Chapter 11
Culverts and Bridges

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1.0   Introduction and Overview

This chapter addresses the hydraulic function of culverts and bridges, i.e., conveyance of surface water through embankments such as roadways and railroads. Structural considerations, such as the design requirements to support loads, are not addressed in this chapter. The chapter is primarily focused on design of culverts with the exception of Section 7.0 which provides a brief overview of considerations with regard to bridges. When designing a culvert or bridge that will include a path, also see the *Stream Access and Recreational Channels* Chapter.

Photograph 11-1. Culverts frequently provide compatible conditions for upstream wetland growth.

A careful approach to design is essential, for new and retrofit situations, because crossings often significantly influence upstream and downstream flood risks, floodplain management, and public safety. Multiple factors have a bearing on the hydraulic capacity and overall performance of a structure. These include the size, shape, slope, material, inlet configuration, outlet protection, and other variables. Sizes and shapes of culverts vary from small circular pipes to extremely large arch sections used in place of a bridge.

In addition to the primary function of conveying flow, culverts can create conditions upstream that are suitable for wetland growth (Photograph 11-1).   Aesthetic considerations should also be incorporated into a design, such as visually integrating a crossing into the surrounding landscape. This can be achieved through thoughtful grading, landscaping and wall design including finishing.

Much of the information and many of the references necessary to design culverts according to the procedure given in this chapter can be found in *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (FHWA 2005a). Examples of charts and nomographs from that publication are given in this chapter for some of the most common culvert scenarios; however, this chapter does not republish many of the nomographs, equations and technical background provided by FHWA's *Hydraulic Design of Highway Culverts* since it is readily available on the internet and provides a level of detail that goes beyond what most typical users of the Urban Storm Drainage Criteria Manual (USDCM) will require. Refer to the FHWA publication for special cases, larger culvert sizes, or specific technical topics not covered in this chapter.

2.0   Required Design Information

The hydraulic design of a culvert or bridge includes determining the types of information described in the following sections:

General Planning Considerations

- Drainage Master Plan

  o How will the proposed structure fit into the relevant major drainageway master plan, and are there multi-purpose objectives that could be satisfied? For example, box culverts can also serve as below-grade crossings, with one cell elevated to convey flows only during larger storm events (see the *Open Channels* chapter for criteria). Additionally, a culvert can be used to discharge at a

controlled flow rate while the area upstream from the culvert is, for example, used for detention storage to reduce a storm runoff peak (in such a case, the embankment that the culvert penetrates should effectively be designed as a dam).

- o Careful consideration should be taken to ensure that upstream and downstream property owners are not adversely affected by new hydraulic conditions. When restricting flow to attenuate major events, evaluate the area of potential flooding upstream of the new culvert. If a culvert is replaced by one with more capacity, the downstream effects of the increased flow must be evaluated. Assure consistency with existing master plans and/or outfall studies.

- ▪ Safety Concerns

  - o Are there specific public safety issues related to the culvert location, such as proximity to parks or other public areas that have a bearing on the culvert design? A key question is whether or not to include a safety/debris grate at the culvert inlet (grates should be avoided at culvert outlets).

  - o Culverts are often located at the bottom of a steep slope. Large box culverts, in particular, can create conditions where there is a significant falling hazard, which poses risk to the public. In such cases, fencing (or guardrails for roadway applications) is recommended for public safety.

Specific Design Considerations

- ▪ Location

  - o Culvert location is an integral part of roadway design. The designer should identify all live stream crossings, springs, low areas, gullies, and impoundment areas created by the new roadway embankment for possible culvert locations.

  - o The culvert should be located as to not change the existing stream alignment and be aligned to give the stream a direct entrance and exit. Abrupt changes in direction at either end may reduce capacity making a larger structure necessary. Bends within a culvert should also be avoided where possible. If necessary, a direct inlet and outlet may be obtained by channel realignment, skewing the culvert, or a combination of these.

  - o Where water must be turned into a culvert, headwalls, wingwalls, and aprons with configurations similar to those in Figure 11-13 should be used as protection against scour and to provide an efficient inlet.

- ▪ Design Flood Frequency and Discharge

  - o The design flood frequency for culverts is closely related to the pavement encroachment and road overtopping criteria presented in Tables 1-2, 1-3 and 1-4 in the *Policy* chapter. Most municipalities within the Urban Drainage and Flood Control District (UDFCD) region have minimum design frequencies related to these tables that require culvert capacity for at least the 10-year event (and in some cases the 25-year event); however, for road and rail crossings of channels that drain a watershed of 130 acres or more, especially for arterial streets, freeways and critical crossings, a 100-year basis of design (plus freeboard above the allowable headwater) is common. Please note that state and federal standards apply to relevant highway projects. The design recurrence interval should be based on the criteria set forth in this manual in conjunction with local requirements and criteria for culvert sizing and road overtopping. The more stringent of the applicable criteria should be applied.

  - o The required hydraulic capacity (i.e., design discharge) is based on the design flood frequency

and the resulting design flow rate calculated for the watershed tributary to the proposed culvert (see the *Runoff* chapter for information on hydrologic calculations). The structure should be designed to operate within acceptable limits of uncertainty of the design discharge.

- o Culverts are frequently designed to overtop in a 100-year event while bridges are typically designed to pass this flow while allowing for freeboard.

▪ Allowable Headwater Depth for Culverts

- o Culverts frequently constrict the natural stream flow, which causes a rise in the upstream water surface. The elevation of the upstream water surface is termed *headwater elevation*. The depth of headwater is measured from the invert of the culvert inlet to the headwater elevation for a known event. In selecting the design headwater elevation, the designer should consider the following:

  1. The headwater depth /culvert diameter ratio (*HW/D*) should not exceed 1.5 for the 100-year event peak flow unless there is justification and sufficient measures are taken to protect the culvert inlet (for example, a concrete headwall). Piping failure can be of concern for deep headwater depths, especially if there are animal burrows in the embankment.

  2. Assess the impacts caused by exceeding the design headwater depth, including:

     a. Hazard to human life and safety.

     b. Potential damage to the culvert, embankment stability and roadway.

     c. Traffic interruption in the event of roadway overtopping.

     d. Anticipated upstream and downstream flood risks, for a range of return frequencies.

  3. The elevation of the watershed divides should be higher than the design headwater elevations in order to prevent the headwater from spilling into adjacent watersheds. In flat terrain, watershed boundaries are often poorly defined, and culverts should be located and designed to minimize disruption of the existing flow paths and avoid spillover into adjacent watersheds due to culvert backwater effects.

▪ Tailwater Depth for Culverts

- o Tailwater is the flow depth in the downstream channel, measured from the invert of the culvert outlet to the water surface (assuming normal (uniform) flow in the channel downstream of the culvert). Knowledge of tailwater depth is critical for culvert design because a submerged outlet may cause the culvert to flow full rather than partially full.

- o Tailwater depth is typically calculated using a computer program, such as HEC-RAS or HY-8, as the water surface profile in the downstream channel, or using an alternative method for computing the normal depth. A field inspection of the downstream channel should be made to determine whether there are obstructions that will influence the tailwater depth. Tailwater depth may be controlled by several factors, including the stage in a contributing stream, headwater/backwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

▪ Allowable Outlet Velocity for Culverts

- o   The outlet velocity of a culvert, measured at the downstream end of the culvert, is usually higher than the maximum velocity that a natural channel can withstand without experiencing significant erosion of the bed and/or banks. Most culverts require adequate outlet protection (typically riprap or a stilling basin), and this is a frequently overlooked issue during design. Use UD-Culvert, available at www.udfcd.org to determine the length of recommended outlet protection.

- o   Permissible velocities at the outlet will depend upon streambed type, and the type of energy dissipation (outlet protection) that is provided. As a general rule, the velocity at the downstream edge of a project right-of -way or downstream constraint should not be greater than the pre-construction velocity.

- o   If the outlet velocity of a culvert is too high, the velocity may be reduced by increasing the barrel roughness, since slope and roughness are the principal factors affecting the outlet velocity. Variations in shape and size of a culvert seldom have a significant effect on the outlet velocity. If changing the barrel roughness does not provide a satisfactory reduction in outlet velocity, it may be necessary to incorporate some type of outlet protection or energy dissipation device.

- ▪   Environmental Permitting

  - o   Environmental permitting constraints often are applicable for new culverts or retrofits as well as for construction of bridges. For example, the Section 404 permit, administered by the United States Army Corps of Engineers (USACE), regulates construction activities in jurisdictional wetlands and "Waters of the United States." The local USACE representative should be consulted when designing a crossing to assess the permitting requirements. Culverts also often have regulatory floodplain implications and a Conditional Letter of Map Revision (CLOMR) and/or Letter of Map Revision (LOMR) is often required when a new culvert is installed.

- ▪   Fish Passage and Culverts

  - o   At some culvert locations, the ability of the structure to accommodate migrating fish is an important design consideration. For such sites, federal and state fish and wildlife agencies (such as the United States Fish and Wildlife Services and the Colorado Division of Wildlife) should be consulted early in the planning process. Some situations may require the construction of a bridge to span the natural stream. However, culvert modifications such as oversizing the diameter or rise of the culvert, placing the culvert below the stream bed and filling the lower portion with native streambed material can often be used to meet the design criteria established by the regulatory agencies and the fish and wildlife agencies.

- ▪   Culvert Details

  - o   Culvert size and shape.

  - o   Culvert material.

  - o   Alignment, grade, and length of culvert.

  - o   Need for protective measures against abrasion and corrosion and type of coating, if required.

  - o   Culvert inlet design.

  - o   Culvert end treatment and erosion protection.

o   Amount and type of cover material required.

Other Design Considerations

Other design considerations include the following:

- What are the impacts of various culvert sizes, dimensions, and materials on upstream and downstream flood risks, including the implications of embankment overtopping?

- What type of sediment load and bed load can be anticipated for the culvert? For streams with a heavy bed load, abrasion and debris blockage can be of concern. For culverts with milder slopes or abrupt changes to a flatter grade within the culvert, filling in of culverts with sediment can be problematic and lead to increased maintenance frequency. If the culvert is in an area where there is potential for significant debris (mountainous terrain, pine beetle kill areas, recently burned areas, etc.), appropriate conservative assumptions for blockage and overflow paths should be applied.

    Deposits in culverts may also occur due to the following conditions:

    o   At moderate flow rates, the culvert cross section may be larger than that of the stream, so the flow depth and sediment transport capacity is reduced.

    o   Point bars form on the inside of stream bends. Culvert inlets placed at bends in the stream will be subject to deposition in the same manner. This effect is most pronounced in multiple-barrel culverts with the barrel on the inside of the curve often becoming almost totally plugged with sediment deposits.

- Structural and geotechnical considerations which are beyond the scope of this chapter.

3.0   Culvert Hydraulics

3.1   Key Hydraulic Principles

For the purposes of this review, it is assumed that the reader has a basic working knowledge of hydraulics and is familiar with the Manning's Equation (Equation 11-1), Continuity Equation (Equation 11-2), and Energy Equation (Equation 11-3):

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2}$$ Equation 11-1

Where:

Q = flow rate or discharge (cfs)
n = Manning roughness coefficient (see Table 11-1)
A = cross-sectional area of flow (ft^2)
R = hydraulic radius (ft)
S = longitudinal slope (ft/ft)

$$Q = v_1 A_1 = v_2 A_2$$ Equation 11-2

Where:

Q = flow rate or discharge (cfs)
v = velocity (ft/s)
A = cross-sectional area of flow (ft^2)

Subscripts refer to two different locations within a culvert or channel between which flow is constant.

$$\frac{v^2}{2g} + \frac{p}{\gamma} + z + \text{losses} = \text{constant}$$

Equation 11-3

Where:

v = velocity (ft/s)
g = gravitational acceleration (32.2 ft/s^2)
p = pressure (lb/ft^2)
γ = specific weight of water (62.4 lb/ft^3)
(Note: p/γ = pressure head or depth of flow [ft])
z = height above datum (ft)

Table 11-1. Manning's roughness coefficients

	Reinforced Concrete Pipe (RCP)	Aluminized Steel Pipe (ASP)	Polymer Coated Steel pipe	Corrugated Aluminum Pipe	Polyvinyl Chloride Pipe (PVC)	High Density Polyethylene Pipe (HDPE)
Manning's Roughness Coefficient	0.013	0.013	0.013	0.013	0.011	0.012

3.1.1   Energy and Hydraulic Grade Lines

The concepts of energy grade line (EGL) and hydraulic grade line (HGL), and related terms, are illustrated for open channel flow (Figure 11-1) and closed conduit flow (Figure 11-2).

Open Channel Flow
The EGL, also known as the line of total head, is the sum of velocity head ($v^2/2g$), the depth of flow or pressure head (p/γ), and elevation above an arbitrary datum represented by the distance (z). The energy grade line slopes downward in the direction of flow by an amount equal to the energy gradient (H_L/L), where H_L equals the total energy loss over the distance L.
The HGL, also known as the line of piezometric head, is the sum of the depth of flow or pressure head (p/γ), and the elevation (z). The HGL does not include the velocity head.

For open channel flow, the term *p/γ* is equivalent to the depth of flow and the hydraulic grade line is the same as the water surface (Point 5 on Figure 11-1).
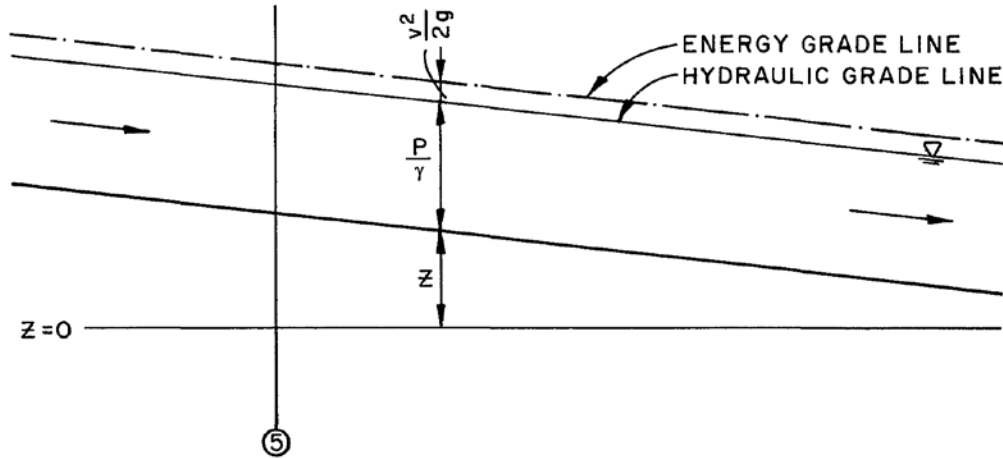
Figure 11-1. Illustration of terms for open channel flow

Closed Conduit Flow
While it is preferable to design culverts for open channel flow conditions (i.e. non-pressurized flow), when the culvert is designed with a headwater depth exceeding the top of the culvert (not uncommon) pressurized flow may develop under some tailwater conditions and/or during events that exceed the design capacity of the culvert. For pressure flow in closed conduits, p/γ is the pressure head and the hydraulic grade line is above the top of the conduit provided that the pressure relative to atmospheric pressure is positive (see Figure 11-2).
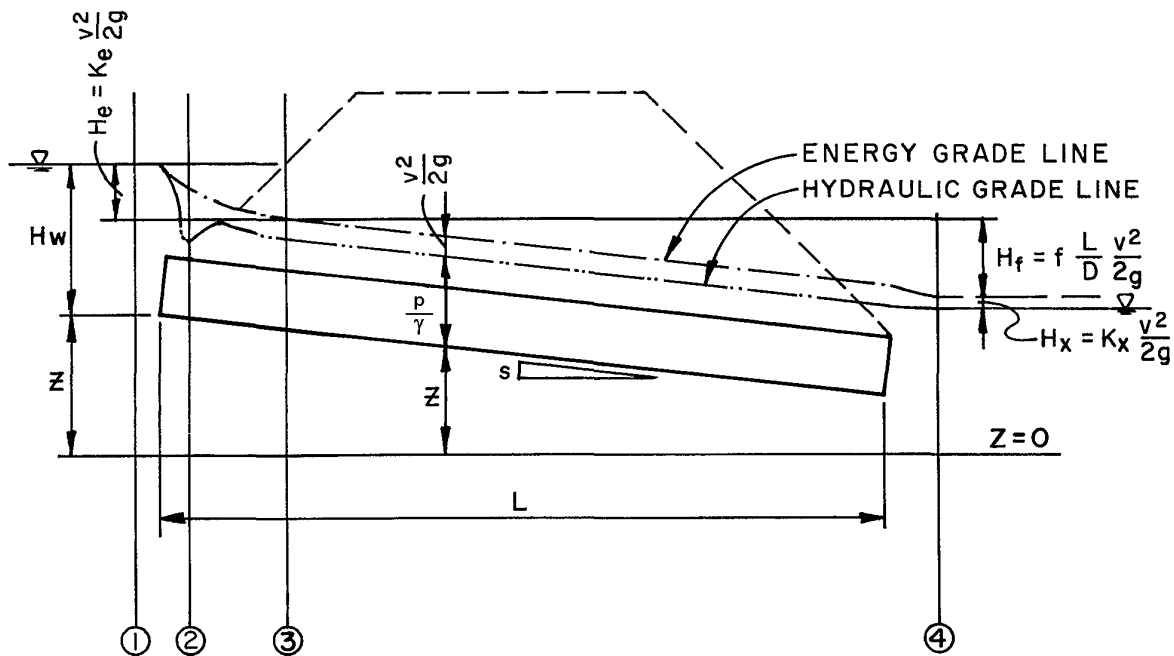
Figure 11-2. Illustration of terms for closed conduit flow

When ponding occurs at the entrance of a culvert (see Point 1 on Figure 11-2) the velocity is considered minimal and the energy grade line and hydraulic grade line are nearly the same. As water enters the culvert at the inlet, the flow is contracted by the inlet geometry causing a loss of energy (see Point 2). As a turbulent velocity distribution is reestablished downstream of the entrance (see Point 3), a loss of energy occurs due to friction and/or resistance from the culvert. In short culverts, the entrance losses are likely to be high relative to the friction loss. At the culvert exit (Point 4), additional losses occur through turbulence as the flow expands and is retarded by the tailwater in the downstream channel.

3.1.2   Inlet and Outlet Control

There are two basic types of flow conditions in culverts: (1) *inlet control* and (2) *outlet control*[1]. For each type of control, a different combination of factors is used to determine the hydraulic capacity of the culvert. The determination of actual flow conditions can be difficult; therefore, the designer must check for both types of control and design for the most adverse condition.

Inlet Control

A culvert operates under inlet control when the flow capacity of the culvert is controlled at the inlet by these factors:

- Depth of headwater.

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[1] "Outlet control" refers to all head loss mechanisms other than the culvert inlet. These outlet control mechanisms include head loss attributed to pipe friction, bends, culvert outlet, and tailwater. "Outlet control" is the common naming convention for these losses, including in FHWA 2005a.

- Inlet edge configuration.

- Cross-sectional area.

- Barrel shape (e.g., circular, elliptical, rectangular, etc.).

With inlet control, the culvert barrel usually flows only partially full. Inlet control for culverts can occur under unsubmerged or submerged conditions.

       Unsubmerged Inlet: The headwater depth is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical (Figure 11-3).
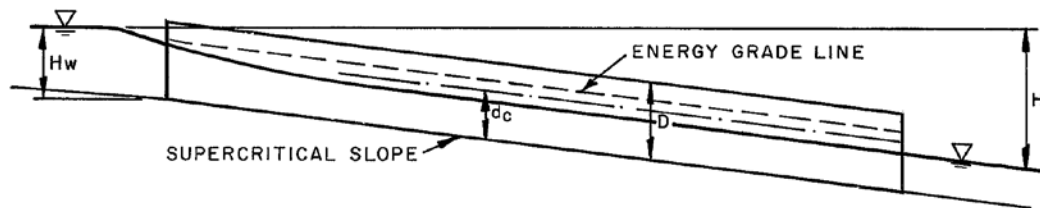

Figure 11-3. Inlet control – unsubmerged inlet

       Submerged Inlet : The headwater submerges the top of the culvert and the pipe does not flow full (Figure 11-4). This is the most common condition of inlet control.
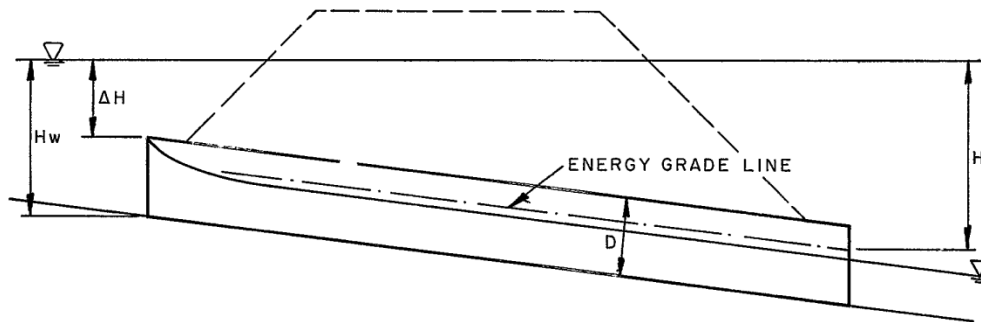
Figure 11-4. Inlet control – submerged inlet

A culvert flowing under inlet control is sometimes referred to as a "hydraulically short" culvert.

Outlet Control
The hydraulic control of a culvert can switch from the inlet to the outlet under several conditions, such as high headwater, relatively flat culvert slope, or sufficiently long culvert length. [2]
With outlet control, culvert hydraulic performance is determined by the following factors:

- Depth of headwater,

- Inlet edge configuration,

- Cross-sectional area,

- Bends (if applicable),

- Culvert shape,

- Barrel slope,

- Barrel length,

- Barrel roughness, and

- Depth of tailwater.

Outlet control for culverts can occur under partially full or full conduit conditions.

> Partially Full Conduit: The headwater depth is insufficient to submerge the top of the culvert, and the culvert slope is subcritical, resulting in the culvert flowing partially full (Figure 11-5). This is the least common condition of outlet control.
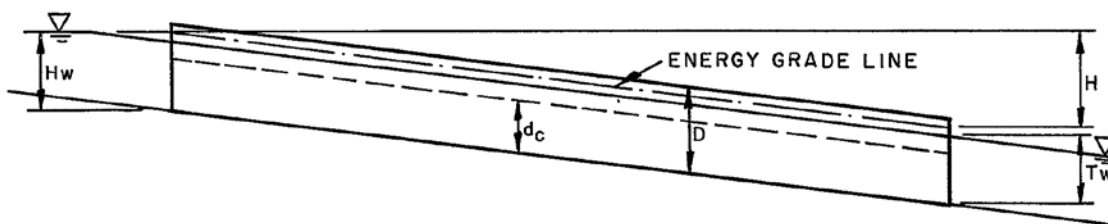
Figure 11-5. Outlet control – partially full conduit

> Full Conduit: The culvert flows full along its length (Figure 11-6). This is the most common condition of outlet control.

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[2] Over a range of event frequencies (and even within an event during dynamic conditions), most culverts experience both inlet and outlet control conditions at times.
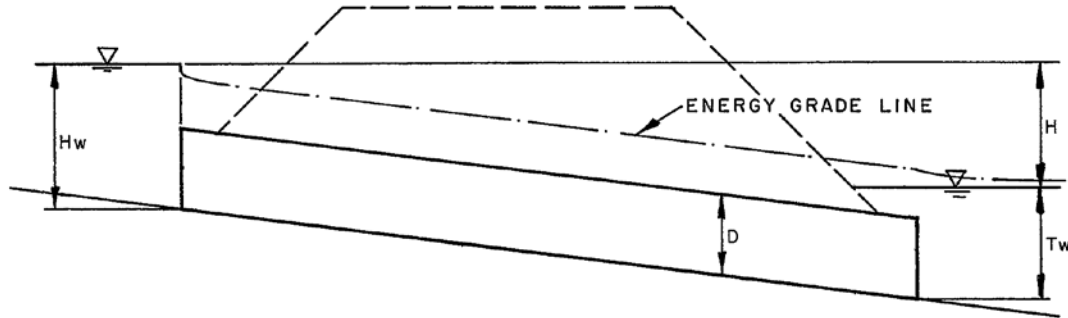
Figure 11-6. Outlet control – full conduit

A culvert flowing under outlet control is sometimes referred to as a "hydraulically long" culvert. With outlet control, factors that may affect performance for a given culvert size and headwater depth are barrel length, barrel roughness, and tailwater depth.

3.2   Energy Losses

The energy losses that must be evaluated to determine the carrying capacity of a culvert are:

- Inlet (or entrance) losses (Section 3.2.1)

- Friction losses (through the culvert) (Section 3.2.2)

- Bend losses (if applicable) (Section 4 of the *Streets, Inlets, and Storm Drains* chapter)

- Outlet (or exit) losses (Section 3.2.3)

It is noteworthy that the entrance losses in a culvert can be as important as the friction losses, particularly in short culverts.

3.2.1   Inlet Losses

For inlet losses, the governing equations are:

$$Q = CA\sqrt{2gH}$$

Equation 11-4

and

$$H_e = K_e \frac{V^2}{2g}$$

Equation 11-5

Where:

Q = flow rate or discharge (cfs)
C = contraction coefficient (dimensionless)
A = cross-sectional area (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)

H = total head (ft)
H_e = head loss at entrance (ft)
K_e = entrance loss coefficient (dimensionless, see Section 5.0)
V = average velocity (ft/s)

Capacity based on headwater relevant to culvert rise.

The Federal Highway Administration (FHWA) has determined that the orifice equation is not valid representation of actual capacity until the headwater (H) is at least 3 times the height (rise) of the culvert. For H less than 0.5(rise), open channel minimum energy equations should be applied, and for 0.5(rise) < H < 3(rise), empirical best-fit equations should be applied. This methodology is programmed into HY-8 and into the UD-Culvert workbook.

3.2.2   Friction Losses

Pipes Flowing Partially Full
Friction head loss for pipes flowing partially full can be determined from the Manning's equation reformulated to calculate head loss:

$$H_f = \left(\frac{29\,n^2\,L}{R^{4/3}}\right)\left(\frac{V^2}{2g}\right)$$

Equation 11-6

Where:

H_f = frictional head loss in culvert barrel (ft)
n = Manning roughness coefficient (dimensionless)
L = culvert length (ft)
R = hydraulic radius (ft, area/wetted perimeter)
A = cross-sectional area of culvert barrel (ft^2)
V = average velocity (ft)
g = acceleration due to gravity (32.2 ft/s^2)

Pipes Flowing Full
Friction head loss for turbulent flow in pipes flowing full can be determined from the Darcy-Weisbach equation.

$$H_f = f\left(\frac{L}{D}\right)\left(\frac{V^2}{2g}\right)$$

Equation 11-7

Where:

H_f = frictional head loss (ft)
f = friction factor (dimensionless)
L = culvert length (ft)
D = pipe diameter (ft)
V = average velocity (ft)
g = acceleration due to gravity (32.2 ft/s^2)

3.2.3   Outlet Losses

For outlet (or exit) losses, the governing equations are related to the difference in velocity head between: a) the pipe flow, and b) the downstream channel at the end of the pipe. The downstream channel velocity is usually neglected, resulting in the outlet losses being equal to the velocity head of full flow in the culvert barrel, given by the following:

$$H_o = \frac{V^2}{2g}$$

Equation 11-8

Where:

H_o = outlet head loss (ft)
V = average velocity in culvert barrel (ft)
g = acceleration due to gravity (32.2 ft/s^2)

3.2.4   Total Losses

Combining the relationships for entrance loss, friction loss, and outlet (or exit) loss, the following equation for total head loss is obtained (i.e., difference in the headwater and tailwater elevations):

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

Equation 11-9

Where:

H = difference in the headwater and tailwater elevations (ft)
K_e = entrance loss coefficient (dimensionless)
n = Manning roughness coefficient (dimensionless)
L = culvert length (ft)
R = hydraulic radius (area/wetted perimeter)
V = average velocity (ft)
g = acceleration due to gravity (32.2 ft/s^2)

4.0   Culvert Sizing and Design

The hydraulic design of culverts can be completed using several different methods, including the following described in this chapter:

▪ Capacity Charts (Section 4.1)

▪ Nomographs (Section 4.2)

▪ Computer Applications (Section 3.04.3)

The capacity charts and nomographs are methods that were frequently used before the widespread use of computers; however, they are older methods that are now less commonly used in lieu of computer applications. The capacity charts and nomographs still have utility for independently sizing culverts or for checking results generated from software packages. Hence, all three of these methods for culvert sizing are addressed in this manual.

4.1   Capacity Charts

Capacity charts can provide a good understanding of how culvert size requirements vary depending on multiple variables. Descriptions are provided below for the application of capacity charts for inlet control (Section 7.17.14.1.1), outlet control (Section 7.17.14.1.2), as well as a procedure for their use (Section 7.17.14.1.3).

It is important to recognize that there are numerous restrictions on the use of capacity charts in terms of culvert entrance and exit conditions. Capacity charts for all of the types of entrance conditions that a designer may encounter are not provided in this manual. For capacity charts for a range of entrance conditions refer to FHWA Hydraulic Engineering Circular No. 10 (FHWA 1972), available for download at: http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec10.pdf. Perhaps most important to recognize is that capacity charts should only be used for free outfall conditions. This is important because for some conditions, such as flood flows on relatively flat slopes, high tailwater conditions will inevitably be encountered and capacity charts would not be suitable.

Examples of capacity charts used for culvert sizing are shown on Figure 11-7. The upper chart is for circular culvert diameters from 18 to 36 inches and the lower chart is for circular culvert diameters from 36 to 66 inches. The discussion below refers to these charts.

Each chart contains a series of curves which show the discharge capacity per culvert barrel (in cfs) for each of several sizes of similar culvert types, given various headwater depths (measured in feet above the culvert invert at the inlet). The curved lines represent the ratio of the culvert length (L) in feet, to 100 times the slope (s) in units of ft/ft. Each culvert size on the chart is described by two lines: one solid and one dashed. The solid line represents the division between outlet and inlet control. The dashed line represents the maximum $L/(100s)$ ratio for which the curve may be used without modification.

4.1.1   Culverts Under Inlet Control

When using the capacity charts, for values of $L/(100s)$ less than that shown on the solid line, the culvert is operating under inlet control. The headwater depth is determined from the $L/(100s)$ value given on the solid line. The inlet control curves (solid) are plotted from model test data. The outlet control curves (dashed) were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed; therefore, tailwater depth is assumed not to influence the culvert performance.
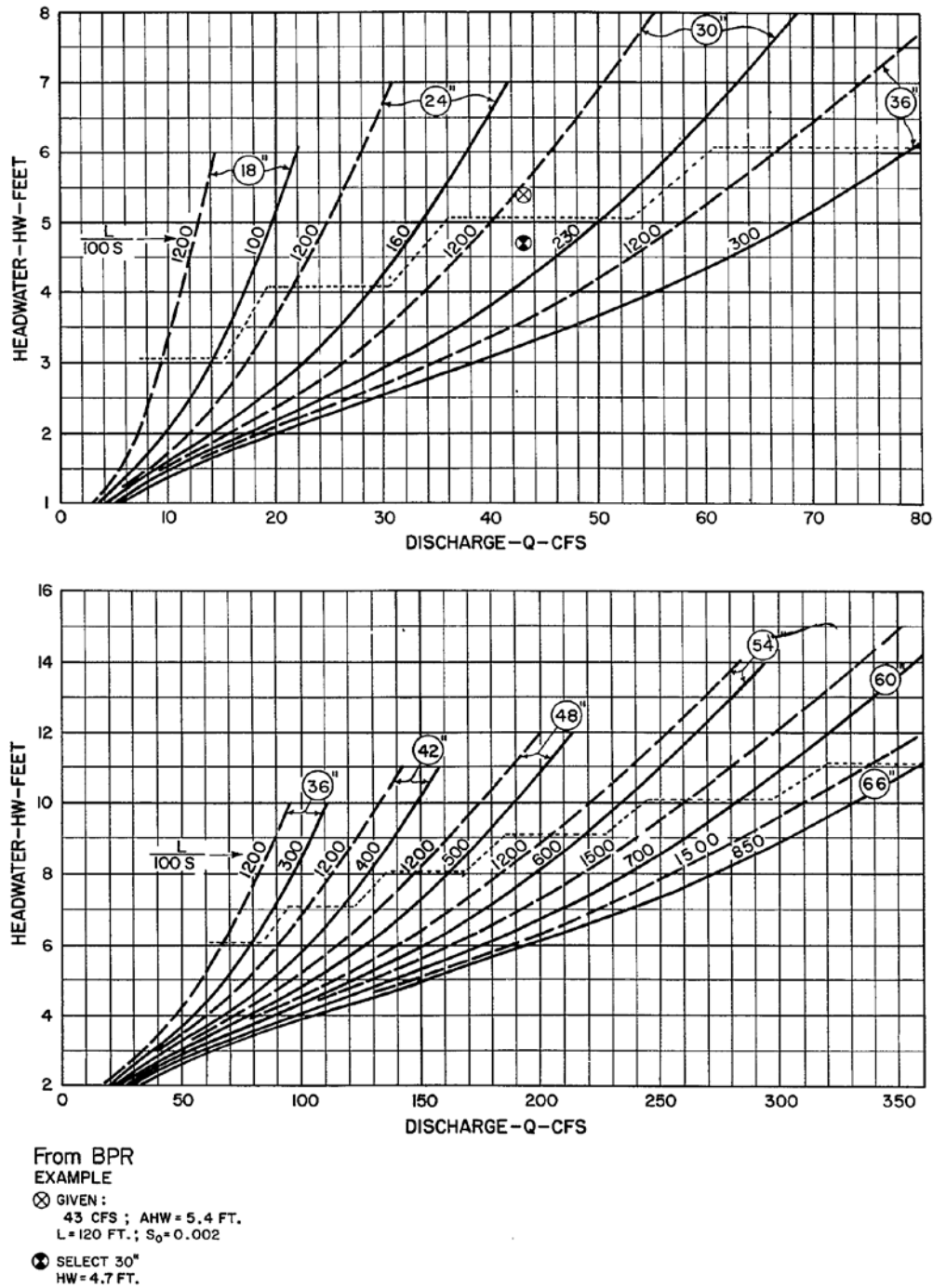
Figure 11-7. Culvert capacity chart—example

(Assumes free outfall conditions and includes elevation plus velocity head in headwater.)

4.1.2   Culverts Under Outlet Control

When using the capacity charts for culverts flowing under outlet control, the head loss at the entrance is not determined by the capacity charts, but is computed using entrance loss coefficients. In addition, the hydraulic roughness of the culvert material is taken into account in computing resistance loss for full or part-full flow, with Manning's *n* values ranging from 0.012 to 0.032, depending on the pipe material (see Table 11-1).

Except for large pipe sizes, headwater depths on the charts extend to 3.0 times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are used in low fills. The dotted line, stepped across the charts, shows headwater depths approximately twice the barrel height and indicates the upper limit of unrestricted use of the charts. Above this line the headwater elevation should be checked with the nomographs (see Section 4.2) or with computer programs (see Section 4.3). Also, as stated in Section 2.2, UDFCD's policy is that the headwater depth/culvert diameter ratio (HW/D) should not exceed 1.5 unless there is justification and sufficient measures are taken to protect the embankment from piping.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head (i.e., the elevation head plus velocity head for flow in the approach channel). In most cases, the water surface upstream from the inlet is so close to the same level as the total head that the chart determination may be used as headwater depth for practical design purposes (assuming minimal velocity head). For practical purposes, approach velocities up to about 3 feet per second can be neglected. However, for approach velocities greater than 3 feet per second, the velocity head should be subtracted from the curve determination of headwater to obtain the actual headwater depth.

4.1.3   Capacity Chart Procedure

The procedure for sizing a culvert using the capacity charts is summarized below. Data can be compiled in the Design Computation Form shown on Figure 11-8.

1. Identify design data and list on the Design Computation Form:

    ▪ Q = flow or discharge rate (cfs) for the design discharge (Q1) and a check discharge (Q2) for a different storm event (e.g., 50-year or 100-year event).

    ▪   Tailwater elevations for both Q1 and Q2 (calculated using HEC-RAS, HY-8 or other method) (ft).

    ▪ L = length of culvert (ft).

    ▪ s = slope of culvert (ft/ft).

    ▪   Allowable Hw = headwater depth (ft).

    ▪   Culvert type and entrance type for the first trial culvert design.

2. Compute $L/(100s)$.

3. Find the design discharge (Q) in the appropriate capacity chart. Locate the appropriate chart (based on culvert size, shape, and entrance condition) in FHWA Hydraulic Engineering Circular No. 10 (HEC 10), *Capacity Charts for the Hydraulic Design of Highway Culverts* (FHWA, 1972), available for download at www.fhwa.dot.gov.

4. Using the design discharge and capacity chart from Step 3, find the L/(100s) value for the smallest pipe that will pass the design discharge. If this value is above the dotted line (the maximum L/(100s) ratio for which the curves may be used without modification), use the nomographs (from FHWA 2005a) to check headwater conditions.

5. If L/(100s) is less than the value of L/(100s) given for the solid line, then the value of Hw is the value obtained from the solid line curve. If L/(100s) is larger than the value for the dashed outlet control curve, then special measures must be taken, and the reader is referred to *Hydraulic Design of Highway Culverts* (FHWA 2005a).

6. Check the headwater depth (Hw) value obtained from the charts with the allowable Hw. If the indicated Hw is greater than the allowable Hw, then check the next largest pipe size to see if the Hw elevation is acceptable (i.e., is less than the allowable *Hw).*

Figure 11-8. Design computation for culverts—blank form

4.2   Nomographs

Examples of nomographs for designing culverts are presented on Figure 11-9 (Inlet Control Nomograph) and Figure 11-10 (Outlet Control Nomograph). A disadvantage of the nomographs is that they require trial and error, whereas the capacity charts described in Section 4.1 are direct.

As noted previously, the capacity charts can be used only when the flow passes through critical depth at the outlet. If the critical depth at the outlet is less than the tailwater depth, then the nomographs or other method must be used.

Nomograph Procedure
The nomograph procedure for culvert design requires the use of both the inlet control and outlet control nomographs (for examples, refer to Figure 11-9 for an inlet control nomograph and Figure 11-10 for an outlet control nomograph). Data can be compiled in the design computation form shown on Figure 11-8. Steps in the nomograph procedure are listed below:

1. List design data on the design computation form:

    ▪ Q (cfs).

    ▪ L (ft).

    ▪   Invert elevations for culvert inlet and outlet (ft).

    ▪   Allowable H_w (ft).

    ▪   Mean and maximum flood velocities and depths in stream (ft/s).

    ▪   Culvert type, shape and entrance type for first selection.

2. Determine a trial size culvert. Assume a maximum average velocity based on channel considerations and use this to compute the culvert's cross-sectional area (A) using the Continuity Equation ($A = Q/V$). Calculate the culvert diameter D that corresponds to A. Round D up to the nearest standard culvert size.

3. Find the headwater depth H_w for a trial size culvert for inlet control and outlet control. Select the appropriate inlet and outlet nomographs, based on the culvert diameter, entrance type, design discharge and allowable headwater, from the *Hydraulic Design of Highway Culverts* (FHWA 2005a). For inlet control (see Figure 11-9 for example inlet control nomograph), connect a straight line through D and Q to scale (1) of the H_w/D scales and project horizontally to the proper scale. (As noted on the nomograph, the different scales correspond to different culvert entrance types). Compute H_w and, if too large or too small, try another culvert size before computing H_w for outlet control.

4. Compute the H_w for outlet control (see Figure 11-10 for example outlet control nomograph). Connect the culvert diameter scale and the culvert length scale with a straight line (select the proper culvert length scale based on the type of culvert entrance). Draw a straight line from the design discharge on the discharge scale through the intersection point of the first drawn line and the turning point line and extend this to the head scale (head loss, H). Compute H_w from the equation:

$$H_w = H + h_o - Ls \qquad\qquad\qquad\qquad \text{Equation 11-10}$$

Where:

H_w= headwater depth (ft)
H = head loss (ft)
h_o = tailwater depth or height of the hydraulic grade line measured from the outlet invert (ft)
L = length of culvert (ft)
s = slope of culvert (ft/ft)

For T_w greater than or equal to the top of the culvert:

$$h_o = T_w$$

Equation 11-11

For T_w less than the top of the culvert:

$$h_o = \frac{(d_c + D)}{2} \text{ or } T_w \text{ (whichever is greater)}$$

Equation 11-12

Where:

h_0 = approximate height of hydraulic grade line above outlet invert (ft)
d_c = critical depth (ft)
D = culvert diameter (ft)
T_w= tailwater depth (ft)

Compare the headwater elevations calculated with the inlet and outlet control nomographs; the higher H_w dictates whether the culvert is under inlet or outlet control. If outlet control governs and the H_w is unacceptable, select a larger trial size culvert and find another H_w with the outlet control nomographs. After a larger pipe size is selected by the outlet control nomograph, it does not need to be re-checked for headwater with the inlet control nomograph, since the smaller size of culvert had previously been evaluated for allowable headwater based on inlet control.
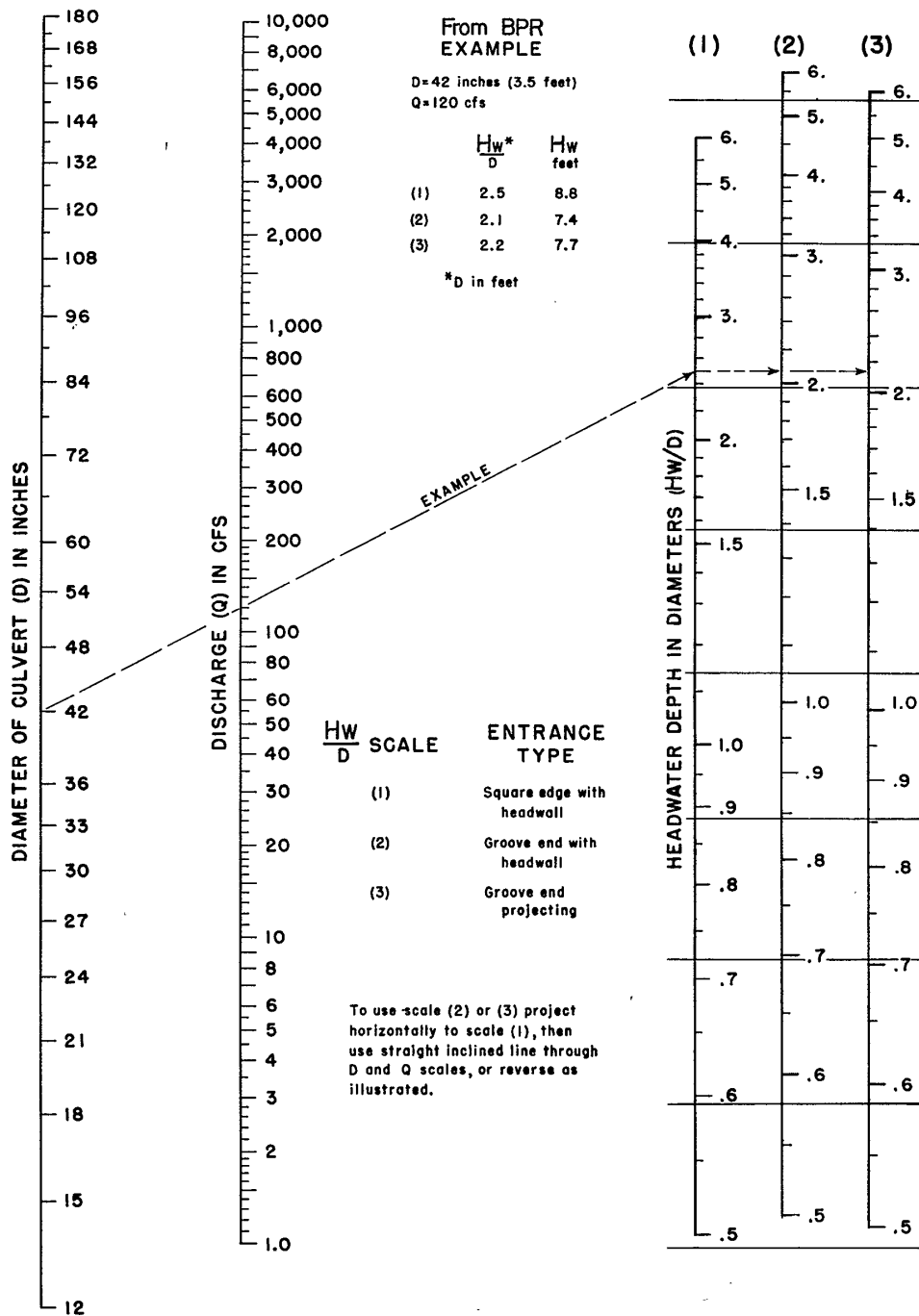
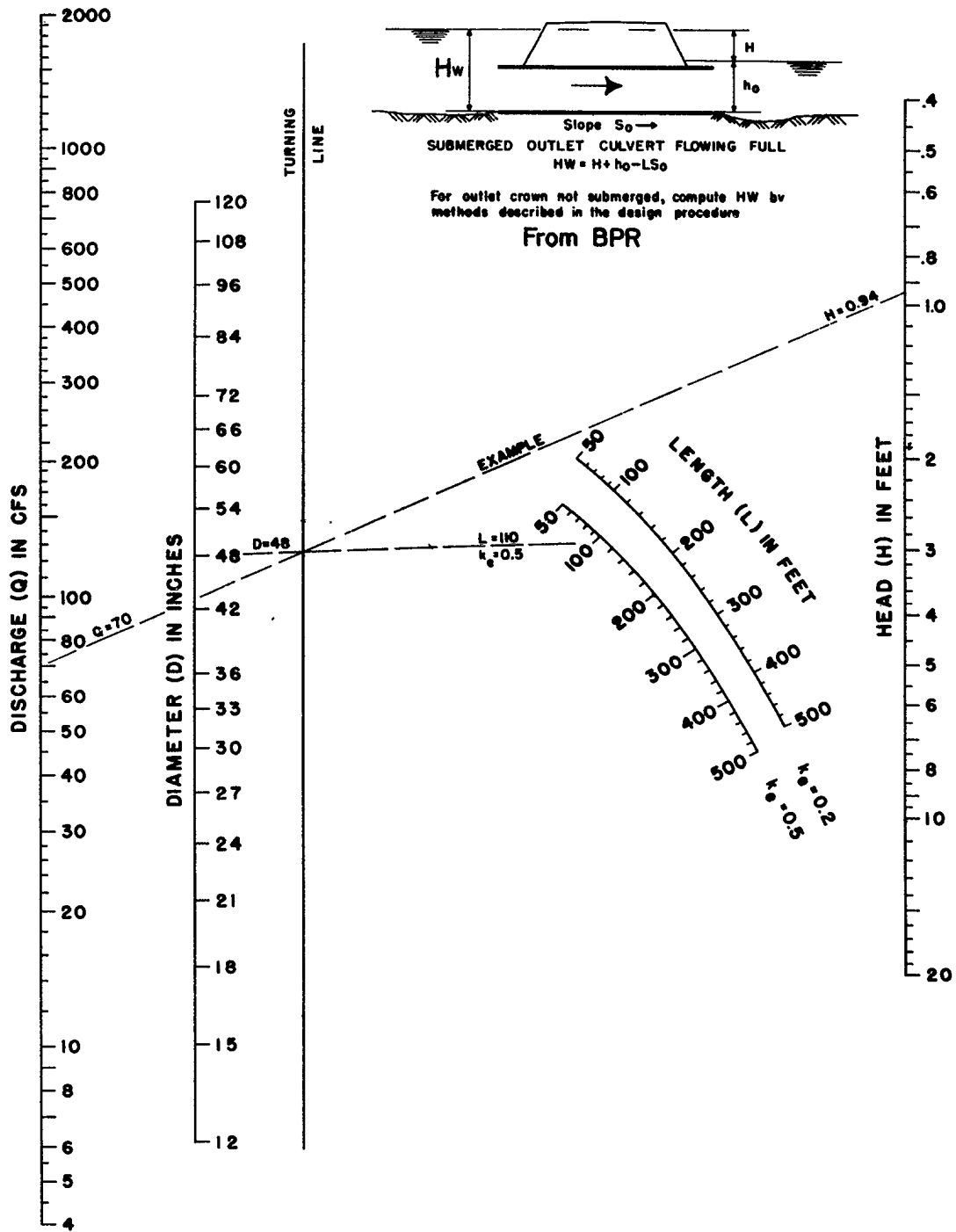**Figure 11-9. Inlet control nomograph—example**

Figure 11-10. Outlet control nomograph—example

4.3   Computer Applications

Although the nomographs continue to be useful tools, especially for engineers who were trained in these methods, engineers increasingly use computer applications for culvert design. Examples of public domain computer applications that are acceptable by UDFCD for the hydraulic design of culverts are listed in the text box at right.

In addition to the public domain computer applications listed in the text box, numerous proprietary computer applications are also available for the hydraulic design of culverts. Proprietary model applications are discouraged because of the costs to municipalities and/or UDFCD to obtain and operate the proprietary software. UDFCD and municipalities may consider on a case-by-case basis whether the use of specific proprietary software may be used.

Computer Applications for the Hydraulic Design of Culverts

The following public domain computer applications are available for free download at the addresses shown and are acceptable for use in conjunction with this chapter:

- UDFCD Excel™ workbook: *UD-Culvert*
   www.udfcd.org

- Federal Highway Administration (FHWA) HY-8 Culvert Analysis program (FHWA 2009).

   http://www.fhwa.dot.gov/engineering/hydraulics/ software/hy8/

- U.S. Army Corps of Engineers Hydrologic Engineering Center - River Analysis System (HEC-RAS):

   http://www.hec.usace.army.mil/software/hec-ras/

4.4   Design Considerations

The design of a culvert installation is more difficult than the process of sizing culverts, since other considerations arise with site-specific factors. The procedure for design in this manual only represents guidelines, since actual design considerations encountered are too varied and too numerous to be generalized. However, the process presented should be followed to ensure that a special problem is not overlooked. Evaluate several combinations of entrance types, invert elevations, and pipe diameters to determine the most economic design that will meet the conditions imposed by topography and engineering.

Specific design considerations are identified and discussed in Sections 0.04.4.1 through 4.6.

4.4.1   Design Computation Forms

The use of design computation forms is a convenient method to use to obtain consistent designs and promote cost-effectiveness. An example form was shown previously on Figure 11-8.

4.4.2   Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate culvert length, culvert invert elevations must be assumed. Significant scour is not likely when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial. Investigate the flow conditions downstream from the culvert to determine if scour is likely and evaluate the area upstream of the planned culvert for the potential of debris and adverse consequences from increased sedimentation. Providing a drop at the outlet of the culvert and including a depressed basin consistent with drop structure details provided in the

Hydraulic Structures chapter provide a location for sedimentation without potential for clogging.

4.4.3   Minimum Culvert Diameter

Since small diameter pipes are often plugged by sediment and debris, UDFCD recommends a minimum pipe diameter of 15 inches for storm drains and culverts.

4.4.4   Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, it is necessary to either oversize the culvert barrel, lower the inlet invert, use an alternate cross section (arch or elliptical), or use a combination of the preceding to increase the discharge rate.

If the inlet invert is lowered, special consideration must be given to headcutting and scour from the acceleration of flow into the culvert. The use of a drop structure, riprap or other type of protection along with headwalls, apron and toe walls should be evaluated to obtain a proper design.

4.4.5   Culvert Outlet

The outlet velocity must be checked to determine if significant scour will occur downstream during the major storm. If scour is indicated, which is frequently the case, refer to the *Outlet Protection* section of this chapter (Section 6.0). Inadequate culvert outlet protection is a common problem. When adequate culvert outlet protection is not provided, the culvert outlet can be undermined and downstream channel degradation can be significant.

4.4.6   Minimum Slope

To minimize sediment deposition in the culvert, the culvert slope must be sufficient to maintain a minimum velocity of 3 feet per second during the average annual flow event. If the minimum velocity is not obtained based on the design slope and average annual flow event, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these employed to increase velocity.

5.0   Culvert Inlets

A culvert cannot convey any more water than can enter the inlet. This is frequently overlooked by engineers who give full consideration to slope, cross section, hydraulic roughness, and other parameters. Culvert designs using uniform flow equations rarely carry their design capacity due to limitations imposed by the inlet.

The longer a culvert is the more important is the design of the entrance. A large culvert unable to flow at the design capacity represents wasted investment. Typically, air vents are necessary immediately downstream of the entrance of a long culvertto allow entrained air to escape and to act as breathers should less-than-atmospheric pressures develop in the pipe.

Where constraints exist such as limited headwater depth, erosion problems, or where sedimentation is likely, a more efficient inlet may be required to obtain the necessary discharge for the culvert. Conversely, if detention or other temporary water storage upstream from the culvert is desirable, an inlet with more limited capacity may be the most desirable choice (in such a case, the embankment should effectively be designed as a dam). The design of a culvert, including both the inlet and the outlet, requires a balance between cost, hydraulic efficiency, purpose, and topography at the proposed culvert site.

The inlet types described in this chapter may be selected to fulfill either of the above requirements. The entrance coefficient, K_e, a variable in Equation 11-5, is a measure of the hydraulic efficiency at the inlet, with lower values indicating greater efficiency. Entrance coefficients recommended for use are given in Table 11-2. Different types of inlets and their suited uses are defined in Section 5.1.

Table 11-2. Entrance loss coefficients

Type of Entrance	Entrance Coefficient, K_e
Pipe entrance with headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Pipe entrance with headwall & 45° wingwall	
Grooved edge	0.20
Square edge	0.35
Headwall with parallel wingwalls spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Special inlets	
Projecting Entrance	
Grooved edge	0.20
Square edge	0.50
Sharp edge, thin wall	0.90

5.1   Types of Inlets

5.1.1   Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used as well as with the orientation of headwalls and wingwalls relative to the direction of flow entering the culvert. Figure 11-11 illustrates an inlet configuration with a headwall and wingwalls.

Photograph 11-2. Culverts at a skew can be sediment traps. Where they cannot be avoided consider how the design can best facilitate maintenance including sediment removal.
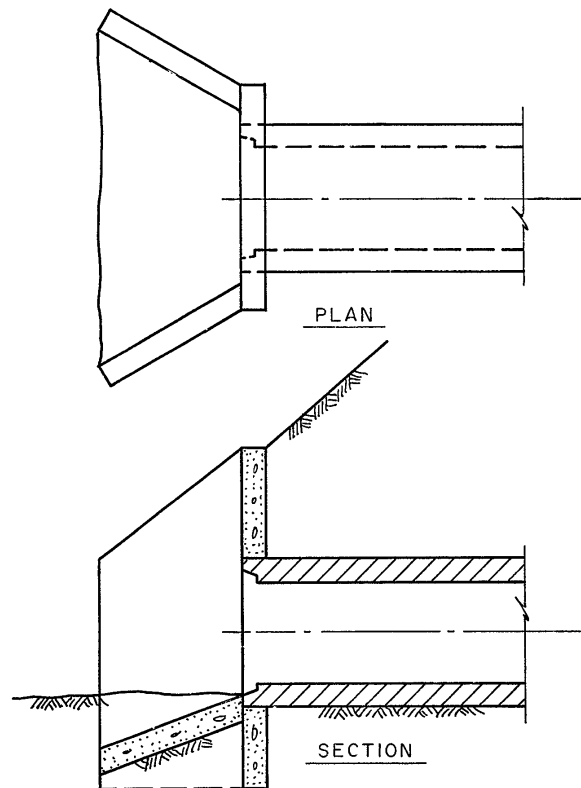
Figure 11-11. Inlet with headwall and wingwalls

Corrugated Metal Pipe
Corrugated metal pipe in a headwall is essentially a square-edged entrance with an entrance coefficient of approximately 0.4. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced as follows:
- Reduce to 0.15 for a rounded edge with a radius equal to 0.15 times the culvert diameter

- Reduce to 0.10 for rounded edge with a radius equal to 0.25 times the diameter of the culvert

Concrete Pipe
For tongue-and-groove or bell-end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reason for using headwalls is for embankment protection and for ease of maintenance. The entrance coefficients for concrete pipe are:
- 0.2 (approximate) for grooved and bell-end pipe

- 0.4 for cut concrete pipe

Wingwalls
Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and/or where the culvert is skewed to the normal channel flow. Wingwalls are often needed to transition from the channel bottom to the embankment slope without creating grades that are too steep. Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use should be justified for reasons other than an increase in hydraulic efficiency. Figure 11-12 illustrates several cases where wingwalls are used. For parallel wingwalls, the minimum distance between wingwalls should be at least 1.25 times the diameter of the culvert pipe.
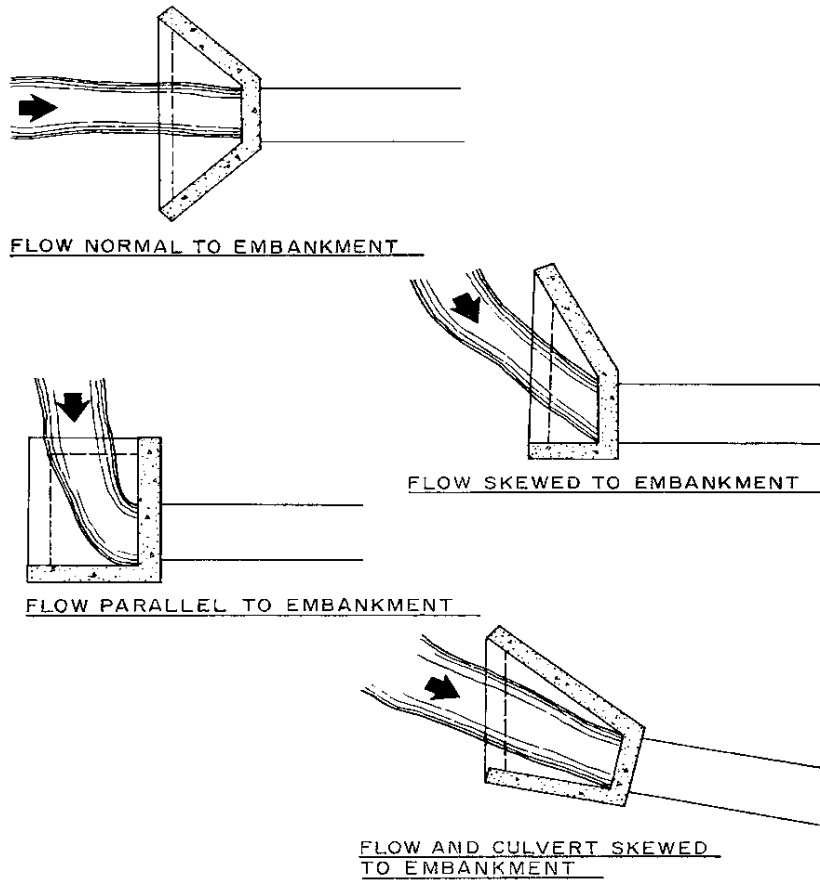
FLOW NORMAL TO EMBANKMENT

FLOW SKEWED TO EMBANKMENT

FLOW PARALLEL TO EMBANKMENT

FLOW AND CULVERT SKEWED
TO EMBANKMENT

Figure 11-12. Typical headwall-wingwall configurations

Aprons

If high headwater depths are to be encountered, or if the approach velocity of the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

Culverts with wingwalls should be designed with a concrete apron extending between the walls. Aprons must be reinforced to control cracking. As illustrated on Figure 11-12, the actual configuration of the wingwalls varies according to the direction of flow and will also vary according to the topographical requirement placed upon them.

For conditions where scour may be a problem due to high approach velocities and special soil conditions, such as alluvial soils, a toe wall/cutoff is often desirable for apron construction.

5.1.2   Special Inlets

A large variety of inlets exist in addition to those described previously. Among these are special end-sections (i.e., flared end sections), which are frequently used at both ends of the culvert. This section discusses special inlets for concrete and corrugated metal pipes, two of the most common pipe materials; although similar improved inlets are manufactured for other pipe types. Because of the difference in requirements due to pipe materials, special end-sections for corrugated metal pipe and concrete pipe are discussed separately. Separate discussions are also provided for mitered inlets and inlets for long conduits.

Corrugated Metal Pipe

Special end-sections for corrugated metal pipe add little to the overall cost of the culvert and have the following advantages:

- Less potential damage and maintenance compared to a projecting entrance.

- Increased hydraulic efficiency. When using design charts, as discussed in Section 4.0, charts for a square-edged opening for corrugated metal pipe with a headwall may be used.

Concrete Pipe

As is the case with corrugated metal pipe, concrete end-sections protect the end of the pipe during maintenance activities, and may aid in increasing the embankment stability or in retarding erosion at the inlet. When properly designed they can also provide an improved appearance compared to a projecting entrance.

The hydraulic efficiency of this type of concrete inlet is dependent on the geometry of the end-section to be used. Where the full contraction to the culvert diameter takes place at the first pipe section, the entrance coefficient, K_e, is equal to 0.5, and where the full contraction to the culvert diameter takes place in the throat of the end-section, the entrance coefficient, K_e, is equal to 0.25.

> ## Other Considerations for Long Culverts
>
> Whenever it is suspected the conduit could operate at Froude Number higher than 0.7 for flows up through the design flow, or when the headwater at the conduits entrance is above the top of the conduit, the engineer must consider installation of adequate air vents along the conduit. These are necessary to minimize major pressure fluctuations that can occur should the flow become unstable. When instabilities occur, air is trapped and less-than-atmospheric pressures have been shown to occur intermittently. Under this condition, air vents can mitigate and reduce structural loads and fluctuating hydraulic capacity in the conduit. Access manholes and storm inlets are useful for permitting air to flow in and out of a conduit as filling and emptying of the conduit occurs. They might also be helpful in providing water ejection ports should the conduit ever inadvertently flow full and cause a pileup of water upstream.
>
> A large rectangular conduit with a special entrance and an energy dissipater at the exit may need an access hole for vehicle use in case major repair work becomes necessary. A vehicle access point might be a large, grated opening just downstream from the entrance. This grated opening can also serve as an effective air vent for the conduit. Equipment may be lowered into the conduit by a crane or A-frame. A long conduit should be easy to inspect, and, therefore, access manholes are desirable at various locations. If a rectangular conduit is situated under a curb, the access manholes may be combined with inlets. Manholes should be aligned with the vertical wall of the box to allow rungs in the riser and box to be aligned.

Mitered Inlets
Mitered inlets are simply culvert pipes cut with the slope of the embankment. They are most commonly used with corrugated metal pipe. The hydraulic efficiency of mitered inlets is dependent on the construction procedure used. If the embankment is not paved, the entrance, in practice, usually does not conform to the side slopes, giving essentially a projecting entrance with $K_e = 0.9$. If the embankment is paved, a sloping headwall is obtained with $K_e = 0.60$ and, by beveling the edges, $K_e = 0.50$.
Uplift is an important factor for a mitered inlet. It is not good practice to use unpaved embankment slopes where a mitered entrance may be submerged to a depth of more than 1.5 times the culvert rise.

Inlets to Long Conduits
While inlets are important in the design of short culverts, such as road crossing, they are even more significant in the economical design of long culverts and pipes. Unused capacity in a long conduit is a wasted investment. Long conduits are costly and require detailed engineering, planning, and design work. The inlets to such conduits are extremely important to the functioning of the conduit and must receive special attention.

Most long conduits require special inlet considerations to meet the particular hydraulic characteristics of the conduit. Generally, on larger conduits, hydraulic model testing will result in better and less costly inlet construction. For additional considerations for long conduits, see the inset.

Inlets to Rectangular Conduits
The entrances take on a special degree of importance for rectangular conduits because the flow must be limited to an extent to ensure against overcharging the conduit. Special maximum-flow limiting entrances are often used with rectangular conduits. These special entrances should reject flow over the design discharge so that, if a runoff larger than the design flow occurs, the excess water will flow via other routes, often overland. A combined weir-orifice design is useful for this purpose. Model tests are needed for dependable design (Murphy 1971).
A second function of the entrance should be to accelerate the flow to the design velocity of the conduit, usually to meet the velocity requirements for normal depth of flow in the upstream reach of the conduit.

For additional considerations for rectangular conduits, see the inset on the following page.

Other Considerations for Rectangular Culverts

The use of rectangular conduits of large flow capacity can sometimes have cost advantages over large-diameter pipe. They can also be poured in place, allowing incorporation of utilities into the floor and roof of the structure.

Major disadvantages of rectangular conduits as storm sewers are:

1. The conduit's capacity drops significantly when the water surface reaches its roof since the wetted perimeter dramatically increases. The drop is 20% for a square cross section and more for a rectangular cross section where the width is greater than the height.

2. The economics of typical structural design usually does not permit any significant interior pressures, meaning that if the conduit reached a full condition and the capacity dropped, there could be a failure due to interior pressures caused by a choking of the capacity (Murphy 1971).

Internal Pressure: An obstruction, or even a confluence with another conduit, may cause the flow in a near-full rectangular conduit to strike the roof and choke the capacity. The capacity reduction may then cause the entire upstream reach of the conduit to flow full, with a resulting surge and pressure head increase of sufficient magnitude to cause a structural failure. When structural design has not been based on internal pressure, internal pressures are typically limited to no more than 2 to 4 feet of head. Surges or conduit capacity choking cannot be tolerated if the structure is not designed for the internal pressure resulting from these conditions. Thorough design is required to overcome this inherent potential problem.

Air Entrainment: Entrained air causes a swell in the volume of water and an increase in depth than can cause flow in the conduit to reach the height of the roof with resulting loss of capacity; therefore, hydraulic design must account for entrained air. In rectangular conduits and circular pipes, flowing water will entrain air at velocities of about 20 ft/sec and higher. Additionally, other factors such as entrance condition, channel roughness, distance traveled, channel cross section, and volume of discharge all have some bearing on air entrainment. Volume swell can be as high as 20% (Hipschman 1970).

Slope and Alignment: Structural requirements and efficiency for sustaining external loads, rather than hydraulic efficiency, usually control the shape of the rectangular conduit. A rectangular conduit should have a straight alignment and should not decrease in size or slope in a downstream direction. It is desirable to have a slope that increases in a downstream direction as an added safety factor against it flowing full. This is particularly important for supercritical velocities that often exist in long conduits.

Curves and Bends: The analysis of curves in rectangular conduits is critical to ensure its hydraulic capacity. When the water surface (normal, standing or reflecting waves) reaches the roof of the conduit, hydraulic losses increase significantly and the capacity drops. Superelevation of the water surface must also be investigated, and allowances must be made for a changing hydraulic radius, particularly in high-velocity flow. Dynamic loads created by the curves must be analyzed to assure structural integrity for the maximum flows. See the *Hydraulic Structures* chapter of the USDCM.

5.1.3   Projecting Inlets

Projecting inlets (protruding pipes at the inlet) should not be used. Headwalls, wingwalls, and flared end sections should be used to maximize efficiency and minimize turbulence, head loss, and erosion. This is especially important for flexible pipe materials (metal or plastic). This condition can cause severe suction and displacement of the pipe.

5.1.4   Improved Inlets

Inlet edge configuration is one of the prime factors influencing the performance of a culvert operating under inlet control. Inlet edges can cause a severe contraction of the flow, as in the case of a thin edge, projecting inlet. In a flow contraction, the effective cross-sectional area of the barrel may be reduced to about one-half of the actual barrel cross-sectional area. As the inlet configuration is improved, the flow contraction is reduced, thus improving the performance of the culvert.

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because the simple beveled edge is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet (Figure 11-13). FHWA (2005a) *Hydraulic Design of Highway Culverts* provides guidance on the design of improved inlets.
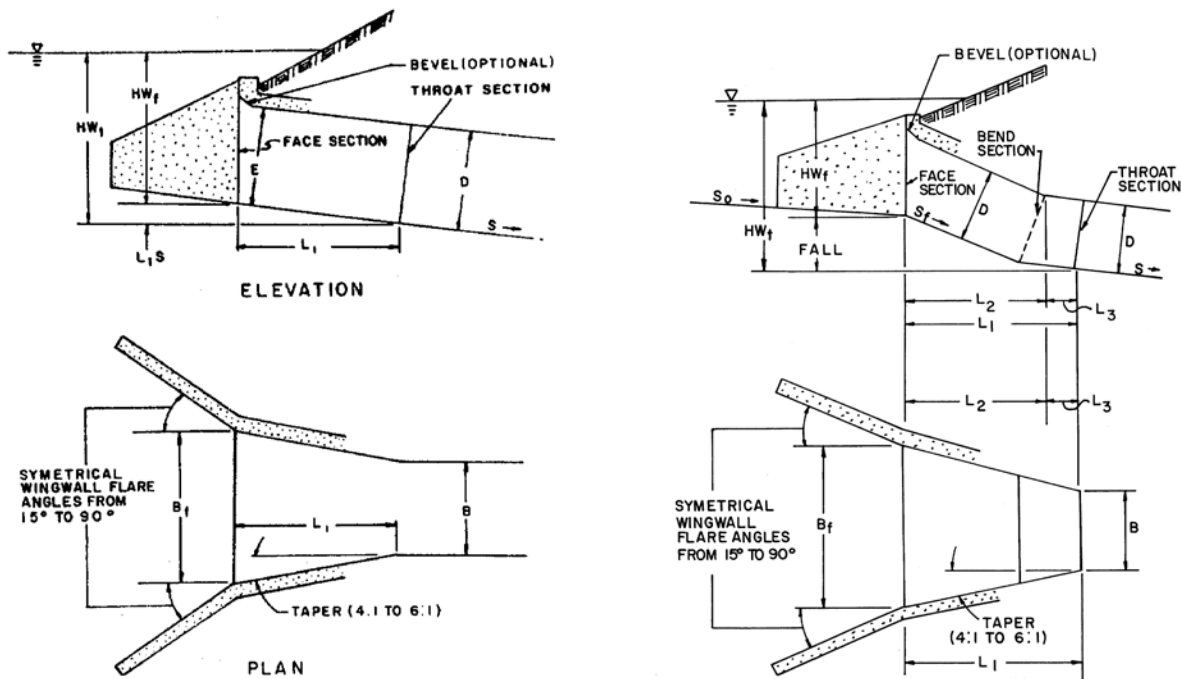

Figure 11-13. Side-tapered and slope-tapered improved inlets

5.2   Inlet Protection

Inlets on culverts, especially on culverts to be installed in live streams, should be evaluated relative to debris control and buoyancy.

5.2.1   Debris Control

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. This may result in damage due to inundation of the road and upstream property. The designer has three general options for addressing the problem of debris plugging a culvert:

Retain the debris upstream of the culvert.

Attempt to pass the debris through the culvert.

Install a bridge to create more cross-sectional area for passage of debris past the embankment.

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. The design of a debris control structure should include a thorough study of the debris problem and should consider the factors listed in the text box below.

Debris Study Considerations

Factors to be considered in a debris study include:

- Type of debris
- Quantity of debris
- Alternate overland flow paths (under plugged conditions)
- Expected changes in type and quantity of debris due to future land use
- Stream flow velocity in the vicinity of culvert entrance
- Maintenance access requirements
- Availability of storage
- Maintenance plan for debris removal
- Assessment of damage due to debris clogging, if protection is not provided

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Hydraulic Engineering Circular No. 9, *Debris Control Structures Evaluation and Countermeasures* (FHWA 2005b), should be used when designing debris control structures.

5.2.2   Buoyancy

The forces acting on a culvert inlet during flows are variable and indeterminate. When a culvert is functioning under inlet control, an air pocket forms just inside the inlet, creating a buoyant effect when the inlet is submerged. The buoyancy forces increase with an increase in headwater depth under inlet control conditions. These forces, along with vortexes and eddy currents, can cause scour, undermine

culvert inlets, and erode embankment slopes, thereby making the inlet vulnerable to failure, especially if deep headwater conditions are present.

In general, installing a culvert in a natural stream channel constricts the normal flow. The constriction is accentuated when the capacity of the culvert is impaired by debris or damage.

The large unequal pressures resulting from inlet constriction are, in effect, buoyant forces that can cause entrance failures, particularly on corrugated metal pipe with mitered, skewed, or projecting ends. The failure potential will increase with steepness of the culvert slope, depth of the potential headwater, flatness of the fill slope over the upstream end of the culvert, and the depth of the fill over the pipe.

Anchorage at the culvert entrance helps to protect against these failures by increasing the deadload on the end of the culvert, protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. Providing a standard concrete headwall or endwall helps to counteract the hydrostatic uplift and to prevent failure due to buoyancy.

Because of a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet, a large bending moment is exerted on the end of the culvert, which may result in failure. This problem has been noted in the case of culverts under high fills, on steep slopes, and with projecting inlets. In cases where upstream detention storage is necessary and headwater depth in excess of 20 feet is required, to restrict discharge it is recommended to reduce the culvert size rather than use an inefficient projecting inlet.

5.3   Safety Grates

Always consider the use of safety grates at inlets to culverts and underground pipes while also evaluating hydraulic forces and clogging potential. Several fatalities can be attributed to the lack of a safety grate on small (< 42-inch) pipes and long culverts (See Table 11-3). At the same time, field experience has shown that undersized or poorly designed grates can become clogged during heavy runoff and the culvert may be rendered ineffective.

Based on UDFCD investigations of culvert related fatalities, safety grating should be required when any of the following conditions are or will be true:

- It is not possible to "see daylight" from one end of the culvert to the other,

- The culvert is less than 42 inches in diameter, or

- Conditions within the culvert (bends, obstructions, vertical drops) or at the outlet are likely to trap or injure a person.

Exceptions to the above criteria consist of street curb-opening inlets and driveway culverts that are subject to a ponding depth of no more than 12 inches at the flow-line and culvert entrances that are made inaccessible to the public by fencing.

The safety grate design process is a matter of identifying all safety hazard aspects and then taking reasonable steps to minimize them while providing adequate inflow capacity to the culvert. Generally, the most common aspect to consider in evaluating the safety hazard of a culvert (or underground pipe) opening is the possibility of a person, especially children, being carried into the culvert or becoming pinned at the culvert entrance by flowing water approaching the inlet. In reviewing hazards, it is necessary to consider depth and velocity of flow, surrounding site features, the appearance of the site

during high water (i.e, what will be visible to someone that may be unfamiliar with the site and what will be hidden), the length and size of the culvert, and other similar factors. Furthermore, in the event that someone is carried to the culvert with the storm runoff, the exposure hazard may in some cases be even greater if the person is pinned to the grating by the hydrostatic pressure of the water rather than being carried through the culvert. Large, sloped grates anchored well in front of the culvert entrance reduce the risk of pinning.

Where public safety and/or debris potential indicate that a safety grate is required, Use Figure OS-1 in Volume 3 of the USDCM to size the grate while separately ensuring that velocity does not exceed 2 feet per second at every stage of flow entering the culvert. The grate should be inclined at a slope no steeper than 3(H):1(V) (flatter is better) and have a clear opening at the bottom of no more than 9 inches to permit passage of debris and bed load at lower flows. Large debris can still become trapped behind the safety grate so it is also important to consider how maintenance personal will access this area when necessary. Access could be via a manhole access behind the headwall, a hatch within the grate, or a hinged grate. Based on site specifics, consider the option to lock access behind the safety grate. The bars on the face of the grate should be parallel to flow and spaced to provide no more than 5-inch clear openings. Transverse support bars located at the back of the grate need to be as few as possible, but sufficient to keep the grate from collapsing under full hydrostatic loads.

Grating should not be installed at the outlet of a culvert or storm drain because a human swept into the culvert will be trapped inside the grate where they will face certain death. Additionally, debris will impinge against the grate and cause significant flow capacity reductions and potential flood upstream areas. Pressurization in the pipe can also result in an unreasonable risk to the health and safety of the public.

Photograph 11-3. Grating at conduit outlets are prone to clogging and will hamper rescue efforts, cause pressurization of the pipe, and potentially flood upstream areas.

Safety Grate Design

- Use Figure OS-1 in Volume 3 of the USDCM to size the grate. This requires an open area at least four times the outlet pipe for outlets having a minimum dimension of 24 inches and greater.

- Ensure velocity does not exceed 2 feet per second.

- Incline the slope of the grate to 3(H):1(V) or flatter.

- Design a clear opening at the bottom of no more than 9 inches.

- Place bars on the face of the grate parallel to flow.

- Limit the openings between bars to no more than 5-inchs clear.

- Design access to the back side of the grate for maintenance and debris removal.

Table 11-3. Pipe and culvert related fatalities

Date	Location	Pipe Diameter	Culvert Length	Fatalities	Age	Survivors	Description
5/5/1996	Kentucky	30-inch	>100 feet	1	9	1, injured	Tried to cross ponded water.
8/4/1998	Illinois	12-inch	0.5 miles	1	6		Playing in ponded water.
9/9/1999	Delaware	unreported	1500 feet	2	11, 12	1 (age 8)	Tried to cross ponded water.
8/17/2000	Colorado	48-inch	900 feet	1	37		Firefighter attempted rescue in ponded water. Ten to 12 feet of headwater at inlet.
9/23/2000	Ohio	14-inch	N/A	2	13		Boys playing in basin. It filled quickly to 15 feet of head on a 14-inch unprotected culvert.
9/20/2009	Illinois	unreported	unreported	1	56		Attempting to clear debris from the inlet of a detention discharge pipe.
2/19/2011	California	Large box culvert	2600 feet	2	16,17		Attempted to raft a small tributary and unintentionally entered a walled section followed by a long box culvert.
5/31/2013	Oklahoma	Large box culvert	1200 feet	5	3 to 21	6	Sought shelter during a tornado warning.
6/20/2014	Nebraska	96-inch	1100 feet	1	29		Drove car into ditch and was swept into the culvert after escaping his partially submerged car.
6/30/2014	Iowa	54-inch	> 1 mile	1	17	1	Attempted to retrieve a flying disc.
5/19/2015	Louisiana	unreported	unreported	1	15		Drowned after being swept into an unprotected irrigation drainage pipe on his property.
5/23/2015	Oklahoma	30-inch	600	1	44	1	Firefighter attempted to cross ponded water, swept into storm drain.
5/24/2015	Texas	24-inch	800 feet	1	14		Unobserved entry.

5.3.1   Collapsible Grating

UDFCD does not generally recommend the use of collapsible grating. On larger culverts where grating is found to be necessary, the use of collapsible grating may be desirable. Such grating must be carefully designed from the structural standpoint so that collapse is achieved with a hydrostatic load of approximately one-half of the maximum allowable headwater. Collapse of the grate should be such that it clears the waterway opening adequately to permit the inlet to function properly.

5.3.2   Upstream Trash Collectors

In lieu of a collapsible grate and where a safety hazard exists, a grate situated a reasonable distance upstream from the actual inlet is often satisfactory. This type of grating may be a series of vertical pipes or posts embedded in the approach channel bottom. If blocking of this grating occurs, the backwater effect causes water to flow over the top of the grating and into the culvert with only minimal upstream backwater effect. The grating must not be so high as to cause the water to rise higher than the maximum allowable elevation.

6.0   Outlet Protection

Scour at culvert outlets is typical and mitigation must be included in the design. This section provides background information and speaks to the complexity of this transitional area. See the *Hydraulic Structures* chapter for detailed discussion and details of outlet protection practices.

Compared to the stream, flow in a culvert barrel is usually confined to a lesser width and greater depth. This results in increased velocity and potentially erosive capabilities as flow exits the barrel. Turbulence and erosive eddies form as the flow expands to conform to the natural channel. However, the velocity and depth of flow at the culvert outlet and the velocity distribution upon reentering the stream are not the only factors that need consideration.

Photograph 11-4. Energy dissipation and outlet protection are essential to promote channel stability.

The characteristics of the stream bed and bank material, velocity, and depth of flow in the stream at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Due to the variation in expected flows and the difficulty in evaluating some of these factors, scour prediction is not a exact science.

As discussed in the *Hydraulic Structures* chapter, riprap channel expansions and concrete aprons stabilize the transition and redistribute or spread the flow. Barrel outlet expansions operate in a similar manner. Headwalls and cutoff walls protect the integrity of the fill. At some locations, use of a rougher culvert material may alleviate the need for a special outlet protection device. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices. These include hydraulic jump basins, impact basins, drop structures, and stilling wells. Design information for the general types of energy dissipators is provided in the *Hydraulic Structures* chapter of the USDCM and in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 1983 and 2000).

7.0   Bridges

7.1   General

Bridges are used to carry roadways, railroads, shared-use paths, and utilities over surface waters. Generally a bridge is defined as having a span of 20 feet or more, as opposed to a culvert. If a bridge is not sized properly with regard to the design flow, overtopping and flooding will occur, leading to public hazards, erosion damage, and possible structural failure. However, bridge design also includes assumption of a certain level of risk that is usually determined by the owner or local jurisdiction. This section provides a brief

Photograph 11-5. Shared-use path over Grange Hall Creek.

overview of hydraulic design of bridges, and includes references for additional design guidance. Structural design is not addressed here – for that information, readers are directed to the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges.

There are many references for bridge hydraulics, some of which are available online. A key source of information is the Federal Highway Administration (FHWA). A listing of references available through their website can be accessed using the following link: http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

Some of the key references for bridge hydraulics published by FHWA and others are provided below:

- Federal Highway Administration, Hydraulics of Bridge Waterways, Hydraulic Design Series No. 1, 1978.

- Federal Highway Administration, Hydraulic Design of Safe Bridges, Hydraulic Design Series No. 7, 2012.

- Federal Highway Administration, River Engineering for Highway Encroachments – Highways in the River Environment, Hydraulic Design Series No. 6 (FHWA HDS-6), December 2010.

- American Association of State Highway and Transportation Officials (AASHTO), Highway Drainage Guidelines, 2007. Chapter 7: Hydraulic Analysis for the Location and Design of Bridges.

- Arizona Department of Water Resources. Design Manual for Engineering Analysis of Fluvial Systems. March 1985.

The Colorado Department of Transportation (CDOT) also provides a good reference on bridge design and hydraulics in Chapter 10 of the CDOT Drainage Design Manual. This is available on their website, www.coloradodot.info/.

Most roadway bridges are designed to pass the 100-year flood event. However, other types of bridges (such as for shared-use paths) may allow a greater risk and lesser design capacity. Designers should always verify the design event with the owner and local jurisdiction.   If the bridge is located within a regulatory floodplain, special consideration must be given to the impacts of the bridge on 100-year floodplain water surface elevations. Contact the local government to determine requirements. At a minimum a floodplain development permit will be required. Impacts to federally designated floodplains may require a Letter of Map Change with FEMA.

7.2   Backwater and Hydraulic Analysis

Bridge openings should be designed to have minimal impact on the flow characteristics and floodplain. However, most bridges and abutments create a constriction of the floodplain. This constriction and losses through the structure create a backwater surface increase on the upstream side of a bridge. Ideally, the backwater elevation remains below that of the bridge deck for critical design discharges. Backwater can be determined with manual calculations or through use of a computer model. The computer program most used is the model HEC-RAS, developed by the U.S. Army Corps of Engineers (USACE) and available online at www.hec.usace.army.mil/.   Other 1- and 2-dimensional hydraulic models include both public and proprietary software programs. FEMA maintains a list of software approved for the basis of map changes on their website, www.fema.org.

HEC-2 was a common computer model used by FEMA for establishing floodplain water surface profiles until 1995, when it was replaced with HEC-RAS. The HEC-RAS User's Manual and Hydraulic Reference Guides (also available through the USACE website) provide a thorough description of the input parameters required for the model. In addition, some considerations to remember in a bridge analysis include:

▪ Proper location of cross sections at the bridge (see Figure 11-14 and Figure 11-15)

▪ Increase in expansion and contraction coefficients upstream and downstream of the bridge.

▪ Definition of ineffective flow areas at the approach to and exit from the bridge (Figure 11-15). Additional cross sections located within the contraction and expansion reaches (as shown in Figure 11-14) should have ineffective flow areas defined based on the locations of the dashed lines within the cross section.
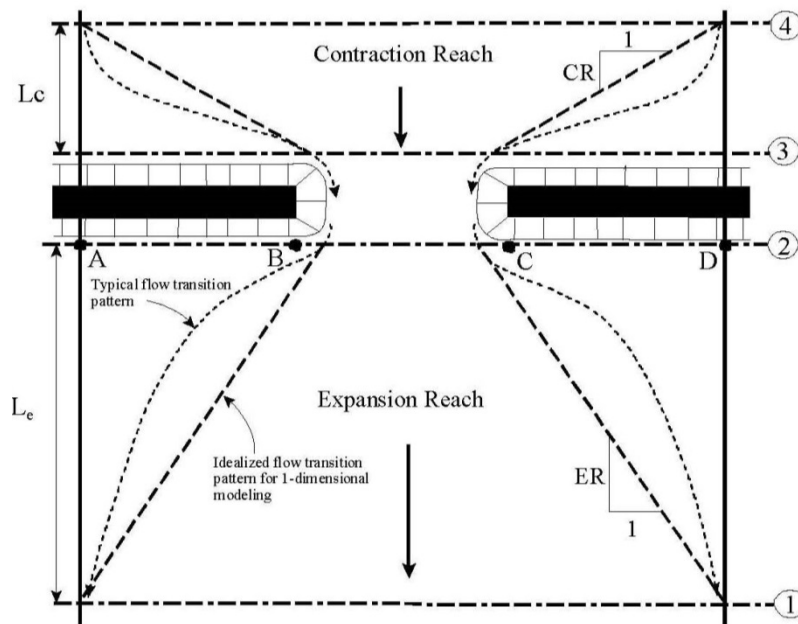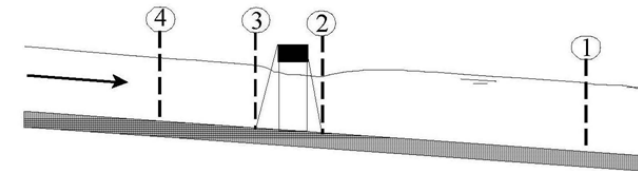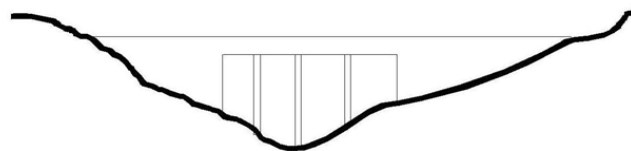
Figure 11-14. Hydraulic cross section locations

(Source: HEC-RAS Hydraulic Reference Manual)

Figure 11-15. Cross section locations and ineffective flow area definition

(Source: HEC-RAS Hydraulic Reference Manual)

7.3   Freeboard

Contrary to culverts which are typically designed with a backwater elevation, freeboard for bridges is critical because the heavy debris flow that can occur during a major flood can permanently damage the structure, potentially leaving an important roadway out of service. Bridge freeboard is the vertical distance between a design water surface elevation and the low chord of the bridge superstructure. It is a key component in bridge hydraulic design. Freeboard accommodates the inherent uncertainty of the design flow rate and also accommodates the passage of ice, debris, and waves during a flood event. Criteria for bridge freeboard vary from 1-foot to 4-feet in Colorado depending on the jurisdiction and risk of debris specific to the channel. Additionally, some criteria define freeboard based on the geometry of the bridge (e.g., Colorado Department of Transportation (CDOT) provides figures for measuring freeboard for bridges with a vertical curve and continuous grade).   When the local jurisdiction does not have criteria regarding to bridge freeboard, refer to CDOT, Colorado Water Conservation Board (CWCB), or AASHTO as appropriate.

7.4   Bridge Scour Analysis

The increased flow velocities at a bridge constriction often leads to scour, which is of particular concern because it can undermine a bridge's foundations and potentially cause collapse of the structure. Established methodologies for estimating scour at bridges are contained in the FHWA guidelines below (both available at www.fhwa.dot.gov/engineering/hydraulics/):

▪   Federal Highway Administration, Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18), Fifth Edition, 2012.

▪   Federal Highway Administration, Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20 (HEC-20), Fourth Edition, 2012.

The methods described in HEC-18 and HEC-20 are incorporated into the HEC-RAS computer program in its Hydraulic Design/Scour module. The program will automatically calculate the needed input parameters to the scour routines from the hydraulic output. However, it is critical to understand what the parameters are, if the program is calculating them correctly, and whether or not the resulting values are reasonable. This can depend on the way data are imported for bridge geometry, bank stations, and other input variables. Localized bridge scour is comprised of:

▪   Contraction scour

▪   Local Scour (Piers)

▪   Local Scour (Abutments)

These 3 components are all added together to arrive at a final scour envelope (Figure 11-17).

FHWA recommends calculation of scour with the absence of riprap at roadway bridges. This includes both piers and abutments. Reliance upon riprap for overall bridge stability and foundation design is not advised. However, riprap is often used as a scour countermeasure. FHWA provides guidance on selecting and designing scour countermeasures, including riprap at bridge piers and abutments:

▪   Federal Highway Administration, Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance. Hydraulic Engineering Circular No. 23, Third Edition, 2009. Volumes 1 and 2.
     http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=23&id=142
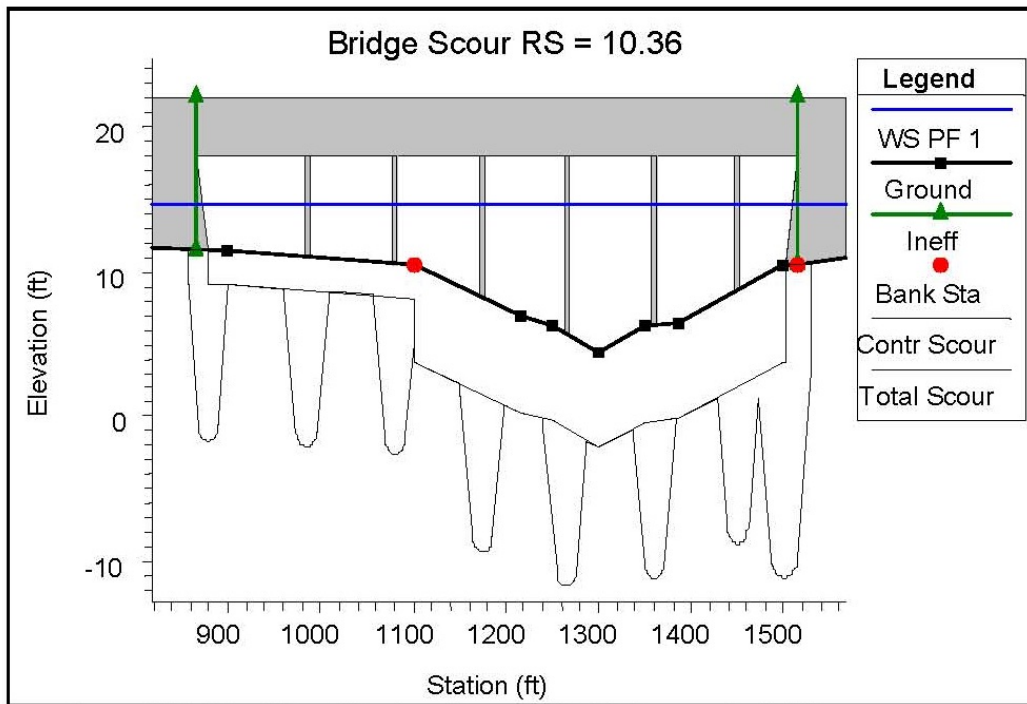
Figure 11-16. Example of scour envelope, as calculated with HEC-RAS

(Source: HEC-RAS Hydraulic Reference Manual)

It is important to note that the above methodology for calculating scour assumes that unconsolidated alluvial material makes up the channel bottom within the scour envelope. If bedrock is located within the scour envelope, or especially if bedrock is exposed at the surface of the channel bottom, other methodology should be used to determine the bedrock erodibility. The Erodibility Index Method was developed to evaluate scour in bedrock and is described in *Scour Technology: Mechanics and Engineering Practice,* 420 pp. (Annandale 2006)

A scour analysis must address long-term patterns of channel change. An understanding of fluvial geomorphology is important in determining this portion of the analysis. This includes evaluation of sediment transport, patterns of channel invert or overbank lowering (degradation), patterns of deposition (aggradation), and lateral migration. Aggradation can lead to a loss of capacity under a bridge, and degradation can cause undermining of a bridge foundation. In the case of long term degradation of a channel, grade control structures downstream of the bridge might be considered. However, it is important to note that local scouring around a bridge's structural elements can still occur even with grade control structures. Long term aggradation indicates the possible need for upstream bed and bank stabilization measures that would reduce sediment loading. These issues are described in the FHWA HDS-6 and in the Arizona Department of Water Resources design manual (both referenced at beginning of this chapter). In addition, many fluvial geomorphic textbooks are available.

FHWA is continually studying scour at bridges as part of its Scour Technology program. Updated information can be found at: http://www.fhwa.dot.gov/engineering/hydraulics/scourtech/index.cfm. Recently, advancements have been made in the methods for estimating scour at bridges under the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board of the National Academies. A list of NCHRP projects can be found at http://www.trb.org/NCHRP/NCHRPProjects.aspx. Bridge scour studies are included in Research Field 24. Such advancements were incorporated into the 2012 version of HEC-18.

8.0   Design Examples

This section demonstrates culvert design using two different methods presented in this chapter. For the purpose of comparison, the following problem is used for both examples:

Size a culvert given the following:

$Q_{5\text{-yr}}$ = 20 cfs, $Q_{100\text{-yr}}$ = 35 cfs, L = 95 feet

The maximum allowable headwater elevation is 5288.5 ft. Channel invert elevations are 5283.5 at the inlet and 5281.5 at the outlet. The tailwater depth is computed as 2.5 feet for the 5-year storm, and 3.0 feet for the 100-year storm. Assume the channel is an excavated channel with gravel (uniform section, clean) and the culvert is circular.

8.1   Example using UD-Culvert

The following example problem for a culvert under an embankment illustrates the culvert design procedures using UD-Culvert workbook. Note that UD-Culvert is based on HY-6.

Solution:

Step 1.   Calculate tailwater elevations:

$T_{w\ 5\text{-yr}}$ = 5,281.5 ft + 2.5 ft = 5,284.0 ft

$T_{w\ 100\text{-yr}}$ = 5,281.5 ft + 3.0 ft = 5284.5 ft

Step 2.   Set invert elevations at natural channel invert elevations to avoid scour. Compute culvert slope and $L/100s$:

$$S = \left(\frac{5283.5 - 5281.5}{95} \right) = 0.021$$

$$\frac{L}{100s} = \left(\frac{95}{2.1} \right) = 45.2$$

Step 3.   Start with an assumed culvert size for the 5-year storm by adopting a velocity of 6.5 ft/s and computing:

$$A = \frac{20}{6.5} = 3.1 \text{ ft}^2$$

This corresponds to a culvert diameter of 2 feet (24 inches):

$$D = 2\sqrt{\frac{A}{\pi}} = 2 \text{ ft}$$

Step 4.   For this example, assume a square edge with headwall ($K_e = 0.5$) and concrete pipe will with a Manning's n of 0.013.

Step 5.   Note that for the 5-year flow rate of 20 cfs for the given input parameters, the workbook indicates that the culvert will be able to pass the design flow rate at an elevation slightly below 5,286.5. However, with the increased tailwater during the 100-year event, a larger culvert will be needed to pass the 100-year design flow below the allowable headwater limit of 5,288.5. A larger culvert size should be selected and analyzed.

Step 6.   Increase the culvert to 27 inches to pass the 100-year flow of 35 cfs. Using the same procedure detailed above, output shows that the culvert continues to be outlet controlled. However, the controlling culvert flow rate at the maximum headwater depth of 5288.5 is adequate to pass the 100-year flow.

Project: **Culvert design example**
Basin ID:
Status:
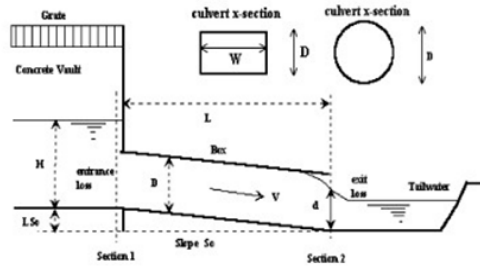
Clear Worksheet

Clear Results

Calculate

Design Information (Input):

Circular Culvert: Barrel Diameter in Inches	D = 24	inches
Inlet Edge Type (choose from pull-down list)	Square End with Headwall	Square End with Headwall ▼
OR:		
Box Culvert: Barrel Height (Rise) in Feet	Height (Rise) =	ft.
Barrel Width (Span) in Feet	Width (Span) =	ft.
Inlet Edge Type (choose from pull-down list)	Square Edge w/ 90-15 Deg. Headwall	Square Edge w/ 90-15 Deg. Headwall ▼
Number of Barrels	No = 1	
Inlet Elevation at Culvert Invert	Inlet Elev = 83.5	ft. elev.
Outlet Elevation at Culvert Invert **OR** Slope of Culvert (ft v./ft h.)	Outlet Elev = 81.5	ft. elev.
Culvert Length in Feet	L = 95	ft.
Manning's Roughness	n = 0.013	
Bend Loss Coefficient	K_b = 0	
Exit Loss Coefficient	K_x = 1	

Design Information (calculated):

Entrance Loss Coefficient	K_e =	0.50
Friction Loss Coefficient	K_f =	1.17
Sum of All Loss Coefficients	K_s =	2.67
Orifice Inlet Condition Coefficient	C_d =	0.86
Minimum Energy Condition Coefficient	KE_{low} =	-0.0342

Calculations of Culvert Capacity (output):

Recalculate

Water Surface Elevation Enter HW Elev (ft., linked)	Tailwater Surface Elevation ft	Culvert Inlet-Control Flowrate cfs	Culvert Outlet-Control Flowrate cfs	Controlling Culvert Flowrate cfs (output)	Inlet Equation Used:	Flow Control Used
84.00	84.00	1.20	0.00	0.00	Min. Energy. Eqn.	N/A
84.50	84.00	4.50	9.31	4.50	Min. Energy. Eqn.	INLET
85.00	84.00	8.30	13.16	8.30	Regression Eqn.	INLET
85.50	84.00	13.00	16.12	13.00	Regression Eqn.	INLET
86.00	84.00	17.10	18.61	17.10	Regression Eqn.	INLET
86.50	84.00	20.50	20.81	20.50	Regression Eqn.	INLET
87.00	84.00	23.40	22.80	22.80	Regression Eqn.	OUTLET
87.50	84.00	25.90	24.62	24.62	Regression Eqn.	OUTLET
88.00	84.00	28.20	26.32	26.32	Regression Eqn.	OUTLET
88.50	84.00	30.30	27.92	27.92	Regression Eqn.	OUTLET
89.00	84.00	32.30	29.43	29.43	Regression Eqn.	OUTLET
89.50	84.00	34.10	30.85	30.85	Regression Eqn.	OUTLET
90.00	84.00	35.80	32.23	32.23	Orifice Eqn.	OUTLET

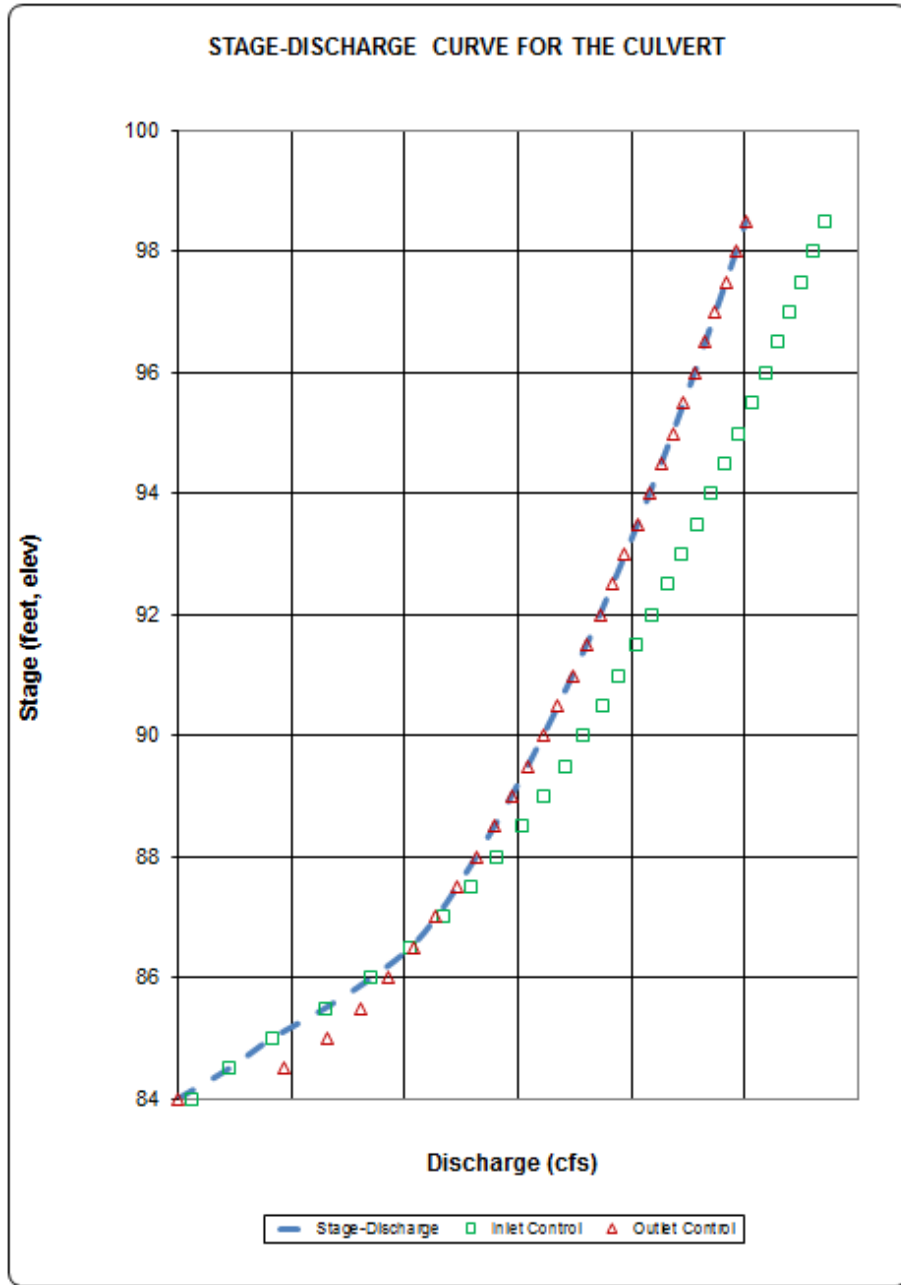**Figure 11-17. Example problem using UD-Culvert (5-year tailwater)**

Figure 11-18. Rating curve generated using UD-Culvert (5-year event)

8.2   Example Using HY-8

The example culvert design presented in section 8.1 is repeated here using the computer program HY-8. Note that UD-Culvert is based on HY-6, thus results will differ slightly from the example in section 8.1.

This section guides the user through the typical steps to set up and run a model in HY-8. To begin, start a new project by adding a "Crossing" with the information in Table 11-4 and the values solved for in the previous example.

Table 11-4. HY-8 program inputs

Parameter	Value
Min Flow (cfs)	0
Design Flow (cfs)	$Q_{5\text{-yr}}$ or $Q_{100\text{-yr}}$
Max Flow (cfs)	35
Channel Type	Trapezoidal Channel
Bottom Width (ft)	10
Side Slope (H:V) (_:1)	2
Channel Slope	0.03 ft/ft
Manning's n (channel)	0.025
Channel Invert Elevation (ft)	5283.5
Roadway Profile Shape	Constant Roadway Elevation
First Roadway Station (ft)	0
Crest Length (ft)	100
Crest Elevation (ft)	5288.5
Roadway Surface	Paved
Top Width (ft)	150
Shape	Circular
Material	Concrete
Diameter	2.0 feet
Embedment Depth	0
Manning's n	0.012
Inlet Type	Conventional
Inlet Edge Condition	Square edge with headwall and Groove end with headwall
Inlet Depression?	No
Inlet Station	0
Inlet Elevation	5283.5
Outlet Station	95
Outlet Elevation	5281.5
Number of Barrels	1

Figure 11-19. Adding a crossing in HY-8

The culvert may now be analyzed using "Analyze Crossing" near the bottom right corner of the box. This should generate an output screen that looks like Figure 11-20. If any critical input values are missing, the program will not execute properly.
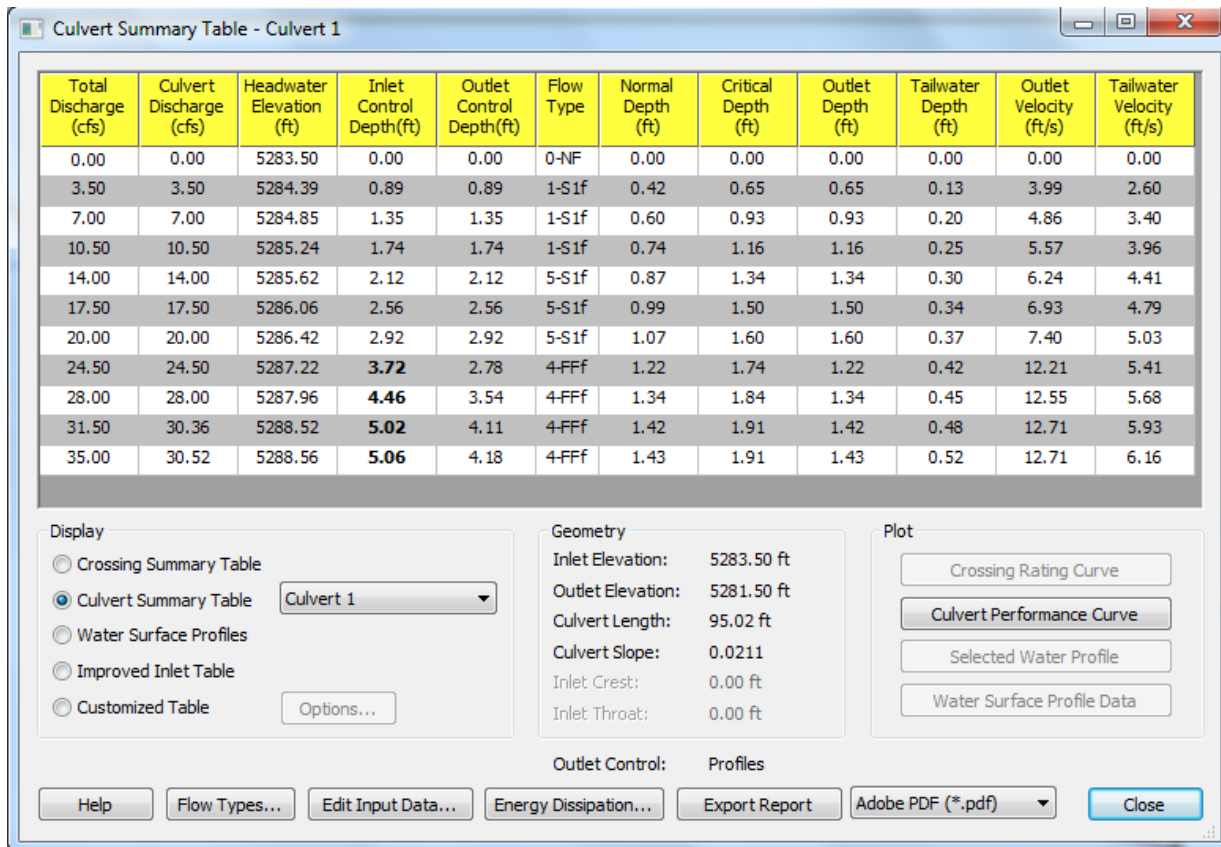
Figure 11-20. HY-8 output

HY-8 provides extensive output for modeled culverts and has options for exporting reports and generating rudimentary figures. Please refer to the HY-8 User's Manual for further interpretation of model output and options for presenting results.

9.0   Checklist

Criterion/Requirement	✓
Culvert diameter should be at least 18 inches.	
HW/D ratio should not exceed 1.5 unless justified and adequate measures are implemented to protect embankment.	
Safety grating is provided when any of the following conditions are or will be true: ▪ It is not possible to "see daylight" from one end of the culvert to the other, ▪ The culvert is less than 42 inches, or ▪ Conditions within the culvert (bends, obstructions, vertical drops) or at the outlet are likely to trap or injure a person.	
Review any proposed changes with local, state, and federal regulators.	
When a culvert is sized such that the overlying roadway overtops during large storms, check the depth of cross flow with the *Streets, Inlets, and Storm Drains* chapter.	
Provide adequate outlet protection in accordance with Section 6.0 of this chapter and the *Hydraulic Structures* chapter.	

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