Techniques of Water-Resources Investigations of the United States Geological Survey

Chapter A3

MEASUREMENT OF PEAK DISCHARGE AT CULVERTS BY INDIRECT METHODS

By G. L. Bodhaine

Book 3

APPLICATIONS OF HYDRAULICS

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: $C(\text{adjusted})=0.98-(0.98-C)m/0.80$. 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

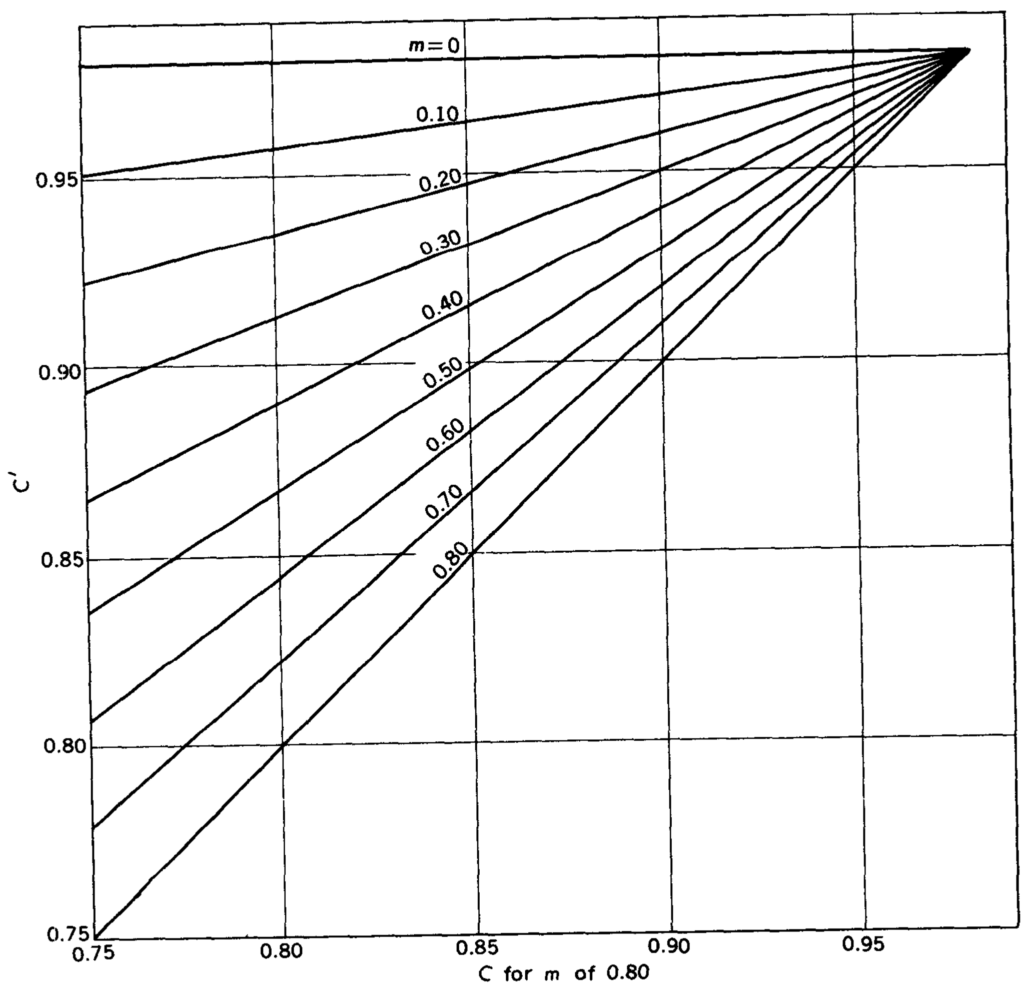


Figure 19.—Adjustment to discharge coefficient for degree of channel contraction.

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

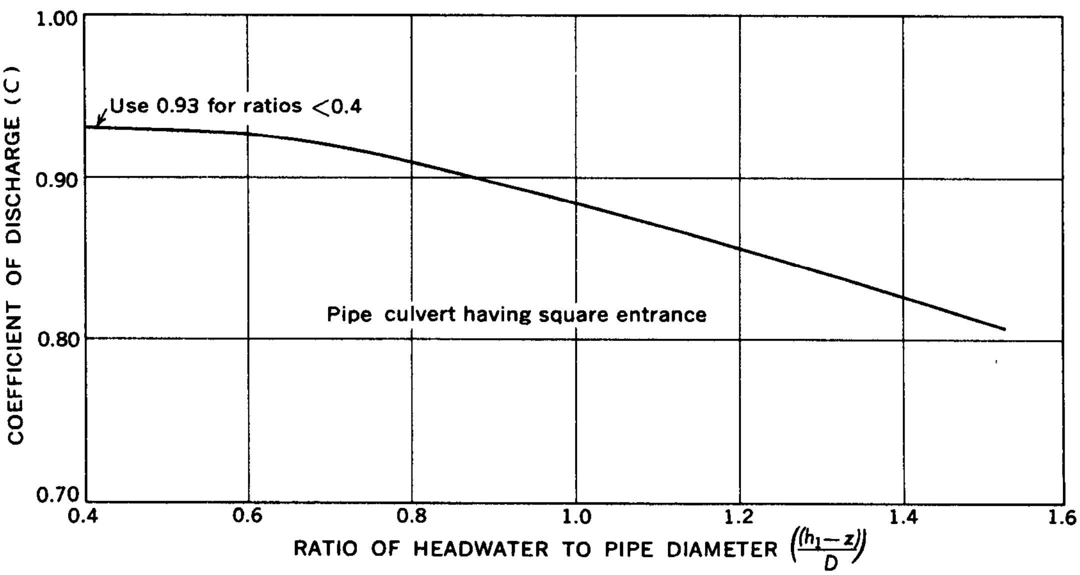


Figure 20.—Base coefficient of discharge for types 1, 2, and 3 flow in pipe culverts with square entrance mounted flush with vertical headwall.

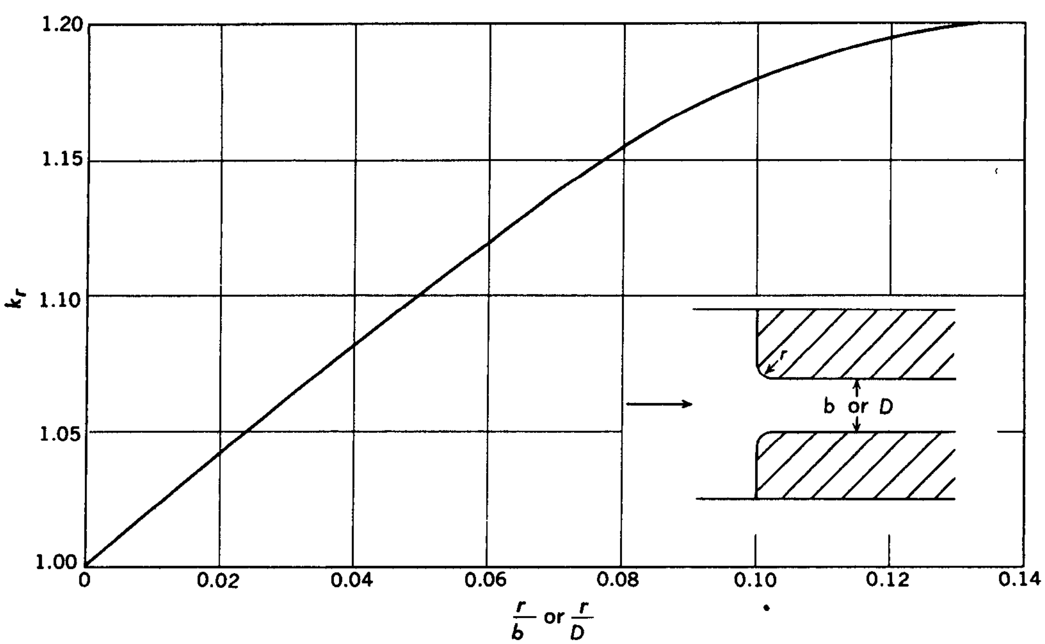


Figure 21.—Variation of the discharge coefficient with entrance rounding, types 1, 2, and 3 flow in box or pipe culverts set flush with vertical headwall.

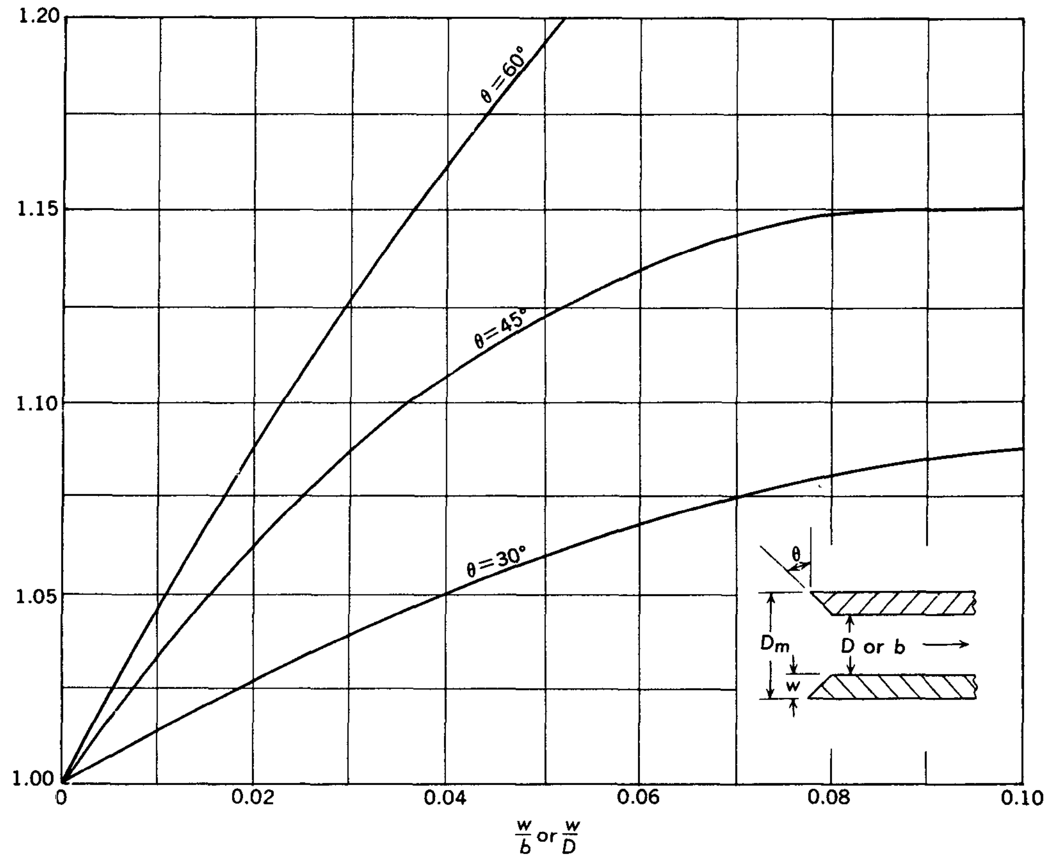


Figure 22.—Variation of the discharge coefficient with entrance beveling, types 1, 2, and 3 flow in box or pipe culverts set flush with vertical headwall.

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

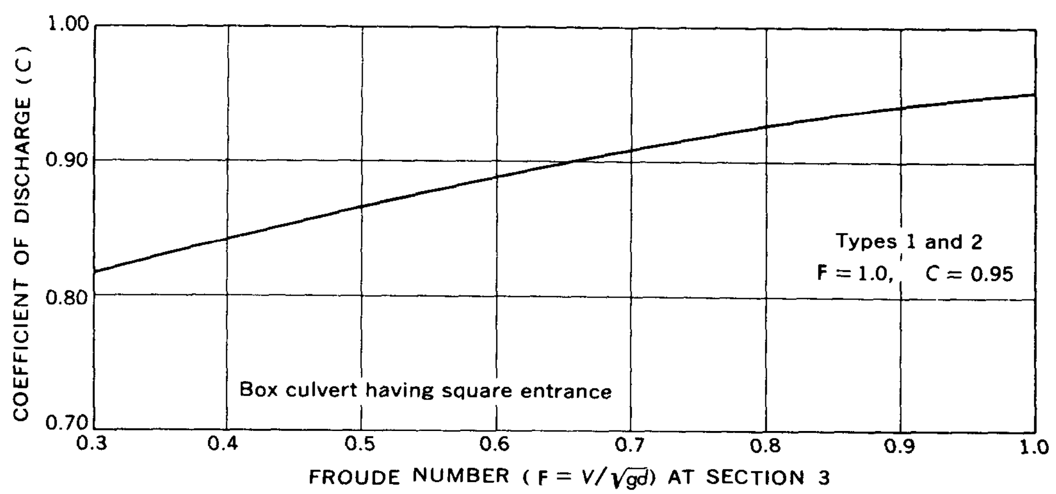


Figure 23.—Base coefficient of discharge for types 1, 2, and 3 flow in box culverts with square entrance mounted flush in vertical headwall.

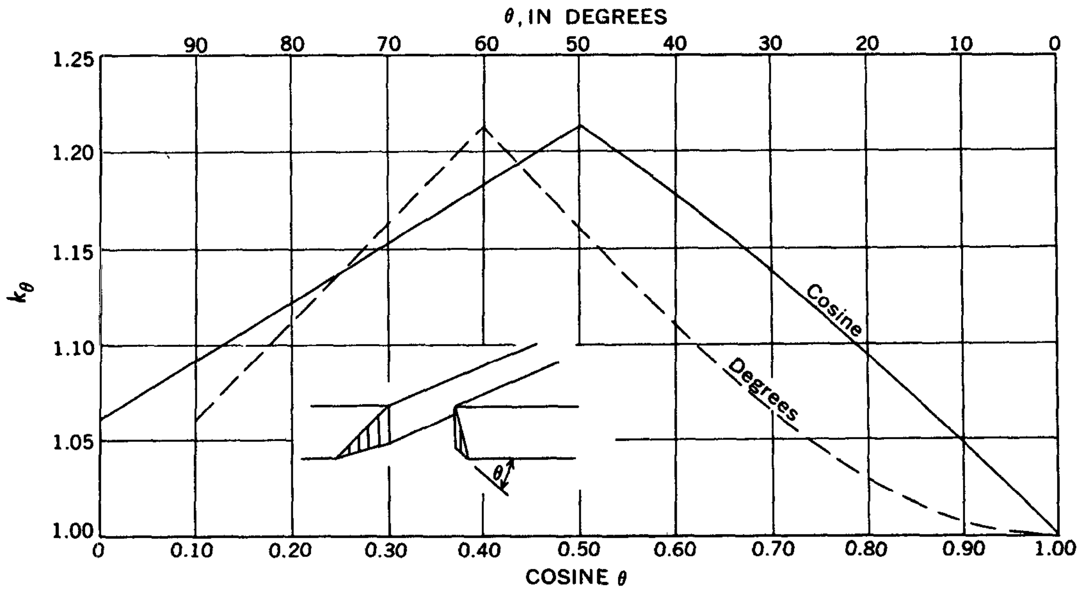


Figure 24.—Variation of discharge coefficient with wingwall angle, types 1, 2, and 3 flow in box culverts with wingwall set flush with sloping embankment.

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 $(h_1-z)/D$.

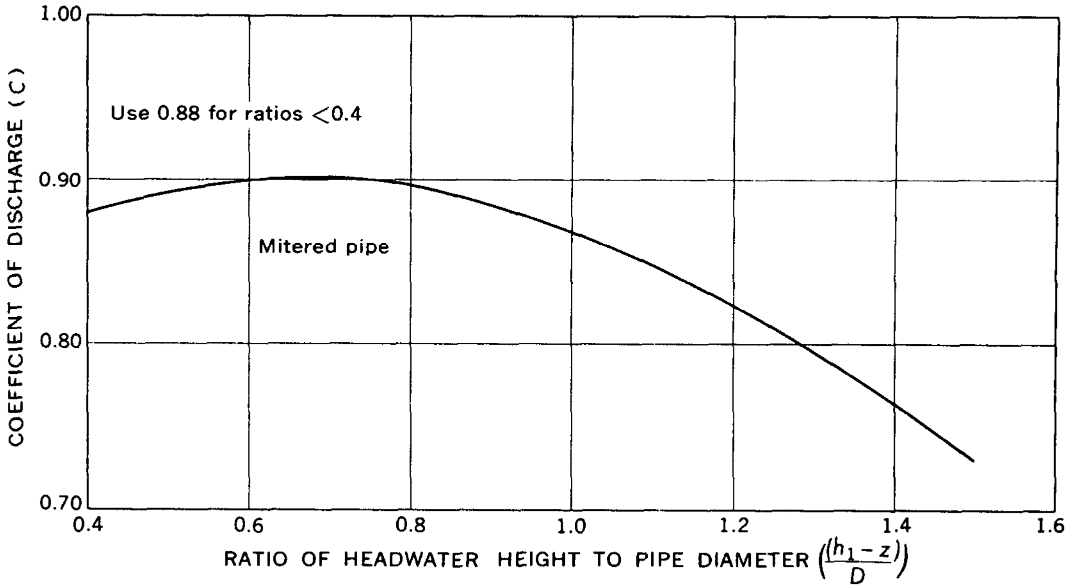


Figure 25.—Variation of discharge coefficient with headwater-diameter ratio, types 1, 2, and 3 flow in mitered pipe set flush with sloping embankment.

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

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

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.

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

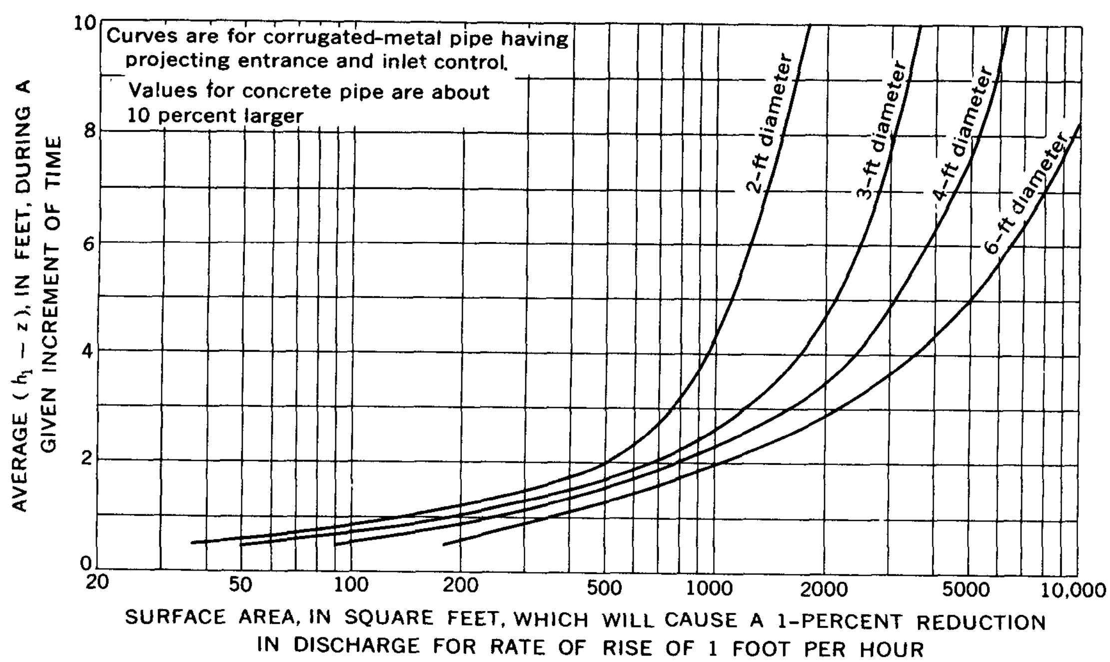


Figure 26.—Variation of headwater with surface area of pond for determining reduction in discharge for various sizes of culverts.

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

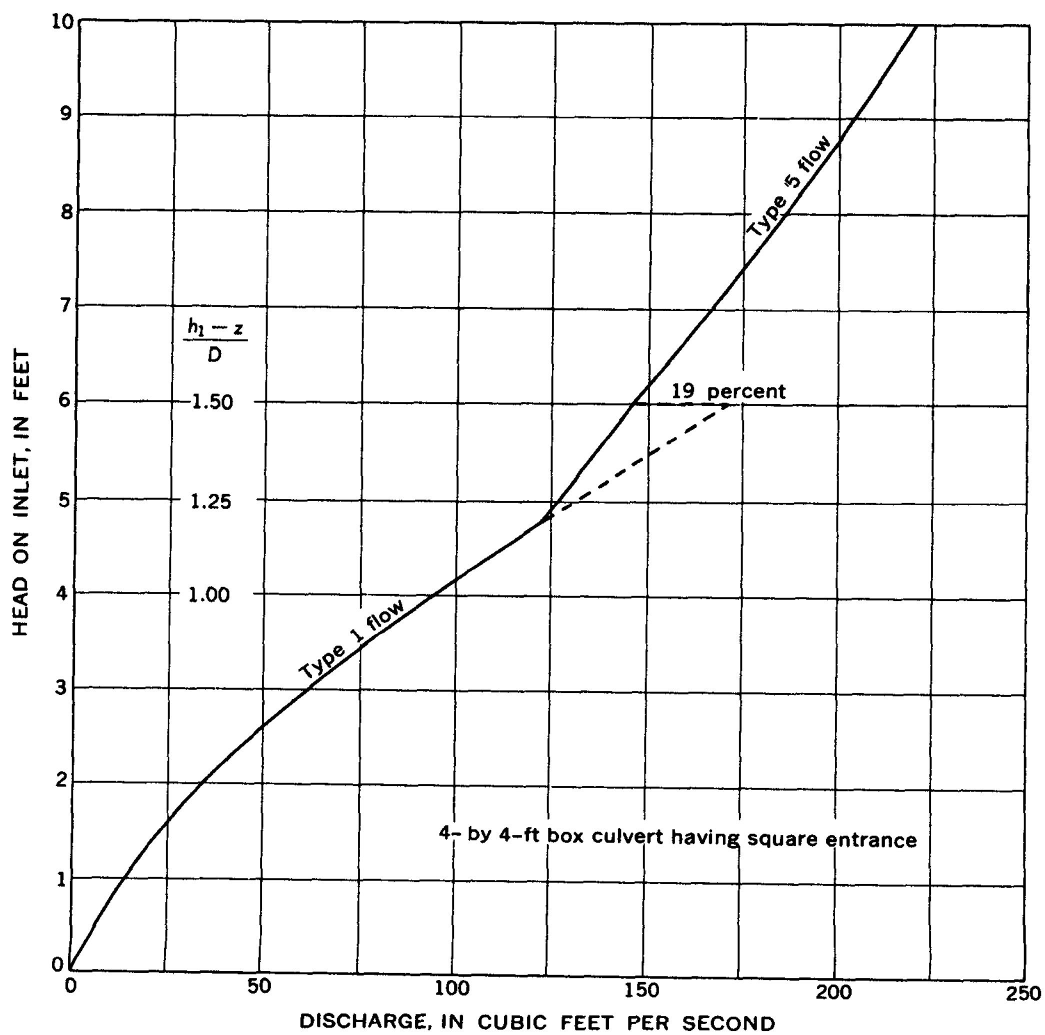


Figure 27.—Rating curve showing transition from type 1 to type 5 flow in a box culvert.

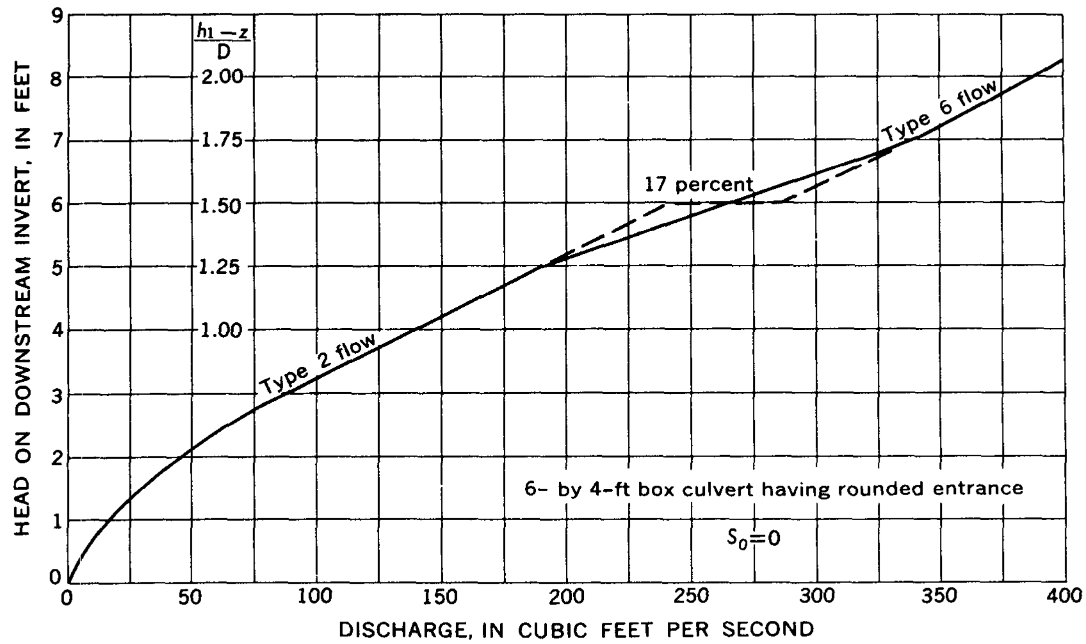


Figure 28.—Rating curve showing transition from type 2 to type 6 flow in a box culvert.

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

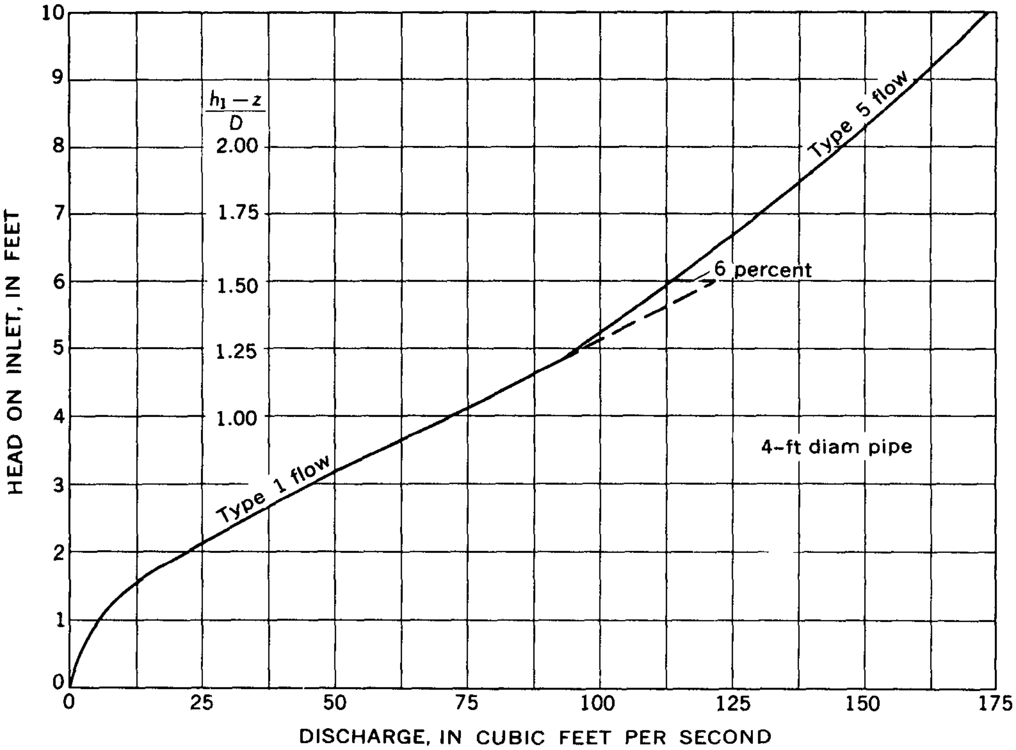


Figure 29.—Rating curve showing transition from type 1 to type 5 flow in a pipe culvert.

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

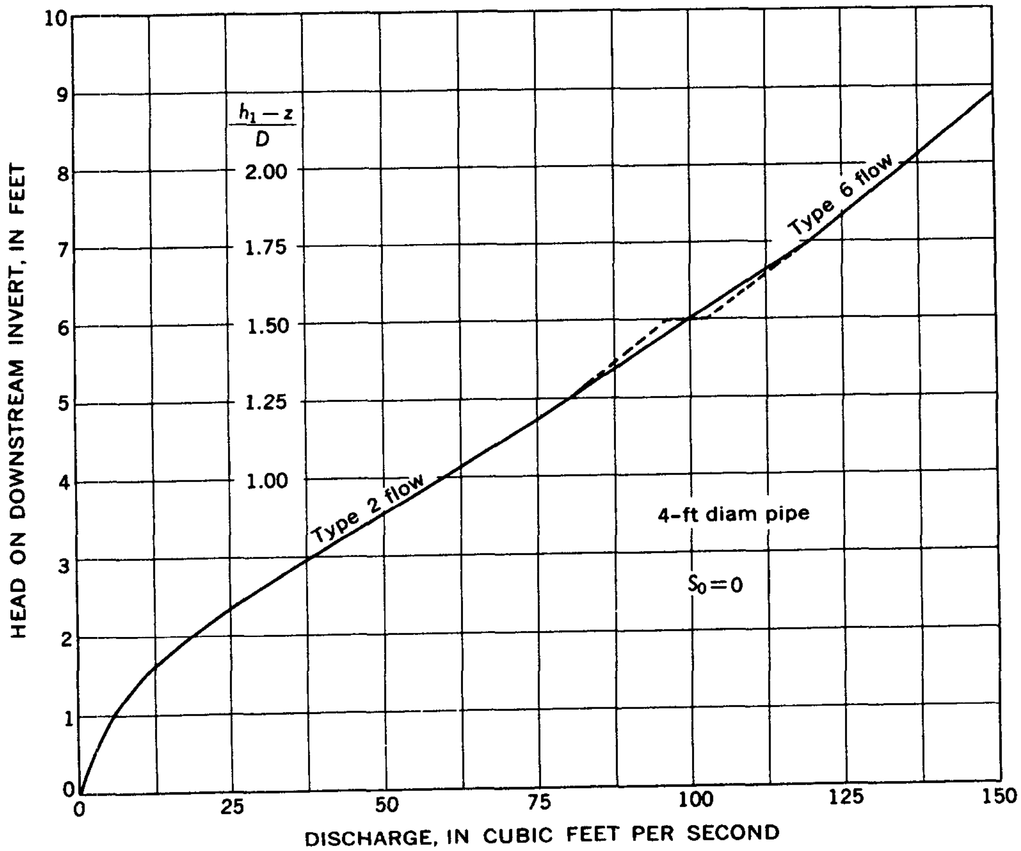


Figure 30.—Rating curve showing transition from type 2 to type 6 flow in a pipe culvert.

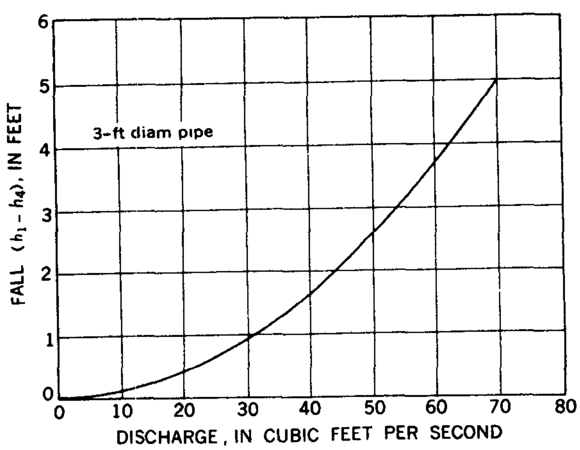


Figure 31.—Rating curve for type 4 flow in a pipe culvert.

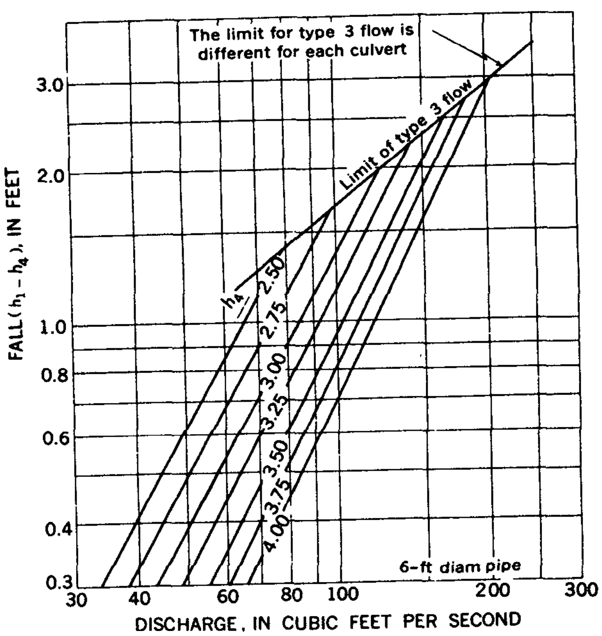


Figure 32.—Rating curve for type 3 flow in a pipe culvert.

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.

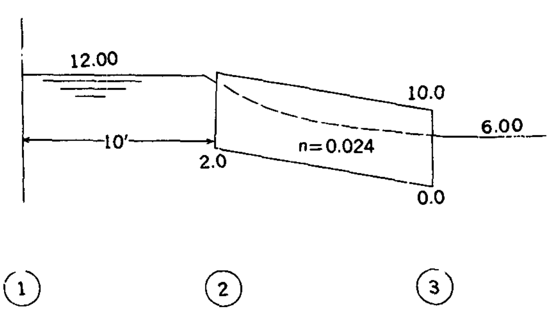
Flow depths below the spring line of pipe-arches may be considerably affected by minor distortions in shape. For this reason, shallow-depth computations are considered less reliable than for conditions of more nearly full flow.

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

   $=0.894\times54.04$

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

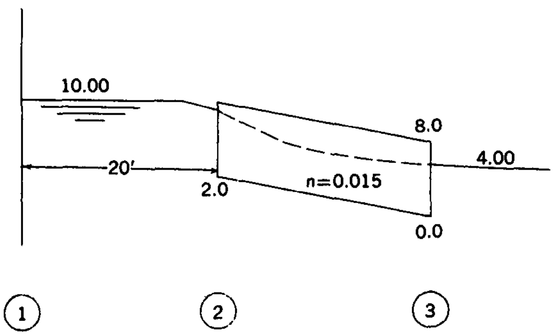
   $=387\sqrt{3.51}=725$ cfs. (Estimated 729 cfs.)

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

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

$S_c<S_0$ 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

$K_1=38{,}900$

$L=100'$

$S_0=0.02$

$d_c=0.66\times 8.00=5.28.$

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

$A=5.28\times 8.00=42.3;$

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

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

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

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. $Q=5.67bd_c^{3/2}=5.67\times 8\times 11.69=530$ cfs.
5. $\frac{V_1^2}{2g}=\frac{1.61^2}{2g}=0.04.$

   $h_{f_{1-2}}=\frac{Q^2L_w}{K_1K_c}=\frac{530^2\times 20}{38{,}900\times 7{,}000}=0.02.$
6. $Q=CA_c\sqrt{2g(h_1-z+V_1^2/2g-d_c-h_{f_{1-2}})}$

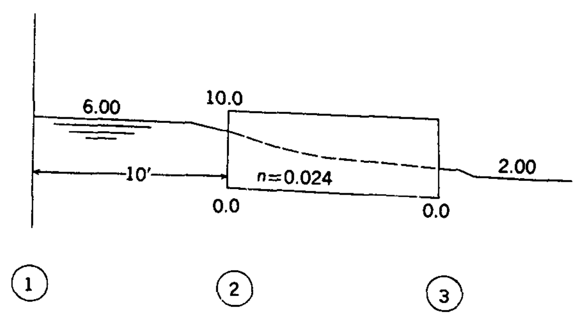
   $=0.95\times 41.1\times 8.02\sqrt{10.00-2.00+0.04-5.14-0.02}$

   $=313\sqrt{2.88}=531$ 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.

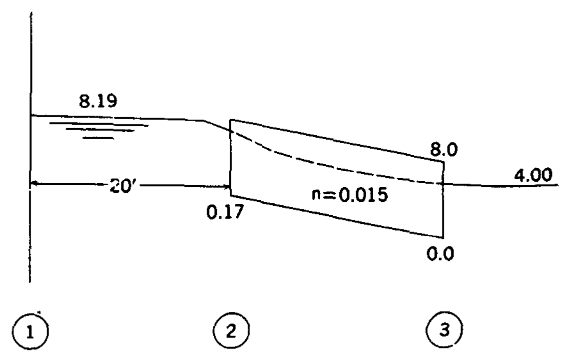
Test for type 2 flow:

$h_c=d_c=3.85$;

$h_4=2.00$;

$S_0<S_c$ 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. $\dfrac{Q^2}{2g(h_1-z)^3b^2C^2}=\dfrac{533^2}{2g\times 8.02^3\times 8^2\times 0.95^2}$
   $=0.149$.

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

   $d_2=0.660\times 8.02=5.30$.

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

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

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

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

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

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

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

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

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

    $d_2=0.75\times 8.02=6.01.$

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

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

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

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:

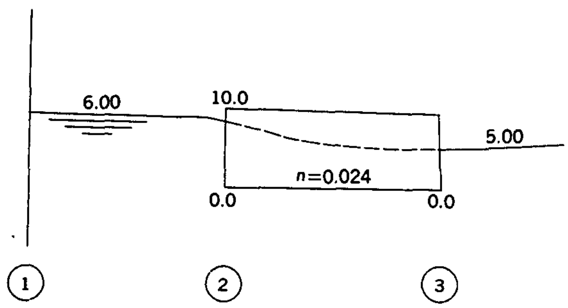
$h_c=d_3=5.07>h_4=4.00;$

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

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

$S_0<S_c$; 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.

$D=10'$

$h_1=6.00'$

$h_4=5.00'$

$L_{2-3}=100'$

$h_3=h_4=5.00'$

$z=0$

$h_1-z=6.00$

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

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

$r/D=0.006$

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

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

$A_3=0.3927\times 10^2=39.27.$

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

$K_3=0.2317\times 19{,}330=4{,}480.$

From figure 20, $C=0.928$.

From figure 21, $k_r=1.012$.

Adjusted $C=1.012\times 0.928=0.939$.

1. Assume $Q=230$ cfs.

   Assume $d_2=5.50$.

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

   $A_2=0.4426\times 10^2=44.26.$

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

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. $K_2=0.2710\times 19{,}330=5{,}240.$

4. $h_{f_{2-3}}=\frac{Q^2L}{K_2K_3}=\frac{230^2\times 100}{5{,}240\times 4{,}480}=0.23.$

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

   $Q=0.939\times 39.27$

   $\times 8.02\sqrt{6.00+0-5.00-0-0.23}$

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

6. Assume $Q=251$ cfs.

   Assume $d_2=5.40$.

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

   $A_2=0.4327\times 10^2=43.27.$

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

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. $K_2=0.2630\times 19{,}330=5{,}090.$

9. $h_{f_{2-3}}=\frac{Q^2L}{K_2K_3}=\frac{251^2\times 100}{5{,}090\times 4{,}480}=0.28.$

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:

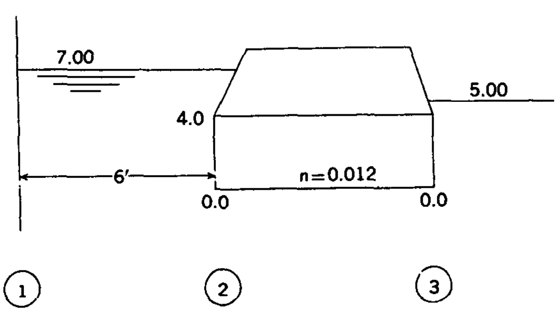
    $S_0=0.00;\ S_c>S_0.$

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

    $\frac{d_c}{D}=0.374;\ d_c=0.374\times 10=3.74=h_c.$

    $h_4>h_c$; 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, $C=0.955$.

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

$$=0.955\times12.6$$

$$\times8.02\sqrt{\frac{7.00-5.00}{1+\frac{29\times0.955^2\times0.012^2\times50}{1.00}}}$$

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

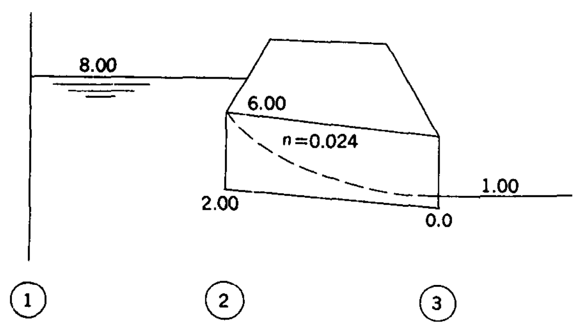
For additional type 4 computations at this site the following equation may be used:

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

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.

$r/D=0.016$

$h_1=8.00'$

$h_4=1.00'$

$h_1-z=6.00'$

$z=2.00'$

$L_{2\text{-}3}=50'$

$D=4.00'$

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).

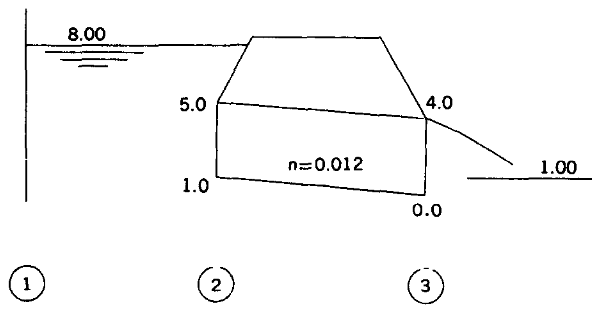
3. $\frac{29n^2(h_1-z)}{R_0^{4/3}}=\frac{29\times0.024^2\times6.00}{1.00}=0.10$.

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. $Q=CA_0\sqrt{2g(h_1-z)}$
   $=0.484\times12.6\times8.02\sqrt{6.00}=120$ cfs.

Example 8. Type 6 flow through a concrete pipe culvert



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

$w/D=0.3/4=0.075$

$h_1=8.00'$

$h_4=1.00'$

$h_1-z=7.00'$

$z=1.0'$

$L_{2\text{-}3}=50'$

$D=4.0'$

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. $\frac{29n^2L}{R_0^{4/3}}=\frac{29\times0.012^2\times50}{1.0}=0.209$.

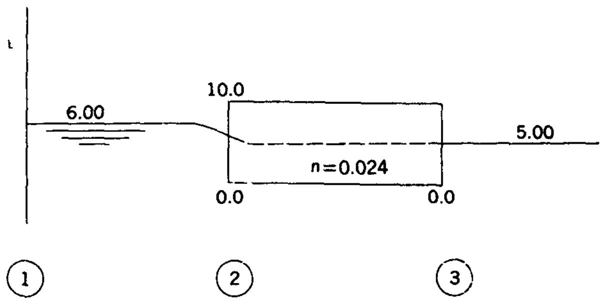
4. From figure 17, factor $k_f=1.54$.

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:

$S_0 = 0.00.$

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

$$Q = C_q D^{5/2};$$

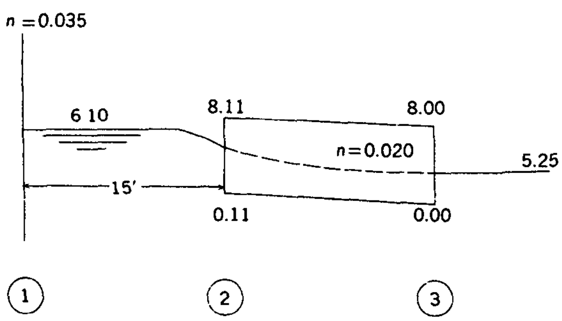
$$C_q = \frac{Q}{D^{5/2}} = \frac{251}{10^{5/2}} = \frac{251}{316} = 0.795.$$

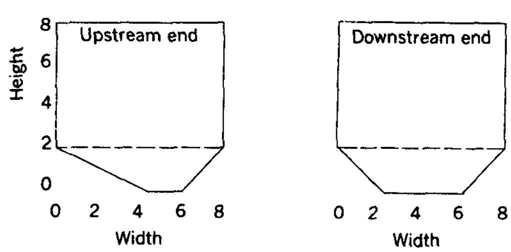
From table 3,

$$\frac{d_c}{D} = 0.374;\ d_c = 0.374 \times 10 = 3.74.$$

$S_c > S_0$ 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

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

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

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

$K_1 = 9{,}210$

$A_3 = (5.25 \times 8) - (2 \times 2)$
$= 42.00 - 4 = 38.0.$

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

$R_3 = 38/16.16 = 2.35;$

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

$K_3 = 5{,}000.$

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

1. Assume type 3 flow.

2. Assume $Q = 250$ cfs.

3. $\mathbf{F}_3 = \dfrac{V_3}{\sqrt{g d_{m_3}}} = \dfrac{6.58}{5.67\sqrt{4.75}} = 0.532.$

   From figure 23, $C = 0.874$.

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

   Adjusted $C = 1.03 \times 0.874 = 0.90$.

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

   (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 $d_{m_2} = 4.45$.

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

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

   $K_2 = 74.3 \times 35.6 \times 1.73 = 4{,}580.$

   $$h_{f_{2\text{-}3}} = \frac{Q^2 L}{K_2 K_3} = \frac{250^2 \times 60}{4{,}580 \times 5{,}000} = 0.16.$$

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

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:

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

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

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

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

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

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

Use $Q=250$ cfs.

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