the tailwater is below the crown at the outlet, and $(h_1-z)/D$ is equal to or greater than 1.5. Approach velocity head and friction loss are included in the computations when appropriate.

The type of flow is dependent largely on the amount of beveling or rounding of the culvert entrance. For this and other reasons previously mentioned, the criteria for identifying type 5 or type 6 flow must be considered approximate. Generally there is a transition from low-head to high-head flow that must be considered. This item is discussed under "Ratings with Transition between Flow Types," on page 47.

Concrete culverts

Figure 15 may be used to classify type 5 or 6 flow in concrete culvert barrels by the procedure outlined below.

1. Compute the ratios L/D, r/D, or w/D, and S_0.
2. Select the curve of figure 15 corresponding to r/D or w/D for the culvert. Sketch in an interpolated curve for the given r/D or w/D, if necessary.
3. Plot the point defined by S_0 and L/D for the culvert.
4. If the point lies to the right of the curve selected in step 2, the flow was type 6; if the point lies to the left, the flow was type 5.

The use of the figure 15 is restricted to square, rounded, or beveled entrances, either with or without wingwalls. Wingwalls do not affect the flow classification, as the rounding effect they provide is offset by a tendency to produce vortexes that supply air to the culvert entrance. For culverts with wingwalls, use the geometry of only the top side of the entrance in computing the effective radius of rounding, r, or the effective bevel, w, in using figure 15.

Corrugated-pipe culverts

Figure 16 may be used to classify type 5 or 6 flow in rough pipes, both circular and pipe-arch sections, mounted flush with a vertical headwall, either with or without wingwalls, as outlined below. Figure 16*A* should be used in classifying the flow if the pipe projects from a headwall or embankment.

1. Determine the ratio r/D for the pipe.
2. From figure 16, select the graph corresponding to the value of r/D for the culvert.
3. Compute the ratio $29n^2(h_1-z)/R_0^{4/3}$ and select the corresponding curve on the graph selected in step 2. Sketch in an interpolated curve for the computed ratio, if necessary.
4. Plot the point defined by S_0 and L/D for the culvert.
5. If the point plots to the right of the curve selected in step 3, the flow was type 6; if the point plots to the left of the curve, the flow was type 5.

Type 5 flow

In type 5 flow the culvert entrance is submerged, and the tailwater is below the crown at the outlet. The flow is rapid near the entrance to the culvert. The discharge may be computed directly from equation 10.

Type 6 flow

In type 6 flow the water surface is assumed to be at the top of the culvert at the outlet, but the culvert is not submerged and free outfall prevails. The following procedure may be used to compute discharge:

1. Compute the ratio h_1/D. Select the discharge coefficient, C, applicable to the culvert geometry.
2. From figure 17, determine the value of $Q/A_0\sqrt{D}$ corresponding to $29n^2L/R_0^{4/3}=1$.
3. Compute the ratio $29n^2L/R_0^{4/3}$ for the culvert under study.
4. From figure 17, using the computed ratio $29n^2L/R_0^{4/3}$ and the coefficient C, find the correction factor, k_f.
5. Multiply the value of $Q/A_0\sqrt{D}$ from step 2 by the value of k_f from step 4, thus determining an adjusted ratio $Q/A_0\sqrt{D}$.
6. Determine the value of Q from the adjusted ratio.

Routing method

The previously described computation procedures cover the standard conditions found in a great majority of culvert installations. The methods for computation of types 1–4 flow are inappropriate for some nonstandard conditions. Some examples of such conditions are:

1. Approach velocity head or friction loss of appreciable amount.

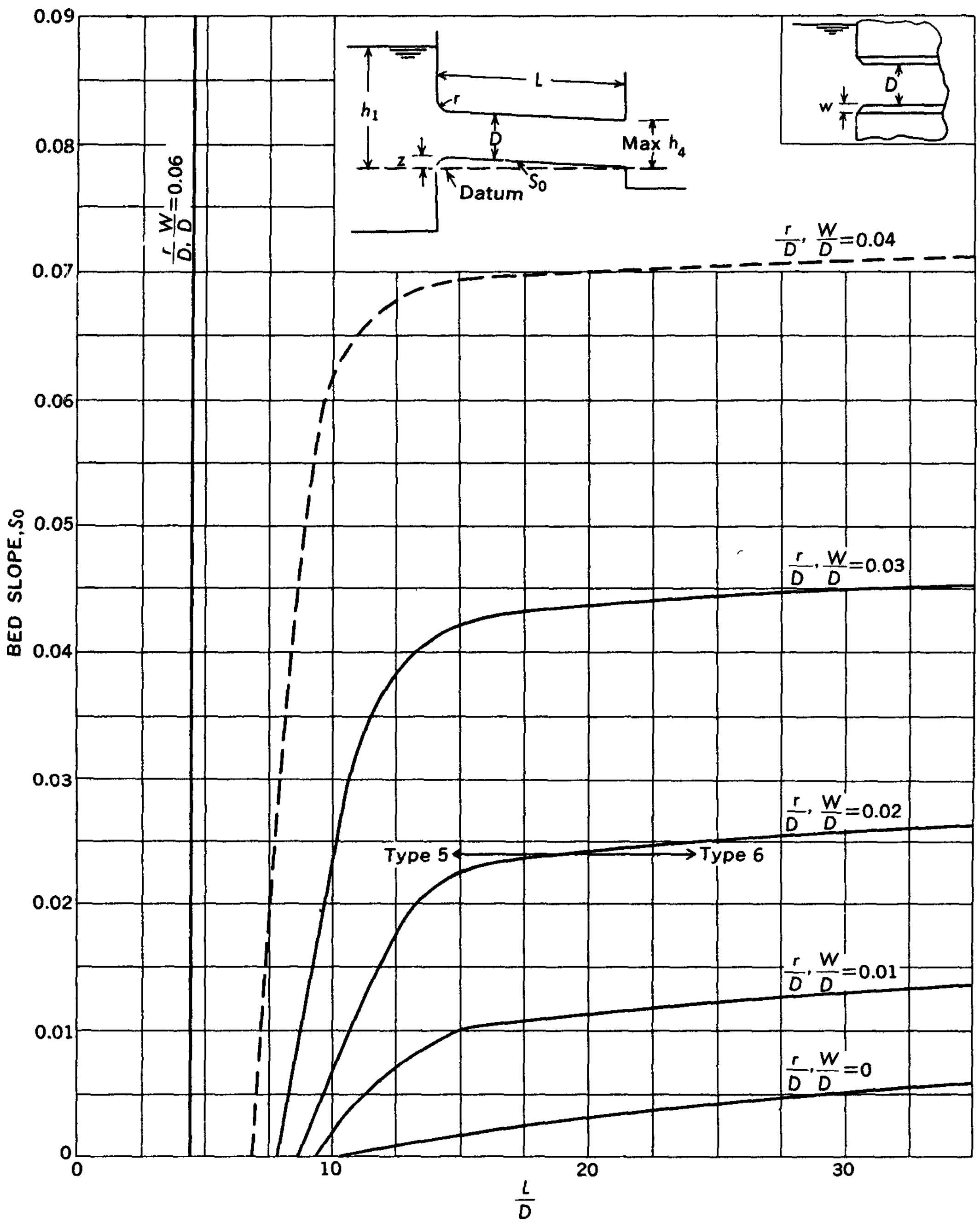


Figure 15.—Criterion for classifying types 5 and 6 flow in box or pipe culverts with concrete barrels and square, rounded, or beveled entrances, either with or without wingwalls.

2. Variation in cross-sectional dimensions through the culvert barrel.
3. Nonuniform slope, break in slope along the culvert barrel, or severe adverse slope.

The standard equations, 5, 7–11, may all be derived by writing the Bernoulli (energy) equation between the control section and the approach section. The routing procedure is a method whereby the energy equation is solved by the following procedures:

1. For flow types 1 and 2 (flow at critical depth), assume a critical depth of flow at the control section, compute the corresponding discharge, then route the energy

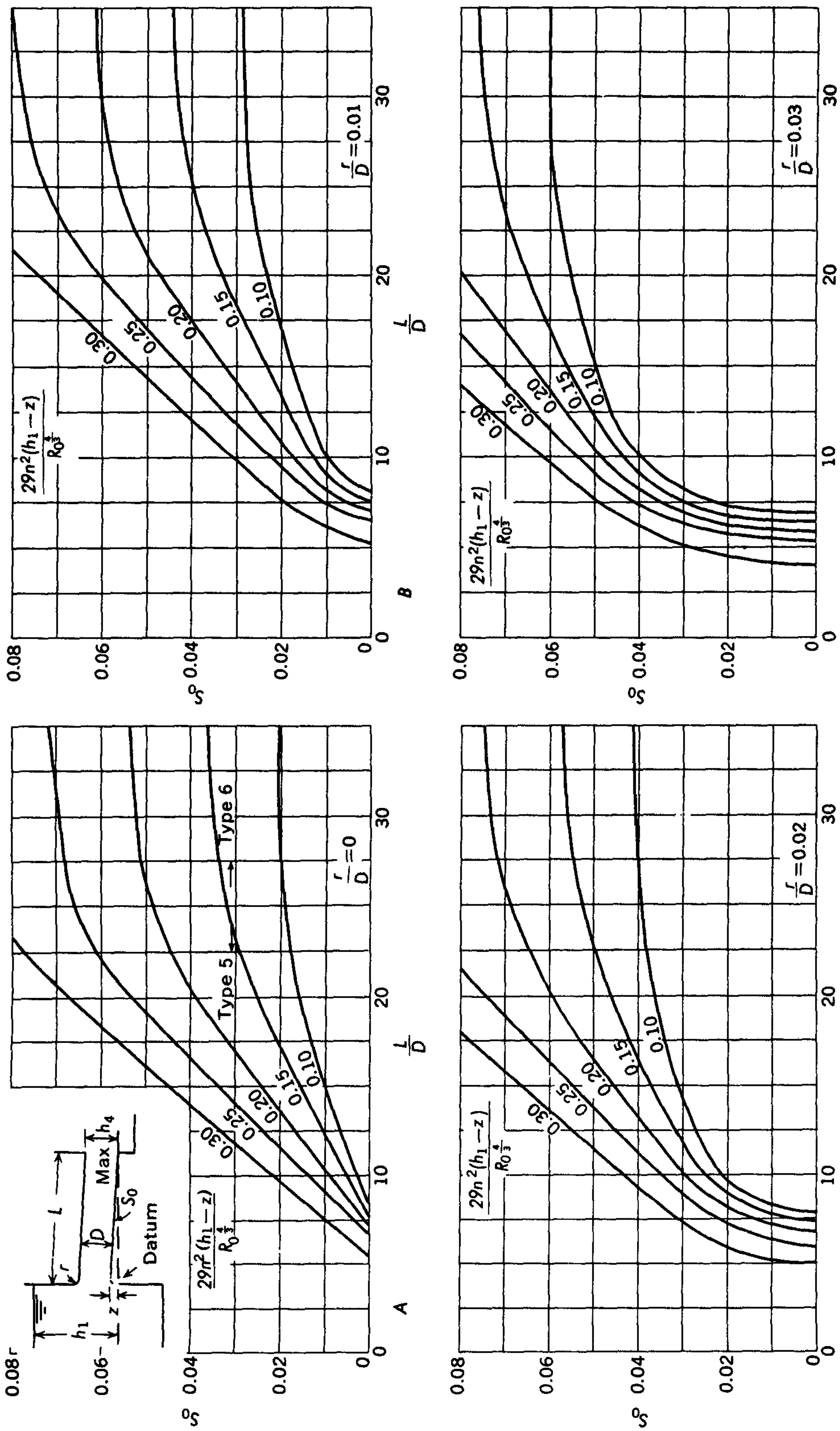


Figure 16.—Criterion for classifying types 5 and 6 flow in pipe culverts with rough barrels.

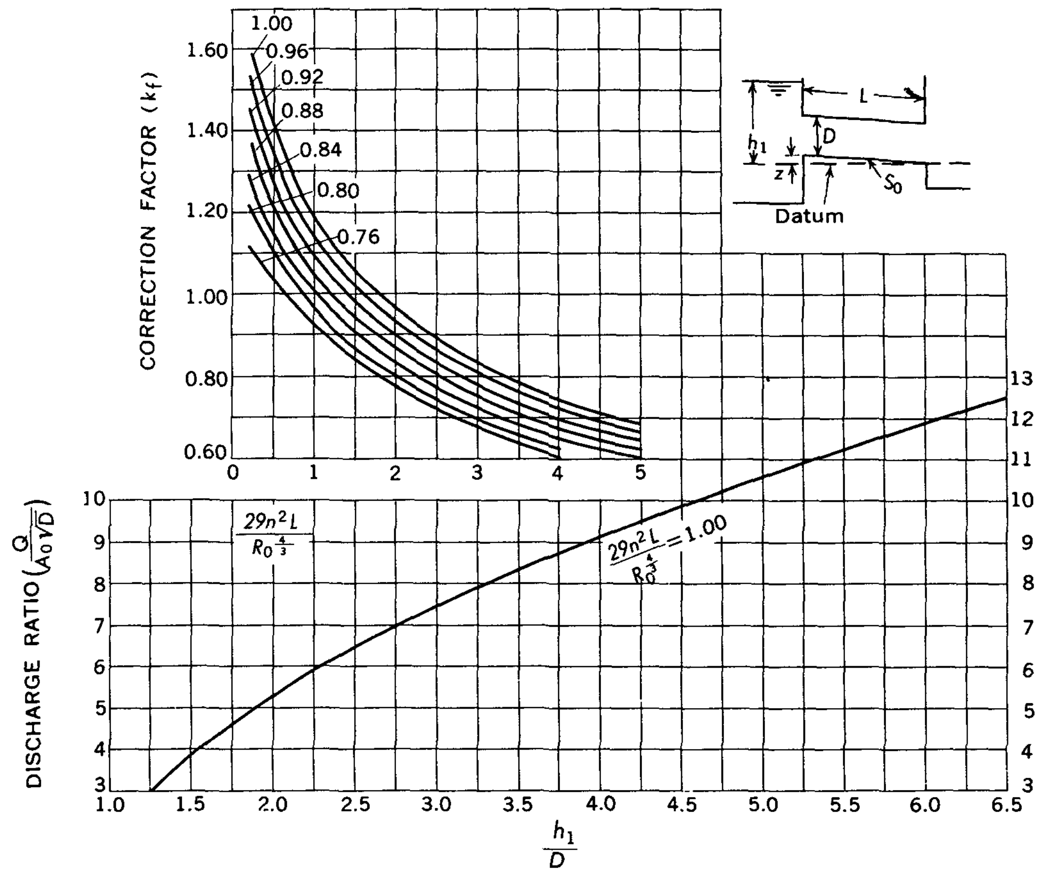


Figure 17.—Relation between head and discharge for type 6 flow.

gradient upstream to the headwater elevation using the computed discharge. If the computed and known elevation there do not agree, assume another critical depth and repeat the procedure.

2. For types 3 and 4 flow (flow with backwater), the discharge is assumed. Then starting with the known tailwater elevation, the energy gradient is routed upstream to the headwater elevation. If the computed headwater elevation differs from that known, repeat the computations using other assumed discharges until agreement is reached.
3. For type 5 flow (flow under high head), the controlling feature is the entrance geometry. The routing procedure cannot be applied here.
4. For type 6 flow (flow under high head), the same procedure is followed as for type 3, except that the starting point for the piezometric head (h_3) must be estimated in some manner. In type 6 flow, h_3 is not measured to the water surface.

(1) For box culverts the line of piezometric head at the outlet may be considered to lie slightly below the centerline for high Froude numbers, gradually increasing to a level about halfway between centerline and top of barrel for a Froude number approaching unity. An average h_3 of $0.65D$ may be used for the range of Froude numbers ordinarily encountered in culvert flow in the field.

(2) For pipe culverts of circular section it is known that the piezometric head usually lies between $0.5D$ and $1.0D$. An average h_3 of $0.75D$ may be used for the range of Froude numbers ordinarily encountered under field conditions. A more exact value, based

on recent experiments with prototype size pipes, may be obtained by use of figure 18.

The routing method can be applied to both standard and nonstandard conditions. Determine type of flow and discharge coefficients from the curves and tables in this manual. Consider the velocity heads at starting and ending points. Compute friction losses separately between points where the bottom slope changes or where contraction occurs. Between any two points 1 and 2, the friction loss is equal to LQ^2/K_1K_2.

Compute entrance loss as $(1/C^2-1)V_3^2/2g$ where C is the discharge coefficient as determined in this chapter and $V_3^2/2g$ is the velocity head at the control or terminal section. Actually the entrance loss occurs through the contracting and expanding portion of the live stream, but it is related to the velocity head at the control or terminal section because of the methods used in determining the discharge coefficients listed in this chapter.

The routing method is performed as follows:

1. Elevation of water surface at the control section.
2. *Plus* velocity head at control section.
3. *Plus* friction loss between control section and entrance.
4. *Plus* entrance loss.
5. *Plus* friction loss between entrance and approach section.
6. *Minus* velocity head at approach section.
7. *Equals* computed headwater elevation.

Compare this computed headwater elevation with the known water-surface elevation and, if different, make successive trials until they reach agreement within 0.02 foot.

For types 2 and 3 flow an intermediate assumption of depth at the culvert entrance is made. Test the assumption by routing to the energy gradient at the entrance (steps 1–3 above), then deducting the velocity head at the entrance. Always verify the type of flow. Remember that it is impossible to route through critical depth.

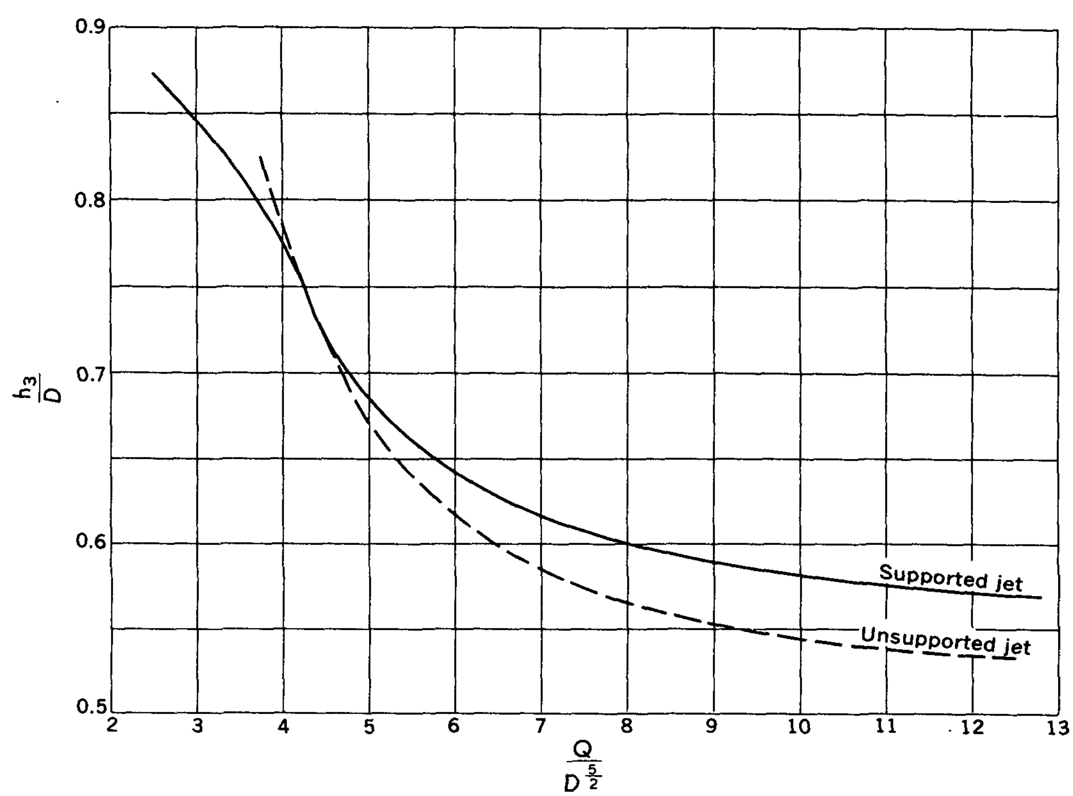


Figure 18.—Relation between outlet pressure lines and discharge for type 6 flow through culverts of circular section. Taken from report by J. L. French, 1956.

Calculations showing the standard and routing methods for various types of flow conditions are given under "Examples."

Unusual conditions

There will be, of course, occasional culvert-flow problems of unusual complexity, examples of which have not been covered in detail in this chapter. In most of the problems, results may be computed by reverting to the basic fundamentals of the subject or to literature covering the special conditions. Undoubtedly there will be occasional problems where no reliable solution is possible.

One exception to the general flow classification is that type 1 flow can occur with h_4 slightly greater than h_c, or with h_4 greater than D. In this case, type 1 flow is proved by trial computation of a backwater curve that extends from the known tailwater surface to the upstream end of the culvert. If the computed water-surface elevation is found to be below d_c, the type 1 flow existed.

Another rare type of part-full flow may sometimes occur where the headwater is below $1.5D$ at the entrance, but the tailwater is above the crown at the outlet.

Type 3 flow can occur with any bottom slope. In a culvert with a steep slope it might occur with $h_4/D>1.0$.

The fact that both $(h_1-z)/D$ and h_4/D are greater than 1.0 does not positively indicate type 4 flow. (h_1-z) must exceed D by an amount equal to $V_0^2/2gC^2$, where V_0 is full culvert velocity, or the pipe will not be full at the upstream end.

A culvert has to be short (L=about 10 diameters) to allow type 5 flow on a mild slope.

In order for type 6 flow to occur on a steep slope, the culvert must have a large r/D ratio, perhaps as great as 0.06.

Examples of unusual conditions that might be experienced are (1) culverts of nonuniform barrel geometry, (2) submerged culverts with flared outlets, (3) culverts with drop inlets, (4) cases where flow is rapid in the approach section, and (5) culverts flowing full part way. Examples 2–4 cannot be satisfactorily computed.

Other conditions which are not so unusual, but which must be given special consideration, are (1) partly buried culverts, (2) culverts in an aggraded section of a stream, (3) culverts partly plugged with debris, and (4) culverts partly plugged with ice.

For a culvert flowing full part way, the length of part-full flow at the downstream end may be determined by writing two equations, each of which will determine the point at which full flow ceases, and solving them simultaneously. Elevation of the culvert crown at the outlet plus the rise in the culvert crown (usually the same as the bottom) in the unknown length must equal the water-surface elevation at the outlet plus the friction loss in the unknown length plus the change in velocity head in the unknown length. The equation is

$$D+S_0x=d_3+Q^2x/K_0K_3+V_3^2/2g-V_0^2/2g, \quad (16)$$

in which x is the length of part-full flow.

Multiple culverts

A multiple-culvert installation is one in which the culvert barrels are separated by more than 0.1 the width or diameter of either barrel. This should not be confused with a multibarrel culvert which generally consists of two or more barrels, separated by thin webs, in a single structure.

Two or more culverts may be used as the drainage structure. In many places the culverts will be (1) made of different materials, (2) laid at different slopes, and (3) installed with different invert elevations. A common occurrence is for different flow types to occur. For example, a small culvert may be flowing under high head while a larger one is flowing as type 3.

At multiple culverts special consideration must be given to the computation of approach friction loss and velocity head. Assume the total discharge and estimate the total culvert area occupied by flow in order to compute these factors. If the assumed values are greatly different from the final result, recompute.

Compute the velocity head for the entire approach section and add to the water-surface elevation to determine the energy head at section 1 applicable to each culvert.

Add the conveyances of the flow sections of all culverts to determine the total conveyance at section 2. Then use this with the total conveyance at section 1 to compute the approach friction loss. Subtract this friction loss from the energy head at section 1 to obtain the energy head at section 2. This energy head is applicable to each of the culverts.

The percent of channel contraction is another factor in which the entire approach area and the combined total of culvert flow areas are used together. Because the total area of flow at the terminal sections of multiple culverts is used, it is possible that one or more of the areas used are located at section 2, and the others are at section 3.

Coefficients of Discharge

Coefficients of discharge, C, for flow types 1–6 were defined by laboratory study and are applicable to both the standard formula and routing methods of computation of discharge. The coefficients vary from 0.39 to 0.98, and they have been found to be a function of the degree of channel contraction and the geometry of the culvert entrance.

For certain entrance geometries the discharge coefficient is obtained by multiplying a base coefficient by an adjustment factor such as k_r or k_w. If this procedure results in a discharge coefficient greater than 0.98, a coefficient of 0.98 should be used as a limiting value in computing the discharge through the culvert.

The coefficients are applicable to both single-barrel and multibarrel culvert installations. If the width of the web between barrels in a multibarrel installation is less than 0.1 of the width of a single barrel, the web should be disregarded in determining the effect of the entrance geometry. Bevels are considered as such only within a range of 0.1 of the diameter, depth, or width of a culvert barrel. Larger sizes are not considered as bevels but as wingwalls.

Laboratory tests also indicate that the discharge coefficient does not vary with the proximity of the culvert floor to the ground level at the entrance. Thus in types 1, 2, and 3 flow, the geometry of the sides determines the value of C; similarly, in types 4, 5, and 6 flow the value of C varies with the geometry of the top and sides. If the degree of rounding or beveling is not the same on both sides, or on the sides and the top, the effect of r or w must be obtained by averaging the coefficients determined for the sides, or for the sides and top, according to the type of flow. One exception is noted: if the vertical sides of the culvert are rounded or beveled and the top entrance is square, multiply the average coefficient (determined by the procedure just described) by 0.90 for type 5 flow and by 0.95 for types 4 and 6 flow, using the coefficient for the square entrance as the lower limiting value.

The discharge coefficient does not vary with culvert skew.

The radius of rounding or degree of bevel of corrugated pipes should be measured in the field. These are critical dimensions that should not be chosen from a handbook and accepted blindly.

The ratio of channel contraction, m, is associated with horizontal contraction typical of flow types 1, 2, and 3. The effect of side contraction becomes negligible for flow types 4, 5, and 6 in which vertical contraction is more important. Therefore, no adjustment for contraction ratios less than 0.80 is warranted for flow types 4, 5, or 6.

In listing the discharge coefficients, it is convenient to divide the six flow types into three groups, each group having a discharge equation of the same general form. Thus, flow types 1, 2, and 3 form one group; types 4 and 6 another; and type 5 a third. The coefficient C is descriptive of the live-stream contraction at the inlet and its subsequent expansion in the barrel of the culvert. Hence, base coefficients for types 1, 2, and 3 flow should be identical for identical geometries, as should coefficients for types 4 and 6.

In a systematic presentation of the coefficients, the entrance geometries have been classified in four general categories: (1) flush setting in vertical headwall, (2) wingwall entrance, (3) projecting entrance, and (4) mitered pipe set flush with sloping embankment. The four classes have been subdivided as necessary, but they all are common to the three flow-type groups.

Types 1, 2, and 3 flow

For culverts, the ratio of channel contraction, m, is defined as $(1-A/A_1)$ where A is the area of flow at the terminal section and A_1 is the area of the approach section. Because the value of m is usually large for flood flows, the laboratory tests placed emphasis on the condition. However, tests on flow through bridge openings demonstrate that the discharge coefficient varies almost linearly between values of m from 0 to 0.80, and that the coefficient reaches a minimum value at $m=0.80$. All coefficients given herein are for an m of 0.80. If the contraction ratio is smaller than 0.80, the value of C may be computed by interpolating between the value of C listed for an m of 0.80 and a value of C of 0.98 for an m of 0. The following formula may be used in place of interpolation: C(adjusted)$=0.98-(0.98-C)m/0.80$. This formula is shown in graph form in figure 19. This adjustment is made as the last step in the computation of the discharge coefficient. Example 10 of the sample computations shows this adjustment.

Flush setting in vertical headwall

Pipe culverts

The discharge coefficient for square-ended pipes set flush in a vertical headwall is a function of the ratio of the headwater height to the pipe diameter, $(h_1-z)/D$. The coefficient for flow types 1, 2, and 3 can be determined from figure 20.

If the entrance to the pipe is rounded or beveled, compute the discharge coefficient by

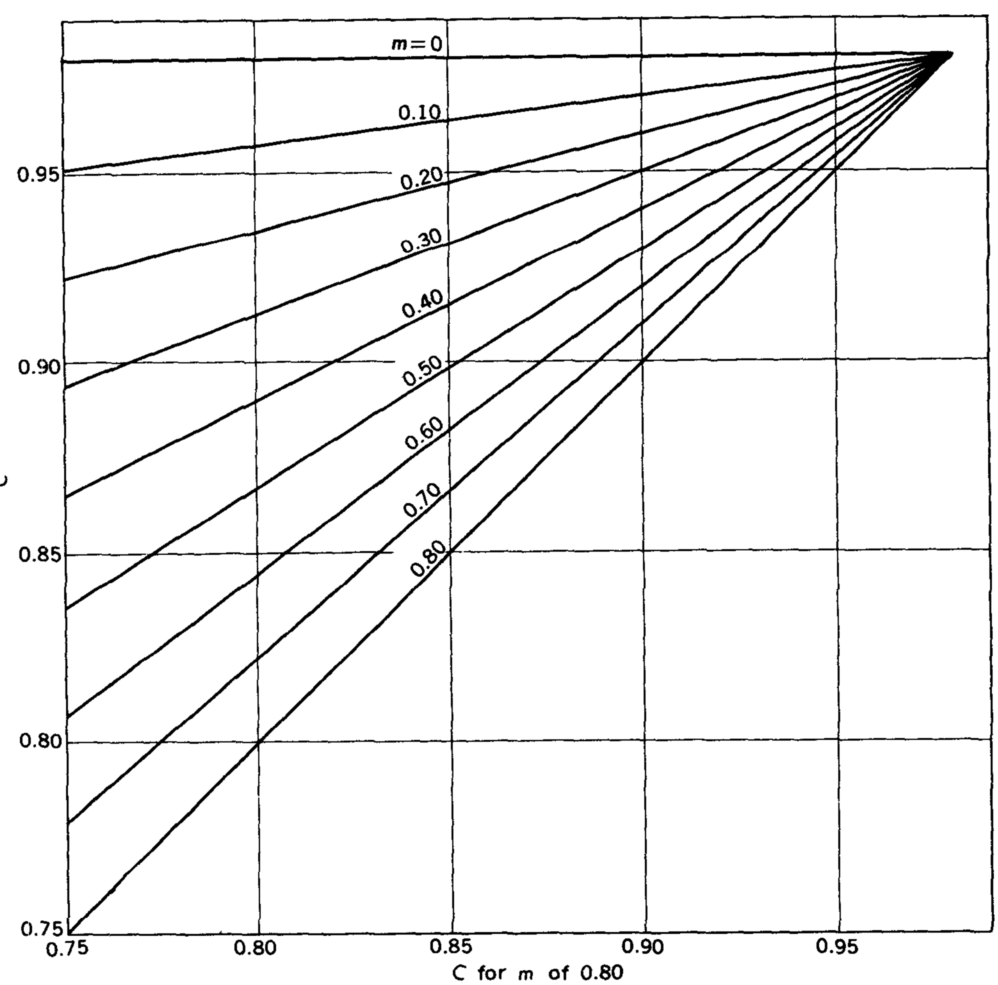


Figure 19.—Adjustment to discharge coefficient for degree of channel contraction.

multiplying the coefficient for the square-ended pipe by an adjustment factor, k_r or k_w. These adjustment factors are a function of the degree of entrance rounding or beveling and these relations, applicable to flow types 1, 2, and 3, are defined in figures 21 and 22.

Machine tongue-and-groove reinforced concrete pipe from 18 to 36 inches in diameter has been tested, and no systematic variation was found between the discharge coefficient and the headwater-diameter ratio. w/D varied from 0.06 to 0.08, and θ averaged 78° with small

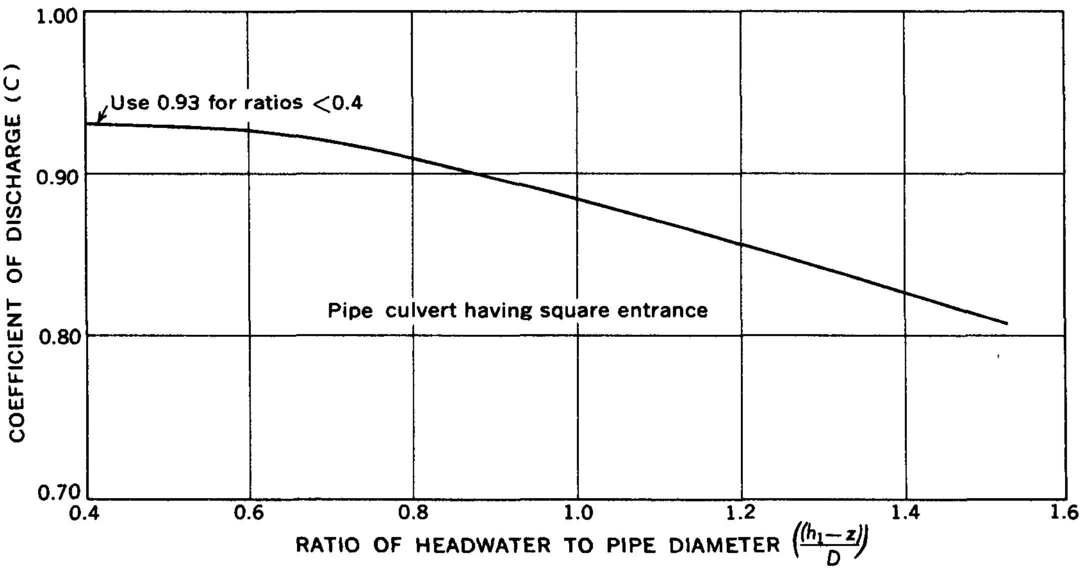


Figure 20.—Base coefficient of discharge for types 1, 2, and 3 flow in pipe culverts with square entrance mounted flush with vertical headwall.

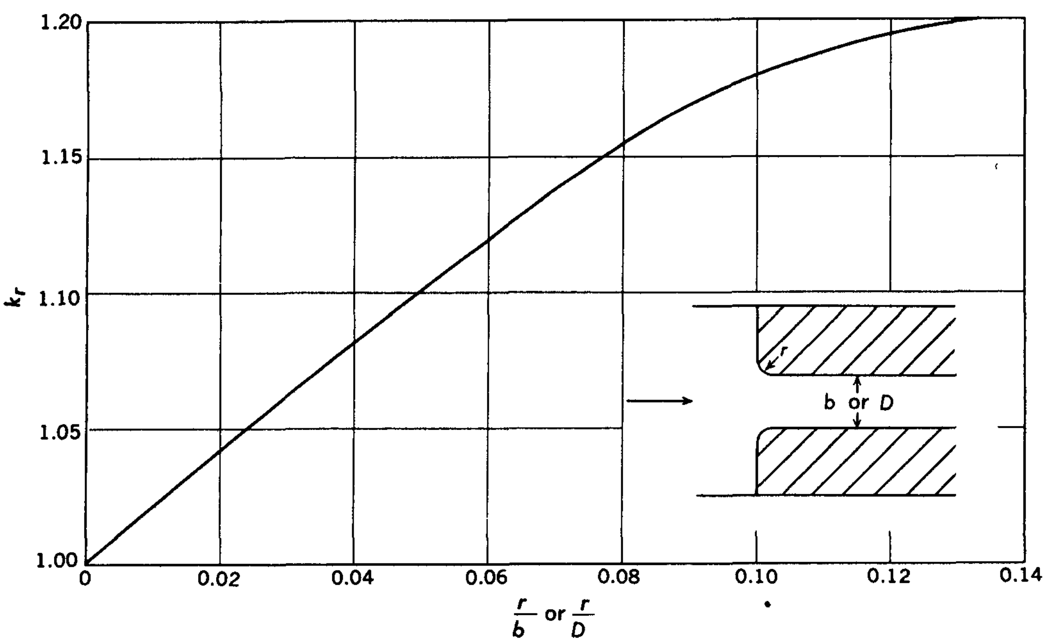


Figure 21.—Variation of the discharge coefficient with entrance rounding, types 1, 2, and 3 flow in box or pipe culverts set flush with vertical headwall.

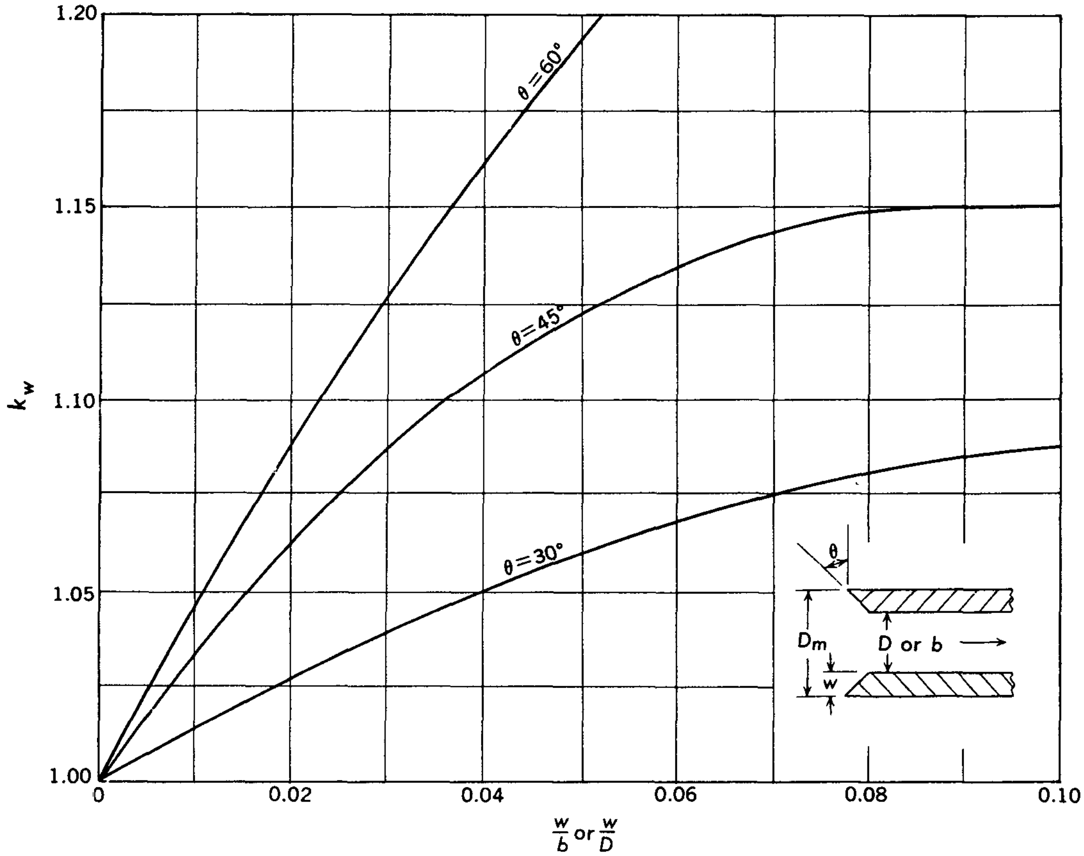


Figure 22.—Variation of the discharge coefficient with entrance beveling, types 1, 2, and 3 flow in box or pipe culverts set flush with vertical headwall.

variation. Therefore, use a C of 0.95 for all sizes of machine tongue-and-groove concrete pipe without regard to $(h_1-z)/D$ for flow types 1, 2, and 3. Bellmouthed precast concrete pipe is considered to be in this category; therefore use a C of 0.95 for it, too.

According to one manufacturer of corrugated-metal pipe, the pipe actually has a beveled rather than a rounded edge. The bevel has an average w (fig. 22) of 0.30 inch (0.025 ft) with a bevel angle of 67°. If the entrance appears to be rounded rather than beveled, the rounding may vary with the gage of the metal, but it will average very nearly 0.80 inch (0.067 ft) for the weights of metal ordinarily used. Occasionally a culvert with a beaded or rolled entrance will be found. The radius of rounding of the bead generally is about 3/8 inch (0.031 ft). Always make exact measurements in the field.

The following list shows values of r/D and w/D for various sizes of standard riveted corrugated-metal pipe.

D (inches)	r/D	w/D
24	0.031	0.0125
36	.021	.0083
48	.016	.0062
60	.012	.0050
72	.010	.0042

Because of the longer pitch in multiplate pipe construction, the entrance is most likely to be considered beveled. The value of w will average about 1.2 inches and θ about 52°. Always measure in the field these factors or the data required to compute them.

Box culverts

The discharge coefficient for box culverts set flush in a vertical headwall is a function of the Froude number. The Froude number for flow types 1 and 2 is always 1.0, and the corresponding discharge coefficient is 0.95. Determine the discharge for type 3 flow from figure 23 after computing the Froude number, $V/\sqrt{gd}$, at the downstream end of the culvert. If necessary, figure 23 may be extrapolated with reasonable safety to Froude numbers of 0.1 to 0.2.

If the entrance to the box is rounded or beveled, compute the discharge coefficient by multiplying the coefficient for the square-ended box by an adjustment factor, k_r or k_w. Determine these adjustment factors, applicable to flow types 1, 2, and 3, from figure 21 or 22, respectively.

Wingwall entrance

Pipe culverts set flush with vertical headwall

The addition of wingwalls to the entrance of pipes set flush in a vertical headwall does not affect the discharge coefficient, which can be determined as shown previously under "Flush Setting in Vertical Headwall," on page 38.

Box culverts

Compute the discharge coefficient for box culverts with a wingwall entrance by first selecting a coefficient from figure 23 and then multiplying this coefficient by an adjustment factor k_θ, which can be determined from figure 24 on the basis of an angle θ of the wingwall. If the angle of the wingwall is not the same on each side, determine the value of C for each side independently and average the results. Where the web between culvert barrels is wide enough (0.1 b or greater) to affect the entrance geometry, treat it as a wingwall. Consider a web corner of less than a right angle as a square entrance.

Projecting entrance

Corrugated-metal pipes and pipe-arches

Determine the discharge coefficient for pipes and pipe-arches that extend beyond a headwall or embankment by first computing a coefficient as outlined for pipes set flush in a vertical headwall and then multiplying the coefficient by an adjustment factor, k_L. The adjustment factor is a function of L_p/D where L_p is the length by which the culvert projects beyond the headwall or embankment. The adjusted C to which k_L is applied must not be greater than 0.98, as this is the limiting value of C.

An acceptable method for determining k_L is to measure L_p at various points around the pipe entrance, between the invert and headwater elevation, then weight L_p for each side of the pipe on the basis of vertical distance and obtain the average L_p before computing k_L. The

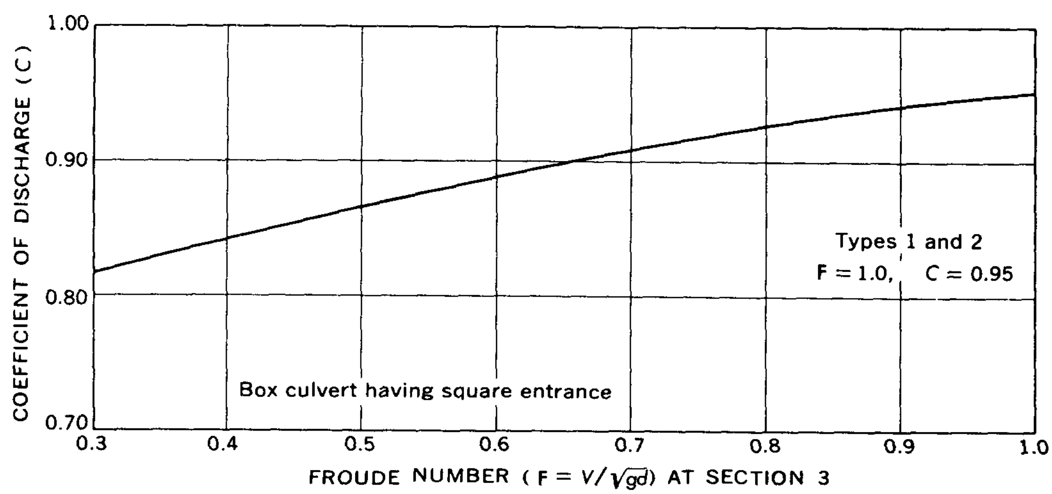


Figure 23.—Base coefficient of discharge for types 1, 2, and 3 flow in box culverts with square entrance mounted flush in vertical headwall.

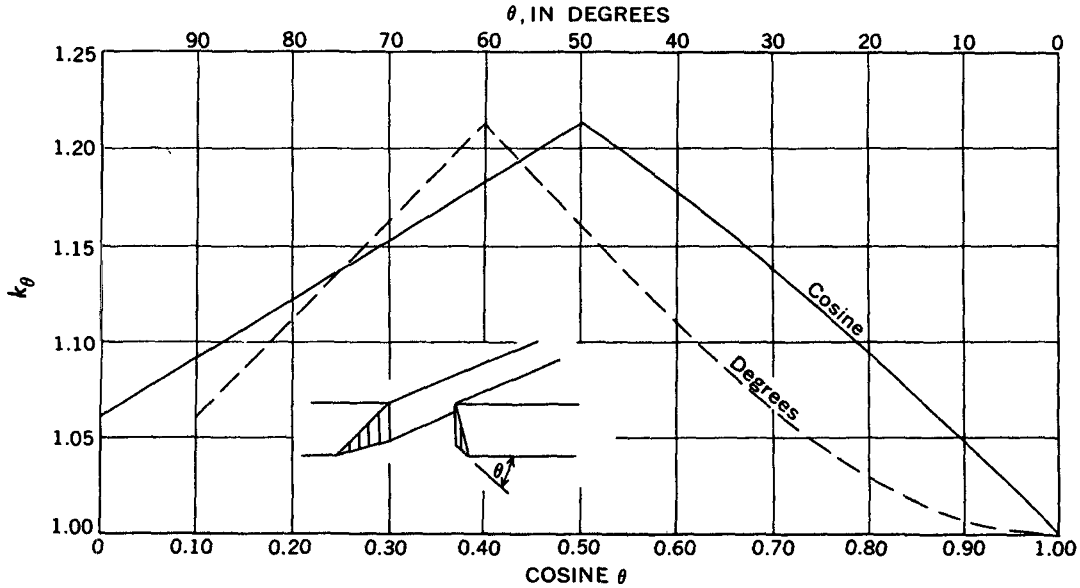


Figure 24.—Variation of discharge coefficient with wingwall angle, types 1, 2, and 3 flow in box culverts with wingwall set flush with sloping embankment.

following list presents values of k_L for various values of L_p/D.

L_p/D	k_L	L_p/D	k_L
0.00	1.00	0.0	1.00
.01	.99	.1	.92
.02	.98	.2	.92
.03	.98	.3	.92
.04	.97	.4	.91
.05	.96	.5	.91
.06	.95	.6	.91
.07	.94	.7	.91
.08	.94	.8	.90
.09	.93	.9	.90
.10	.92	$\geqq$1.0	.90

Concrete pipes with beveled end

The discharge coefficient for projecting entrances for concrete pipes with a beveled end is the same as for flush entrances.

Mitered pipe set flush with sloping embankment

The discharge coefficient for mitered pipes set flush with a sloping embankment is a function of the ratio of headwater height to pipe diameter and can be determined from figure 25.

For a projecting mitered pipe with a thin wall (like corrugated metal), adjust the discharge coefficient in the same manner as any other projecting barrel. Do not adjust for rounding or beveling.

Types 4 and 6 flow

Flush setting in vertical headwall

Box or pipe culverts

Select the discharge coefficient for box or pipe culverts set flush in a vertical headwall from table 5. This includes square-ended pipes or boxes, corrugated pipes, corrugated pipe-arches, corrugated pipes with a standard conical entrance, concrete pipes with a beveled or bellmouthed end, and box culverts with rounded or beveled sides.

Table 5.—Discharge coefficients for box or pipe culverts set flush in a vertical headwall; types 4 and 6 flow

r/b, w/b, w/D, or r/D	C
0	0.84
.02	.88
.04	.91
.06	.94
.08	.96
.10	.97
.12	.98

The discharge coefficient for flared pipe end sections is 0.90 for all diameters and all values of $(h_1-z)/D$.

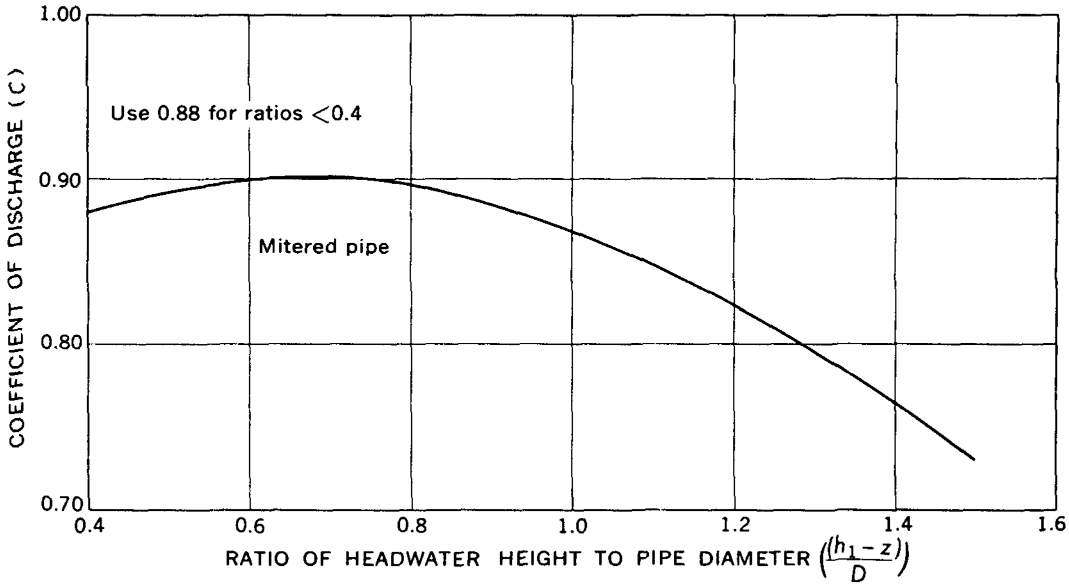


Figure 25.—Variation of discharge coefficient with headwater-diameter ratio, types 1, 2, and 3 flow in mitered pipe set flush with sloping embankment.

Wingwall entrance

Pipe culverts set flush with vertical headwall

The addition of wingwalls to the entrance of pipes set flush with a vertical headwall does not affect the discharge coefficient, which can be determined from table 5.

Box culverts

For box culverts with wingwalls and a square top entrance the discharge coefficient is 0.87 for wingwall angles, θ, of 30–75° and is 0.75 for the special condition when θ equals 90°. If the top entrance is rounded or beveled, and θ is between 30° and 75°, select a coefficient from table 5 on the basis of the value of w/D or r/D for the top entrance, but use 0.87 as the lower limiting value. For the special case when θ equals 90°, if the top entrance is rounded or beveled, multiply the base coefficient (0.75) by k_r or k_w from figure 21 or 22. For angles between 75° and 90°, interpolate between 0.87 and 0.75 to obtain the base coefficient and apply the adjustment for rounding or beveling as described above.

Projecting entrance

Corrugated-metal pipes and pipe-arches

Determine the discharge coefficient for corrugated-metal pipes and pipe-arches that extend past a headwall or embankment by first selecting the coefficient from table 5 that corresponds to the particular value of r/D and then multiplying this coefficient by an adjustment factor k_L.

Concrete pipes with beveled end

The discharge coefficient for concrete pipes with a beveled end that have a projecting entrance is the same as for those with a flush entrance and can be determined from table 5.

Mitered pipe set flush with sloping embankment

The discharge coefficient for pipes mitered and set flush with a sloping embankment is 0.74. For corrugated-metal pipes and pipe-arches that project beyond the embankment, multiply 0.74 by the adjustment factor k_L.

Type 5 flow

Flush setting in vertical headwall

Box or pipe culverts

Determine the discharge coefficient for box or pipe culverts set flush in a vertical headwall from table 6. This includes square-ended pipe or box, corrugated pipe, corrugated pipe-arch, concrete pipe with a beveled end, and box culverts with rounded or beveled sides.

Type 5 flow usually cannot be obtained when flared pipe end sections are installed. Only for L/D ratios less than 6 and culvert slopes greater

Table 6.—Discharge coefficients for box or pipe culverts set flush in a vertical headwall with variation of head and entrance rounding or beveling; type 5 flow

$\frac{h_1-z}{D}$	r/b, w/b, r/D, or w/D: 0	0.02	0.04	0.06	0.08	0.10	0.14
1.4	0.44	0.46	0.49	0.50	0.50	0.51	0.51
1.5	.46	.49	.52	.53	.53	.54	.54
1.6	.47	.51	.54	.55	.55	.56	.56
1.7	.48	.52	.55	.57	.57	.57	.57
1.8	.49	.54	.57	.58	.58	.58	.58
1.9	.50	.55	.58	.59	.60	.60	.60
2.0	.51	.56	.59	.60	.61	.61	.62
2.5	.54	.59	.62	.64	.64	.65	.66
3.0	.55	.61	.64	.66	.67	.69	.70
3.5	.57	.62	.65	.67	.69	.70	.71
4.0	.58	.63	.66	.68	.70	.71	.72
5.0	.59	.64	.67	.69	.71	.72	.73

than 0.03 will type 5 flow occur. Even under these conditions the flow may eventually translate to type 6 flow. If type 5 flow is believed to exist, the following discharge coefficients are applicable:

$(h_1-z)/D$	C	$(h_1-z)/D$	C
1.4	0.48	2.0	0.57
1.5	.50	2.5	.59
1.6	.52	3.0	.61
1.7	.53	3.5	.63
1.8	.55	4.0	.65
1.9	.56	5.0	.66

Wingwall entrance

Pipe culverts set flush with vertical headwall

For pipes set flush with a vertical headwall, the addition of wingwalls to the entrance does not affect the discharge coefficient, which can be determined from table 6.

Box culverts

Determine the discharge coefficient for box culverts with wingwalls and a square top entrance from table 7. If the top entrance is rounded or beveled, select the coefficient from table 6 on the basis of w/D or r/D for the top entrance, but use the coefficient from table 7 as a lower limiting value.

Projecting entrance

Corrugated-metal pipes and pipe-arches

Determine the discharge coefficient for pipes and pipe-arches that extend past a headwall or embankment by first selecting a coefficient from table 6 and then multiplying by an adjustment factor k_L.

Table 7.—Discharge coefficients for box culverts with wingwalls with variation of head and wingwall angle, θ; type 5 flow

$\frac{h_1-z}{D}$	Wingwall angle, θ: 30°	45°	60°	75°	90°
1.3	0.44	0.44	0.43	0.42	0.39
1.4	.46	.46	.45	.43	.41
1.5	.47	.47	.46	.45	.42
1.6	.49	.49	.48	.46	.43
1.7	.50	.50	.48	.47	.44
1.8	.51	.51	.50	.48	.45
1.9	.52	.52	.51	.49	.46
2.0	.53	.53	.52	.49	.46
2.5	.56	.56	.54	.52	.49
3.0	.58	.58	.56	.54	.50
3.5	.60	.60	.58	.55	.52
4.0	.61	.61	.59	.56	.53
5.0	.62	.62	.60	.58	.54

Concrete pipe with beveled end

Determine the discharge coefficient for a concrete pipe with a beveled end directly from table 6.

Mitered pipe set flush with sloping embankment

Determine the discharge coefficient for mitered pipes set flush with a sloping embankment by first selecting a coefficient from table 6 for a square-ended pipe and then multiplying this coefficient by 0.92. If the mitered pipe is thin walled (such as corrugated metal) and projects beyond the embankment, the adjustment factor k_L should be applied also.

Unusual culvert entrances

The coefficient of discharge for the commercial flared opening described in figure 8 is 0.95 for flow types 1, 2, and 3 for all diameters of pipe and all values of $(h_1-z)/D$. The properties of these flared end sections are shown in figure 8.

For culvert entrances of unusual shape, estimate the discharge coefficient on the basis of known values for the more common shapes (reentrant, sharp, 45-degree wingwalls). For the six types of flow discussed in this manual, this entrance coefficient can usually be estimated with sufficient accuracy. In high-head flow the entrance shape is very important because it may mean the difference between a culvert flowing full or partly full.

Remember that the effect of side contraction becomes negligible for flow types 4, 5, and 6 and that vertical contraction is very important.

General Remarks

Storage

The storage effect of pondage upstream from a culvert reduces the peak discharge that normally would result if there were no embankment acting as a dam. If small-area sites are being selected for regional hydrologic studies, it is advisable to select sites where the ponded area is a negligible part of the total drainage area.

The two main factors to be considered in storage are the rate of rise in the pond and the relationship of size of culvert to size of pond. For any given rate of rise, the size (surface area) of the pond required to reduce the outflow a selected percent can be computed. If the surface area is computed for several stages, holding the rate of rise and percent difference between inflow and outflow constant, a curve can be developed. The curves of figure 26 were developed for corrugated-metal pipes with projecting entrances and inlet control. For concrete pipes the surface areas of figure 26 should be increased about 10 percent.

The reduction in flow is directly proportional to the surface area of the pond and to the rate of rise. The curves of figure 26, therefore, can be used for other rates of rise or percentage reductions in flow if treated correctly mathematically. For example, if the surface area is doubled at a given stage and rate of rise held constant, the reduction in flow will be doubled, or increased to 2 percent. If the rate of rise is doubled, the reduction in flow at a given surface area will be doubled.

These curves may be used in two ways: (1) to determine how much the discharge would be affected by a pond of known surface area and (2) to determine the allowable surface area for a given reduction in discharge. In considering item 2, if a reduction in flow of less than 1 percent is desired, a site should be selected where the surface area of the pond is less than the allowable value shown on figure 26.

All the above factors except rate of rise can be determined easily at most sites. Rate of rise

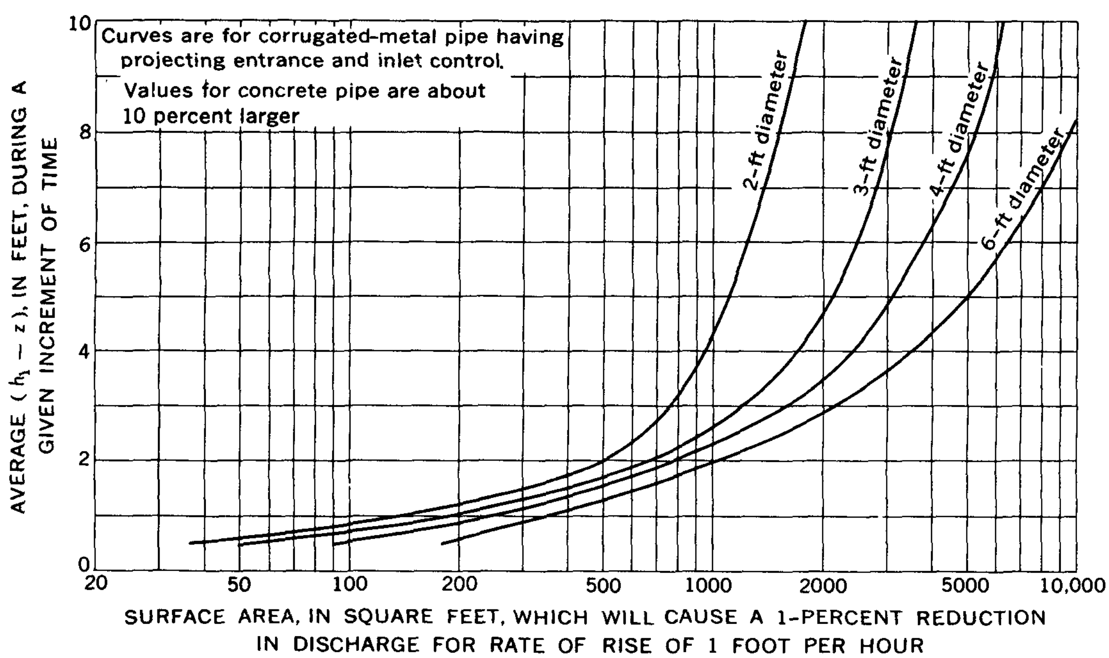


Figure 26.—Variation of headwater with surface area of pond for determining reduction in discharge for various sizes of culverts.

is dependent on (1) inflow, (2) size of pipe, and (3) size of pond. This factor can be determined only by continual reading of a gage or from a recorder chart. Because continuous records of stage usually are not available at culvert sites, an estimate must be made from recorder graphs of nearby streams.

Rates of rise in ponds subject to cloudburst runoff have been recorded as high as 10 feet per hour where the outflow was through a small pipe. The ponds at these places had surface areas of 10–15 acres. If this same flow were ponded at an ordinary crest-stage site, the rise could be considerably greater.

The reduction in peak flow through any given culvert varies with stage as shown in figure 26. In general, pondage does not exist to any great degree at low stages. However, if the culvert is set high in the fill, then a sizeable pond may form before flow through the pipe begins. For this situation many low peaks may be completely absorbed in storage.

Stage-discharge relationships for culverts

Many small-area, high-water gaging stations instrumented with crest-stage gages are located at culverts because of the convenient means of measuring peak discharge. At many such installations experience has shown that either type 1, 2, or type 4 flow may be expected through a considerable range of discharge. At high heads, types 1 and 2 flow will usually change to either type 5 or 6 flow, respectively. For example, a steep culvert with free getaway might always support type 1 flow until the headwater-diameter ratio reaches 1.5, when flow will become either type 5 or 6. A flat culvert with free getaway might always have type 2 flow at headwater-diameter ratios less than 1.5. With an intermediate slope and free getaway, a transition from type 1 to type 2 flow might always occur at a relatively fixed upstream stage. Also, a culvert may always be submerged at both ends, so that type 4 flow will occur at all high stages.

Gages should always be installed at the approach section and along the downstream embankment if tailwater can be significant. A stage-discharge relation, often called a rating curve, can be prepared in advance of actual flood peaks, but several field verifications of the accuracy of the crest-stage recordings and of the constancy of flow type should be made before the rating curve is used. For types 1 and 2 flow various critical depths can be assumed, and the corresponding discharges and headwater elevations computed. For type 4 flow, discharge can be computed for various falls, because discharge is a function of the difference in water-surface elevation between headwater and tailwater. Type 3 flow does not lend itself to a direct computation of the rating curve, but a fairly reliable rating can be developed in the manner discussed below by making numerous computations using assumed values. Field data are very valuable, however, in determining the usable range of values. There is no need for making many computations in the range where the curves will never be used.

In a steep channel the point of zero flow at the gage may be higher than the point of zero flow at the culvert entrance. In this case the channel is the control at low stages, and the theoretical curve cannot be used until the culvert becomes the control. Unless the Froude number at the approach section is less than about 0.70, there is no assurance that the culvert is the control. Even then, a field check may be necessary to ensure against a sharp break in channel slope just above the culvert.

The effect of a changing approach section must be considered in drawing ratings for low-head flow. Rating curves should not be used if the approach channel shifts sufficiently to alter the approach velocity head or friction loss. Small changes in areas will have no effect provided the velocity head and friction loss are small, but they may have considerable effect where these items are large. When the approach channel is fairly stable, curves of area and conveyance are helpful in making computations of discharge.

The rating curves shown in the section under transitions are combinations of curves representing certain types of flow. These figures are used to show rating curves for flow types 1–6, as well as the transitions between certain combinations.

Current-meter measurements should be made to help define rating curves. These are especially valuable at low stages if there is a possibility of critical flow occurring between the approach section and the culvert and at stages where high-head flow is likely to occur.

Ratings with transition between flow types

For rectangular culverts a very pronounced break in the computed rating curve occurs between types 1 and 5 flows. This discharge may vary as much as 35 percent between the 2 methods of computation at a headwater-diameter ratio of 1.5. Because an instantaneous reduction in discharge probably does not occur, a gradual transition is expected between the two types of flow. This is shown by the head-discharge curves determined from the data of various experiments.

Laboratory data show that an unstable condition exists after the headwater-diameter ratio becomes 1.2 and before it reaches 1.5 where flow usually becomes high-head. It is recommended that the transition curve in rating for types 1–5 flow be represented by a straight line between the discharge computed by low-head methods at a ratio of 1.2 and the discharge computed by high-head methods at a ratio of 1.5. Figure 27 is an example of a rating curve for a box culvert where a transition from type 1 to type 5 flow occurs. This curve is for a particular site, as are the other examples shown below. The rating curve for any given site will reflect the unique features of that site.

A curve for a rectangular culvert with a transition from type 2 to type 6 flow is shown in figure 28, where the limiting ratios are considered

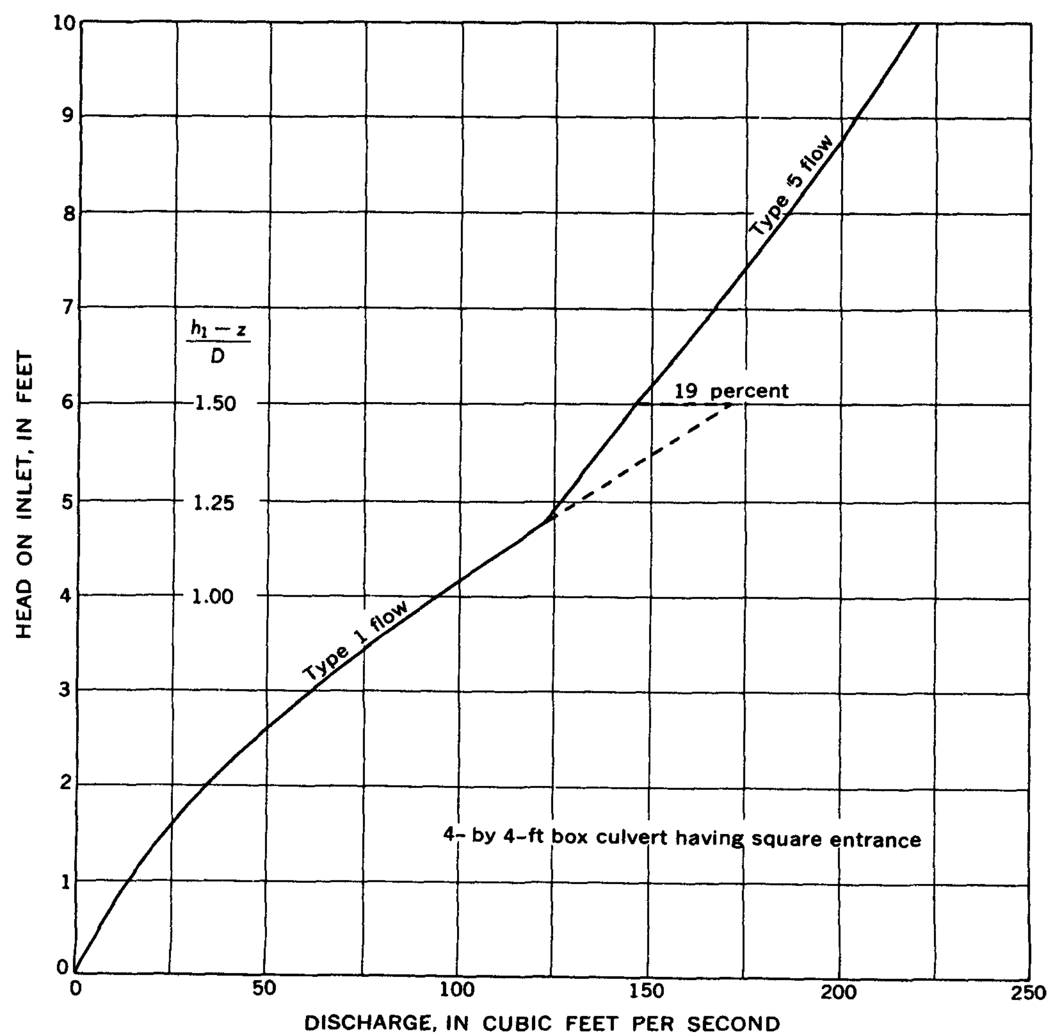


Figure 27.—Rating curve showing transition from type 1 to type 5 flow in a box culvert.

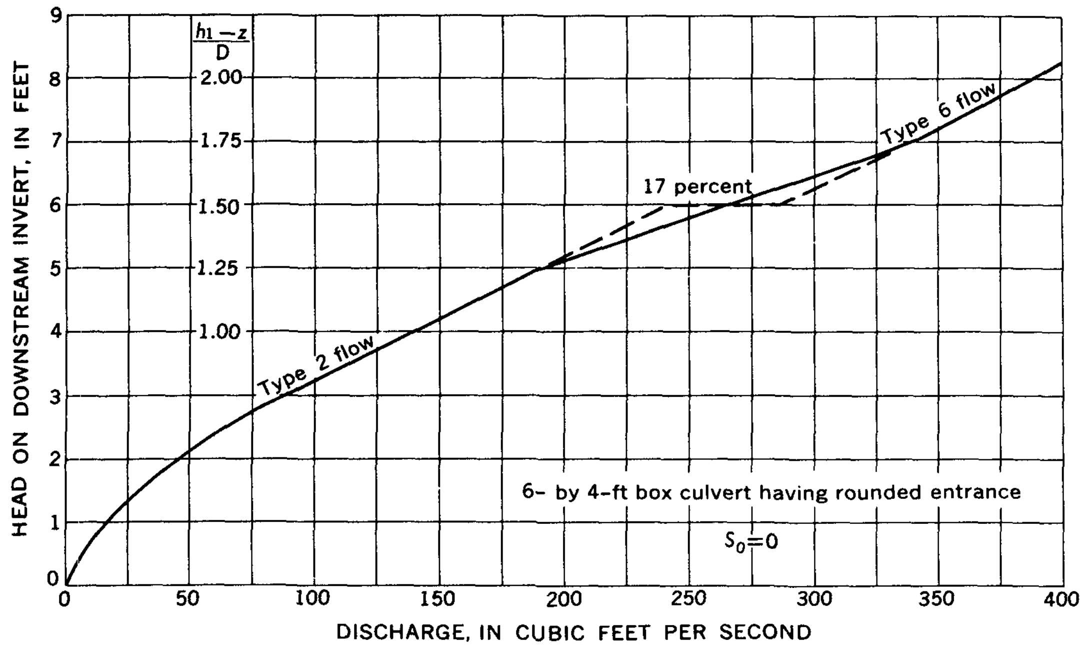


Figure 28.—Rating curve showing transition from type 2 to type 6 flow in a box culvert.

to be 1.25 and 1.75. A straight line is drawn between discharges computed at these ratios.

The break in a rating curve at a headwater-diameter ratio of 1.5 for a circular culvert is not nearly so severe as for a rectangular culvert. This is due to the gradual contraction and reduction in area per unit of culvert diameter as the top of the pipe is approached. Figure 29 is an example of a rating curve for a pipe culvert showing the relatively small difference in discharge between types 1 and 5 flow condition at a headwater-diameter ratio of 1.5. In this example the area of questionable flow type apparently lies between headwater-diameter ratios of 1.2 and 1.5. Therefore, a straight line drawn between these two points will provide a satisfactory transition.

The transition between types 2 and 6 flow through circular culverts can be made in the same manner as for box culverts. The spread in discharge at a headwater-diameter ratio of 1.5 is very small. Therefore a straight line drawn between headwater-diameter ratios of 1.25 and 1.75 is an average line. Figure 30 shows a transition from type 2 to type 6 flow in a circular culvert.

The above are some of the more common transitions. There are many special cases, such as between 5 and 6, or borderline between 4 and 5, or between 4 and 6, that must be treated on individual bases.

It is recommended that the foregoing procedures also be applied to discharges computed at miscellaneous sites. The discharge should be interpolated between the limiting values of the transition whenever the headwater-diameter ratio falls in that range.

Type 4 flow

A rating curve for type 4 flow is shown in figure 31. The development of a rating for either box or pipe culverts is a simple procedure. In any given culvert the discharge coefficient is constant, and the pipe is flowing full. The only two variables in the discharge equation are the fall (h_1-h_4) and the discharges. Therefore a constant can be computed to represent the remaining factors in the equation. The rating curve is determined simply by multiplying the constant by various values of the square root of the fall.

Type 3 flow

A rating curve for type 3 flow is not readily developed, because the discharge is a function

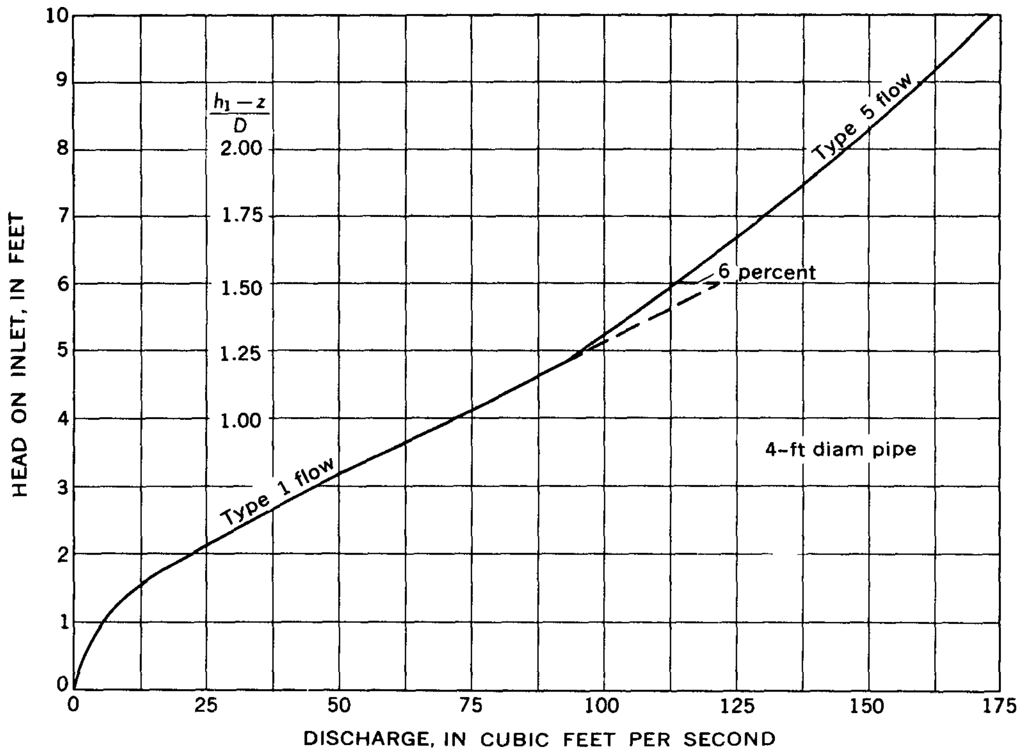


Figure 29.—Rating curve showing transition from type 1 to type 5 flow in a pipe culvert.

of both the outlet area (A_3) and the fall ($h_1 - h_4$) between the headwater and tailwater pools. Figure 32 is an example of a family of curves that define the rating for type 3 flow through a 6-foot-diameter pipe. To develop the rating the discharge must be computed for several combinations of tailwater elevation and fall. The tailwater elevation can be used because A_3 for a given culvert is a direct function of h_4. These discharges are then plotted against fall, and curves drawn connecting points of equal h_4. Logarithmic scales have been used in the example because the curves become very nearly straight lines; but with sufficient definition, any graph scale can be used. As many as 15 or 20 computations may be required to define these curves adequately. It may not be worthwhile to do this for a station where backwater is a factor only occasionally. These curves may be developed from assumed values, but a few field observations are helpful in determining the practical limits of h_4 and fall.

At some sites gravel in varying amounts is deposited at the downstream end of the culvert. When this condition exists at rectangular culverts, the discharge curves may be used if they are drawn using points of equal mean depth at the outlet. If the cross section at the entrance also changes, the curves can be used only if friction losses in the culvert constitute a small proportion of the fall in water surface between the headwater and tailwater. The area and conveyance of a circular section which is partly filled with gravel are not direct functions of depth. Therefore, curves are not satisfactory for the determination of discharge at these sites, but such curves may aid in making first assumptions of discharge for computation.

Slope-area measurement within a culvert

A slope-area measurement may be made within a culvert barrel if certain conditions are met. Where types 1 and 2 flow occur, this method may be used only near the lower end of a culvert that is long enough for normal depth to have been attained. For type 3 flow the

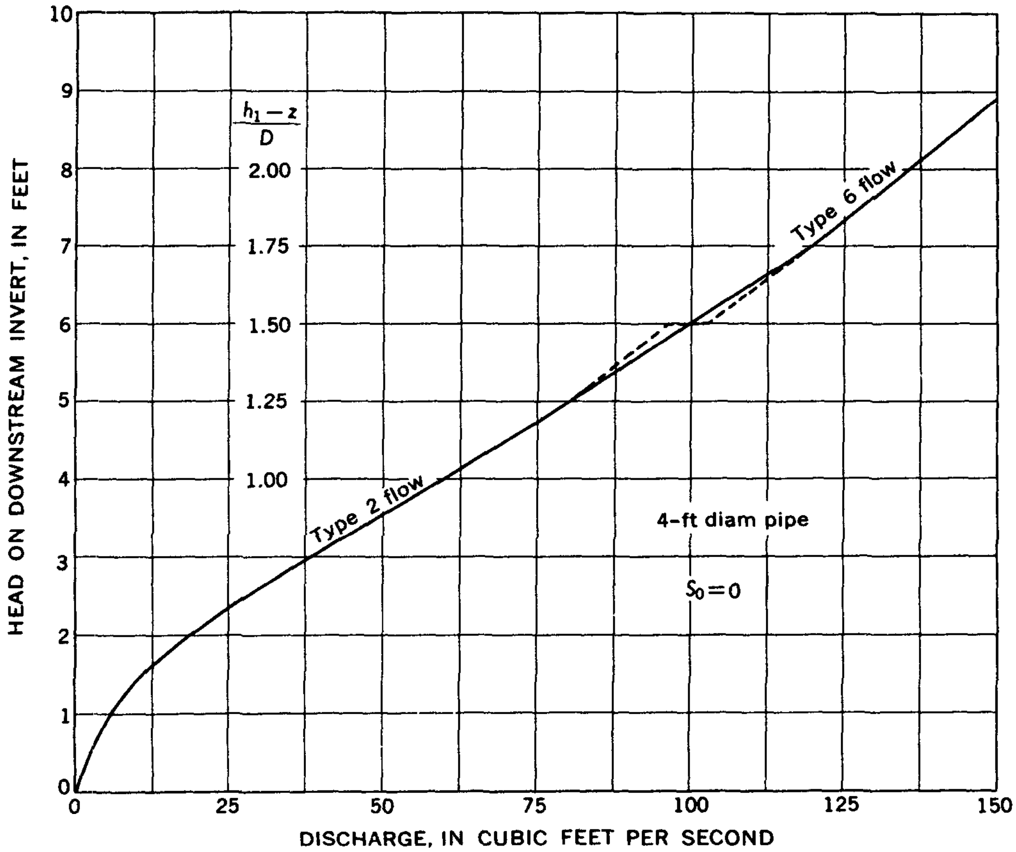


Figure 30.—Rating curve showing transition from type 2 to type 6 flow in a pipe culvert.

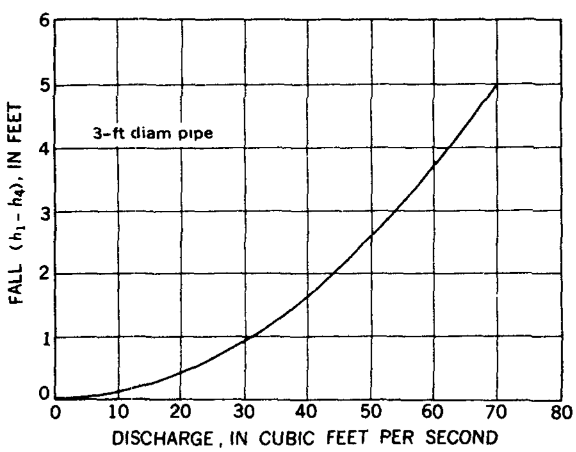


Figure 31.—Rating curve for type 4 flow in a pipe culvert.

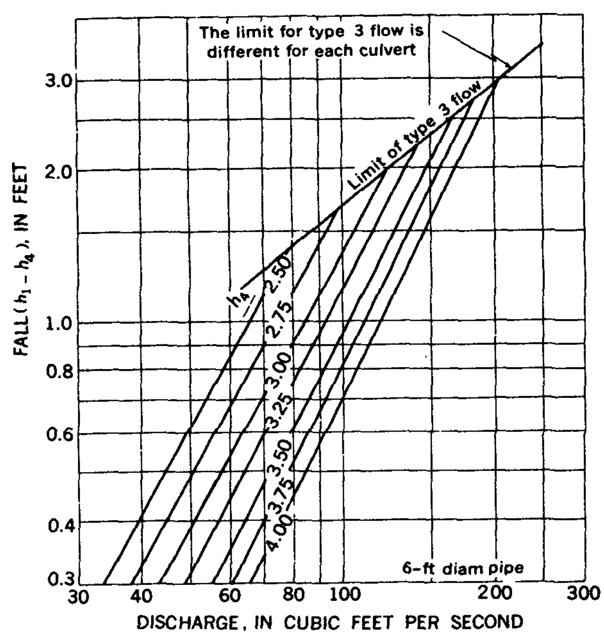


Figure 32.—Rating curve for type 3 flow in a pipe culvert.

slope-area reach should be located at the lower end of the culvert when there is a contraction at the upper end, or it may be located anywhere in a culvert that does not appreciably contract the natural stream.

The most common type of high-water line found in a culvert is a mud line. At times, seed lines may be found if the velocity is low. In some places water-soluble paint has been used as a means of obtaining water-surface elevations, but a good practical and economical method for field use is not known.

Experience has shown that conditions favorable for slope-area measurements in culverts occur infrequently. It is very seldom that reliable high-water marks can be found. A slope-area measurement within a culvert is very sensitive to differences in values of n, whereas a computation involving the critical-depth method is not greatly affected by the values of n assigned. Also, there is evidence that the fluctuating water surface and high velocities within a culvert may leave high-water marks which do not truly represent the effective water-surface profile. In contrast, critical-depth computations utilize headwater elevations that are generally obtained in areas of tranquil flow upstream from the culvert.

Verification of culvert flow

Computation of flow through culverts should be verified whenever possible by current-meter measurements. This includes high stages as well as low stages, although many times it is not possible to find a usable measuring section at high stages because of high velocities and depths too great for wading. Even at low stages it sometimes is necessary to improve the channel before measurements can be made.

Where to measure

Where there is no ponding, the best place to make a current-meter measurement is at the approach section. Current-meter measurements may be made upstream from the culvert even though there is appreciable pondage. The size of the pond and the change in stage during the measurement can be used to compute adjustments to the measured discharge needed because of excessive buildup or reduction in flow by ponding. Gages should always be read before and after the measurement, the same as for a regular gaging station.

Another good place to make a discharge measurement is downstream from the culvert, because then the actual culvert outflow is measured. Care must be taken to exclude inflow from side ditches at the downstream end of the culvert.

If the velocities are not too high, a good measurement can usually be made in or at the downstream end of a culvert. Measuring at the upstream end is not recommended because of the curving streamlines resulting from drawdown into the culvert.

Pipes flowing full

Tests on 36-inch-diameter pipes (Straub and others, 1960) show a fairly constant relationship between mean velocity and velocity at the center of pipes flowing full. For concrete pipe, the mean velocity is approximately 0.86 of the center velocity. For corrugated pipe, this factor is about 0.74.

Under ideal conditions a reliable discharge can be measured in this manner. However, if the velocities are extremely high or if there is air-entrained flow, this method should not be used. In addition to high velocities that may be encountered in type 6 flow, care must be taken to assure full pipe flow; often the water breaks away from the top of the pipe before reaching the outlet.

Accuracy of culvert computations

Under most field conditions, the computation of peak discharge through culverts should provide reliable results. The more ideal the field conditions, the more reliable the computed discharge will be.

In low-head flow very good results may be expected up to headwater-diameter ratios of 1.25 except where critical depth occurs between the approach section and the culvert entrance. Good results may be also expected for high-head flow when the type is definitely known and the headwater-diameter ratio is greater than 1.75. For type 6 flow, good results may always be expected.

In the range of transition between types of flow, better results may be expected from culverts of circular shape than from box culverts, but the results probably should not be rated better than "fair" in either case.

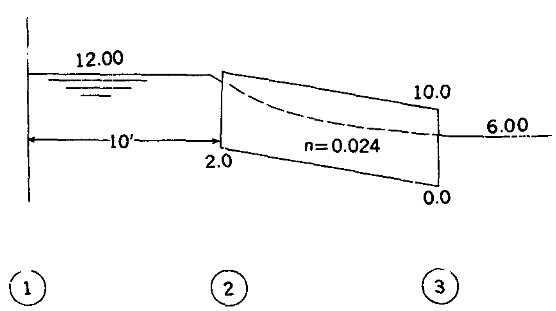
Flow depths below the spring line of pipe-arches may be considerably affected by minor distortions in shape. For this reason, shallow-depth computations are considered less reliable than for conditions of more nearly full flow.

As in other kinds of indirect measurements, the quality of the field data will be a factor in rating the measurement. Factors to be considered are (1) accuracy to which headwater and tailwater elevations can be determined, (2) stability of the approach channel, (3) closeness of the entrance conditions to a standard, (4) the shape and condition of the culvert, (5) scour or fill in the culvert, and (6) the possibility of the culvert being partly plugged by debris at time of peak.

Examples

Ten examples of computation of peak discharge through culverts are given in the following pages. These examples illustrate the procedures used to identify the type of flow and the selection of the proper equation and discharge coefficient.

Example 1. Type 1 flow through a corrugated-metal pipe culvert



10′ diameter corrugated-metal pipe set in concrete headwall.

$r/D=0.006$

$h_1=12.00'$

$z=2.00'$

$h_1-z=10.00'$

$h_4=6.00'$

$A_1=1{,}000$ sq ft

$K_1=300{,}000$

$L=100'$

$S_0=0.02$

$$\frac{S_0D^{1/3}}{n^2}=\frac{0.02\times10^{1/3}}{0.024^2}=74.8.$$

1. From figure 20, $C=0.883$.

   From figure 21, $k_r=1.012$.

   Adjusted $C=0.883\times1.012=0.894$.

   From figure 9, type 1 flow is indicated.

2. From figure 10, $d_c/D=0.65$.

   $d_c=6.50$; $d_c+z=6.50+2.0=8.50$;

   $h_4<d_c+z$; type 1 flow is fairly well assured.

3. $Q=C_qD^{5/2}=2.307\times316=729$ cfs.

4. $A_c=C_aD^2=0.5404\times10^2=54.04$.

   $K_c=C_kD^{8/3}/n=(0.3501\times464)/0.024=6{,}770.$

   $$\frac{V_1^2}{2g}=\frac{0.729^2}{2g}=0.008.$$

   $$h_{f_{1-2}}=\frac{Q^2L_w}{K_1K_c}=\frac{729^2\times10}{300{,}000\times6{,}770}=0.0026.$$

5. $Q=CA_c\sqrt{2g(h_1-z+V_1^2/2g-d_c-h_{f_{1-2}})}$

   $=0.894\times54.04$

   $\times8.02\sqrt{12.00-2.00+0.01-6.50-0}$

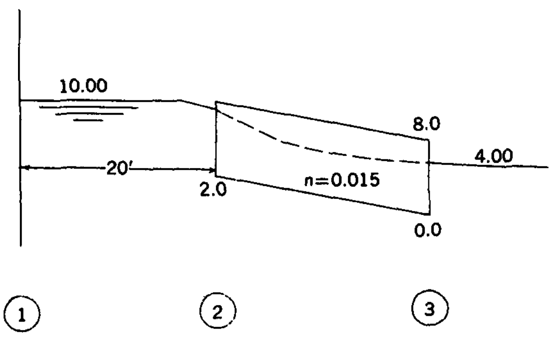
   $=387\sqrt{3.51}=725$ cfs. (Estimated 729 cfs.)

$$S_c=\left(\frac{Q}{K_c}\right)^2=\left(\frac{725}{6{,}770}\right)^2=0.0115.$$

$h_c=d_c+z=6.50+2.00=8.50.$

$S_c<S_0$ and $h_4<h_c$; type 1 flow proved.

Example 2. Type 1 flow through a concrete box culvert



8′ square concrete box culvert, square-edged entrance.

$h_1 = 10.00'$

$z = 2.00'$

$h_1 - z = 8.00'$

$h_4 = 4.00'$

$A_1 = 330$ sq ft

$K_1 = 38{,}900$

$L = 100'$

$S_0 = 0.02$

$d_c = 0.66 \times 8.00 = 5.28.$

$h_c = d_c + z = 5.28 + 2.00 = 7.28.$

$A = 5.28 \times 8.00 = 42.3;$

$P = 8 + (2 \times 5.28) = 18.6;$

$R = \frac{42.3}{18.6} = 2.28.$

$$\frac{14.5 n^2 d_c}{R^{4/3}} = \frac{14.5 \times 0.015^2 \times 5.28}{2.28^{4/3}} = \frac{0.01723}{2.99} = 0.00576.$$

From figure 11, type 1 flow is indicated; also, $h_4 < h_c$.

1. From figure 23, $C = 0.95$.
2. From page 25, the d_c factor $= 0.643$.
3. Assume $d_c = 0.643(h_1 - z) = 0.643 \times 8.00 = 5.14;$

   $A_c = 5.14 \times 8.00 = 41.1;$

   $P_c = 8.00 + 2(5.14) = 18.3;$

   $R = \frac{41.1}{18.3} = 2.24;$

   $R^{2/3} = 1.72.$

   $K_c = 99.1 \times 41.1 \times 1.72 = 7{,}000.$
4. $Q = 5.67 b d_c^{3/2} = 5.67 \times 8 \times 11.69 = 530$ cfs.
5. $\frac{V_1^2}{2g} = \frac{1.61^2}{2g} = 0.04.$

   $h_{f_{1-2}} = \frac{Q^2 L_w}{K_1 K_c} = \frac{530^2 \times 20}{38{,}900 \times 7{,}000} = 0.02.$
6. $Q = C A_c \sqrt{2g(h_1 - z + V_1^2/2g - d_c - h_{f_{1-2}})}$

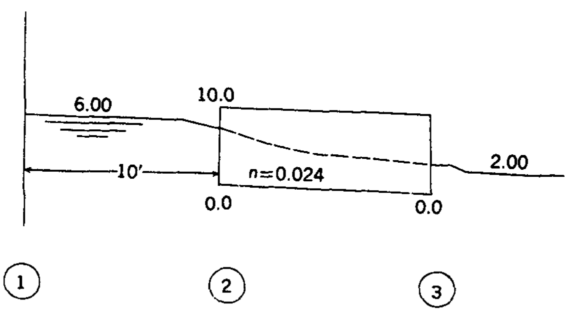
   $= 0.95 \times 41.1 \times 8.02 \sqrt{10.00 - 2.00 + 0.04 - 5.14 - 0.02}$

   $= 313\sqrt{2.88} = 531$ cfs. (Estimated 530 cfs.)

$S_c = \left(\frac{Q}{K_c}\right)^2 = \left(\frac{531}{7{,}000}\right)^2 = 0.00576.$

$S_c < S_0$ and $h_4 < h_c$; type 1 flow proved.

Example 3. Type 2 flow through a corrugated-metal pipe culvert



Ponded conditions. 10′ diameter corrugated-metal pipe set in vertical headwall.

$r/D = 0.006$

$D = 10'$

$h_1 = 6.00'$

$h_4 = 2.00'$

$L_{2-3} = 100'$

$z = 0$

$h_{f_{1-2}} = 0$

$\frac{V_1^2}{2g} = 0$

1. $\frac{h_1-z}{D}=\frac{6.00}{10.0}=0.60.$

From figure 20, $C=0.928$.

From figure 21, $k_r=1.012$.

$C=0.928\times1.012=0.0939$.

2. From figure 10, $d_c/D=0.42$.

$0.95\times0.42=0.399$, and $d_c=3.99$;

$d_c/D=0.399$.

3. $Q=C_QD^{5/2}=0.906\times316=286$ cfs.

4. $$\frac{Q^2}{2gC^2(h_1-z)D^4}=\frac{286^2}{2g\times0.939^2\times6.0\times10^4}=0.0241.$$

From figure 12, $d_2/D=0.520$; $d_2=5.20$.

5. $\frac{\alpha_1V_1^2}{2g}=0$; $h_{f_{1-2}}=0$.

$K=C_k\frac{D^{8/3}}{n}$;

$K_2=0.2472\times19{,}330=4{,}780,$

$K_c=0.1554\times19{,}330=3{,}010.$

$$h_{f_{2-3}}=\frac{Q^2L}{K_2K_c}=\frac{286^2\times100}{3{,}010\times4{,}780}=0.57.$$

6. $H=h_1+\frac{\alpha_1V_1^2}{2g}-h_{f_{1-2}}-h_{f_{2-3}}=6.00+0-0$
$-0.57=5.43.$

7. $\frac{\text{Step 6 value}}{D}=\frac{5.43}{10}=0.543.$

8. From figure 10, $d_c/D=0.385$.

$d_c=0.385\times10=3.85.$

9. $Q=C_QD^{5/2}=0.846\times316=268$ cfs.

10. $$\frac{Q^2}{2gC^2(h_1-z)D^4}=\frac{268^2}{2g\times0.939^2\times6\times10^4}$$
$=0.0211.$

11. From figure 12, $\frac{d_2}{D}=0.53$; $d_2=5.30$.

12. $K=C_k\left(\frac{D^{8/3}}{n}\right)$;

$K_2=0.2556\times19{,}330=4{,}940,$

$K_c=0.1454\times19{,}330=2{,}810.$

$$h_{f_{2-3}}=\frac{Q^2L}{K_2K_c}=\frac{268^2\times100}{4{,}940\times2{,}810}=0.52.$$

$A_c=0.2788\times10^2=27.88.$

13. $$Q=CA_c\sqrt{2g\left(h_1+\frac{V_1^2}{2g}-d_c-h_{f_{1-2}}-h_{f_{2-3}}\right)}$$

$=0.939\times27.88$

$\times8.02\sqrt{6.00+0-3.85-0-0.52}$

$=210\sqrt{1.63}=268$ cfs.

(Estimate was 268 cfs.)

Use 268 cfs.

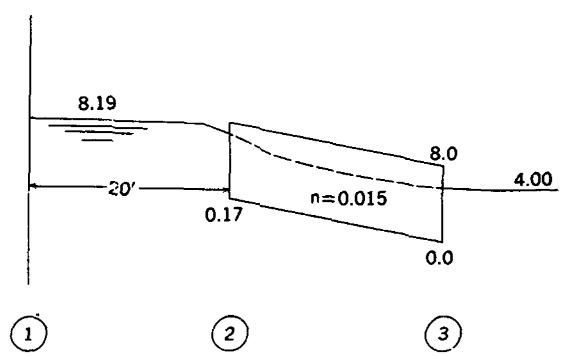
Test for type 2 flow:

$h_c=d_c=3.85$;

$h_4=2.00$;

$S_0<S_c$ and $h_c>h_4$; type 2 flow proved.

Example 4. Type 2 flow through a concrete box culvert



8′ square concrete box culvert, square-edged entrance.

$h_1=8.19$

$z=0.17$

$h_1-z=8.02$

$L_{1-2}=20$

$L_{2-3}=60$

$h_4=4.00$

1. From figure 23, $C=0.95$.

2. d_c factor$=0.643$ from table on page 25.

3. Assume $d_c=0.643(h_1-z)=0.643\times 8.02$ $=5.16$.

4. $Q=5.67bd_c^{3/2}=5.67\times 8\times 11.75=533$ cfs.

5. $\frac{Q^2}{2g(h_1-z)^3b^2C^2}=\frac{533^2}{2g\times 8.02^3\times 8^2\times 0.95^2}$ $=0.149$.

From figure 14, $\frac{d_2}{h_1-z}=0.660$;

$d_2=0.660\times 8.02=5.30$.

6. $\frac{V_1^2}{2g}=0.04$ (computation not shown).

$h_{f_{1-2}}=0.02$ (computation not shown).

$R_2=\frac{8\times 5.30}{8+(2\times 5.30)}=\frac{42.4}{18.60}=2.28,$

$R_c=\frac{8\times 5.16}{8+(2\times 5.16)}=\frac{41.28}{18.32}=2.25.$

$K_2=99.1\times 42.4\times 1.73=7{,}260,$

$K_c=99.1\times 41.3\times 1.72=7{,}040;$

$h_{f_{2-3}}=\frac{Q^2L}{K_2K_c}=\frac{533^2\times 60}{7{,}260\times 7{,}040}=0.33,$

7. $H=h_1+V_1^2/2g-h_{f_{1-2}}-h_{f_{2-3}}$

$=8.19+0.04-0.02-0.33=7.88.$

8. $d_c=0.643(7.88)=5.07$.

9. $Q=5.67bd_3^{3/2}=5.67\times 8\times 11.40=517$ cfs.

10. $\frac{Q^2}{2g(h_1-z)^3b^2C^2}=\frac{517^2}{2g(8.02)^3\times 8^2\times 0.95^2}=0.140.$

From figure 14, $\frac{d_2}{h_1-z}=0.75$;

$d_2=0.75\times 8.02=6.01.$

$K_2=99.1\times 48.08\times 2.40^{2/3}=8{,}540,$

$K_c=99.1\times 40.48\times 2.23^{2/3}=6{,}860.$

11. $h_{f_{2-3}}=\frac{517^2\times 60}{8{,}540\times 6{,}860}=0.27.$

12. $Q=CA_c\sqrt{2g(h_1+V_1^2/2g-d_c-h_{f_{1-2}}-h_{f_{2-3}})}$

$=0.95\times 40.48$

$\times 8.02\sqrt{8.19+0.04-5.07-0.02-0.27}$

$=308.6\sqrt{2.87}=523$ cfs. (Estimated 517 cfs.)

Use 523 cfs.

Test for type 2 flow:

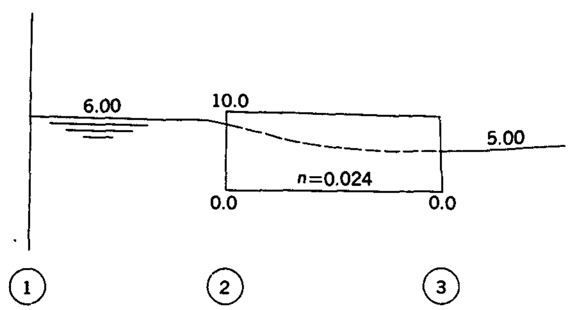
$h_c=d_3=5.07>h_4=4.00;$

$S_0=\frac{0.17}{60}=0.0028;$

$S_c=\left(\frac{Q}{K_c}\right)^2=\left(\frac{523}{6{,}860}\right)^2=0.0058;$

$S_0<S_c$; type 2 flow proved.

Example 5. Type 3 flow through a corrugated-metal pipe culvert



Ponded conditions. 10′ diameter corrugated-metal pipe, square-edged entrance.

$D=10'$

$h_1=6.00'$

$h_4=5.00'$

$L_{2-3}=100'$

$h_3=h_4=5.00'$

$z=0$

$h_1-z=6.00$

$\frac{V_1^2}{2g}=0$

$h_{f_{1-2}}=0$

$r/D=0.006$

$\frac{h_1-z}{D}=\frac{6.00}{10}=0.60.$

$\frac{d_3}{D}=\frac{5.00}{10}=0.500.$

$A_3=0.3927\times10^2=39.27.$

$K=C_k\left(\frac{D^{8/3}}{n}\right),$

$K_3=0.2317\times19{,}330=4{,}480.$

From figure 20, $C=0.928$.

From figure 21, $k_r=1.012$.

Adjusted $C=1.012\times0.928=0.939$.

1. Assume $Q=230$ cfs.

   Assume $d_2=5.50$.

   $\frac{d_2}{D}=\frac{5.50}{10}=0.55.$

   $A_2=0.4426\times10^2=44.26.$

   $\frac{V_2^2}{2gC^2}=\frac{5.20^2}{2g(0.939)^2}=0.48.$

2. From equation 13,

   $$d_2+\frac{V_2^2}{2gC^2}=h_1-z+\frac{V_1^2}{2g}-h_{f_{1-2}}$$

   $5.50+0.48=6.00+0-0$

   $5.98=6.00.$

3. $K_2=0.2710\times19{,}330=5{,}240.$

4. $h_{f_{2-3}}=\frac{Q^2L}{K_2K_3}=\frac{230^2\times100}{5{,}240\times4{,}480}=0.23.$

5. $$Q=CA_3\sqrt{2g\left(h_1+\frac{V_1^2}{2g}-h_3-h_{f_{1-2}}-h_{f_{2-3}}\right)}$$

   $Q=0.939\times39.27$

   $\times8.02\sqrt{6.00+0-5.00-0-0.23}$

   $=296\sqrt{0.77}=260$ cfs. (Assumed 230 cfs.)

6. Assume $Q=251$ cfs.

   Assume $d_2=5.40$.

   $\frac{d_2}{D}=\frac{5.40}{10}=0.54.$

   $A_2=0.4327\times10^2=43.27.$

   $\frac{V_2^2}{2gC^2}=\frac{5.80^2}{2g(0.939)^2}=0.59.$

7. From equation 13,

   $$d_2+\frac{V_2^2}{2gC^2}=h_1-z+\frac{V_1^2}{2g}-h_{f_{1-2}}$$

   $5.40+0.59=6.00+0-0$

   $5.99=6.00.$

8. $K_2=0.2630\times19{,}330=5{,}090.$

9. $h_{f_{2-3}}=\frac{Q^2L}{K_2K_3}=\frac{251^2\times100}{5{,}090\times4{,}480}=0.28.$

10. $Q=296\sqrt{6.00+0-5.00-0-0.28.}$

    $=296\sqrt{0.72}=251$ cfs. (Assumed 251 cfs.)

    Check for type 3 flow:

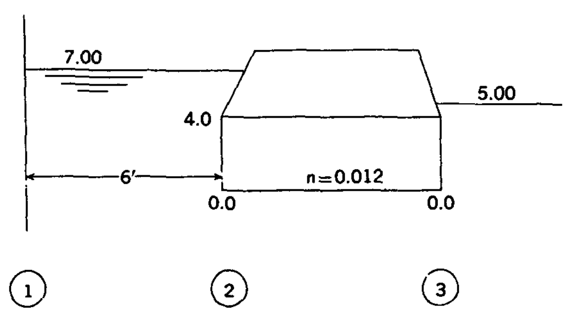
    $S_0=0.00;\ S_c>S_0.$

    $C_q=\frac{Q}{D^{5/2}}=\frac{251}{10^{5/2}}=\frac{251}{316}=0.795.$

    $\frac{d_c}{D}=0.374;\ d_c=0.374\times10=3.74=h_c.$

    $h_4>h_c$; type 3 flow proved.

Example 6. Type 4 flow through a concrete pipe culvert



Ponded conditions. Given: 4′ diameter concrete pipe, bell entrance.

$w/D=0.3/4=0.075$

$h_1=7.00'$

$h_4=5.00'$

$D=4.00'$

$z=0$

$L_{2-3}=50'$

$A_0=12.6$

$R_0=0.25D=1.00$

$R_0^{4/3}=1.00$

From table 5, $C=0.955$.

$$Q=CA_0\sqrt{\frac{2g(h_1-h_4)}{1+\frac{29C^2n^2L}{R_0^{4/3}}}}$$

$$=0.955\times 12.6$$

$$\times 8.02\sqrt{\frac{7.00-5.00}{1+\frac{29\times 0.955^2\times 0.012^2\times 50}{1.00}}}$$

$$=96.5\sqrt{\frac{2.00}{1+0.19}}=96.5\sqrt{1.68}=125 \text{ cfs.}$$

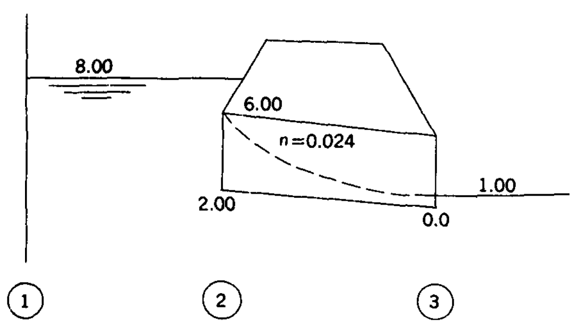
For additional type 4 computations at this site the following equation may be used:

$$Q=K'\sqrt{h_1-h_4}=88.5\sqrt{h_1-h_4},$$

where

$$K'=CA_0\sqrt{\frac{2g}{1+\frac{29C^2n^2L}{R_0^{4/3}}}}.$$

Example 7. Type 5 flow through a corrugated-metal pipe culvert



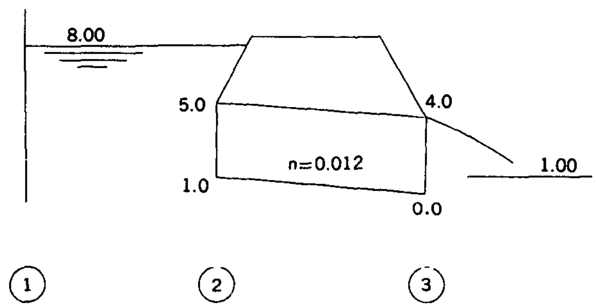
Ponded conditions. Given: 4′ diameter corrugated-metal pipe, rounded entrance.

$$r/D=0.016$$
$$h_1=8.00'$$
$$h_4=1.00'$$
$$h_1-z=6.00'$$
$$z=2.00'$$
$$L_{2-3}=50'$$
$$D=4.00'$$

1. $\frac{L}{D}=\frac{50}{4}=12.5$; $\frac{w}{D}=0$; $S_0=\frac{z}{L}=\frac{2.00}{50}=0.040$.
2. See figure 16 for type of flow (charts b and c).
3. $\frac{29n^2(h_1-z)}{R_0^{4/3}}=\frac{29\times 0.024^2\times 6.00}{1.00}=0.10$.
4. $\frac{h_1-z}{D}=1.50$; type 5 flow is indicated.
5. From table 6, $C=0.484$.
   From table 2, $A_0=12.6$.
6. $Q=CA_0\sqrt{2g(h_1-z)}$

   $=0.484\times 12.6\times 8.02\sqrt{6.00}=120$ cfs.

Example 8. Type 6 flow through a concrete pipe culvert



Ponded conditions. 4′ diameter concrete pipe, beveled entrance.

$$w/D=0.3/4=0.075$$
$$h_1=8.00'$$
$$h_4=1.00'$$
$$h_1-z=7.00'$$
$$z=1.0'$$
$$L_{2-3}=50'$$
$$D=4.0'$$

1. $\frac{L}{D}=\frac{50}{4}=12.5$; $\frac{w}{D}=0.075$; $S_0=\frac{1.00}{50}=0.020$.

   From figure 15, type 6 flow is indicated.

   From table 5,

   $$C=0.955;\ \frac{h_1}{D}=\frac{8.00}{4.00}=2.00.$$

2. From figure 17,

   $$\frac{Q}{A_0\sqrt{D}}=5.40.$$

3. $\frac{29n^2L}{R_0^{4/3}}=\frac{29\times 0.012^2\times 50}{1.0}=0.209$.

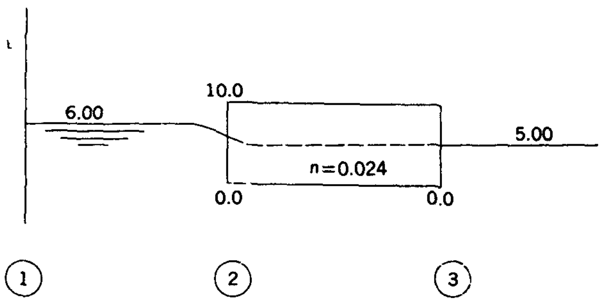
4. From figure 17, factor $k_f=1.54$.

5. Adjusted $\frac{Q}{A_0\sqrt{D}}=k_f\frac{Q}{A_0\sqrt{D}}$

$=1.54\times5.40=8.31.$

6. $Q=8.31A_0\sqrt{D}=8.31\times12.6\times2.0=209$ cfs.

Example 9. Computation of flow by the routing method



Ponded conditions. 10′ diameter corrugated-metal pipe, square-edged entrance.

$$r/D=0.006$$

$$D=10'$$

$$h_1=6.00'$$

$$h_4=5.00'$$

$$L_{2\text{-}3}=100'$$

$$h_3=h_4=5.00'$$

$$z=0$$

$$h_1-z=6.00$$

$$\frac{V_1^2}{2g}=0$$

$$h_{f_{1\text{-}2}}=0$$

1. Assume type 3 flow.

2. Assume $Q=251$ cfs.

$$\frac{d_3}{D}=\frac{5.00}{10}=0.500.$$

$$A_3=0.3927\times10^2=39.27.$$

$$K_3=0.2317\times19{,}330=4{,}470.$$

$$\frac{V_3^2}{2g}=\frac{6.39^2}{2g}=0.64.$$

3. From figure 20, $C=0.928$.
From figure 21, $k_r=1.012$.
Adjusted $C=1.012\times0.928=0.939$.

4. Assume $d_2=5.40$; water-surface elevation $=5.40'$.

$$\frac{d_2}{D}=\frac{5.40}{10}=0.54.$$

$$A_2=0.4327\times10^2=43.27.$$

$$K_2=0.2630\times19{,}330=5{,}090.$$

$$\frac{V_2^2}{2g}=\frac{5.80}{2g}=0.52.$$

$$h_{f_{2\text{-}3}}=\frac{Q^2L}{K_2K_3}=\frac{251^2\times100}{4{,}470\times5{,}090}=0.28.$$

5.

Elevation of water surface at section 3	= 5.00
Velocity head at section 3	=+.64
Friction loss between 2 and 3	=+.28
Velocity head at section 2	=−.52
Water surface at section 2	= 5.40; assumed 5.40.

6. Entrance loss$=\left(\frac{1}{C^2}-1\right)\frac{V_3^2}{2g}$

$$=\left(\frac{1}{0.939^2}-1\right)0.64$$

$$=0.113\times0.64=0.09$$

$h_{f_{1\text{-}2}}$ has been determined as 0.00

$\frac{V_1^2}{2g}$ has been determined as 0.00.

7.

Water surface at section 3	= 5.00
Velocity head at section 3	=+.64
$h_{f_{2\text{-}3}}$	=+.28
Entrance loss	=+.09
$h_{f_{1\text{-}2}}$	=+.00

Velocity head at section 1 $= -.00$

Computed headwater $=6.01$; actual $=6.00$. $Q=25$ cfs.

8. Check for type 3 flow:

$S_0=0.00$.

$$S_c=\left(\frac{Q}{K_3}\right)^2=\left(\frac{251}{4{,}470}\right)^2=0.0031.$$

$Q=C_qD^{5/2}$;

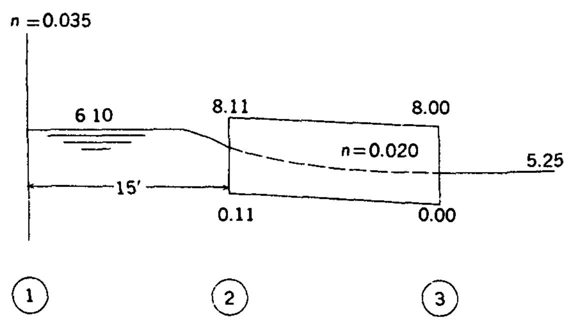
$$C_q=\frac{Q}{D^{5/2}}=\frac{251}{10^{5/2}}=\frac{251}{316}=0.795.$$

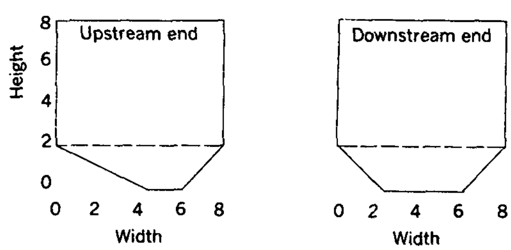
From table 3,

$$\frac{d_c}{D}=0.374;\ d_c=0.374\times10=3.74.$$

$S_c>S_0$ and $h_4>d_c$; type 3 flow proved.

Example 10. Computation of type 3 flow through an irregularly shaped culvert





Square-edged entrance. Wingwall angle $=20°$.

$h_1=6.10'$

$z=0.11'$

$h_4=5.25'$

$A_1=95$ sq ft

$L_{1\text{-}2}=15$

$L_{2\text{-}3}=60$

$n_{2\text{-}3}=0.020$

$K_1=9{,}210$

$A_3=(5.25\times8)-(2\times2)$
$=42.00-4=38.0.$

$P=4.0+3.25+3.25$
$+2.83+2.83=16.16.$

$R_3=38/16.16=2.35$;

$P_3^{2/3}=1.77.$

$K_3=5{,}000.$

$$d_{m_3}=\frac{38.0}{8}=4.75.$$

1. Assume type 3 flow.
2. Assume $Q=250$ cfs.
3. $\mathbf{F}_3=\dfrac{V_3}{\sqrt{gd_{m_3}}}=\dfrac{6.58}{5.67\sqrt{4.75}}=0.532.$

   From figure 23, $C=0.874$.

   From figure 24, $k_\theta=1.03$.

   Adjusted $C=1.03\times0.874=0.90$.

   $$m=\left(1-\frac{A_3}{A_1}\right)=\left(1-\frac{38}{95}\right)=0.60$$

   (adjusted for channel contraction).

   Adjusted $C=0.98-(0.98-C)m/0.80$

   $$=0.98-(0.98-0.90)\ \frac{0.60}{0.80}$$

   $=0.98-0.06=0.92.$

4. Assume $d_2=5.20$; $A_2=35.6$ and $d_{m_2}=4.45$.

   $P=2.0+2.83+4.47+3.20+3.20=15.70.$

   $R_2=2.27$; $R_2^{2/3}=1.73.$

   $K_2=74.3\times35.6\times1.73=4{,}580.$

   $$h_{f_{2\text{-}3}}=\frac{Q^2L}{K_2K_3}=\frac{250^2\times60}{4{,}580\times5{,}000}=0.16.$$

   $$\frac{V_3^2}{2g}=\frac{6.58^2}{2g}=0.67;\qquad \frac{V_2^2}{2g}=\frac{7.02^2}{2g}=0.77.$$

5. Water surface at section 3 $= 5.25$
   Velocity head at section 3 $=+.67$
   $h_{f_{2-3}}$ $=+.16$
   Velocity head at section 2 $=-.77$

   Water surface at section 2 $= 5.31$; assumed 5.31.

6. Entrance loss$=\left(\frac{1}{C^2}-1\right)\frac{V_3^2}{2g}$

$$=\left(\frac{1}{0.92^2}-1\right)0.67=0.12.$$

$$h_{f_{1-2}}=\frac{Q^2L}{K_1K_2}=\frac{250^2\times 15}{9{,}210\times 4{,}580}=0.022.$$

$$\frac{V_1^2}{2g}=\frac{2.63^2}{2g}=0.11.$$

7. Water surface at section 3 $= 5.25$
   Velocity head at section 3 $=+.67$
   $h_{f_{2-3}}$ $=+.16$
   Entrance loss $=+.12$
   $h_{f_{1-2}}$ $=+.02$
   Velocity head at section 1 $=-.11$

   Water surface at section 1 $= 6.11$; actual$=6.10$.

8. Check for type 3 flow:

$$S_0=\frac{0.11}{60}=0.00183.$$

$$S_{c_2}=\left(\frac{Q}{K_2}\right)^2=\left(\frac{250}{4{,}580}\right)^2=0.00298.$$

$S_{c_2}>S_0$; type 1 flow cannot occur.

Compute d_c at section 3 for $Q=250$ cfs:

$$Q=A_c^{3/2}\sqrt{\frac{g}{T}}=A_c^{3/2}\sqrt{\frac{32.2}{8}}=2.00A_c^{3/2}.$$

$$A_c=\left(\frac{Q}{2.0}\right)^{2/3}=\left(\frac{250}{2.0}\right)^{2/3}=125^{2/3}=25.0;$$

$$A_c=(d_c\times 8)-\left(\frac{2\times 2}{2}\right)-\left(\frac{2\times 2}{2}\right)=8d_c-2-2=8d_c-4.$$

$$d_c=\frac{A_c+4}{8}=\frac{25.0+4.0}{8}=\frac{29.0}{8}=3.63.$$

$$h_4=5.25;\quad h_{c_3}=d_c+z=3.63+0.00=3.63.$$

$S_c>S_0$ and $h_4>h_c$; type 3 flow proved.

Use $Q=250$ cfs.

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