The interception efficiency is then

$$E = R_f E_0 + R_s(1 - E_0) = [1 \times 0.35 + 0.20(1 - 0.35)] = 0.48$$

The interception capacity is $Q_i = EQ = 0.48 \times 10 = 4.8$ ft³/s, and $Q_b = 10 - 4.8 = 5.2$ ft³/s. A berm could be placed downstream of the grate inlet for total interception of flow in the ditch.

16.2 HYDRAULIC DESIGN OF CULVERTS

Culverts are hydraulically short closed conduits that convey streamflow through a road embankment or some other type of flow obstruction. The flow in culverts may be full flow over all its length or partly full, resulting in pressurized flow and/or open-channel flow. The characteristics of flow in culverts are very complicated because the flow is controlled by many variables, including inlet geometry, slope, size, flow rate, roughness, and approach and tailwater conditions.

Culverts have numerous cross-sectional shapes, including circular, box (rectangular), elliptical, pipe arch, and arch. Shape selection is typically based upon cost of construction, limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance. Culverts are also made of numerous materials, depending upon structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. Concrete, corrugated aluminum, and corrugated steel are the three most common.

Various types of inlets are also used for culverts, including both prefabricated and constructed-in-place inlets. Some of the commonly used inlets are illustrated in Figure 16.2.1. Inlet design is important because the hydraulic capacity of a culvert may be improved by the appropriate inlet selection. Natural channels are usually much wider than the culvert barrel, so that the inlet is a flow contraction and can be the primary flow control.

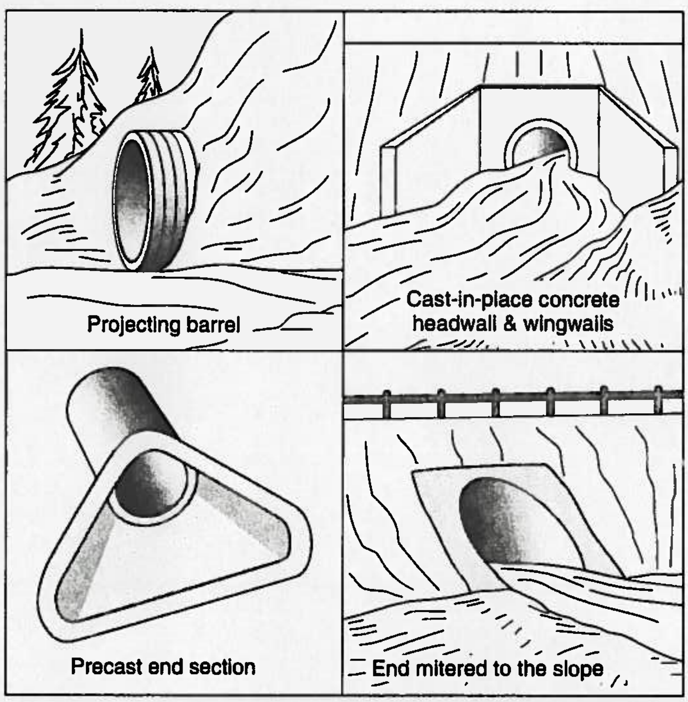


Figure 16.2.1 Four standard inlet types (schematic) (from Normann et al. (1985)).

16.2.1 Culvert Hydraulics

16.2.1.1 Types of Control

There are two basic types of flow control in culverts: ***inlet control*** and ***outlet control.*** Culverts with inlet control have high-velocity shallow flow that is supercritical, as shown in Figure 16.2.2. The control section is at the upstream end (inlet) of the culvert barrel. Culverts with outlet control have lower velocity, deeper flow that is subcritical as shown in Figure 16.2.3. The control section is at the downstream end (outlet) of the culvert barrel. Tailwater depths are either critical depth or higher.

Figure 16.2.2 illustrates four different examples of inlet control that depend on the submergence of the inlet and outlet ends of the culvert. In Figure 16.2.2a, neither end of the culvert is submerged. Flow passes through critical depth just downstream of the culvert entrance with supercritical flow in the culvert barrel. Partly full flow occurs throughout the length of the culvert, approaching normal depth at the outlet.

In Figure 16.2.2b, the outlet is submerged and the inlet is unsubmerged. The flow just downstream of the inlet is supercritical and a hydraulic jump occurs in the culvert barrel. In Figure 16.2.2c, the inlet is submerged and the outlet is unsubmerged. Supercritical flow occurs throughout the length of the culvert barrel, with critical depth occurring just downstream of the culvert entrance. Flow approaches normal depth at the downstream end. This flow condition is typical of design conditions. Figure 16.2.2d shows an unusual condition in which submergence occurs at both ends of the culvert with a hydraulic jump occurring in the culvert barrel. Note the median inlet, which provides ventilation of the culvert barrel.

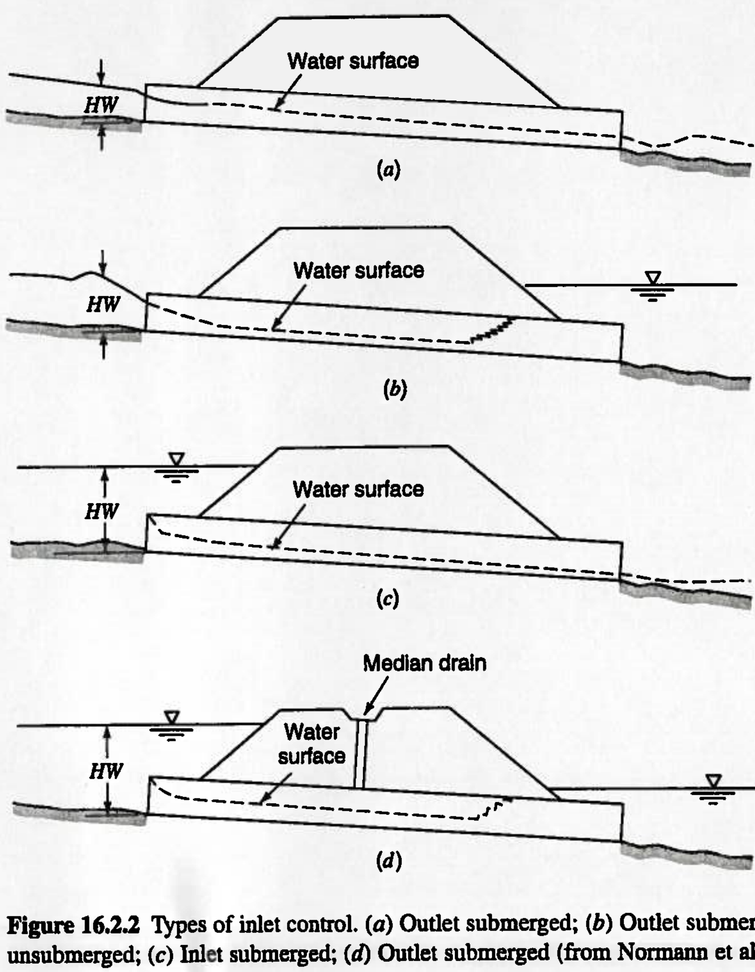


Figure 16.2.2 Types of inlet control. (*a*) Outlet submerged; (*b*) Outlet submerged, inlet unsubmerged; (*c*) Inlet submerged; (*d*) Outlet submerged (from Normann et al. (1985)).

Figure 16.2.3 illustrates five flow conditions for outlet control. Subcritical flow occurs for the partly full flow conditions. Figure 16.2.3a is the classic condition with both the inlet and the outlet submerged, with pressurized flow throughout the culvert. In Figure 16.2.3b, the outlet is submerged and the inlet is unsubmerged. In Figure 16.2.3c, the entrance is submerged enough that full flow occurs throughout the culvert length but the exit is unsubmerged. Figure 16.2.3d is a typical condition in which the entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. The culvert barrel flows partly full part of the length with subcritical flow and passes through critical just upstream of the outlet. Figure 16.2.3e is another typical condition in which neither the inlet nor the outlet is submerged. The flow is subcritical and partly full throughout the length of the culvert barrel.

16.2.1.2 Inlet-Control Design Equations

A culvert under inlet-control conditions performs as an orifice when the inlet is submerged and as a weir when it is unsubmerged. The (*submerged*) *orifice discharge equation* is computed using

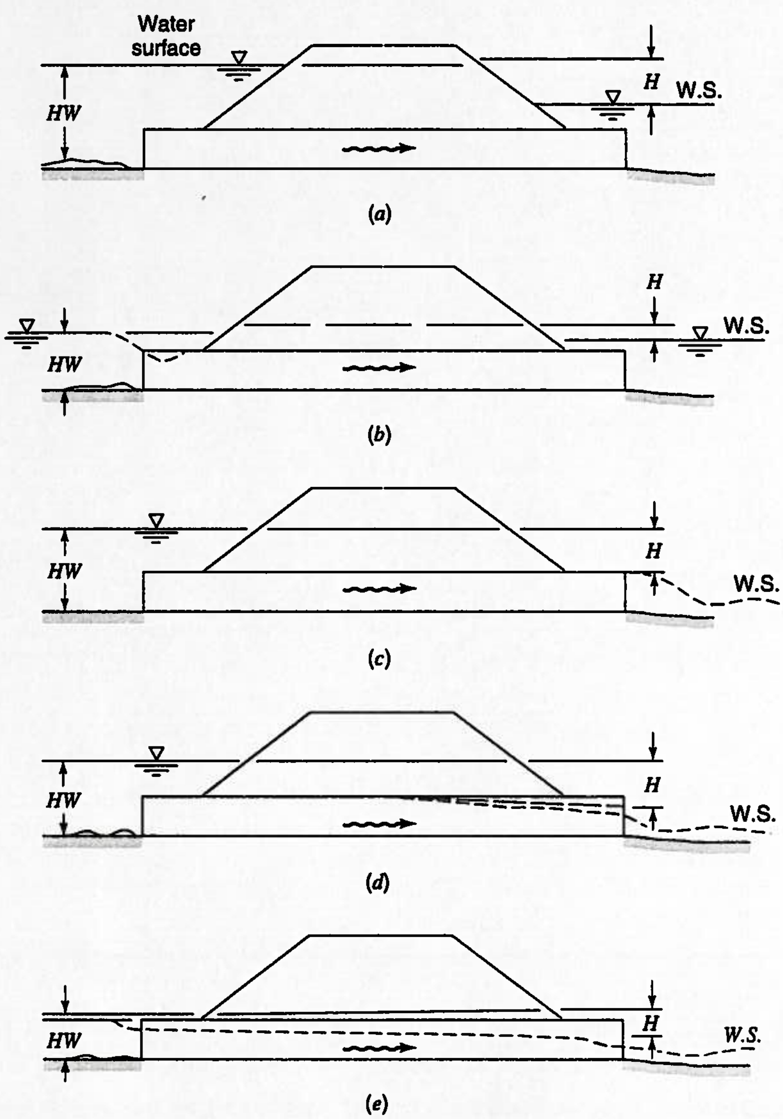


Figure 16.2.3 Types of outlet control (from Normann et al. (1985)).

$$\left[\frac{HW}{D}\right] = C\left[\frac{Q}{AD^{0.5}}\right]^2 + Y + Z \quad \text{for} \quad \left[\frac{Q}{AD^{0.5}}\right] \geq 4.0 \qquad (16.2.1)$$

where HW is the headwater depth above the inlet control section invert (ft), D is the interior height of the culvert barrel (ft), Q is the discharge (ft^3/s), A is the full cross-sectional area of the culvert barrel in (ft^2), S_0 is the culvert barrel slope (ft/ft), C and Y are constants from Table 16.2.1, and Z is the slope correction factor where $Z = -0.5S_0$ in general and $Z = +0.7S_0$ for mitered inlets.

The *(unsubmerged) weir discharge equation* is (Form 1):

$$\left[\frac{HW}{D}\right] = \left[\frac{H_c}{D}\right] + K\left[\frac{Q}{AD^{0.5}}\right]^M + Z \quad \text{for} \quad \left[\frac{Q}{AD^{0.5}}\right] \leq 3.5 \qquad (16.2.2)$$

where H_c is the specific head at critical depth ($H_c = d_c + V_c^2/2g$) (ft), d_c is the critical depth (ft), V_c is the critical velocity (ft/s), and K and M are constants in Table 16.2.1. A simpler equation to use for the unsubmerged condition is (Form 2):

$$\left[\frac{HW}{D}\right] = K\left[\frac{Q}{AD^{0.5}}\right]^M + Z \quad \text{for} \quad \left[\frac{Q}{AD^{0.5}}\right] \leq 3.5 \qquad (16.2.3)$$

Form 2 is easier to apply and is the only documented form for some of the design inlet control nomographs in Normann et al. (1985).

Equations (16.2.1) to (16.2.3) are implemented by assuming a culvert diameter D and using it on the right-hand side of these equations and solving for $[HW/D]$. The headwater depth is then obtained by multiplying $D[HW/D]$. Typical inlet-control nomographs are presented in Figures 16.2.4 and 16.2.5.

Table 16.2.1 Constants for Inlet Control Design Equations

Chart[a] No.	Shape and Material	Nomograph Scale	Inlet EdgeDescription	Equation Form[b]	Unsubmerged K	Unsubmerged M	Submerged C	Submerged Y
1	Circular concrete	1	Square edge w/headwall	1	0.0098	2.0	0.0398	0.67
		2	Groove and w/headwall		.0078	2.0	.0292	.74
		3	Groove and projecting		.0045	2.0	.0317	.69
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69
		2	Mitered to slope		.0210	1.33	.0463	.75
		3	Projecting		.0340	1.50	.0553	.54
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74
		B	Beveled ring, 33.7° bevels		.0018	2.50	.0243	.83
8	Rectangular box	1	30° to 75° wingwall flares		.026	1.0	.0385	.81
		2	90° and 15° wingwall flares	1	.061	0.75	.0400	.80
		3	0° wingwall flares		.061	0.75	.0423	.82
9	Rectangular box	1	45° wingwall flare d = .0430	2	.510	.667	.0309	.80
		2	18° to 33.7° wingwall flare d = .0830		.486	.667	.0249	.83
10	Rectangular box	1	90° headwall w/3/4" chamfers	2	.515	.667	.0375	.79
		2	90° headwall w/45° bevels		.495	.667	.0314	.82
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865
11	Rectangular box	1	3/4" chamfers; 45° skewed headwall	2	.522	.667	.0402	.73
		2	3/4" chamfers; 30° skewed headwall		.533	.667	.0425	.705
		3	3/4" chamfers; 15° skewed headwall		.545	.667	.04505	.68
			45° bevels; 10°–45° skewed headwall		.498	.667	.0327	.75

Table 16.2.1 Constants for Inlet Control Design Equations (*continued*)

Chart[a] No.	Shape and Material	Nomograph Scale	Inlet EdgeDescription	Equation Form[b]	Unsubmerged K	Unsubmerged M	Submerged C	Submerged Y
12	Rectangular	1	45° non offset wingwall flares	2	.497	.667	.0339	.805
	box	2	18.4° non offset wingwall flares		.493	.667	.0361	.806
	3/4" chamfers	3	18.4° non offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71
13	Rectangular	1	45° wingwall flares — offset	2	.495	.667	.0302	.835
	box	2	33.7° wingwall flares — offset		.493	.667	.0252	.881
	Top bevels	3	18.4° wingwall flares — offset		.497	.667	.0227	.887
16–19	CM boxes	1	90° headwall	1	.0083	2.0	.0379	.69
		2	Thick wall projecting		.0145	1.75	.0419	.64
		3	Thin wall projecting		.0340	1.5	.0496	.57
29	Horizontal	1	Square edge with headwall	1	.0100	2.0	.0398	.67
	ellipse	2	Groove end with headwall		.0018	2.5	.0292	.74
	concrete	3	Groove end projecting		.0045	2.0	.0317	.69
30	Vertical	1	Square edge with headwall	1	.0100	2.0	.0398	.67
	ellipse	2	Groove end with headwall		.0018	2.5	.0292	.74
	concrete	3	Groove end projecting		.0095	2.0	.0317	.69
34	Pipe arch	1	90° headwall	1	.0083	2.0	.0379	.69
	18" corner	2	Mitered to slope		.0300	1.0	.0463	.75
	radius CM	3	Projecting		.0340	1.5	.0496	.57
35	Pipe arch	1	Projecting	1	.0296	1.5	.0487	.55
	18" corner	2	No. bevels		.0087	2.0	.0361	.66
	radius CM	3	33.7° bevels		.0030	2.0	.0264	.75
36	Pipe arch	1	Projecting	1	.0296	1.5	.0487	.55
	31" corner		No. bevels		.0087	2.0	.0361	.66
	radius CM		33.7° bevels		.0030	2.0	.0264	.75
40–42	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69
		2	Mitered to slope		.0300	1.0	.0463	.75
		3	Thin wall projecting		.0340	1.5	.0496	.57
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.89
		2	Rough tapered inlet throat		.519	.64	.0289	.90
56	Elliptical	1	Tapered inlet—beveled edges	2	.536	.622	.0368	.83
	Inlet face	2	Tapered inlet—square edges		.5035	.719	.0478	.80
		3	Tapered inlet—thin edge projecting		.547	.80	.0598	.75
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97
58	Rectangular	1	Side tapered—less favorable edges	2	.56	.667	.0466	.85
	concrete	2	Side tapered—more favorable edges		.56	.667	.3978	.87
59	Rectangular	1	Slope tapered—less favorable edges	2	.50	.667	.0466	.65
	concrete		Slope tapered—more favorable edges		.50	.667	.0378	.71

[a]Chart number in Normann et al. (1985)
[b]Form 1 is equation (16.2.2)
Form 2 is equation (16.2.3)

Source: Normann et al. (1985).

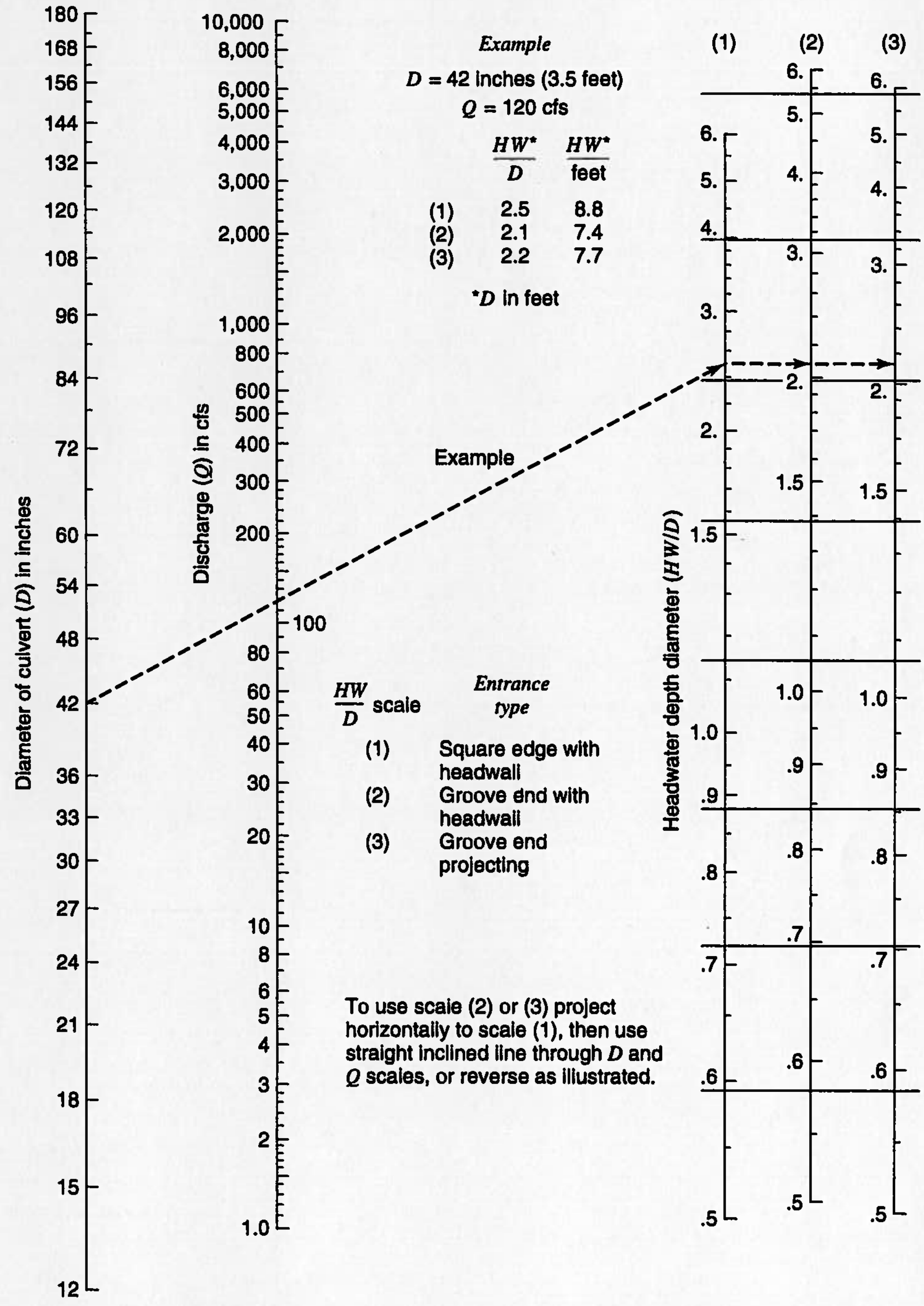


Figure 16.2.4 Headwater depth for concrete pipe culverts with inlet control (from Normann et al. (1985)).

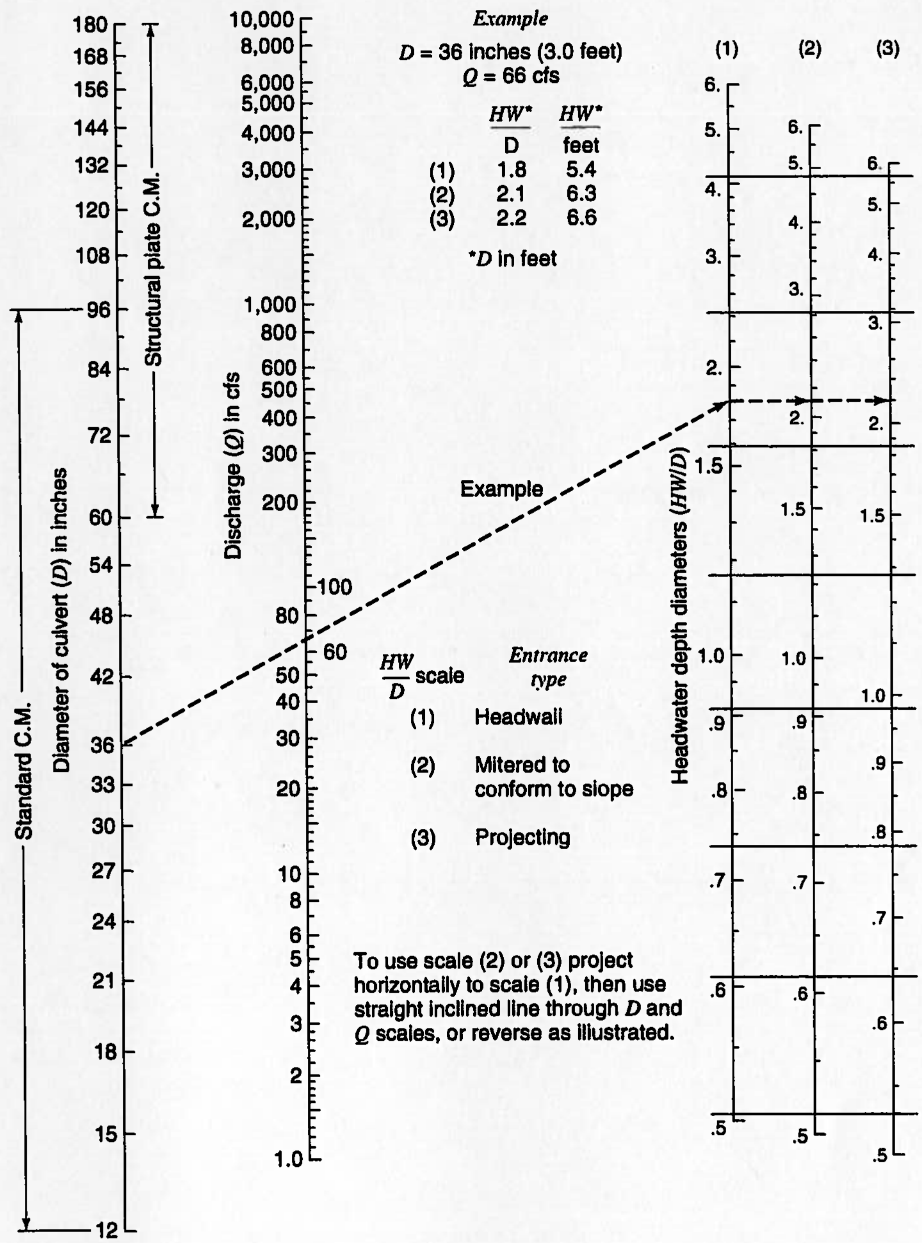


Figure 16.2.5 Headwater depth for CM pipe culverts with inlet control (from Normann et al. (1985)).

16.2.1.3 Outlet-Control Design Equations

A culvert under outlet-control conditions has either subcritical flow or full-culvert flow, so that outlet-control flow conditions can be calculated using an energy balance. For the condition of full-culvert flow, considering entrance loss H_e, friction loss (using Manning's equation) H_f, and exit loss H_0, the total headloss H is

$$H = H_0 + H_e + H_f \tag{16.2.4a}$$

and in U.S. customary units is

$$H = \left[1 + K_e + \left(\frac{29n^2L}{R^{1.33}}\right)\right]\frac{V^2}{2g} \tag{16.2.4b}$$

or in SI units is

$$H = \left[1 + K_e + \left(\frac{20n^2L}{R^{1.33}}\right)\right]\frac{V^2}{2g} \tag{16.2.4c}$$

where K_e is the entrance loss coefficient, n is Manning's roughness coefficient, R is the hydraulic radius of the full-culvert barrel in ft (m), V is the velocity in ft/s (m/s), and L is the culvert length in ft (m). Other losses such as bend losses H_b, junction losses H_j, and grate losses H_g can also be added to equation (16.2.4). Table 16.2.2 lists common values of Manning's n values for culverts. Table 16.2.3 lists entrance loss coefficients for outlet control, full or part full flow.

Table 16.2.2 Manning n Values for Culverts*

Type of Conduit	Wall Description	Manning n
Concrete pipe	Smooth walls	0.010–0.013
Concrete boxes	Smooth walls	0.012–0.015
Corrugated metal pipes and boxes, annular or helical pipe (Manning n varies with barrel size)	2 2/3" by 1/2" corrugations	0.022–0.027
	6" by 1" corrugations	0.022–0.025
	5" by 1" corrugations	0.025–0.026
	3" by 1" corrugations	0.027–0.028
	6" by 2" structural plate corrugations	0.033–0.035
	9" by 2 1/2" structural plate corrugations	0.033–0.037
Corrugated metal pipes, helical corrugations, full circular flow	2 2/3" by 1/2" corrugations	0.012–0.024
Spiral rib metal pipe	Smooth walls	0.012–0.013

*Note: The values indicated in this table are recommended Manning n design values. Actual field values for older existing pipelines may vary depending on the effects of abrasion, corrosion, deflection, and joint conditions. Concrete pipe with poor joints and deteriorated walls may have n values of 0.014 to 0.018. Corrugated metal pipe with joint and wall problems may also have higher n values, and in addition may experience shape changes which could adversely affect the general hydraulic characteristics of the pipeline.

Source: Normann et al. (1985).

Table 16.2.3 Entrance Loss Coefficients for Outlet Control, Full or Partly Full $H_e = K_e\,[V^2/2g]$

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, concrete	
Mitered to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Socket end of pipe (groove-end)	0.2
Projecting from fill, socket end (groove-end)	0.2
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2

Table 16.2.3 Entrance Loss Coefficients for Outlet Control, Full or Partly Full $H_e = K_e\,[V^2/2g]$ *(continued)*

Type of Structure and Design of Entrance	Coefficient K_e
Pipe or pipe-arch, corrugated metal	
Projecting from fill (no headwall)	0.9
Mitered to conform to fill slope, paved or unpaved slope	0.7
Headwall or headwall and wingwalls square-edge	0.5
*End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, reinforced concrete	
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Wingwalls at 10° to 25° or 30° to 75° to barrel	
Square-edged at crown	0.5
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Side- or slope-tapered inlet	0.2

*Note: "End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both *inlet* and *outlet* control. Some end sections, incorporating a *closed* taper in their design, have superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source: Normann et al. (1985).

Figure 16.2.6 illustrates the energy and hydraulic grade lines for full flow in a culvert. Equating the total energy at section 1 (upstream) and section 2 (downstream) gives

$$HW_0 = \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_f + H_e + H_0 \tag{16.2.5}$$

where HW_0 is the headwater depth above the outlet invert and TW is the tailwater depth above the outlet invert. Neglecting the approach velocity head and the downstream velocity head (Figure 16.2.6), equation (16.2.5) reduces to

$$HW_0 = TW + H_f + H_e + H_0 \tag{16.2.6}$$

For full flow $TW \geq D$; however, for partly full flow, the headloss should be computed from a backwater analysis. An empirical equation for the head loss H for this condition is

$$H = HW_0 - h_0 \tag{16.2.7}$$

where $h_0 = \max\,[TW, (D + d_c)/2]$.

The outlet-controlled headwater depth can be computed by first determining the tailwater depth from backwater computations where TW is measured above the outlet invert. By using equation (16.2.4) for full-flow conditions the headloss H is obtained. With equation (16.2.7) the *required outlet-controlled headwater elevation* H is obtained as

$$HW = H + h_0 - LS_0 \tag{16.2.8}$$

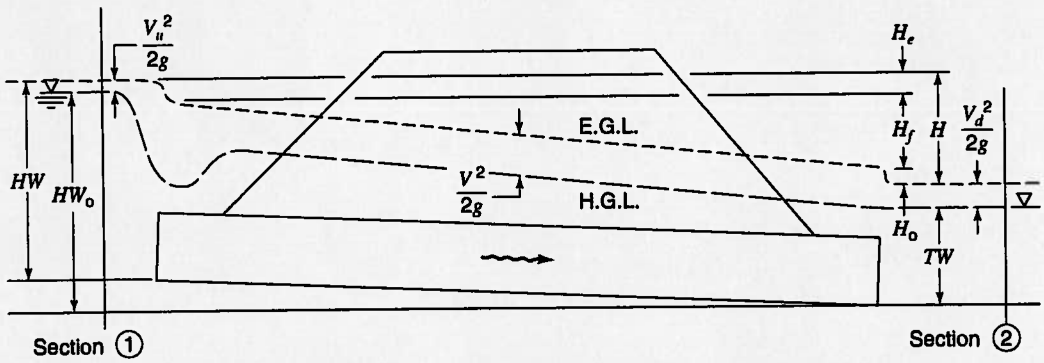


Figure 16.2.6 Full-flow energy and hydraulic grade lines (from Normann et al. (1985)).

Sample outlet-control nomographs are shown in Figures 16.2.7 and 16.2.8. Using the value of H from these nomographs, equation (16.2.8) can be implemented to compute HW_0. For Manning's n value different from that of the outlet nomograph, a modified length L_1 is used as the length scale:

$$L_1 = L\left(\frac{n_1}{n}\right)^2 \tag{16.2.9}$$

where L is the actual culvert length, n_1 is the desired Manning's n, and n is the Manning n from the chart.

The larger of the headwater elevation, obtained from the inlet- and outlet-control calculation, is adopted as the design headwater elevation. If a design headwater elevation exceeds the permissible headwater elevation, a new culvert configuration is selected and the process is repeated. Under outlet-control conditions a larger barrel is necessary since inlet improvement may have only limited effect. In the case of very large culverts, the use of multiple culverts may be required with the new design discharge taken as the ratio of the original discharge to the number of culverts. Figure 16.2.9 illustrates computation of the outlet velocity under inlet control and outlet control.

EXAMPLE 16.2.1

Analyze a 6 ft × 5 ft square-edged reinforced concrete box culvert (designed for outlet control) for a roadway crossing to pass a 50-year discharge of 300 ft³/s with the following site conditions (adapted from Normann et al., 1985):

Shoulder elevation = 113.5 ft

Stream bed elevation at culvert face = 100.0 ft

Natural stream slope = 2%

Tailwater depth = 4.0 ft

Approximate culvert length = 250 ft

Maximum allowable upstream water surface (head) elevation = 110 ft (based on adjacent structures)

The inlet is not to be depressed (no fall). Refer to Figure 16.2.10 for further details.

SOLUTION

Consider an outlet control and determine the headwater elevation (EL_{h0}) in steps 1–8.

Step 1 The tailwater depth is specified as 4.0 ft, which is obtained from backwater computations or from normal depth calculations.

Step 2 The critical depth is computed as $d_c = \sqrt[3]{\dfrac{(300/6)^2}{32.2}} = 4.27$ ft.

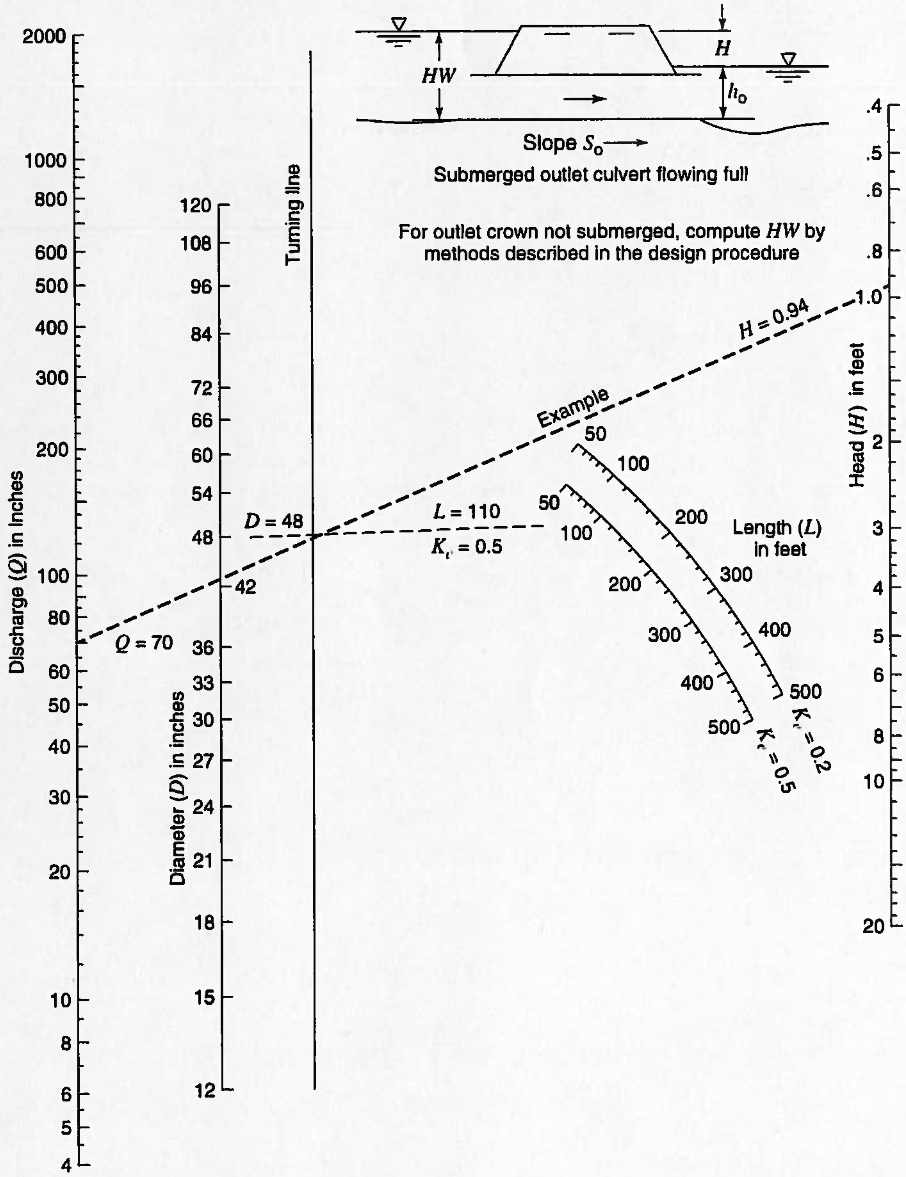


Figure 16.2.7 Head for concrete pipe culverts flowing full, n = 0.012 (from Normann et al. (1985)).

Step 3 $\dfrac{d_c + D}{2} = \dfrac{4.27 + 5.0}{2} = 4.64$ ft.

Step 4 $h_0 = TW$ or $(d_c + D)/2$, whichever is larger. For this problem $h_0 = 4.64$ ft.

Step 5 Use Table 16.2.3 to obtain the entrance loss coefficient. For the square-edged entrance, $K_e = 0.5$.

Step 6 Determine headlosses through the culvert barrel; use equation (16.2.4):

$$H = \left[1 + K_e + \left(\frac{29n^2L}{R^{1.33}}\right)\right]\frac{V^2}{2g}$$

where $A = 6 \times 5 = 30$ ft^2, $V = 300/30 = 10$ ft/s, $R = A/P = 30/(6 + 6 + 5 + 5) = 1.36$ ft. For

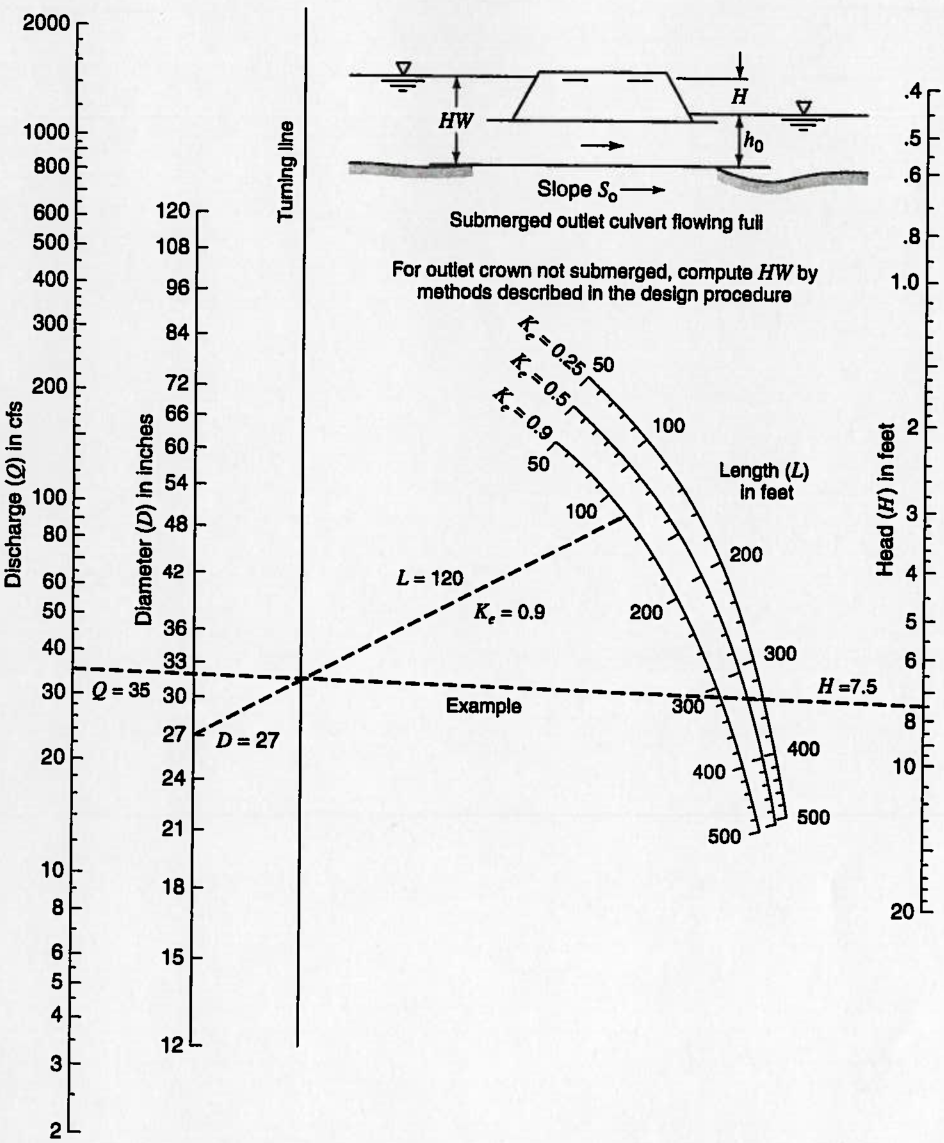


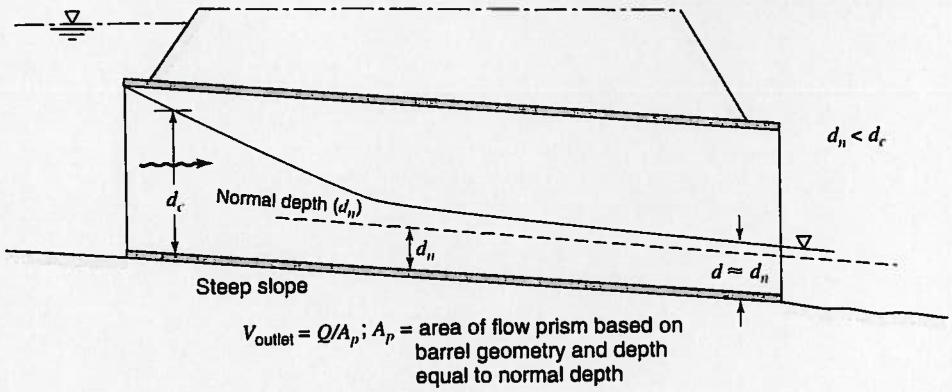
Figure 16.2.8 Head for standard CM pipe culverts flowing full, $n = 0.024$ (from Normann et al. (1985)).

concrete box culvert, take $n = 0.012$. So, $H = \left[1 + 0.5 + \left(\frac{29(0.012)^2(250)}{1.36^{1.33}}\right)\right]\frac{10^2}{2(32.2)} = 3.41$ ft

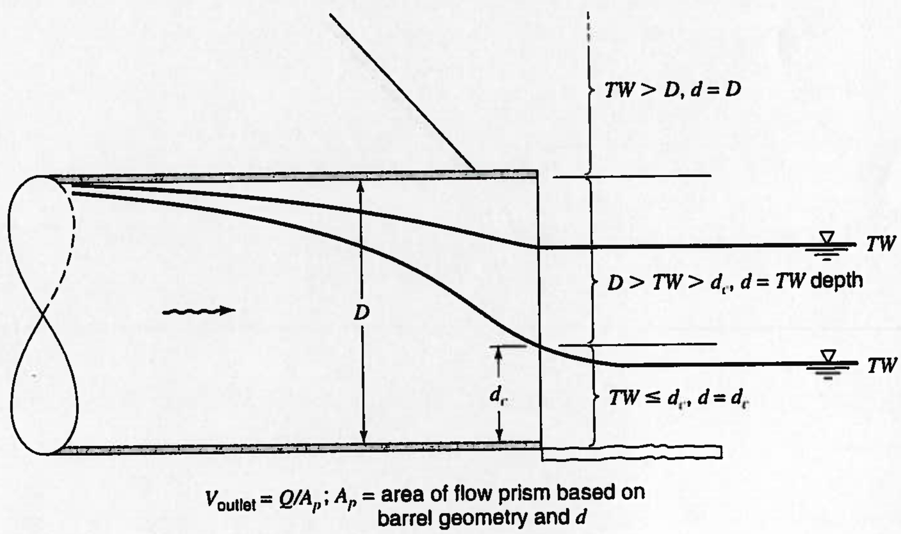
Because $TW < D$ there is only partly full flow at the exit. The headlosses would be more accurately computed from a backwater analysis.

Step 8 Determine the required outlet control head water elevation (EL_{h0}), $EL_{h0} = EL_0 + H + h_0$, where EL_0 is the invert elevation at the outlet:

$EL_0 = EL_i - S_0 L = 100 - 0.02(250) = 95$ ft



(a)



(b)

Figure 16.2.9 Outlet velocity for (a) inlet and (b) outlet control (from Normann et al. (1985)).

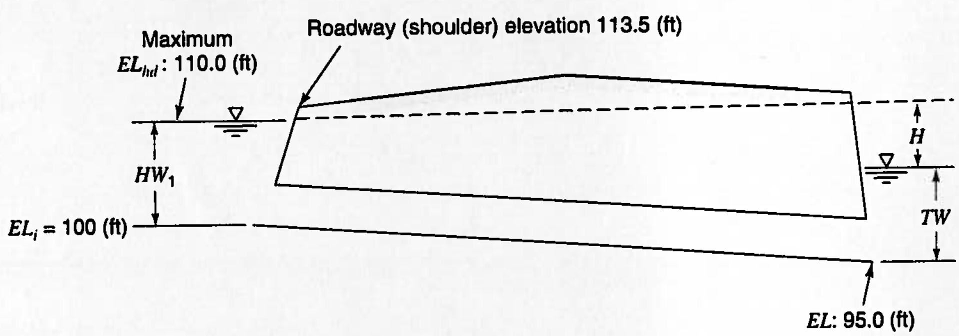


Figure 16.2.10 Details of culvert for Example 16.2.1.

Then $EL_{h0} = 95 + 3.41 + 4.69 = 103$ ft. Also the approach velocity head $(V_u^2/2g)$ and the downstream velocity head can be considered in the calculation of EL_{h0} by adding $V_d^2/2g$ and subtracting $V_u^2/2g$ from the right-hand side of the above equation for EL_{h0}. Consider inlet control and determine headwater elevation, EL_{hi}.

Step 9 The design headwater elevation is now computed as $EL_{hi} = HW_i + EL_i$, so HW_i must be computed using equation (16.2.1), where

$$\frac{HW}{D} = C\left[\frac{Q}{AD^{0.5}}\right]^2 + Y + Z$$

and C and Y are obtained from Table 16.2.1 as $C = 0.0423$ and $Y = 0.82$ for a rectangular box culvert with 0° wing wall flares. $Z = -0.5S_0 = -0.5(0.02) = -0.01$

$$\frac{HW}{D} = 0.0423\left[\frac{300}{30(5)^{0.5}}\right]^2 + 0.82 - 0.01 = 1.66$$

To check,

$$\left[\frac{Q^2}{AD^{0.5}}\right] = 4.47 > 4$$

$$HW_i = D\left[\frac{HW}{D}\right] = 5(1.66) = 8.28 \text{ ft}$$

$$EL_{hi} = 8.28 + 100 = 108.28 \text{ ft}$$

The design headwater elevation of 108.28 ft exceeds the outlet-control headwater elevation of 103 ft. Also, the headwater elevation is less than the roadway shoulder elevation of 113.5 ft.

This design is OK; however, a smaller culvert could be considered. In fact, for this problem a 5 ft × 5 ft reinforced concrete culvert with either a square-edged entrance or a 45° beveled-edge entrance will work, as shown by Normann et al. (1985).

16.2.2 Culvert Design

The hydrologic analysis for culverts involves estimation of the design flow rate based upon the climatological and watershed characteristics. Chapters 7 through 9 and 15 cover the various methods used. The previous section described the use of the hydraulic equations and nomographs for the design of culverts under inlet and outlet conditions. This section concentrates on the use of performance curves for the design process. *Performance curves* are relationships of the flow rate versus the headwater depth or elevation for different culvert designs, including the inlet configuration. Both inlet and outlet performance curves are developed.

An overall performance curve can be developed using the following procedure (Norman et al., 1985):

1. Select a range of flow rates and determine the corresponding headwater elevation for the culvert. The flow rate should cover a range of flows of interest above and below the design discharge. Both inlet and outlet control headwater are computed.
2. Combine the inlet- and outlet-control performance curves into a single curve.
3. For roadway overtopping (culvert headwater elevation > roadway crest), compute the equivalent upstream water depth above the roadway crest for each flow rate using the weir equation

$$Q_0 = C_d L(HW_r)^{1.5} \tag{16.2.10}$$

where Q_0 is the overtopping flow rate in ft³/s (m³/s), C_d is the discharge coefficient ($C_d = k_t C_r$, see Figure 16.2.11), L is the length of roadway crest overtopped in ft (m), and HW_r is the

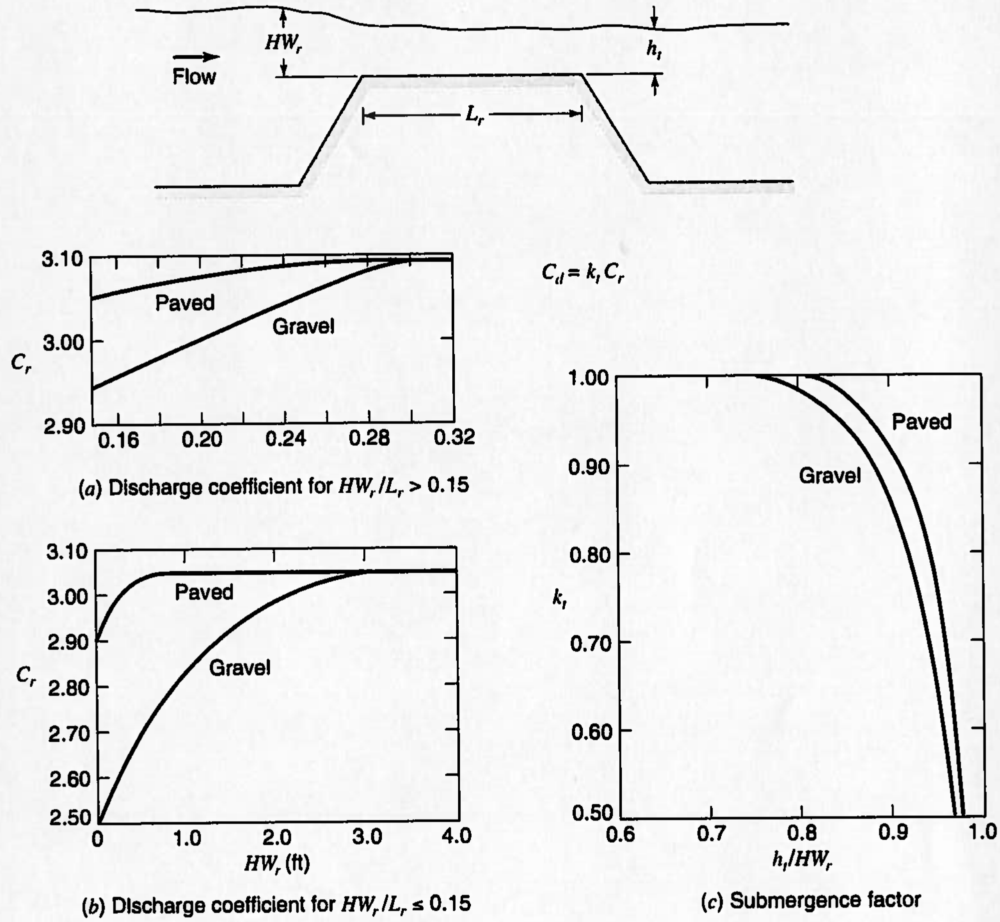


Figure 16.2.11 Discharge coefficients for roadway overtopping (from Normann et al. (1985)).

upstream depth measured from the roadway crest to the water surface upstream of the weir drawdown in ft (m).

4. Add the culvert flow and roadway overtopping flow for the corresponding headwater elevations to obtain the overall culvert performance curve. Figure 16.2.12 shows a culvert performance curve with roadway overtopping, showing the outlet-control portion and the inlet-control portion.

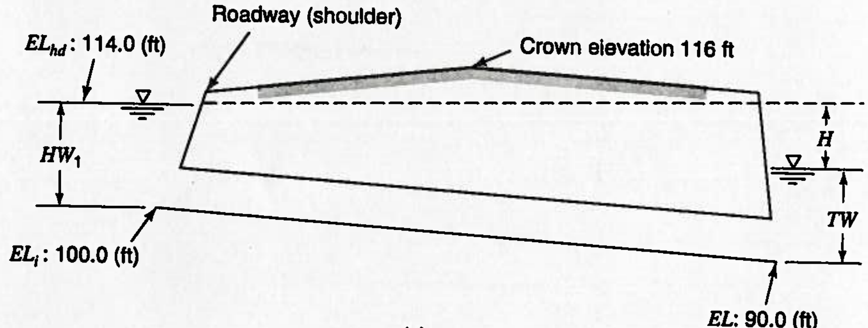
Tuncok and Mays (1999) provide a brief review of various computer models for culverts including HYDRAIN (www.fhwa.dot.gov) by the Federal Highway Administration and CAP (http://water.usgs.gov/software/) by the U.S. Geological Survey.

EXAMPLE 16.2.2

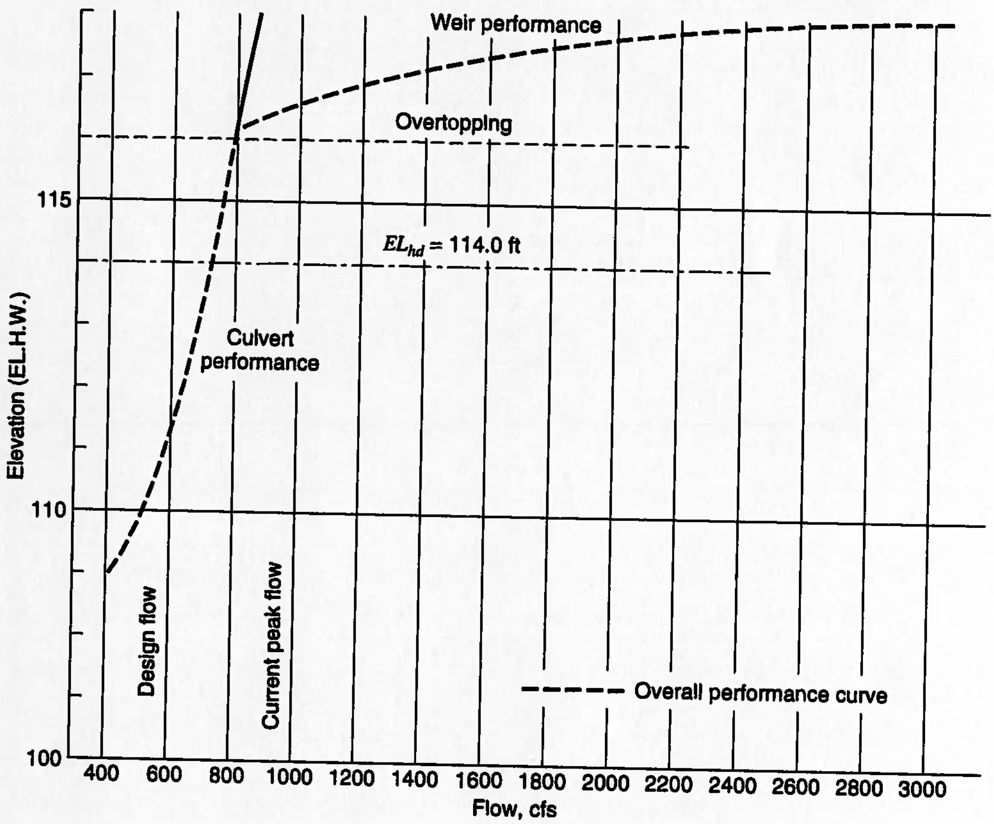
The objective is to develop the performance curve for an existing 7-ft by 7-ft and 200-ft long concrete box culvert on a 5 percent slope that was designed for a 50-year flood of 600 ft^3/s at a design headwater elevation of 114 ft (refer to Figure 16.2.12 for further details). The roadway is a 40 ft wide gravel road that can be approximated as a broad-crested weir with centerline elevation of 116 ft. The culvert inlet invert elevation is 100 ft. The tailwater depth-discharge relationship is:

Q (ft^3/s)	400	600	800	1000	1200
TW (ft)	2.6	3.1	3.8	4.1	4.5

(modified from Normann et al. (1985)).



(a)



(b)

Figure 16.2.12 (a) Culvert profile; (b) Performance curve (from Johnson and Chang (1984)).

The flow width on the roadway for various elevations are:

Elevation (ft)	116.0	116.5	117.0	117.5	118.0	119.0
Flow width (ft)	0	100	150	200	250	300

SOLUTION

The same basic type of calculations performed in example 16.2.1 are followed in Table 16.2.4 for various discharges ranging from 400 ft^3/s to 1000 ft^3/s. From Figure 16.2.11, we find

$$C_d = 2.70 \text{ @ } HW_r = 0.57$$
$$Q = C_d(HW_r)^{1.5}$$
$$C_d = 2.97 \text{ @ } HW_r = 1.9$$
$$k_t = 1$$

The performance curve computations are given in Table 16.2.5. The resulting performance curve is shown in Figure 16.2.12b.

Table 16.2.4 Discharge-headwater Computations for Culvert Flow—Example 16.2.2

Total Flow	Flow Per Barrel	Headwater Calculations: Inlet Control			Outlet Control								Control Head Elevation
Q (cfs)	Q/N	HW_i/D	HW_i	EL_{hi}	TW	d_c	$\frac{d_c+D}{2}$	h_0	K_e	H	EL_{h_0}	
400	400	1.15	8.1	108.1	2.6	4.6	5.8	5.8	0.5	1.95	97.8	108.1
600	600	1.65	11.6	111.6	3.1	6.1	6.6	6.6	0.5	4.4	101.0	111.6
700	700	1.95	13.7	113.7	3.5	6.8	6.9	6.9	0.5	6.0	102.9	113.7
800	800	2.35	16.5	116.5	3.8	>7	7.0	7.0	0.5	7.9	104.9	116.5
850	850	2.55	17.9	117.9	3.9	>7	7.0	7.0	0.5	9.0	106.0	117.9
1000	1000	3.21	22.5	122.5	4.1	>7	7.0	7.0	0.5	1.26	109.6	122.5

Table 16.2.5 Performance Curve Computations—Example 16.2.2

Q_c Culvert flow	EL_h	H_0	Q_0 Overtopping flow	Q Total flow
400	108.1	—	—	400
600	111.6	—	—	600
700	113.7	—	—	700
800	116.5	0.5	191	991
850	117.9	1.9	1556	2406

PROBLEMS

16.1.1 Determine the time of concentration for an overland flow length of 100 m on a turf surface (n = 0.4) with an average slope of 0.02 for a design frequency of 10 years in Phoenix, AZ (see Figure P15.2.1).

16.1.2 Determine the time of concentration for an overland flow length of 200 m on a bare sand (n = 0.01) with an average slope of 0.003 m/m for a design frequency of 10 years in Phoenix, AZ (see Figure P15.2.1).

16.1.3 Determine the time of concentration for an overland flow length of 200 ft on an area (n = 0.10) in Colorado Springs, CO, for a design frequency of 25 years. The rainfall intensity-duration relationship for a 25-year frequency is as given below. Take the average slope of the area as 0.005 ft/ft.

Duration (min)	5	10	15	30	60
Rainfall intensity (in/hr)	7.3	5.7	4.8	3.3	2.1

16.1.4 Determine the time of concentration for an overland flow length of 400 ft on an area of bluegrass sod (n = 0.45) in Charlotte, NC, for a 5-year recurrence interval. The 5-year rainfall intensity-duration relationship (i in in/hr and t_D in min) is

$$i = \frac{57}{(t_D + 12)^{0.77}}$$

Take the average slope of the overland flow area as 0.010 ft/ft.

16.1.5 Determine the peak runoff from 500 ft of pavement (32 ft wide) that drains toward a gutter (for a 10-year frequency storm) in Phoenix, AZ. The pavement slope is 0.005, n = 0.016, and C = 0.9.

16.1.6 Rework problem 16.1.5 for a 25-year storm.

16.1.7 Determine the runoff from 600 ft of pavement (32 ft wide) that drains toward a gutter for a 25-year frequency storm in exceed the shoulder width of 10 ft, the cross-slope is $S_x = 0.05$, and $K = 130$. (a) Determine the location of flanking inlets if they are to function in relief of the inlet at the low point when depth at the curb exceeds design depth. (b) Determine the location of flanking inlets when the depth at the curb is 0.2 ft less than the depth at design spread.

16.1.39 Consider a 2 ft by 2 ft P-1-7/8 grate that is to be placed in a flanking inlet location in a sag vertical curve that is 250 ft downgrade from the most downstream curved vane inlet in problem 16.1.32 ($Q_b = 2.37$ ft^3/s, $S_x = 0.03$, $T_w = 8$ ft, $n = 0.016$, $i = 10.7$ in/hr). The slope on the curve at the inlet is $S = 0.006$. Determine the spread at the flanking inlet and at $S = 0.003$.

16.1.40 Rework example 16.1.16 to determine the interception capacity of a larger median drop inlet ($W = 2$ ft, $L = 4$ ft).

16.2.1 Rework example 16.2.1 using a 5 ft × 5 ft culvert with a square-edged entrance.

16.2.2 Rework example 16.2.1 using a 5 ft × 5 ft culvert with a 45° beveled-edged entrance.

16.2.3 Rework example 16.2.1 for a discharge of 200 ft^3/s.

16.2.4 A culvert is to be designed for a new roadway crossing for a 25-year peak discharge of 200 ft^3/s. Use a circular corrugated metal pipe culvert with standard 2-2/3 by 1/2 corrugations and beveled edges. The site conditions include: elevation at culvert face = 100 ft; natural stream bed slope = 1%; tailwater for 25-year flood = 3.5 ft; approximate culvert length = 200 ft; shoulder elevation = 110 ft. Base the design headwater on the shoulder elevation with a 2-ft freeboard (elevation of 108 ft). Set the inlet invert at the natural stream bed elevation (no fall). Analyze the design.

16.2.5 Rework problem 16.2.4 using a concrete pipe with a grooved end.

16.2.6 Rework example 16.2.2 to develop the performance curve for an 8 ft by 7 ft concrete box culvert with a square-edged entrance. All other conditions are the same as in example 16.2.2.

16.2.7 Determine if a 1.5 m × 1.5 m square-edged reinforced concrete box culvert is adequate for a roadway crossing to pass a discharge of 8.50 m^3/s. The inlet is not depressed (no fall). The site conditions are as follows:

Shoulder elevation = 34.6 m

Stream bed elevation at culvert face = 30.5 m

Natural stream slope = 2%

Tailwater depth = 1.2 m

Approximate culvert length = 76.2 m

Upstream water surface (head) elevation = 33.5 m

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