

CHAPTER 7. CULVERT AND BRIDGE HYDRAULIC DESIGN

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EXECUTIVE SUMMARY

Purpose of the Chapter

The purpose of this chapter is to provide guidance for culvert and bridge hydraulic design. The primary objective of a culvert or bridge is to convey stormwater flows, based on a design flow rate, through embankments or under roadways without causing damage to adjacent properties and developments, the roadway, or to the drainage structure. Specifically, this chapter provides information on the criteria and methodology necessary to design culverts and bridges according to City requirements.

Chapter Summary

The function of culverts and bridges is to convey surface water under a highway, city street, railroad, recreation trail, or other embankment. In addition to the hydraulic function, the culverts and bridges must carry construction, highway, railroad, or other traffic and earth loads. Therefore, culvert and bridge design involves both hydraulic and structural design considerations. The hydraulic aspects and design loading criteria of culvert and bridge design are set forth in this chapter.

Culverts

Culverts are available in a variety of sizes, shapes, and materials. These factors, along with several others, affect their capacity and overall performance. Sizes and shapes may vary from small circular pipes to extremely large arch sections that are sometimes used in lieu of bridges.

The most commonly used culvert shape is circular, but arches, boxes, and elliptical shapes are used, as well. Pipe arch, elliptical, and rectangular shapes are generally used in lieu of circular pipe where there is limited cover. Arch culverts have application in locations where less obstruction to a waterway is a desirable feature, and where foundations are adequate for structural support. Box culverts can be designed to pass large flows and to fit nearly any site condition. A box or rectangular culvert lends itself more readily than other shapes to reduced allowable headwater situations since the height may be decreased and the span increased to satisfy the location requirements.

The material selected for a culvert is dependent upon various factors, such as durability, structural strength, roughness, bedding condition, abrasion and corrosion resistance, and water tightness. The more common culvert materials used are concrete and steel (smooth and corrugated).

Another factor that significantly affects the performance of a culvert is its inlet configuration. The culvert inlet may consist of a culvert barrel projecting from the roadway fill or mitered to the embankment slope. Other inlets have headwalls, wingwalls, and apron slabs or standard end-sections of concrete or metal.

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A careful approach to culvert design is essential, both in new land development and retrofit situations, because culverts often significantly influence upstream and downstream flood risks, floodplain management and public safety. Culverts can be designed to provide beneficial upstream conditions and to avoid negative visual impact.

Bridges

Bridge openings shall be designed to have as little effect on the flow characteristics as reasonable, consistent with good bridge design and economics. The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the major storm runoff. The design of a bridge opening generally determines the overall length of the bridge. The hydraulic engineering in the design of bridges has more impact on the bridge cost than does the structural design. **All structural calculations shall be in compliance with the AASHTO LRFD Bridge Design Specifications (current edition) and stamped by a structural engineer licensed in the State of Arkansas. Trail bridges shall be designed according to the LRFD Guide Specifications for Design of Pedestrian Bridges (current edition) and stamped by a structural engineer licensed in the State of Arkansas. The construction specifications shall be ARDOT's specifications modified appropriately to reflect Rogers as the owner rather than ARDOT.**

A majority of bridge failures are the result of scour. The added cost of reducing a bridge's vulnerability to damage from scour is small in comparison to the total cost of a bridge failure. Scour investigation is required by the AASHTO LRFD Bridge Design Specifications.

Critical Design Criteria

The summary below outlines some of the most critical design criteria essential to design engineers for proper drainage design of streets, inlets, and storm sewers according to City of Rogers requirements. The information below contains exact numerical criteria as well as general guidelines that must be adhered to during the design process. This section is meant to be a summary of critical design criteria for this section; however, the engineer is responsible for all information in this chapter. It should be noted that any design engineer who is not familiar with Rogers' Drainage Criteria Manual and its accepted design techniques and methodology should review the entirety of this chapter. If additional specific information is required, it will be necessary to review the appropriate section as needed.

Required Design Information

Information necessary for the design of culverts is summarized below:

- Design flood frequency and the corresponding design flow rate that the culvert must convey.
- Identify the impacts of various culvert sizes and dimensions on upstream and downstream flood risks, including the implications of embankment overtopping.

- Determine how the proposed culvert will fit into the relevant major drainageway master plan, and determine if there are multipurpose objectives that should be satisfied.
- Identify the necessary alignment, grade, and length of culvert.
- Determine the culvert size and type (material and shape).
- Determine the headwater depth, outlet velocity, and end treatment.
- Determine the inlet and outlet design and the need for special considerations.
- Determine the amount and type of cover.
- Identify public safety issues, including the key question of whether or not to include a safety/debris rack; handrails and/or guardrail.
- Identify the need for protective measures against abrasion and corrosion.
- Identify potential structural and geotechnical considerations that need to be addressed (these are beyond the scope of this chapter). The City may require a structural or geotechnical analysis.

Culvert Shapes and Sizes

- Refer to [Section 3.3.2](#) of this chapter for more detailed information/explanation.
- Box
- Circular
- Elliptical
- Arch

Culvert Sizes

- Refer to [Section 3.3.2](#) of this chapter for more detailed information/explanation.
- Minimum Pipe Size = 18 inches
- Minimum Box Size = "W" x 18 inches (width x height)

Culvert Material

- All pipe shall be installed per the manufacturer's specifications.
- Reinforced Concrete Pipe (RCP)
 - RCP ASTM Class III shall be used in all areas unless otherwise required due to fill heights; use ARDOT standards to determine.
 - Shall be used in all right-of-way areas and under all traffic areas (including parking lots, driveways, etc.)

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- RCP shall conform to:
 - ◆ Circular Pipe – AASHTO M170/ASTM C76
 - ◆ Arch-shaped Pipe – AASHTO M206/ASTM C506
 - ◆ Elliptical Pipe – AASHTO M207/ASTM C507.
- All storm sewer pipe having a diameter or hydraulically equivalent pipe size diameter of 36-inches or greater must be RCP.
- Minimum one-foot cover.
- Reinforced Concrete Box (RCB)
 - Box culverts shall be structurally designed to accommodate the earth and live loads to be imposed upon the culvert.
 - Shall comply with ARDOT's Reinforced Concrete Box Culvert Standard Drawings.
 - When installed within public right of way, all culverts shall be capable of withstanding a minimum HL-93 loading.
- **Materials other than reinforced concrete shall be approved by the City.**
- Corrugated Metal Pipe (CMP) [including Smooth Lined (SLCMP)]
 - CMP can only be used in areas outside of street right-of-way, but shall not be used under traffic areas.
 - CMP shall conform to shall conform to the following:
 - ◆ Galvanized Steel - AASHTO M218/ASTM A929; AASHTO M36/ASTM A760 and AASHTO Section 12/ASTM A796
 - ◆ Aluminized Steel Type 2 – AASHTO M274/ASTM A929; AASHTO M36/ASTM A760 and AASHTO Section 12/ASTM A796
 - ◆ Aluminum – AASHTO M197/ASTM B744; AASHTO M196/ASTM B745 and AASHTO Section 12/ASTM B790.
 - CMP shall have a minimum cover of 2-feet.
- Corrugated Polyethylene Pipe (CPP) [including Smooth Lined (SLCPP)]

- CPP may not be used:
 - ◆ in City right-of-way
 - ◆ under traffic areas
 - ◆ in City drainage easements
 - ◆ to convey water through a development from properties upstream
 - ◆ on properties where drainage structures are maintained by a residential POA
- CPP can only be used in situations where it is not draining off-site properties, after approval by the City.
- CPP up to 30-inches can be used in areas outside of the right-of-way and outside of city drainage easements.
- CPP shall conform to AASHTO M 294, Type S specification or ASTM F2648, ASTM D3350 and ASTM F2306.
- CPP shall have a minimum cover of 2-feet.

Culvert Physical and Operational Constraints

- Maximum Allowable Discharge Velocity:

Downstream Condition	Maximum Allowable Discharge Velocity (ft/sec)
Grass	5
Riprap	12
Concrete	18
Turf Reinforcement Mat	Manufacturer's Specs.

- Culvert flow velocity (minimum) = 3-ft/sec (when flowing full per HEC-22)
- Three methods of energy dissipation/erosion control ([Section 6.2](#) of this chapter) are Drop Structures, Turf Reinforcement Mats and Riprap. Riprap must be approved by City prior to use.
- Design Storm Frequency and Freeboard Policy:

Description	Design Storm Frequency	Minimum Freeboard (ft)
Culverts (Local Street)	10	1

Culverts (Collector)	25	1
Culverts (Minor Arterial & Major Arterial)	50	1
Bridges (Local & Collector Roadways)	50**	1*
Bridges (Arterial & Critical Service Access Roadways/Drives)	100	1*

* – from “Low Chord” / “Low Steel”

** – must pass 100 year water surface elevation below “Low Chord”/“Low Steel”

Refer to Chapter 4 Storm Sewer System Design Table ST-1 for allowable pavement encroachment and gutter depths.

1. CULVERTS INTRODUCTION AND OVERVIEW

The function of a culvert is to convey surface water under a roadway, railroad, trail, or other embankment. In addition to the hydraulic function, the culvert must carry construction, highway, railroad, or other traffic

and earth loads. Therefore, culvert design involves both hydraulic and structural design considerations. The hydraulic aspects of culvert design are set forth in this chapter.

Culverts are available in a variety of sizes, shapes, and materials. These factors, along with several others, affect their capacity and overall performance. Sizes and shapes may vary from small circular corrugated metal pipes to large concrete box sections that are sometimes used in lieu of bridges.

A careful approach to culvert design is essential, both in new land development and retrofit situations, because culverts often significantly influence upstream and downstream flood risks, floodplain management and public safety. Culverts can be designed to provide beneficial upstream and downstream conditions and to simultaneously avoid creating a negative visual impact.

The information and references necessary to design culverts according to the procedure given in this chapter can be found in FHWA's Hydraulic Design Series, No. 5 (HDS-5 2005 - <http://isddc.dot.gov/.../FHWA>), Hydraulic Design of Highway Culverts.

1.1. Required Design Information

The hydraulic design of a culvert consists of an analysis of the required performance of the culvert to convey flow from one side of an embankment to the other. The designer must select a design flood frequency, estimate the design discharge for that frequency, and set an allowable headwater elevation based on the selected design flood and headwater considerations. These criteria are dictated by the City of Rogers. The culvert size and type can be selected after the design discharge, controlling design headwater, slope, tailwater, and allowable outlet velocity have been determined.

The design of a culvert requires that the following be determined:

- Impacts of various culvert sizes and dimensions on upstream and downstream flood risks, including the implications of embankment overtopping.
- How will the proposed culvert/embankment fit into the relevant major drainageway master plan, and are there multipurpose objectives that should be satisfied?
- Alignment, grade, and length of culvert.
- Size, type, end treatment, headwater, and outlet velocity.
- Amount and type of cover.
- Public safety issues, including the key question of whether or not to include a safety/debris rack.
- Pipe material.

- Need for protective measures against abrasion and corrosion.
- Need for specially designed inlets or outlets.
- Structural and geotechnical considerations, which are beyond the scope of this chapter. The City may require a structural or geotechnical analysis.

1.1.1. Discharge

The discharge used in culvert design is usually estimated on the basis of a preselected storm recurrence interval, and the culvert is designed to operate within acceptable limits of risk at that flow rate. The design recurrence interval shall be based on the criteria set forth in [Section 3.1.1](#) of this chapter. Peak discharge rates for the design storm can be calculated using design methods described in Chapter 4 – *Determination of Stormwater Runoff*.

1.1.2. Headwater

Culverts generally constrict the natural stream flow, which causes a rise in the upstream water surface. The elevation of this water surface is termed *headwater elevation*, and the total flow depth in the stream measured from the culvert inlet invert is termed *headwater depth*.

In selecting the design headwater elevation, the designer shall consider the following:

- Roadway elevation above the structure and low point in roadway grade line.
- Elevation at which water will flow to the next cross drainage.
- Anticipated upstream and downstream flood risks, for a range of return frequency events.
- Potential damage to the culvert and the roadway caused by various headwater depths.
- Traffic interruption caused by overtopping a roadway with flood flows.
- Hazard to human life and safety caused by roadway or trail overtopping.
- Headwater/Culvert Depth (HW/D) ratio.
- Relationship to stability of embankment that culvert passes through.

The headwater elevation for the design discharge shall be consistent with the freeboard and overtopping criteria in [Section 3.1.1 \(Table CB-2\)](#) of this chapter and Chapter 5 – *Storm Sewer System Design*. The designer shall verify that the watershed divides are higher than the design headwater elevations. In flat

terrain, drainage divides are often undefined or nonexistent and culverts shall be located and designed for the least disruption of the existing flow distribution.

1.1.3. Tailwater

Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full, which affects the capacity of the culvert.

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 the stage in a contributing stream, headwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

1.1.4. Outlet Velocity

The outlet velocity of a culvert is the velocity measured at the downstream end of the culvert. The outlet velocity is usually higher than the maximum natural stream velocity and can cause streambed scour and bank erosion downstream from the culvert outlet. Permissible velocities at the outlet will depend upon streambed characteristics, and the type of energy dissipation (outlet protection) that is provided.

Variations in shape and size of a culvert seldom have a significant effect on the outlet velocity. Slope and roughness of the culvert barrel are the principal factors affecting the outlet velocity.

2. CULVERT HYDRAULICS

This section describes key hydraulic principles that are pertinent to the design of culverts. Application of these principles is presented in [Section 3.0](#) of this chapter.

2.1. Key Hydraulic Principles

For purposes of the following review, it is assumed that the reader has a basic working knowledge of hydraulics and is familiar with the Manning's, continuity and energy equations, which are presented in Chapter 7 – *Open Channel Flow Design*:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \quad \text{(Equation CB-1)}$$

where:

Q = Flow rate or discharge (ft³/sec)

n = Manning's Roughness Coefficient

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A = Flow Area (ft²)

R = Hydraulic Radius (ft)

S = Channel Slope (ft/ft)

$$Q = v_1 A_1 = v_2 A_2$$

(Equation CB-2)

where:

Q = Flow rate or discharge (ft³/sec)

v = Velocity (ft/sec)

A = Flow Area (ft²)

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

(Equation CB-3)

where:

v = Velocity (ft/sec)

g = Gravity (32.2 ft/sec²)

p = Pressure (lb/ft²)

γ = Specific weight of water (62.4 lb/ft³)

z = Height above datum (ft)

2.1.1. Energy and Hydraulic Grade Lines

[Figures CB-1](#) and [CB-2](#) illustrate the energy grade line (EGL) and hydraulic grade line (HGL) and related terms.

Energy Grade Line

The energy grade line, also known as the line of total head, is the sum of velocity head $\frac{v^2}{2g}$, the depth of flow or pressure head $\frac{p}{\gamma}$, and the elevation above an arbitrary datum represented by the distance Z (see

[Figure CB-1](#)). 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 .

Hydraulic Grade Line

The hydraulic grade line is the sum of the elevation Z and the depth of flow or pressure head $\frac{p}{\gamma}$.

For open channel flow, the term $\frac{p}{\gamma}$ is equivalent to the depth of flow and the hydraulic grade line is the same as the water surface (see [Figure CB-1](#)). For pressure flow in closed conduits (e.g., culverts), $\frac{p}{\gamma}$ is the pressure head and the hydraulic grade line falls above the top of the conduit as long as the pressure relative to atmospheric pressure is positive.

Figure CB-1 – Definition of Terms for Closed Conduit Flow

(UDFCD USDCM, 2001)

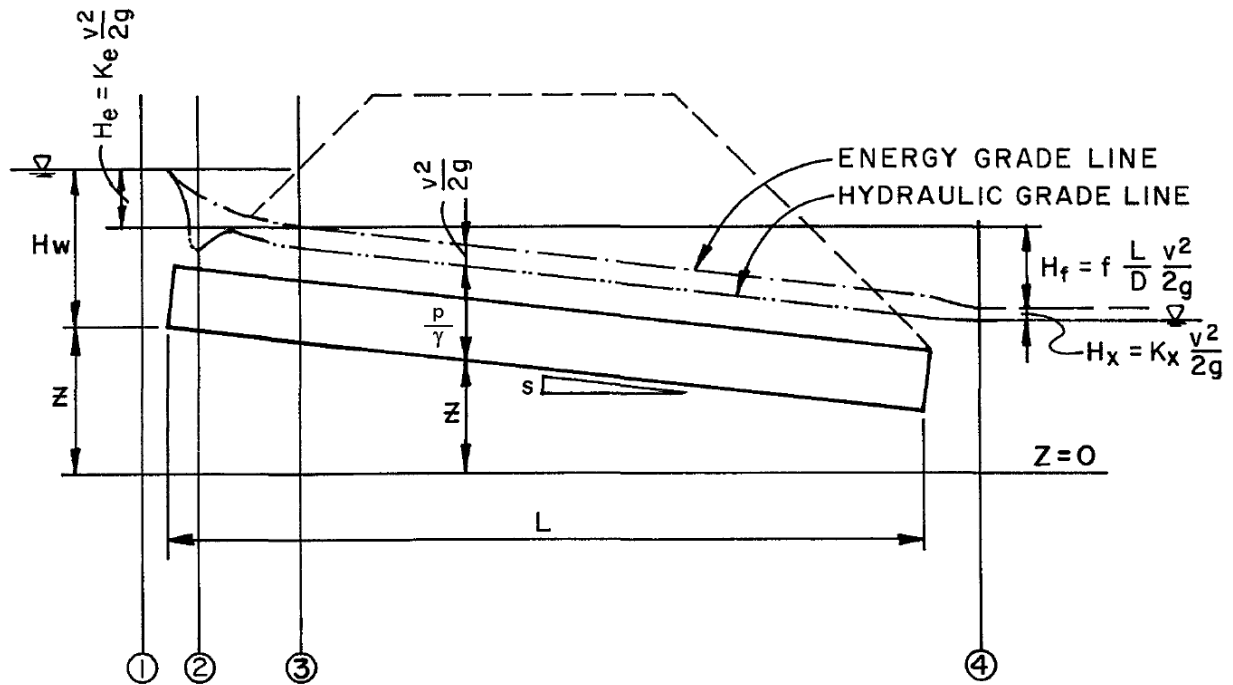
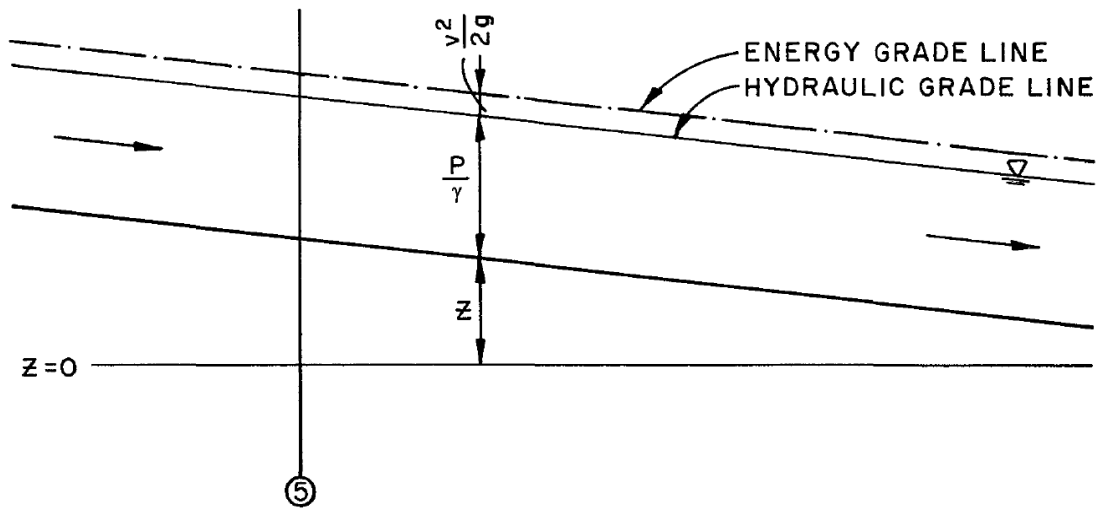


Figure CB-2 – Definition of Terms for Open Channel Flow

(UDFCD USDCM, 2001)



Approaching the entrance to a culvert (refer to Point 1 of [Figure CB-1](#)) the flow is essentially uniform and the hydraulic grade line and energy grade lines are almost the same. As water enters the culvert at the inlet, the flow is first contracted and then expanded by the inlet geometry, which causes a loss of energy at Point 2. As normal turbulent velocity distribution is reestablished downstream of the entrance at Point 3, a loss of energy is incurred through friction or from resistance. In short culverts, the entrance losses are likely to be high relative to the friction loss. At the exit, Point 4, an additional loss is incurred through turbulence as the flow expands and is retarded by the water in the downstream channel. At Point 5 of [Figure CB-2](#) open channel flow is established and the hydraulic grade line is the same as the water surface.

2.1.2. Culvert Flow Conditions

There are two major types of flow conditions in culverts: (1) inlet control and (2) outlet control. For each type of control, a different combination of factors is used to determine the hydraulic capacity of a 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 and outlet control are described in the following sections.

2.1.2.1. Inlet Control

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

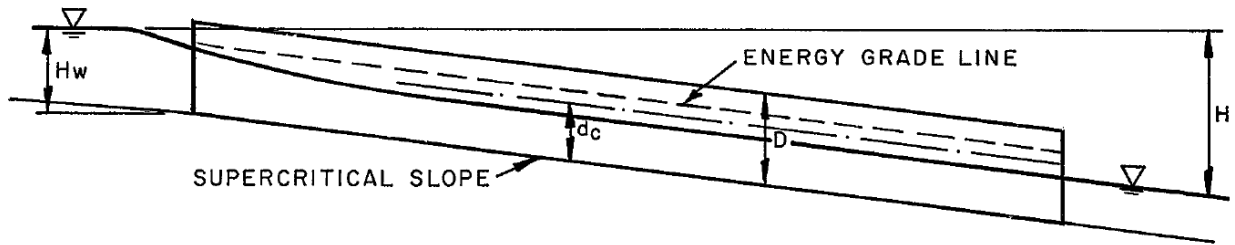
- Depth of headwater
- Culvert cross-sectional area at inlet
- Inlet edge configuration
- Barrel shape

When a culvert operates under inlet control, headwater depth and the inlet edge configuration determine the culvert capacity, with the culvert barrel usually flowing only partially full.

Inlet control for culverts may occur in two ways. The least common occurs when the headwater depth is not sufficient to submerge the top of the culvert and, concurrently, the culvert invert slope is supercritical as shown in [Figure CB-3](#).

Figure CB-3 – Inlet Control—Unsubmerged Inlet

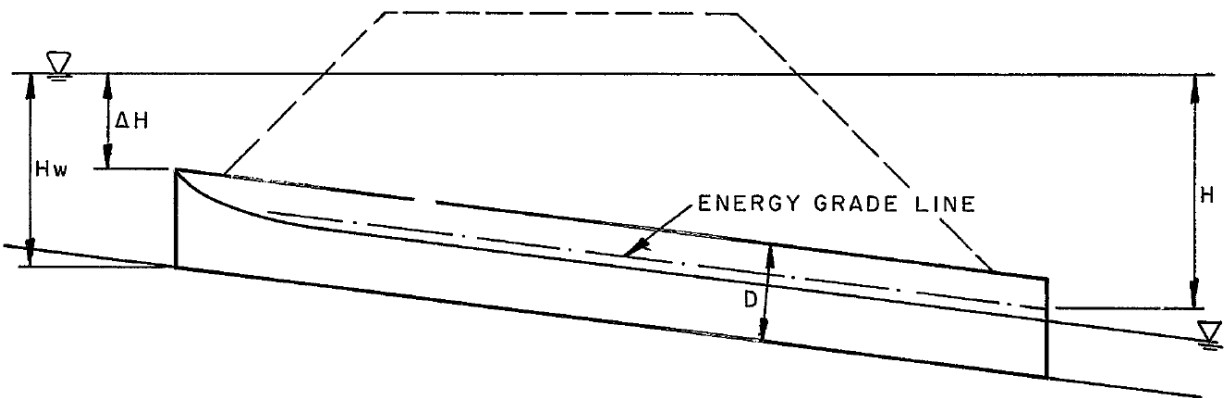
(UDFCD USDCM, 2001)



The most common occurrence of inlet control is when the headwater submerges the top of the culvert (Figure CB-4), and the pipe does not flow full. A culvert flowing under inlet control is defined as a hydraulically short culvert.

Figure CB-4 – Inlet Control—Submerged Inlet

(UDFCD USDCM, 2001)



For a culvert operating with inlet control, the roughness, slope, and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance.

2.1.2.2. Outlet Control

If the headwater is high enough and the culvert is sufficiently long and flat, the control will shift to the outlet. In outlet control, the discharge is a function of the inlet losses, the headwater depth, the culvert roughness, the culvert length, the barrel diameter, the culvert slope, and sometimes the tailwater elevation.

In outlet control, culvert hydraulic performance is determined by these factors:

- Depth of headwater
- Culvert cross-sectional area
- Inlet edge configuration
- Culvert shape
- Barrel slope
- Barrel length
- Barrel roughness
- Depth of tailwater

Outlet control will exist under two conditions: 1) the most common condition occurs when the culvert is flowing full (Figure CB-6), and 2) the least common condition occurs where the headwater is insufficient to submerge the top of the culvert and, concurrently, the culvert slope is subcritical (Figure CB-5). A culvert flowing under outlet control is defined as a hydraulically long culvert.

Figure CB-5 – Outlet Control—Partially Full Conduit
(UDFCD USDCM, 2001)

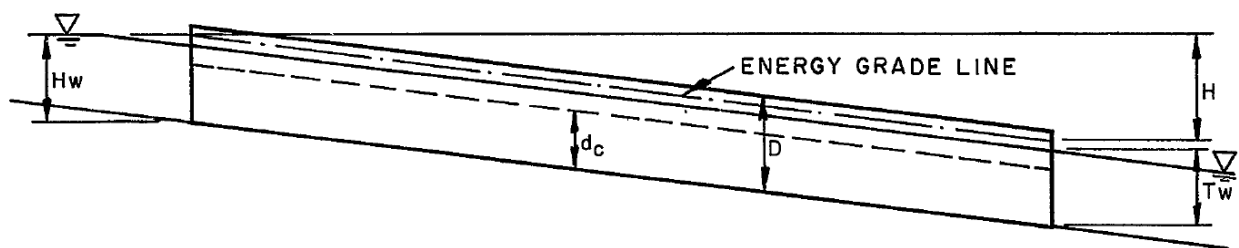
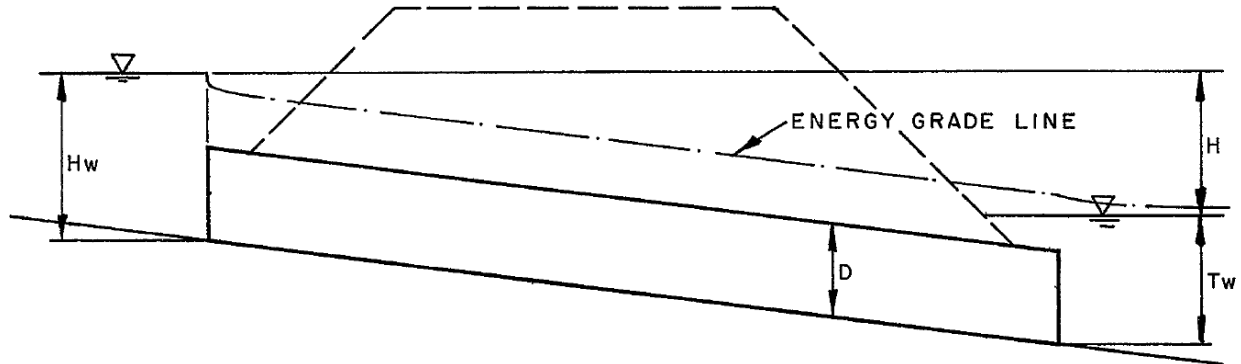


Figure CB-6 – Outlet Control—Full Conduit

(UDFCD USDCM, 2001)



Culverts operating under outlet control may flow full or partly full depending on various combinations of the factors described above. In outlet control, factors that may affect performance appreciably for a given culvert size and headwater are barrel length and roughness, and tailwater depth.

2.2. Energy Losses

In short conduits, such as culverts, the losses caused by the entrance can be as important as the friction losses through the conduit. The losses that must be evaluated to determine the carrying capacity of the culverts consist of inlet (or entrance) losses, friction losses along the length of the culvert and outlet (or exit) losses. These losses are described in [Sections 2.2.1](#) through [2.2.3](#) of this chapter, respectively.

2.2.1. Inlet Losses

For inlet losses, the governing equations are:

$$Q = CA\sqrt{2gH} \quad \text{(Equation CB-4)}$$

$$H_e = K_e \frac{v^2}{2g} \quad \text{(Equation CB-5)}$$

where:

Q = Flow rate or discharge (ft³/sec)

C = Contraction coefficient (dimensionless) (see [Table CB-1](#) below)

A = Cross-sectional area (ft²)

g = Acceleration due to gravity, 32.2 (ft/sec²)

H = Total head (ft)

H_e = Head loss at entrance (ft)

K_e = Entrance loss coefficient (dimensionless)

v = Average velocity (ft/sec)

Table CB-1 – Contraction Coefficient

Transition Description	Contraction Coefficient, C
No transition loss computed	---
Gradual transitions	0.3
Intermediate transitions	0.5
Abrupt transitions	0.8

2.2.2. Outlet Losses

For outlet losses, the governing equations are related to the difference in velocity head between the pipe flow and that in the downstream channel at the end of the pipe.

2.2.3. Friction Losses

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

$$H_f = f \left(\frac{L}{D} \right) \left(\frac{v^2}{2g} \right) \quad \text{(Equation CB-6)}$$

where:

H_f = Frictional head loss (ft)

f = Friction factor (dimensionless)

L = Length of culvert (ft)

D = Hydraulic diameter of culvert (ft) (internal diameter for circular pipe)

v = Average velocity (ft/sec)

g = Acceleration due to gravity, 32.2 (ft/sec²)

The friction factor has been determined empirically and is dependent on relative roughness, velocity, and barrel diameter. Moody diagrams can be used to determine the friction factor. The friction losses for culverts are often expressed in terms of Manning's n (see Table ST-9 in Chapter 5 – *Storm Sewer System Design*), which is independent of the size of pipe and depth of flow. Another common formula for pipe flow is the Hazen-Williams formula. Standard hydraulic texts should be consulted for the limitations of these formulas.

3. CULVERT SIZING AND DESIGN

HDS-5 (FHWA 2005 - <http://isddc.dot.gov/.../FHWA>) provides valuable guidance for the design and selection of drainage culverts. This particular circular explains inlet and outlet control and the procedure for designing culverts. Culvert design is iterative and consists of the following steps:

1. Determine the flow rate of water the culvert must carry.
2. Select a culvert shape, type, and size with a particular inlet end treatment.
3. Determine a headwater depth from the relevant charts for both inlet and outlet control for the design discharge, the grade and length of culvert, and the depth of water at the outlet (tailwater).
4. Compare the largest depth of headwater (as determined from either inlet or outlet control) to the design criteria. If the design criteria are not met, continue trying other culvert configurations until one or more configurations are found to satisfy the design parameters.
5. Estimate the culvert outlet velocity and determine if there is a need for any special features such as energy dissipators or armoring of the downstream channel.

These steps are described in [Sections 3.1](#) through [3.5](#) of this chapter.

3.1. Determination of Design Flow Rate

The first step to consider in the hydraulic design of a culvert is the determination of the flow rate that the culvert must convey. There is no single method for determining peak discharge that is applicable to all watersheds. The method chosen should be a function of drainage area size, availability of data, and the degree of accuracy desired.

The following methods described in Chapter 4 – *Determination of Stormwater Runoff*, shall be used to generate peak discharge:

Rational Method – used for drainage areas less than 30 acres.

Soil Conservation Method – used for drainage areas between 30 and 2000 acres.

3.1.1. Design Frequency and Freeboard Criteria

The storm frequencies and freeboard used as the basis for culvert design are summarized in [Table CB-2](#):

Table CB-2 – Design Storm Frequencies and Minimum Freeboard

Description	Design Storm Frequency	Minimum Freeboard (ft)
Trails	2	1
Local Street	10	1
Collector	25	1
Minor Arterial & Major Arterial	50	1
Bridges (Local & Collector Roadways)	50	1*
Bridges (Arterial & Critical Service Access Roadways/ Drives)	100	1*

* – from “Low Chord” / “Low Steel”

3.2. Computer Applications

Although nomographs can still be used for design, the majority of engineers currently design culverts using computer applications. Among these applications are the FHWA’s HY8 Culvert Analysis (Ginsberg 1987) and numerous proprietary applications such as CulvertMaster. FHWA’s HY8 Culvert Analysis (Version 7.2) is located FHWA’s webpage (<http://www.fhwa.dot.gov>) for download.

In addition, the City of Rogers has developed spreadsheets to aid in the sizing and design of culverts. Use of the RDM-Culvert spreadsheet application is required when sizing and designing culverts in the City of Rogers.

3.3. Design Considerations

The actual design of a culvert installation is more complex than the simple process of sizing culverts because of problems arising from topography and other considerations. Since the problems encountered are too varied and too numerous to be generalized, the information in the design procedure presented below is only a guide to design. Several combinations of entrance types, invert elevations, and pipe

diameters should be evaluated to determine the most economic design that will meet the conditions imposed by topography and engineering. Descriptions of different variables that must be evaluated are presented in [Sections 3.3.1](#) through [3.3.2](#) of this chapter.

3.3.1. Invert Elevations

After determining the allowable headwater elevation, tailwater elevation, and approximate culvert length, the culvert invert elevations must be assumed. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade shall be used as a first trial.

For natural channels, the flow conditions in the channel upstream from the culvert should be investigated to determine if scour will occur. For more information on scour, see [Section 6.1](#) of this chapter.

3.3.2. Culvert Shape, Size and Material

After the invert elevations have been assumed, the shape of the culvert must be selected. The permissible shapes of culverts under all roadways and embankments are box, circular, elliptical and arch.

Next, the diameter of pipe that will meet the headwater requirements should be determined. Because small diameter pipes are often plugged by sediment and debris, the minimum size of pipe for all culverts is 18-inches or the equivalent sized elliptical pipe or arch pipe. The minimum size box culvert shall have a minimum height of 18-inches and a width (“W”) designed to meet the loading (vehicular/overburden) and hydraulic requirements for the desired application.

Reinforced concrete shall be used for all culverts under roadways and running parallel to the roadway in the street right of way and under all traffic and parking areas. Materials other than reinforced concrete must have City approval prior to use.

3.4. Culvert Discharge Velocity

The outlet velocity must be checked to determine if significant scour will occur downstream during the major storm. If scour is indicated (which is normally the case), refer to [Section 6.0](#) of this chapter for guidance on outlet protection. The maximum allowable discharge velocities from culverts for particular downstream conditions are listed in [Table CB-3](#):

Table CB-3 – Maximum Allowable Discharge Velocities

Downstream Condition	Maximum Allowable Discharge Velocity (ft/sec)
Grass	5
Riprap	12
Concrete	18
Turf Reinforcement Mat	Manufacturer’s Specs.

3.5. Minimum Slope

To minimize sediment deposition in the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity of 3-ft/sec flowing full as recommended in FHWA [HEC-22](#). The slope should be checked for each design, and if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these measures implemented.

4.

CULVERT INLETS

The capacity of culverts to convey water is limited by the capacity of the inlet. This is frequently overlooked by designers. Culverts and open channels are often carefully designed with full consideration given to slope, cross section, and hydraulic roughness, but without regard to the inlet limitations. Culvert designs based on uniform flow equations rarely can convey their design capacity due to limitations imposed by the inlet.

The design of a culvert, including the inlet and the outlet, requires a balance between hydraulic efficiency, purpose, and topography at the proposed culvert site. Where there is sufficient allowable headwater depth, the choice of inlets may not be critical, but where headwater depth is limited, erosion is a problem, or sedimentation is likely, a more efficient inlet may be required to obtain the necessary discharge capacity for the culvert.

Although the primary purpose of a culvert is to convey flows, a culvert may also be used to restrict flow, such as in cases where a controlled amount of water is discharged while the area upstream from the culvert is used for detention storage to reduce the peak discharge rate. In this case, an inlet with limited capacity may be the appropriate choice.

The inlet types described in this chapter may be selected to fulfill either of the above requirements depending on the topography or conditions imposed by the designer. The entrance coefficient, K_e , as defined for [Equation CB-5](#), is a measure of the hydraulic efficiency at the inlet, with lower values indicating greater efficiency. Inlet coefficients are given in [Table CB-4](#).

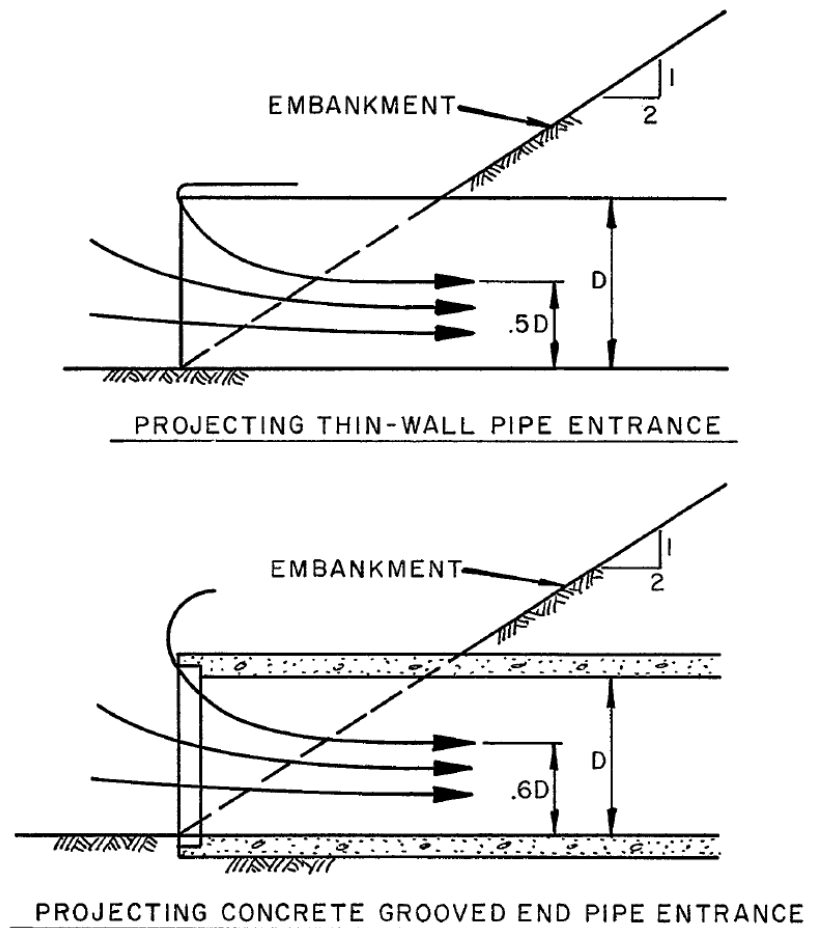
**Table CB-4 – Entrance Loss Coefficients for Outlet Control,
Full or Partly Full Flow
(FHWA HDS-5, 2005)**

Circular Culvert	Coefficient, K_e
Square End Projection	0.2
Square End with Headwall	0.5
Grooved End Projection	0.2
Grooved End with Headwall	0.2
1.1 : 1 Beveled Edge	0.2
1.5 : 1 Beveled Edge	0.2
Box Culvert	Coefficient, K_e
Square Edge w/ 90-15 Degree Headwall	0.5
1.5 : 1 Bevel w/ 90 Degree Headwall	0.2
1 : 1 Bevel w/ Headwall	0.2
Square Edge w/ 30-78 Degree Flared Wingwall	0.4
Square Edge w/ 90-15 Degree Flared Wingwall	0.5
Square Edge w/ 0 Degree Flared Wingwall	0.7
1.5 : 1 Bevel w/ 18-34 Degree Flared Wingwall	0.2
1.5 : 1 Bevel w/ 45 Degree Flared Wingwall	0.2

4.1. Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. [Figure CB-7](#) illustrates this type of inlet.

**Figure CB-7 – Common Projecting Culvert Inlets
(USFCD USDCM, 2001)**



Corrugated metal pipe projecting inlets have limitations which include low hydraulic efficiency, damage resulting from maintenance of the channel and the area adjacent to the inlet, and restrictions imposed on maintenance crews to work around the inlet. In contrast, concrete grooved or bell-end pipe has hydraulic efficiency that is superior to corrugated metal pipe and, therefore, the primary restriction placed on the use of concrete pipe for projecting inlets is the requirement for maintenance of the channel and the embankment surrounding the inlet. Where equipment will be used to maintain the embankment around the inlet, it is not recommended that a projecting inlet of any type be used.

4.1.1. Corrugated Metal Pipe

A projecting entrance of corrugated metal pipe is equivalent to a sharp-edged entrance with a thin wall and has an entrance coefficient of 0.9 (see [Table CB-4](#)).

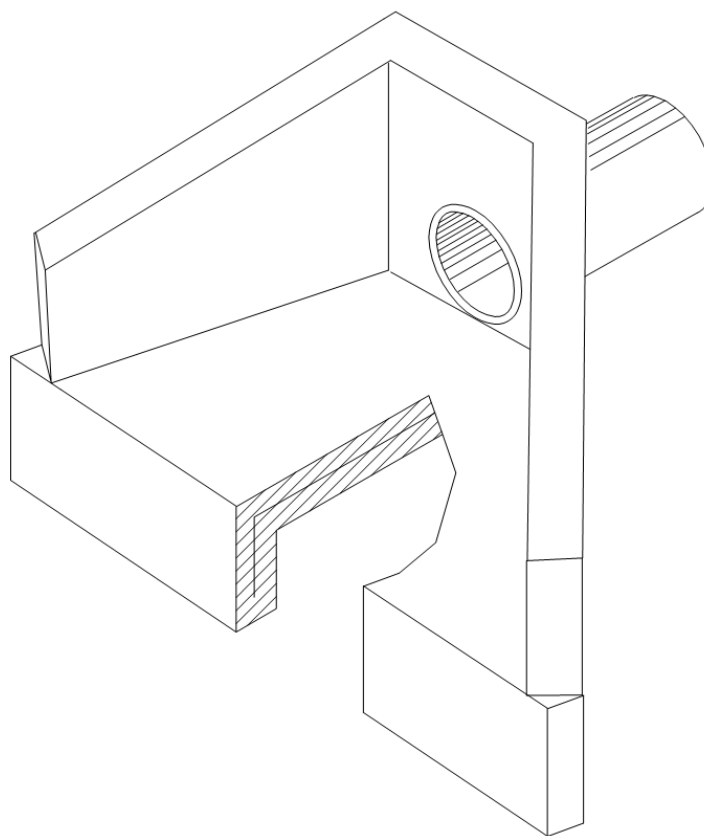
4.1.2. Concrete Pipe

Bell-and-spigot concrete pipe or tongue-and-groove concrete pipe with the bell-end or grooved-end used as the inlet section, are relatively efficient hydraulically with an entrance coefficient of 0.2. For concrete pipe that has been cut, the entrance is square edged, and the entrance coefficient is 0.5 (see [Table CB-4](#)).

4.2. 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. The range of inlet coefficients for different headwall configurations is summarized in [Table CB-4](#). Different configurations of pipe with headwalls are described in [Sections 4.2.1](#) through [4.2.4](#) of this chapter. [Figure CB-8](#) illustrates a headwall with wingwalls.

Figure CB-8 – Inlet with Headwall and Wingwalls



4.2.1. Corrugated Metal Pipe

Corrugated metal pipe in a headwall is characterized as a square-edged entrance with an entrance coefficient of 0.5. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced to 0.15 for a rounded edge with a radius equal to 0.15 times the culvert diameter, and to 0.10 for rounded edge with a radius equal to 0.25 times the diameter of the culvert.

4.2.2. 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 coefficient is equal to about 0.2 for a tongue-and-grooved and bell-end pipe, and equal to 0.5 for cut concrete pipe.

4.2.3. Wingwalls

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and where the culvert is skewed to the normal channel flow. 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 CB-9](#) illustrates several cases where wingwalls are used. For parallel wingwalls, the minimum distance between wingwalls shall be at least 1.25 times the diameter of the culvert pipe.

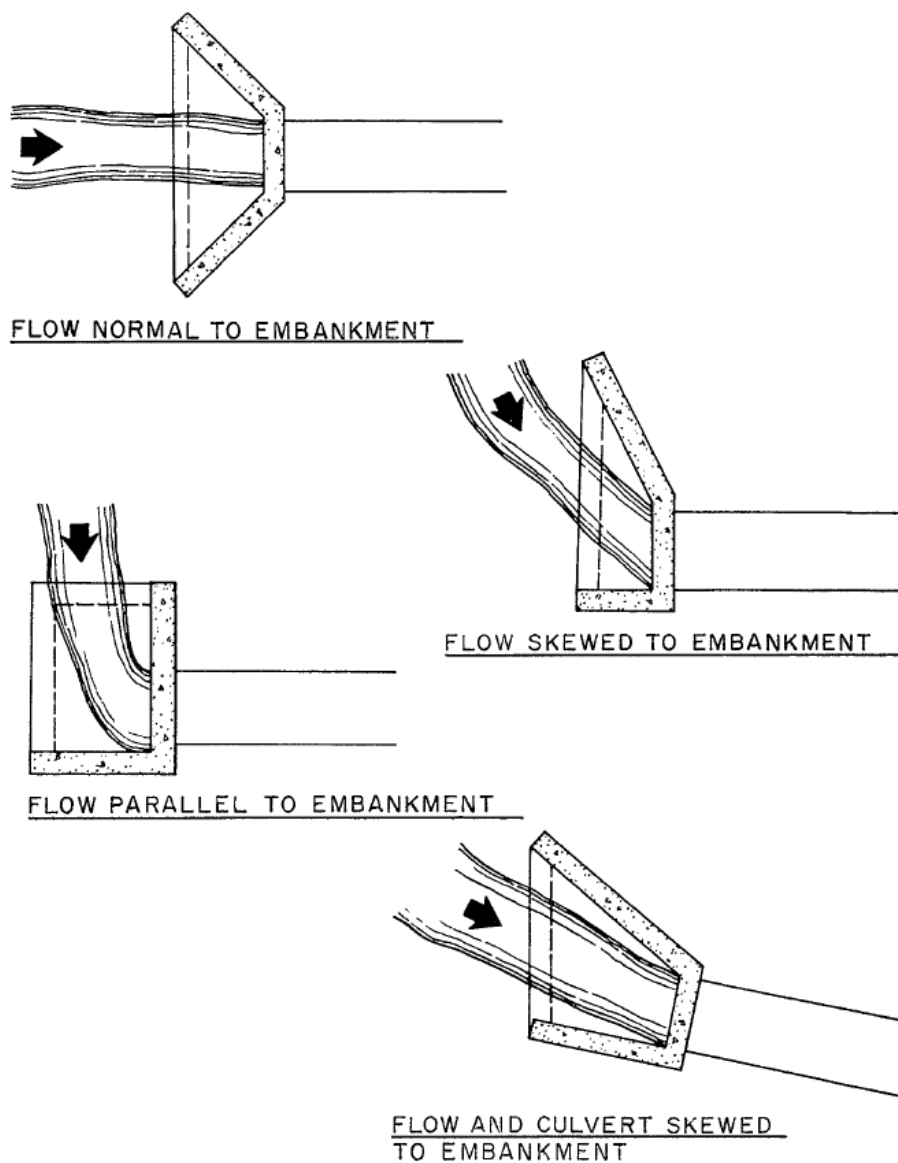
4.2.4. Aprons

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

Culverts with wingwalls shall be designed with a concrete apron extending between the walls. Aprons must be reinforced to control cracking. As illustrated in [Figure CB-9](#), the actual configuration of the wingwalls varies according to the direction of flow and will also vary according to the topographical constraints of the site.

For conditions where scour may be a problem because of high approach velocities and/or the soil conditions, a toe wall is required for apron construction.

Figure CB-9 – Typical Headwall-Wingwall Configurations
 (UDFCD USDCM, 2001)



4.3. Special Inlets

In addition to the common inlets described above, a large variety of other special inlet types exist. Among these are special end-sections, which serve as both outlets and inlets and are available for both corrugated metal pipe and concrete pipe. Because of the difference in requirements due to pipe materials, the special end-sections are addressed according to pipe material. Mitered inlets are discussed in [Section 4.3.3](#) of this chapter.

4.3.1. Corrugated Metal Pipe

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

1. Require less maintenance around the inlet.
2. Sustain less damage from maintenance work and from accidents compared to a projecting entrance.
3. Provide increased hydraulic efficiency.

4.3.2. Concrete Pipe

As in the case of corrugated metal pipe, special concrete flared end-sections, similar to flared end-sections for corrugated metal pipe, may increase the embankment stability and retard erosion at the inlet. They should be used where maintenance equipment must be used near the inlet or where, for aesthetic reasons, a projecting entrance is considered too unsightly.

The hydraulic efficiency of a concrete flared end section 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 0.2.

4.3.3. Mitered Inlets

Mitered inlets are predominantly used with corrugated metal pipe and their hydraulic efficiency 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, resulting in a projecting entrance with $K_e = 0.9$. If the embankment is paved, a sloping headwall is obtained with $K_e = 0.7$ and, by beveling the edges, $K_e = 0.2$.

Uplift is an important factor for mitered inlets. It is not good practice to use unpaved embankment slopes where a mitered entrance may be submerged above the top of the pipe to an elevation one-half the diameter of the culvert.

4.3.4. Long Conduit Inlets

Inlets are important in the design of culverts for road crossings and other short sections of conduit; however, inlets are even more significant in the economical design of long culverts and pipes. Unused capacity in a long conduit will result in 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.

4.3.5. 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 approximately one-half of the actual available barrel 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 (or culvert throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because a simple beveled edge inlet is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet. HDS-5 (FHWA 2005 - <http://isddc.dot.gov/.../FHWA>) provides guidance on the design of improved inlets.

5. INLET PROTECTION

Inlets on culverts, especially on culverts to be installed in live streams, should be evaluated relative to debris control and buoyancy. The following section discusses this further.

5.1. Debris Control

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. This can result in damage caused by overtopping of the roadway and/or inundation of the upstream property. Three main options exist to address the debris problem:

1. Retain the debris upstream of the culvert.
2. Attempt to pass the debris through the culvert.
3. Install a bridge.

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 shall include a thorough study of the debris problem. Factors to be considered in a debris study include the following:

- Type of debris
- Quantity of debris
- 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

FHWA's Hydraulic Engineering Circular, No. 9 (HEC-9 2005 - <http://www.fhwa.dot.gov/engineering/>), Debris Control Structures, shall be referenced when designing debris control structures.

5.2. Buoyancy

When a culvert is functioning with inlet control, an air pocket forms, just inside the inlet, that creates 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 vortices and eddy currents, can cause scour, undermine culvert inlets, and erode embankment slopes, thereby making the inlet vulnerable to failure, especially with deep headwater.

The large unequal pressures resulting from inlet constriction, which are accentuated when the capacity of the culvert is impaired by debris or damage, 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 dead load on the end of the culvert, protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. When inlet control conditions are present a standard concrete headwall or endwall will be provided unless otherwise approved by the City 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 due to separation, 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 requires headwater depth

in excess of 20 feet, reducing the culvert size is required to limit the discharge rate rather than using an inefficient projecting inlet.

6. OUTLET PROTECTION

Scour at culvert outlets is a common occurrence and must be accounted for. The natural channel flow is usually confined to a lesser width and greater depth as it passes through a culvert barrel. Increased flow velocity typically results with potentially erosive capabilities as it 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 natural channel are not the only factors that need consideration. Other factors to consider with respect to scour potential include the characteristics of the channel bed and bank material, velocity, and depth of flow in the channel at the culvert outlet, and the amount of sediment and other debris conveyed in the flow. Due to the variation in expected flows and the difficulty in evaluating the variables described above, scour prediction is an inexact science.

6.1. Scour

Protection against scour at culvert outlets varies from limited riprap placement to complex and expensive energy dissipation devices. At some locations, use of a rougher culvert material may alleviate the need for a special outlet protection device. Pre-formed scour holes (approximating the configuration of naturally formed holes) dissipate energy while providing a protective lining to the streambed. Methods for predicting scour hole dimensions are provided in FHWA's Hydraulic Engineering Circular, No. 14 ([HEC-14](#), 2006), Hydraulic Design of Energy Dissipators for Culverts and Channels.

6.2. Energy Dissipation/Erosion Control

Riprap-armored channel expansions and concrete aprons protect the channel and redistribute or spread the flow. Barrel outlet expansions operate in a similar manner. Headwalls and cutoff walls protect the integrity of the fill. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices. Design information for the general types of energy dissipators can be found in [HEC-14](#) (FHWA 2006).

Four examples of energy dissipators and erosion control are given below: Drop Structures, Turf Reinforcement Mats, Hydraulic Jump Energy Dissipators and Riprap (requires City approval).

6.2.1. Drop Structures

Drop structures are commonly used for flow control and energy dissipation. Changing the channel slope from steep to mild, by placing drop structures at intervals along the channel reach, changes a continuous

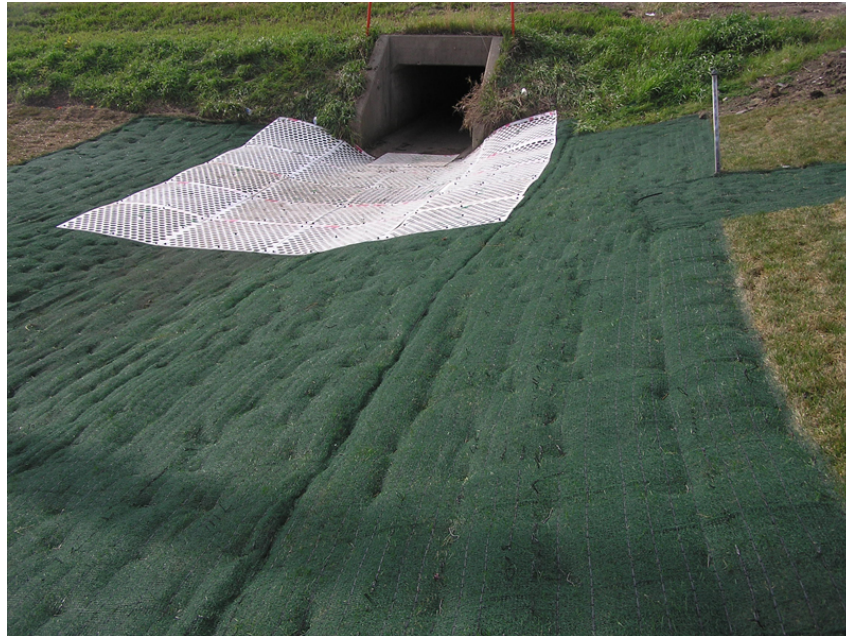
steep slope into a series of gentle slopes and vertical drops. Instead of slowing down and transferring high velocities that produce erosion into low non-erosive velocities, drop structures control the slope of the channel in such a way to prevent high, erosive velocities from developing. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by a specially designed apron or stilling basin. [HEC-14](#) (FHWA 2006) provides guidance for the design and selection of drop structures.

6.2.2. Turf Reinforcement Mat

Turf reinforcement mat (TRM) is a long term non-biodegradable biotechnical alternative for hard armor such as riprap. It is mechanically-anchored polymer matting designed with voids throughout the structure which enables vegetative growth to cover the material while still providing mechanical protection in areas where design discharges exert velocities and shear stresses that exceed the limits of natural vegetation. TRMs are used to extend the performance limits of natural vegetation by, retaining soil particles and vegetative seeds, promoting conditions for accelerated vegetative growth, and reinforcement of the vegetative cover. The EPA has documented TRMs as useful BMPs for stormwater runoff. The EPA's Storm Water Technology Fact Sheet for TRMs can be found at <http://www.epa.gov/owm/mtb/turfrein.pdf> and provides a useful general discussion on the benefits, specific locations for use, and other general information for TRMs.

Many different manufacturers produce TRMs, each with its own patented methods and material combinations. Some of the manufacturers/distributors of TRMs include *ScourStop*, *North American Green*, *Propex*, *ShoreMax*, among others. TRMs shall be selected, designed, and installed according to the manufacturer's recommendation. When attempting to implement such reinforcement into the design of energy dissipation/erosion control, it will be the responsibility of the design engineer to provide the City with appropriate material specifications and design information. Enough information needs to be provided to ensure the product selected and specified in a design is adequately suited for the situation in the field.

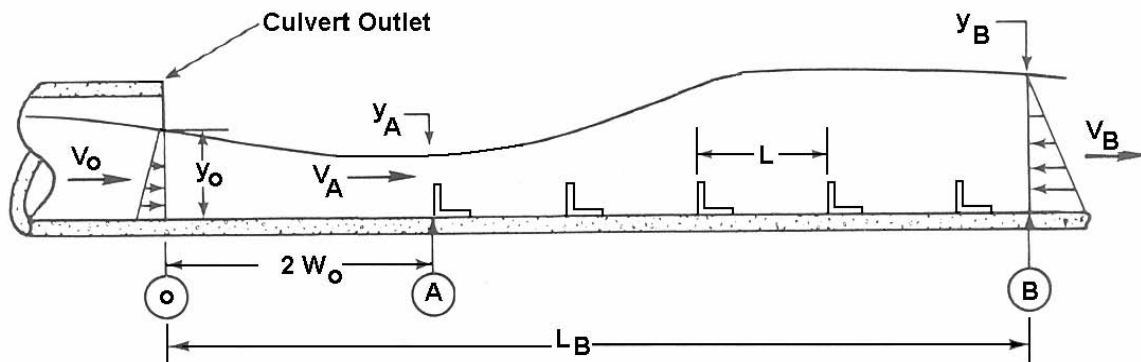
Figure CB-10 – Typical Turf Reinforcement Mat Application
(Scourstop.com)



6.2.3. Hydraulic Jump Energy Dissipators

Hydraulic Jump Energy Dissipators create a hydraulic jump by placing staggered rows of blocks at the culvert outlet. The block height (h) shall be 0.31 to 0.91 of the approach flow’s average depth y_A and the ratio of L/h shall be equal to 6 or 12. The design of these dissipators is based on the momentum equation as shown in [Equation CB-7](#). This equation is applicable for slopes up to 10%; see [HEC-14](#) (FHWA 2006) for design methods on slopes greater than 10%.

**Figure CB-11 – Hydraulic Jump Energy Dissipators
(FHWA – HEC 14, 2006)**



$$\rho V_o Q + C_p \gamma (y_o^2 / 2) W_o = C_B A_F N \rho V_A^2 / 2 + \rho V_B Q + \gamma Q^2 / (2 V_B^2 W_B) \tag{Equation CB-7}$$

where:

- y_0 = depth at the culvert outlet (ft)
- V_0 = velocity at the culvert outlet (ft/s)
- W_0 = culvert width at the culvert outlet (ft)
- V_A = approach velocity at two culvert widths downstream of the culvert outlet (ft/s)
- V_B = exit velocity, just downstream of the last row of roughness elements (ft/s)
- W_B = basin width, just downstream of the last row of roughness elements (ft/s)
- N = total number of roughness elements in the basin
- A_F = frontal area of one full roughness element (ft²)
- C_B = basin drag coefficient
- C_P = momentum correction coefficient for the pressure at the culvert outlet
- γ = unit weight of water (62.4 lbs/ft³)
- ρ = density of water (1.94 slugs/ft³)

Table CB-5 shows empirical drag coefficients C_B for the basin configurations shown in Figure CB-12.

Figure CB-12 – Basin Configurations
(FHWA HEC 14, 2006)

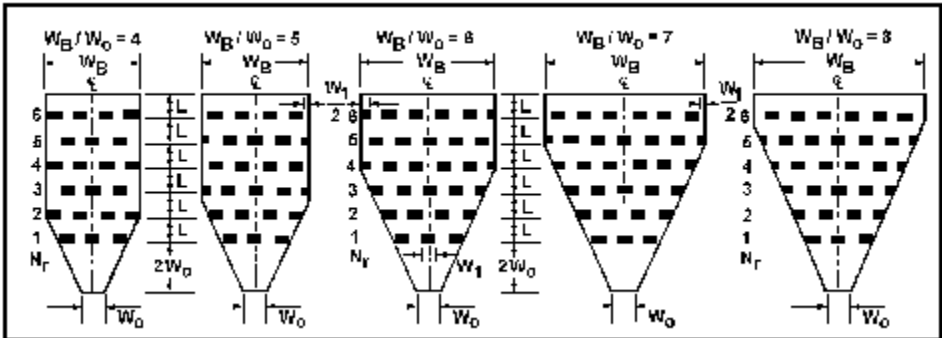


Table CB-5 – Design Values for Roughness Elements

(FHWA HEC-14, 2006)

W_B/W_o		2 to 4			5			6			7		8	
W_1/W_o		0.57			0.63			0.6			0.58		0.62	
Rows (N_r)		4	5	6	4	5	6	4	5	6	5	6	6	
Elements (N)		14	17	21	15	19	23	17	22	27	24	30	30	
RECTANGULAR	h/y_A	L/h	Basin Drag Coefficient, C_B											
	0.91	6	0.32	0.28	0.24	0.32	0.28	0.24	0.31	0.27	0.23	0.26	0.22	0.22
	0.71	6	0.44	0.40	0.37	0.42	0.38	0.35	0.40	0.36	0.33	0.34	0.31	0.29
	0.48	12	0.60	0.55	0.51	0.56	0.51	0.47	0.53	0.48	0.43	0.46	0.39	0.35
	0.37	12	0.68	0.66	0.65	0.65	0.62	0.60	0.62	0.58	0.55	0.54	0.50	0.45
CIRCULAR	0.91	6	0.21	0.20	0.48	0.21	0.19	0.17	0.21	0.19	0.17	0.18	0.16	
	0.71	6	0.29	0.27	0.40	0.27	0.25	0.23	0.25	0.23	0.22	0.22	0.20	
	0.31	6	0.38	0.36	0.34	0.36	0.34	0.32	0.34	0.32	0.30	0.30	0.28	
	0.48	12	0.45	0.42	0.25	0.40	0.38	0.36	0.36	0.34	0.32	0.30	0.28	
	0.37	12	0.52	0.50	0.18	0.48	0.46	0.44	0.44	0.42	0.40	0.38	0.36	

6.2.4. Riprap as Outlet Protection

Riprap can be an effective measure for erosion and scour protection, but is a nuisance to maintain and an eyesore to the public. Information regarding the sizing of riprap is provided in Chapter 7 – *Open Channel Flow Design*. Riprap can only be used with City approval and riprap must also be grouted in place. Riprap shall not be the first choice for energy dissipation/erosion control and City approval will be dependent upon the design engineer showing the ineffectiveness of other types of energy dissipation devices for the specific situation under consideration. The City requires the use of the **RDM-Culvert** spreadsheet for calculating the size and extents of riprap as outlet protection for circular and equivalent diameter noncircular pipe. The *Riprap Outlet Protection* tab in the **RDM-Culvert** spreadsheet calculates the minimum riprap size (d_{50}) and minimum apron length required for riprap outlet protection for circular and equivalent diameter noncircular pipe. However, should riprap as outlet protection need to be designed for an outlet and/or culvert scenario not applicable to the conditions set forth in the *Riprap Outlet Protection* tab in the **RDM-Culvert** spreadsheet, the following sections shall be used in the design of riprap as outlet protection. In all cases the thickness/structural layer of riprap as outlet protection shall be constructed as shown in [Figure CB-14](#).

6.2.4.1. Length of Protection

Riprap, when used as an outlet velocity control measure, shall be applied to the channel area immediately downstream of the culvert outlet for a length, L_p , determined using one of the following formula(s):

$$L_p = \left(\frac{1.7 * Q}{D_o^{3/2}} \right) + 8 * D_o \quad \text{if culvert is flowing } < \text{ half full} \quad \text{(Equation CB-8)}$$

$$L_p = \left(\frac{3.0 * Q}{D_o^{3/2}} \right) \quad \text{if culvert is flowing } \geq \text{ half full} \quad \text{(Equation CB-9)}$$

where:

L_p = Length of protection (length of riprap apron) (ft)

Q = Design discharge (ft³/sec)

D_o = Maximum inside culvert width (ft) (use diameter for circular culverts)

In no instance shall L_p be less than $3 * D_o$ nor does L_p need to be greater than $20 * D_o$.

6.2.4.2. Width of Protection

Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel. Where no well-defined channel exists immediately downstream of the outlet area, the outlet protection width, W , shall be determined using the following formula(s):

$$W = 3 * D_o + 0.4 * L_p \quad \text{if } TW \geq \text{elevation of center of culvert} \quad \text{(Equation CB-10)}$$

$$W = 3 * D_o + L_p \quad \text{if } TW < \text{elevation of center of culvert} \quad \text{(Equation CB-11)}$$

where:

W = Width of outlet protection (width of riprap apron) (ft)

L_p = Length of protection (length of riprap apron) (ft)

D_o = Maximum inside culvert width (ft) (use diameter for circular culverts)

In no instance shall W be less than $3 * D_o$. See [Figure CB-13](#) for additional details on outlets that don't have a well-defined channel downstream.

6.2.4.3. Thickness and Stone Size/Gradation

The riprap blanket thickness shall be at a minimum two-times (2x) d_{50} for the initial half of L_p immediately after the culvert discharge and at least one-and-a-half-times (1.5x) d_{50} for the final half of L_p . Furthermore, the riprap blanket shall extend up the side slopes at least 1-foot above the design tailwater elevation, but no lower than two-thirds of the vertical culvert dimension above the culvert invert. The riprap thickness on

the side slopes shall be at least one-and-a-half-times (1.5x) d_{50} . A geotextile fabric or stone filter (as outlined in *Section 3.4 – Riprap-Lined Channels* in Chapter 7 – *Open Channel Flow Design*) must be placed under the riprap to prevent undermining of the soil beneath the riprap layer. See [Figure CB-14](#) for additional details on riprap extents.

The median stone diameter, d_{50} , shall be based on the following equation:

$$d_{50} = \left(\frac{0.02}{TW} \right) * \left(\frac{Q}{D_o} \right)^{4/3} \tag{Equation CB-12}$$

where:

d_{50} = Median stone size (ft)

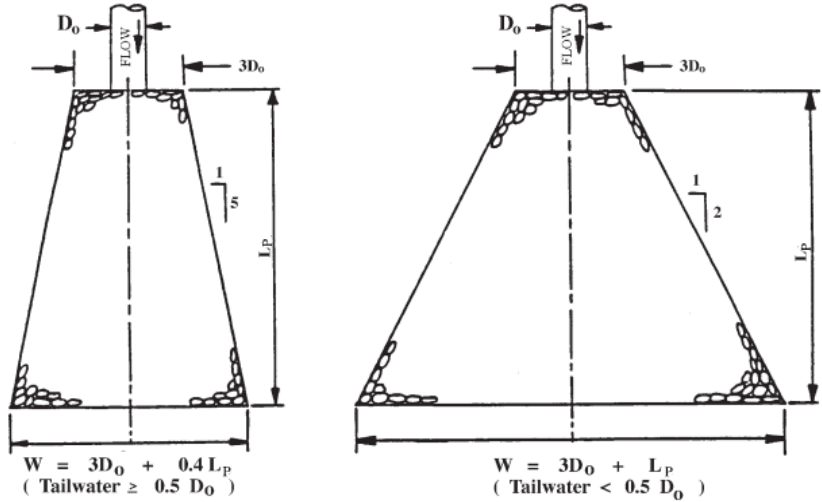
TW = Tailwater depth above culvert invert (ft), in areas where TW cannot be computed, use $TW=0.20*D_o$

Q = Design discharge (ft³/sec)

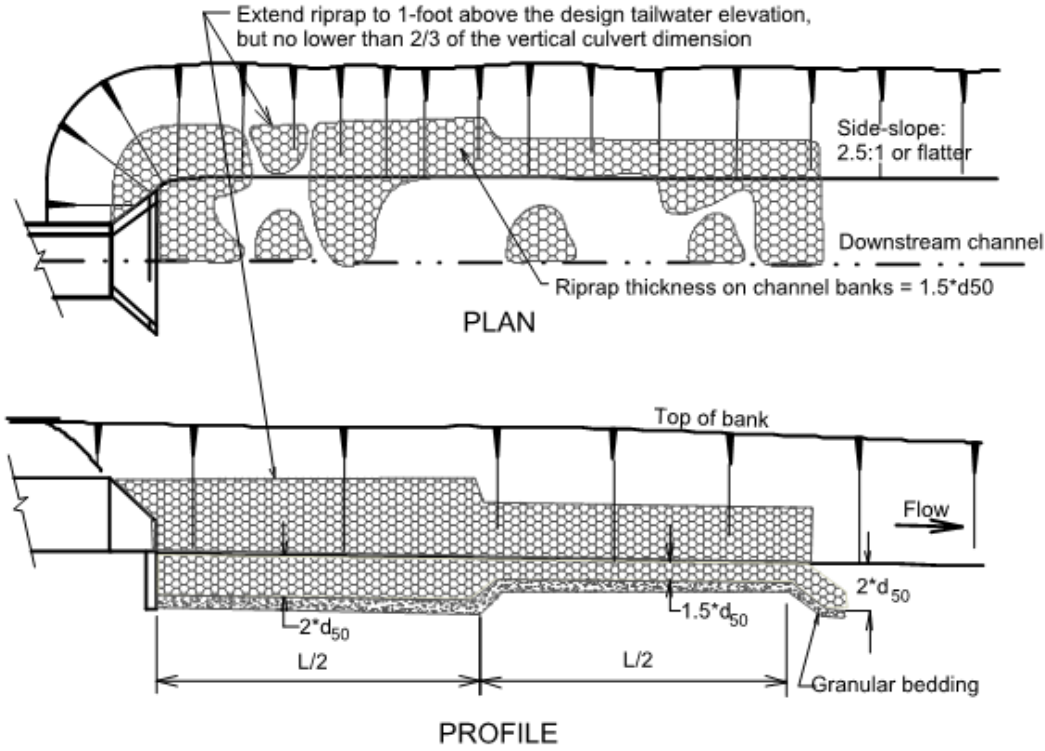
D_o = Maximum inside culvert width (ft) (use diameter for circular culverts)

Where required riprap size calculated from [Equation CB-12](#) exceeds those as defined in Table OC-10 of Chapter 7 of this *Manual*, alternate energy dissipation / erosion control devices shall be used such as stilling basins, baffle chutes, streambed level dissipators, drop structures, etc. (see [HEC-14](#)).

Figure CB-13 – Configuration of Conduit Outlet Protection for un-Defined Channel Downstream (NJDOT SESCS, 2008)



**Figure CB-14 – Culvert and Pipe Outlet Erosion Protection
(UDFCD USDCM 2002)**



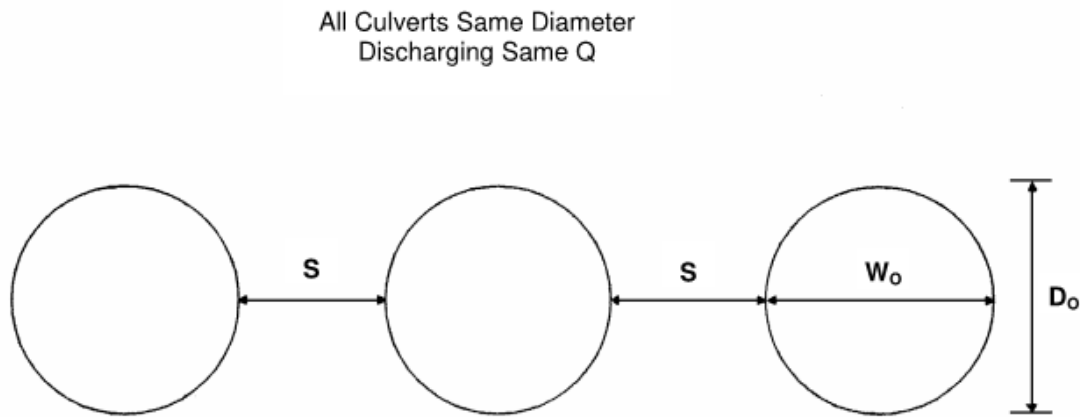
- NOTES:
1. Headwall with wingwalls or flared end section required at all culvert outlets.
 2. Cutoff wall required at end of wingwall aprons and end section.
Minimum depth of cutoff wall = $2 \cdot d_{50}$ or 3-feet, whichever is deeper.
 3. Provide joint fasteners for flared end sections.

6.2.4.4. Multiple Culverts Outlets

When more than one culvert outlet exists at the same location, use the guidelines below to size the riprap protection apron. See [Figure CB-15](#) for additional information.

- When the spacing between the culverts is less than of the width of one culvert, the riprap size and apron dimensions for one culvert shall accommodate all culverts.
- When the spacing between the culverts is greater than the width of one culvert, the riprap size and apron dimensions shall be 25% larger than the dimensions for one culvert.

Figure CB-15 – Guidance for Outlet Protection for Multiple Culverts (NJDOT, SESCS 2008)



- | | |
|------------------------------|---|
| For $S < \frac{1}{4} W_o$ | Size riprap & length for 1 pipe.
Width shall accommodate all culverts. |
| For $S \geq \frac{1}{4} W_o$ | Size riprap & length for 1 pipe and
increase values by 25%. |

7. GENERAL CONSIDERATIONS

7.1. Culvert Location

Culvert location is an integral part of the total design. The main purpose of a culvert is to convey storm water drainage across the roadway section expeditiously and effectively. 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. Note that environmental permitting constraints will often apply for new culverts or retrofits, such as a Section 404 permit that regulates construction activities in jurisdictional wetlands and “Waters of the United States.”

Culverts shall be located on existing stream alignments and aligned to give the stream a direct entrance and a direct exit. Abrupt changes in direction at either end may retard the flow and make a larger structure necessary. If necessary, a direct inlet and outlet may be obtained by means of a channel change, skewing the culvert, or a combination of these. The choice of alignment should be based on environmental concerns, hydraulic performance, and/or maintenance considerations.

If possible, a culvert shall have the same alignment as its channel. Often this is not practical and where the water must be turned into the culvert, headwalls, wingwalls, and aprons with configurations similar to those in [Figure CB-9](#) shall be used as protection against scour and to provide an efficient inlet.

7.2. Sedimentation

Deposits usually occur within the culvert barrels at flow rates smaller than the design flow. The deposits may be removed during larger floods depending upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert, and other factors.

Culvert location in both plan and profile is of particular importance to the maintenance of sediment-free culvert barrels. Deposits occur in culverts because the sediment transport capacity of flow within the culvert is often less than in the stream.

Deposits in culverts may also occur because of the following conditions:

- At moderate flow rates the culvert cross section is larger than that of the stream, so the flow depth and sediment transport capacity is reduced within the culvert compared to the stream.
- 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.
- Abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce sedimentation. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

7.3. Open Channel Inlets

Entrances to open channels often require the same careful planning and design as is needed for culverts and long conduits if the necessary hydraulic balance is to be achieved. The energy grade line shall be analyzed by the designer to provide proper balanced energy conversion, velocity control, energy loss, and other factors that control the downstream flow. Channel confluences, in particular, require careful hydraulic design to eliminate scour, reduce oscillating waves, and minimize upstream backwater effects.

7.4. Transitions

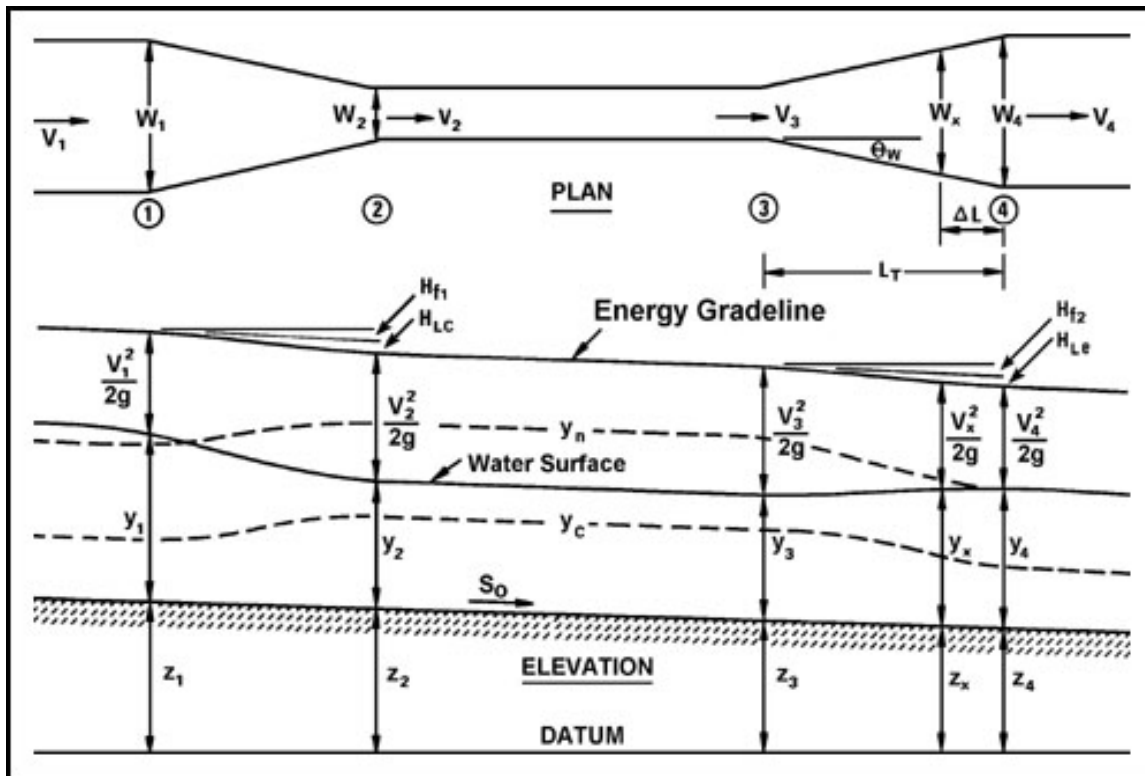
Transitions from pipe flow to open channels, between different rigid channels, and from slow flow to supercritical flow must be designed using the concepts of conservation of energy and open channel hydraulics. Primarily, a transition is necessary to change the shape or cross section of flowing water.

Normally, the designer will have as an objective the avoidance of excessive energy losses, cross waves, and turbulence. It is also necessary to provide against scour and overtopping.

Supercritical flow transitions must receive more attention than is generally provided to subcritical flow transitions. Care must be taken to prevent unwanted hydraulic jumps or velocities that cause critical depth. Froude numbers between 0.8 and 1.2 must be avoided.

In general, the rate at which the flow prism may be changed shall not exceed perhaps 5 to 12½ degrees, depending upon velocity. Sharp angles shall be avoided. The water surface hydraulic grade line shall normally be smooth. More information on transitions is available in [HEC-14](#) (FHWA 2006).

Figure CB-16 – Subcritical Flow Transition
(HEC-14, 2006)



7.5. Culvert Replacements

When installing or replacing a culvert, careful consideration should be taken to ensure that upstream and downstream property owners are not adversely affected by the new hydraulic conditions. The potential upstream flooding impacts associated with the backwater from the calculated headwater depth must be considered and the determination of the available headwater should take into account the area inundated at the projected water surface elevation. If a culvert is replaced by one with more capacity, the downstream effects of the additional flow must be factored into the analysis. Assuring consistency with existing major drainageway master plans and/or outfall studies is important.

7.6. Fencing for Public Safety

Culverts are frequently located at the base of steep slopes. Large box culverts, in particular, can create conditions where there is a significant drop, which poses risk to the public. In such cases, handrail or fencing (or a guardrail configuration) is required for public safety. A handrail or fence shall be placed to provide a barrier between adjacent pedestrian areas and culvert openings when the culvert height/drop is ≥ 30 -inches and ≤ 10 -feet from the edge of the closest travel way.

Typical culvert inlets consist of concrete headwalls and wingwalls for larger structures and beveled-end sections for smaller pipes that may represent an obstacle to motorists who run off the road. This type of design may result in either a fixed object protruding above an otherwise traversable embankment or an opening into which a vehicle can drop causing an abrupt stop. The options available to a design engineer to minimize these obstacles are: use a traversable design, extend the structures so it is less likely to be hit, shield the structures (guardrail, concrete barrier wall, etc.), or delineate the structure if the other alternatives are not appropriate. Guidance for when to use which option is located in Section 3.4.2 Cross-Drainage Structures of the AASHTO Roadside Design Guide (2002).

7.7. Cover, Fill Heights and Bedding for Culverts

Refer to Chapter 5 for pipe and

The minimum cover for reinforced concrete pipe shall be one-foot and shall meet minimum ASTM Class III specifications. The minimum cover for metal pipe is two-feet. Minimum cover less than these values shall be fully justified in writing and approved by the City Engineer prior to proceeding with final plans. Maximum fill heights and bedding descriptions for pipes are shown on [Tables CB-6](#) and [CB-7](#).

Box culverts shall be structurally designed to accommodate earth and live load to be imposed upon the culvert. Refer to the Arkansas State Highway and Transportation Department’s Reinforced Concrete Box Culvert Standard Drawings. When installed within public right of way, all culverts shall be capable of withstanding minimum HL-93 loading.

When culverts under railroad facilities are necessary, the designer shall obtain approval from the affected railroad.

**Table CB-6 – Maximum Heights of Fill Over RCP Culverts
(ARDOT, Standard Drawing PCC-1)**

Installation Type	Class of Pipe		
	Class III	Class IV	Class V
	Feet		
Type 1	21	32	50
Type 2	17	27	41
Type 3	13	20	32

Note: If fill height exceeds 50 feet, a special design concrete pipe will be required using Type 1 Installation.

**Table CB-7 – Pipe Bedding Installation Types
(ARDOT, Standard Drawing PCC-1)**

Installation Type	Material Requirements for Haunch and Structural Bedding
Type 1	Aggregate Base Course (Class 5 or Class 7)
Type 2	Selected Materials (Class SM-1, SM-2 or SM-3) or Type 1 Installation material
Type 3	AASHTO Classification A-1 thru A-6 Soil or Type 1 or 2 Installation material

Note: Material listed in this table corresponds to the ARDOT Standard Specifications for Highway Construction, Latest Edition. Materials shall not include organic materials or stones larger than 3-inches.

8. BRIDGES INTRODUCTION AND OVERVIEW

Bridges are important roadway hydraulic structures that are vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of stream crossings must be recognized and considered in all phases of roadway development, construction and maintenance.

There are extensive manuals on bridges that are available and should be used in bridge hydraulic studies and river stability analysis. Some of the best include:

1. *Hydraulics of Bridge Waterways* Hydraulic Design Series No. 1 (FHWA 1978). This is a good basic reference.
2. *Highway in the River Environment* (Richardson 1988 draft with appendices and 1974). This is particularly good for hydraulics, geomorphology, scour, and degradation.
3. *Hydraulic Analysis Location and Design of Bridges* Volume 7 (AASHTO 1987). This is a good overview document.
4. *Technical Advisory on Scour at Bridges* (FHWA 1988). This presents information similar to references 2, 3, and 4 above, but in a workbook format, and perhaps oversimplified.

Bridges are required across nearly all open urban channels sooner or later and, therefore, sizing the bridge openings is of paramount importance. Open channels with improperly designed bridges will either have excessive scour or deposition or not be able to carry the design flow.

All structural calculations shall be in compliance with the AASHTO LRFD Bridge Design Specifications (current edition) and stamped by a structural engineer licensed in the State of Arkansas. Trail bridges shall be designed according to the LRFD Guide Specifications for Design of Pedestrian Bridges (current edition) and stamped by a structural engineer licensed in the State of Arkansas. The construction specifications shall be ARDOT's specifications modified appropriately to reflect Rogers as the owner rather than ARDOT.

8.1. Coordination with Other Agencies

Numerous local, State and Federal agencies have vested interests in surface waters. These agencies represent interests in water rights, flood control, drainage, conservation, navigation and maintenance of navigation channels, recreation, floodplain management and safety of floodplain occupancy, fish and wildlife, preservation of wetlands, and regulation of construction for the protection of environmental values. Other local, State and Federal agencies have vested interest in historic bridge structures and archeological resources. Early coordination with other agencies will reveal areas of mutual interest and offer opportunities to conserve public funds by resolving conflicts between roadway plans and water resources plans.

8.2. Basic Criteria

Bridge openings shall be designed to have as little effect on the flow characteristics as reasonable, consistent with good bridge design and economics. However, with respect to supercritical flow with a lined channel, the bridge shall not affect the flow at all—that is, there shall be no projections into the design water prism that could create a hydraulic jump or flow instability in the form of reflecting and standing waves.

8.2.1. Design Approach

The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the major storm runoff. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge should be determined. In urban cases this shall not exceed a backwater effect of more than 12 inches.

Velocities under the bridge and downstream of the bridge must receive consideration when choosing the size of the bridge opening. Velocities exceeding those permissible will necessitate special protection of the bottom and banks.

For supercritical flow, the clear bridge opening shall permit the flow to pass under the bridge unimpeded and unchanged in cross section.

8.2.2. Bridge Opening Freeboard

The distance between the design flow water surface and the bottom of the low steel / low chord of the bridge will vary from case to case. However, the debris that may be expected must receive full consideration in setting the freeboard. The minimum allowable freeboard for an arterial/critical service bridge or local/collector bridge is 1-foot for a 100-year and 50-year design storm, respectively. In no case shall any local/collector bridge overtop in the 100-year event no matter the allowable freeboard. Any ARDOT requirements for freeboard shall be adhered to on all state and interstate highways. Refer to ARDOT's freeboard policy in its *Roadway Design Drainage Manual* at <http://www.arkansashighways.com/>.

8.3. Hydraulic Analysis

The hydraulic analysis procedures described below are suitable, although the use of HEC-RAS is preferred.

The design of a bridge opening generally determines the overall length of the bridge. The length affects the final cost of the bridge. The hydraulic engineering in the design of bridges has more impact on the bridge cost than does the structural design.

The reader is referred to *Hydraulics of Bridge Waterways* (U.S. Bureau of Public Roads 1978) for more guidance on the preliminary hydraulic assessment approach described below. In working with bridge openings, the designer may use the designation shown in [Figure CB-17](#).

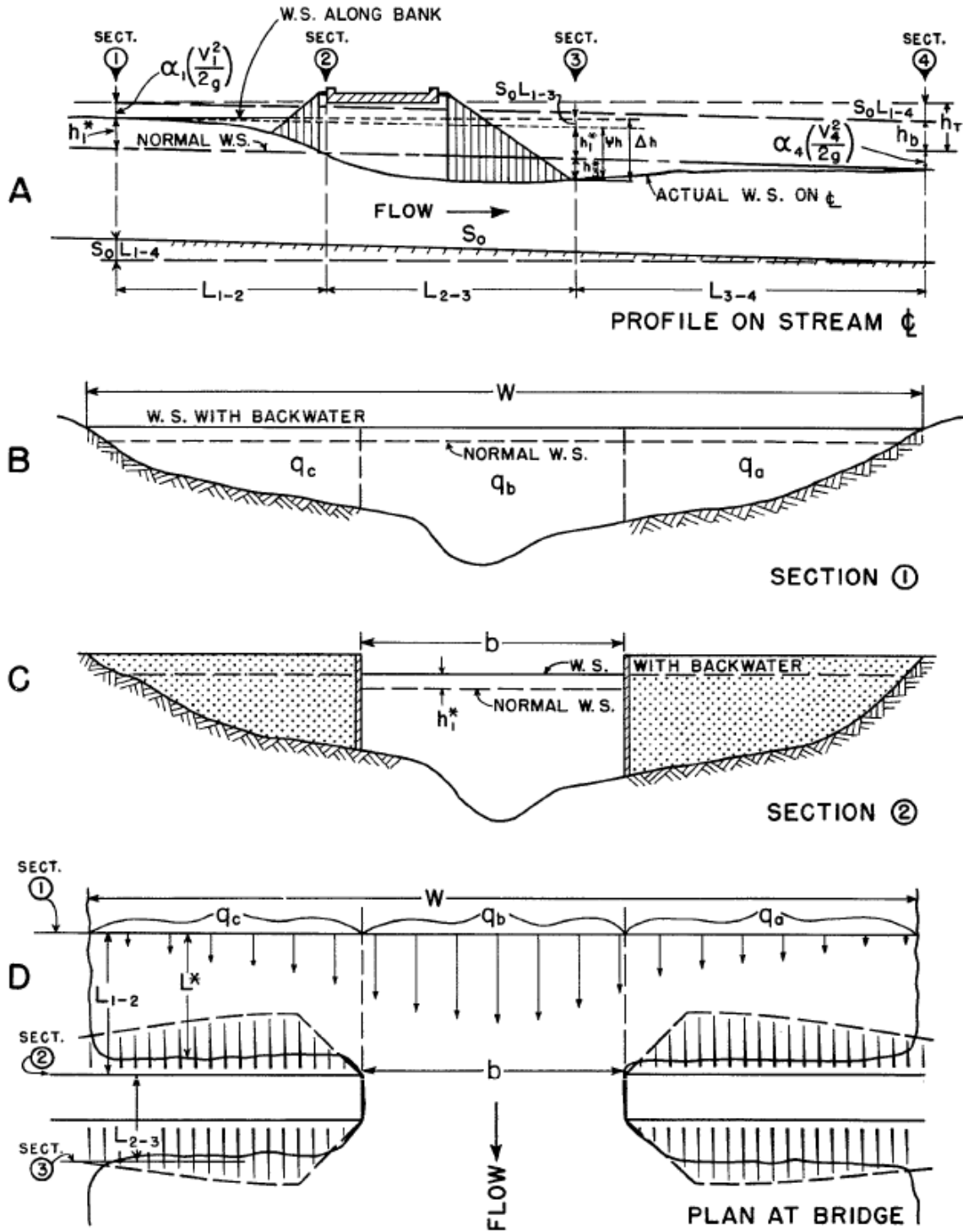
8.3.1. Backwater

Backwater is the increment of increased flood depth upstream of a roadway crossing over a waterway. Backwater should not be used as the sole criterion for judging the acceptability of an alternative design. It is, instead, an aid that can be used in selecting the waterway opening, the crossing profile, and to assess the risk costs of incremental flooding caused by the crossing facility.

8.3.2. Expression for Backwater

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been reestablished, as shown in Sections 1 and 4, respectively, of [Figure CB-17](#). The expression is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical.

Figure CB-17 – Normal Bridge Crossing Designation
(FHWA HDS-1, 1978)



The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_1^* = K^* \left(\frac{V_{n2}^2}{2g} \right) + \alpha 1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad \text{(Equation CB-13)}$$

in which:

h_1^* = Total backwater (ft)

K^* = Total backwater coefficient

$\alpha 1$ = Kinetic energy coefficient

A_{n2} = Gross water area in constriction measured below normal stage (ft²)

V_{n2} = Average velocity in constriction or Q/A_{n2} (ft/sec). The velocity V_{n2} is not an actual measurable velocity but represents a reference velocity readily computed for both model and field structures.

A_4 = Water area at Section 4 where normal stage is reestablished (ft²)

A_1 = Total water area at Section 1 including that produced by the backwater (ft²)

g = Acceleration of gravity (32.2 ft/sec²)

To compute backwater by [Equation CB-13](#), it is necessary to obtain the approximate value of h_1^* by using the first part of the equation:

$$h_1^* = K^* \left(\frac{V_{n2}^2}{2g} \right) \quad \text{(Equation CB-14)}$$

The value of A_1 in the second part of [Equation CB-13](#), which depends on h_1^* can then be determined.

This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $V_{n2}^2/2g$. [Equation CB-14](#) may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$$M > 0.7, \text{ where } M = \text{bridge opening ratio} = b/W \text{ (Figure CB-17)}$$

$$V_{n2} < 7 \text{ ft/sec}$$

$$K^* \left(\frac{V_{n2}^2}{2g} \right) < 0.5 \text{ ft}$$

If values meet all three conditions, the backwater obtained from [Equation CB-14](#) can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use [Equation CB-13](#) in its entirety. The use of the guides is further demonstrated in the examples given in *Hydraulics of Bridge Waterways* (FHWA, HDS-1 1978) that should be used in all bridge design work.

8.3.3. Backwater Coefficient

The value of the overall backwater coefficient K^* , which was determined experimentally, varies with:

1. Stream constriction as measured by bridge opening ratio, M .
2. Type of bridge abutment: wingwall, spill through, etc.
3. Number, size, shape, and orientation of piers in the constriction.
4. Eccentricity or asymmetric position of bridge with the floodplains.
5. Skew (bridge crosses floodplain at other than 90 degree angle).

The overall backwater coefficient K^* consists of a base curve coefficient, K_b , to which are added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K^* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

8.3.4. Effect of M and Abutment Shape (Base Curves)

[Figure CB-18](#) shows the base curve for backwater coefficient, K_b , plotted with respect to the opening ratio, M , for several wingwall abutments and a vertical wall type. Note how the coefficient K_b increases with

channel constriction. The several curves represent different angles of wingwalls as can be identified by the accompanying sketches; the lower curves represent the better hydraulic shapes.

[Figure CB-18](#) shows the relation between the backwater coefficient, K_b , and M for spill-through abutments for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. [Figures CB-18](#) and [CB-19](#) are “base curves” and K_b is referred to as the “base curve coefficient.” The base curve coefficients apply to normal crossings for specific abutment shapes but do not include the effect of piers, eccentricity, or skew.

8.3.5. Effect of Piers (Normal Crossings)

The effect on the backwater from introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p , which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient, ΔK_p , is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M , and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p , to the gross water area of the constriction, A_{n2} , both based on the normal water surface, has been assigned the letter J . In computing the gross water area, A_{n2} , the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from [Figure CB-20](#). The procedure is to enter Chart A, [Figure CB-20](#), with the proper value of J and read ΔK and obtain the correction factor σ from Chart B, [Figure CB-20](#), for opening ratios other than one (1.0). The incremental backwater coefficient is then

$$\Delta K_p = \Delta K \sigma \quad \text{(Equation CB-14)}$$

The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing but should be increased if there are more than 5 piers in a bent. A bent with 10 piers should be given a value of ΔK_p about 20% higher than those shown for bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figures CB-18 or CB-19) } + \Delta K_p \text{ (Figure CB-20)} \quad \text{(Equation CB-15)}$$

8.3.6. Scour

A majority of bridge failures are the result of scour. The added cost of reducing a bridge’s vulnerability to damage from scour is small in comparison to the total cost of a bridge failure. As required by the AASHTO LRFD Bridge Design Specifications Article 3.7.5, scour at bridge foundations is investigated for two conditions.

The first condition is for the design flood for scour, the streambed material in the scour prism above the total scour line shall be assumed to have been removed for design conditions. The design flood storm surge, tide, or mixed population flood shall be the more severe of the 100-year events or from an overtopping flood of lesser recurrence interval.

The second condition is for the check flood for scour, the stability of bridge foundation shall be investigated for scour conditions resulting from a designated flood storm surge, tide, or mixed population flood not to exceed the 500-year event or from an overtopping flood of lesser recurrence interval. Excess reserve beyond that required for stability under this condition is not necessary. The extreme event limit state shall apply.

If the site conditions and low tailwater conditions near stream confluences dictate the use of a more severe flood event for either the design or the check flood for scour, the engineer may use such flood event.

For additional guidance and requirements, refer to the AASHTO LRFD Bridge Design Specifications.

8.4. Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.
2. Determine the stage of the stream at the bridge site for the design discharge.
3. Plot a representative cross section of the stream for design discharge at Section 1, if not already done under Step 2. If the stream channel is essentially straight and the cross section substantially uniform in the vicinity of the bridge, the natural cross section of the stream at the bridge site may be used for this purpose.
4. Subdivide the above cross section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient, n , to each subsection. Careful judgment is necessary in selecting these values. Refer to Table OC-7 in Chapter 7 – *Open Channel Flow Design*.
5. Compute conveyance and then discharge in each subsection.
6. Determine the value of the kinetic energy coefficient.
7. Plot the natural cross section under the proposed bridge based on normal water surface for design discharge and compute the gross water area (including area occupied by piers).

8. Compute the bridge opening ratio, M , observing modified procedure for skewed crossings.
9. Obtain the value of K_b from the appropriate base curve.
10. If piers are involved, compute the value of J and obtain the incremental coefficient, ΔK_p .
11. If eccentricity is severe, compute the value of eccentricity and obtain the incremental coefficient, ΔK_e (FHWA, HDS-1 1978).
12. If a skewed crossing is involved, observe proper procedure in previous steps, and then obtain the incremental coefficient, ΔK_s , for proper abutment type.
13. Determine the total backwater coefficient, K^* , by adding incremental coefficients to the base curve coefficient, K_b .
14. Compute the backwater by [Equation CB-14](#).
15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in *Hydraulics of Bridge Waterways* (FHWA, HDS-1 1978).

8.5. Inadequate Openings

The engineer will often encounter existing bridges and culverts that have been designed for storms having return periods less than 100 years. In addition, bridges will be encountered which have been improperly designed. Often the use of the orifice formula will provide a quick determination of the adequacy or inadequacy of a bridge opening:

$$Q_m = C_b A_b \sqrt{2gH_{br}} \quad \text{(Equation CB-17)}$$

or

$$H_{br} = 0.04 \left(\frac{Q_m}{A_b} \right)^2 \quad \text{(Equation CB-18)}$$

in which:

Q_m = The major storm discharge (ft³/s)

C_b = The bridge opening coefficient (0.6 assumed in Equation CB-17)

A_b = The area of the bridge opening (ft²)

g = Acceleration of gravity (32.2 ft/s²)

H_{br} = The head, that is the vertical distance from the bridge opening center point to the upstream water surface about 10H upstream from the bridge, where H is the height of the bridge, in feet. It is approximately the difference between the upstream and downstream water surfaces where the lower end of the bridge is submerged.

These expressions are valid when the water surface is above the top of the bridge opening.

Figure CB-18 – Base Curves for Wingwall Abutments
(UDFCD USDCM, 2001)

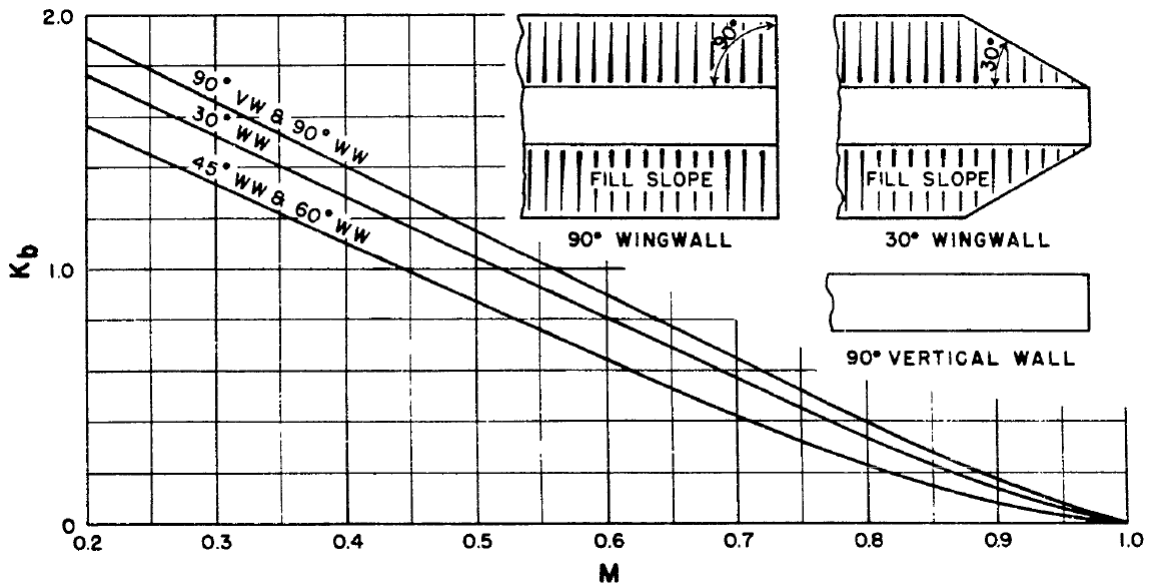


Figure CB-19 – Base Curves for Spillthrough Abutments
(UDFCD USDCM, 2001)

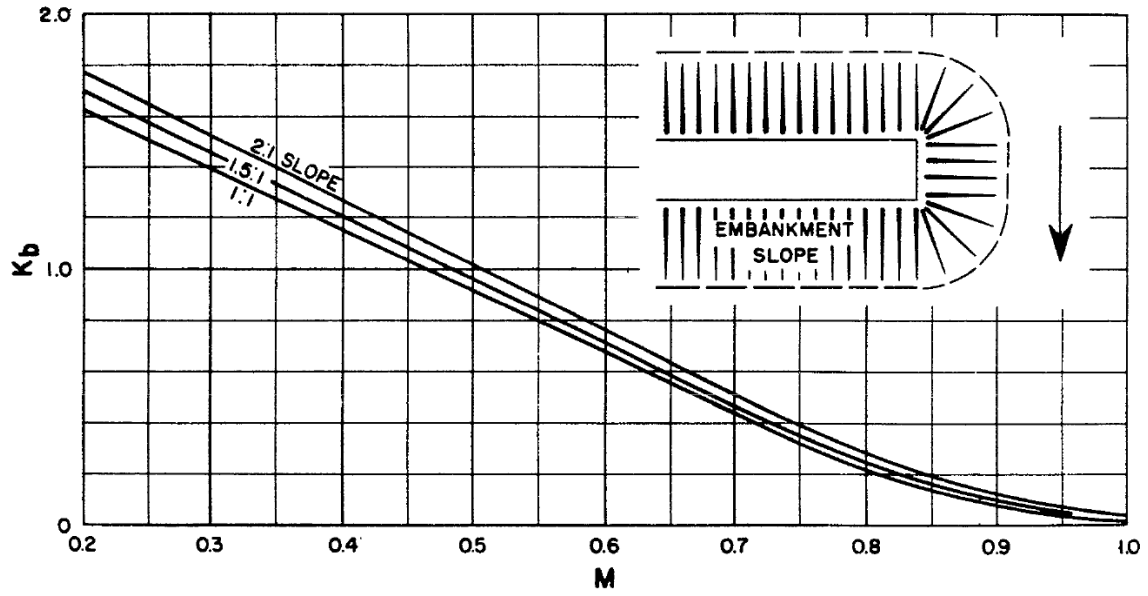
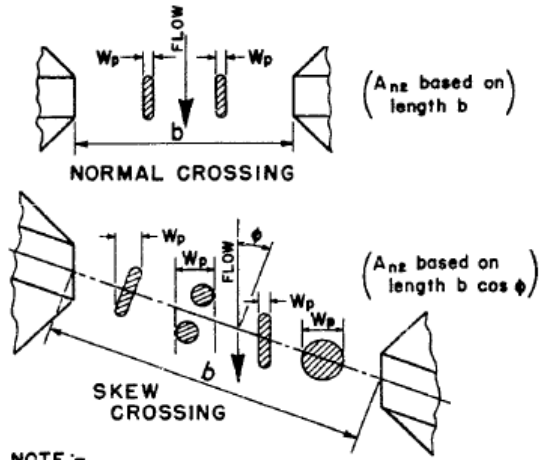


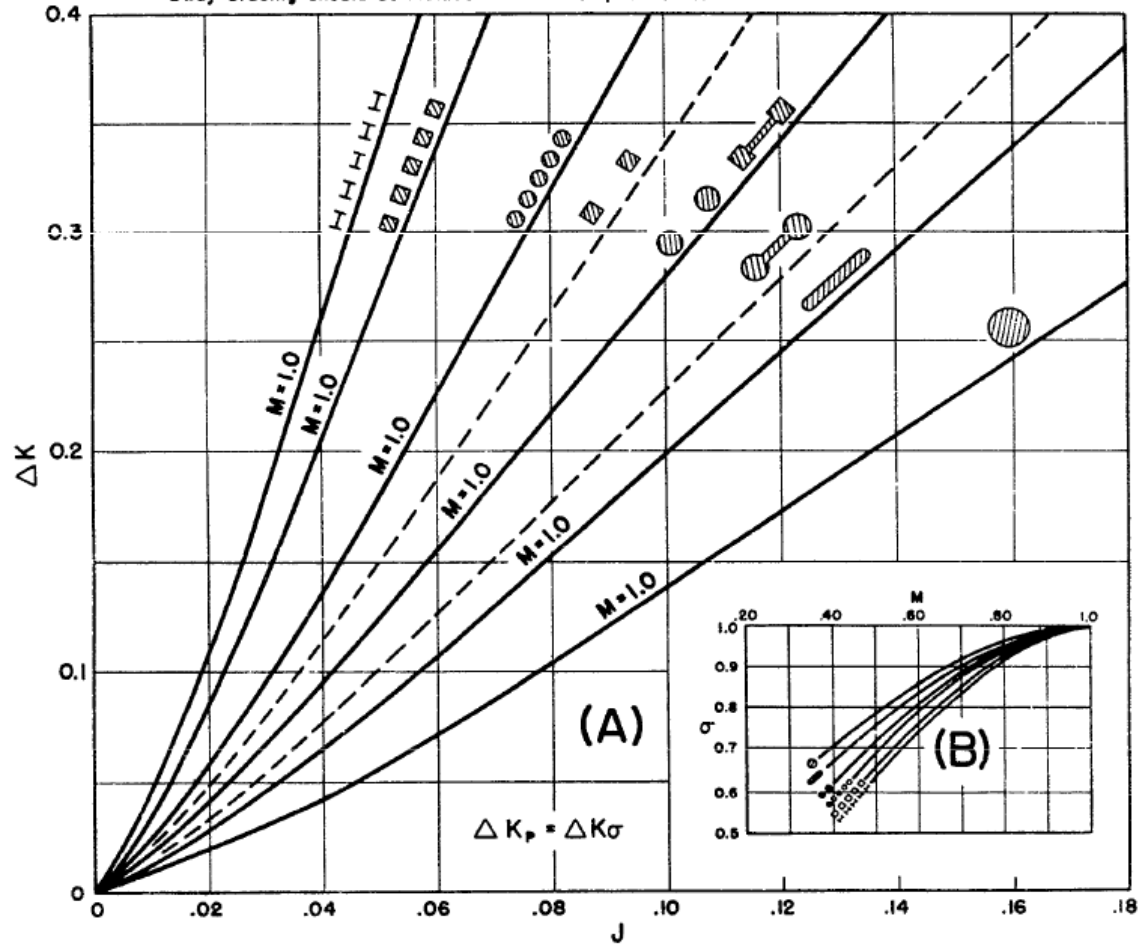
Figure CB-20 – Incremental Backwater Coefficient for Pier

(FHWA HDS-1, 1978)



- W_p = Width of pier normal to flow - feet
- h_{nz} = Height of pier exposed to flow - feet
- N = Number of piers
- $A_p = \sum^N W_p h_{nz}$ = total projected area of piers normal to flow - square feet
- A_{nz} = Gross water cross section in constriction based on normal water surface. (Use projected bridge length normal to flow for skew crossings)
- $J = \frac{A_p}{A_{nz}}$

NOTE:- Sway bracing should be included in width of pile bents.



9. DESIGN EXAMPLE

The following example problem illustrates the culvert design procedures using the *RDM-Culvert* spreadsheet application.

9.1. Culvert Under an Embankment

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

The maximum allowable headwater elevation is 5288.5. The natural 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.

Solution:

Step 1 Gather all crucial design information:

- Design discharge (Q) for desired events:
 - $Q_{5\text{-yr}} = 20$ cfs & $Q_{100\text{-yr}} = 35$ cfs
- Culvert Length (L):
 - $L = 95.00'$
- Invert elevations
 - Inlet Invert = 5283.50'
 - Outlet Invert = 5281.50'
- Calculate pipe slope
 - $$\frac{5283.50' - 5281.50'}{95.00'} = 0.0211 \text{ ft / ft}$$
- Determine acceptable headwater and tailwater elevations:
 - Headwater = 5288.50
 - Tailwater_{5-yr} = 5281.50' + 2.50' = 5284.00'
 - Tailwater_{100-yr} = 5281.50' + 3.00' = 5284.50'

- Step 2** **Select culvert shape and material:**
- Pipe
 - Concrete
 - Manning's n-value = 0.013
- Step 3** **Enter information into BC-Culvert Spreadsheet under the 'Pipe' tab leaving the Diameter (D) cell blank for the 5 year storm ([Figure CB-16](#)).**
- Step 4** **Follow prompts given by spreadsheet to select an appropriate size of pipe.**
- D = 24" Pipe
- Step 5** **Check headwater elevation by entering information into 'Culvert' tab.**
- D = 24 inches
 - Square End with Headwall
 - 1 Barrel
 - Inlet Invert Elevation = 5283.50'
 - Culvert Slope = 0.0211 ft/ft
 - L = 95.00'
 - n = 0.012
 - $K_b = 0.00$
 - $K_x = 1.00$
 - Tailwater Elevation = 5284.00'
 - Start Headwater Elevation = 5285.00' and increase by 0.25'
- Step 6** **Examine the results from the Calculations of Culvert Capacity (output) table. Determine whether culvert is large enough based on the Controlling Culvert Flowrate (cfs) given the design discharge (Q) and maximum allowable headwater elevation. Make adjustments as needed to accommodate the design discharge and maximum allowable headwater elevation.**

- @ Headwater = 5288.50', Controlling Culvert Flowrate = 26.32 cfs
- 26.32 cfs > 20.00 cfs; therefore, proceed to next step

Step 7 **Check the culvert size against additional design discharges (if multiple design storms are required) by repeating Steps 3 – 6. Make adjustments to the culvert until it can handle all design discharges at or below the maximum allowable headwater elevation.**

- In order for the culvert to handle 35 cfs from the 100 year storm, the pipe size must be increased to at least 28" ([Figure CB-17](#)).

Step 8 **Develop multiple alternatives for analysis.**

Step 9 **Compute outlet velocities for each acceptable alternate.**

Step 10 **Make recommendations.**

Figure CB-21 – BC-Culvert Spreadsheet Pipe Tab

	A	B	C	D	E	F	G																																																																				
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2	CIRCULAR CONDUIT FLOW (Normal & Critical Depth Computation)																																																																										
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10															<table border="1"> <thead> <tr> <th colspan="2">Design Information (Input)</th> </tr> </thead> <tbody> <tr> <td>Pipe Invert Slope</td> <td>So = 0.0211 ft/ft</td> </tr> <tr> <td>Pipe Manning's n-value</td> <td>n = 0.0120</td> </tr> <tr> <td>Pipe Diameter</td> <td>D = 24.00 inches</td> </tr> <tr> <td>Design discharge</td> <td>Q = 20.00 cfs</td> </tr> <tr> <td colspan="2">Full-flow Capacity (Calculated)</td> </tr> <tr> <td>Full-flow area</td> <td>Af = 3.14 sq ft</td> </tr> <tr> <td>Full-flow wetted perimeter</td> <td>Pf = 6.28 ft</td> </tr> <tr> <td>Half Central Angle</td> <td>Theta = 3.14 radians</td> </tr> <tr> <td>Full-flow capacity</td> <td>Qf = 35.70 cfs</td> </tr> <tr> <td colspan="2">Calculation of Normal Flow Condition</td> </tr> <tr> <td>Half Central Angle (0<Theta<3.14)</td> <td>Theta = 1.64 radians</td> </tr> <tr> <td>Flow area</td> <td>An = 1.71 sq ft</td> </tr> <tr> <td>Top width</td> <td>Tn = 2.00 ft</td> </tr> <tr> <td>Wetted perimeter</td> <td>Pn = 3.28 ft</td> </tr> <tr> <td>Flow depth</td> <td>Yn = 1.07 ft</td> </tr> <tr> <td>Flow velocity</td> <td>Vn = 11.68 fps</td> </tr> <tr> <td>Discharge</td> <td>Qn = 20.00 cfs</td> </tr> <tr> <td>Percent Full Flow</td> <td>Flow = 56.02% of full flow</td> </tr> <tr> <td>Normal Depth Froude Number</td> <td>Fr_n = 2.22 supercritical</td> </tr> <tr> <td colspan="2">Calculation of Critical Flow Condition</td> </tr> <tr> <td>Half Central Angle (0<Theta-c<3.14)</td> <td>Theta-c = 2.22 radians</td> </tr> <tr> <td>Critical flow area</td> <td>Ac = 2.70 sq ft</td> </tr> <tr> <td>Critical top width</td> <td>Tc = 1.59 ft</td> </tr> <tr> <td>Critical flow depth</td> <td>Yc = 1.61 ft</td> </tr> <tr> <td>Critical flow velocity</td> <td>Vc = 7.40 fps</td> </tr> <tr> <td>Critical Depth Froude Number</td> <td>Fr_c = 1.00</td> </tr> </tbody> </table>							Design Information (Input)		Pipe Invert Slope	So = 0.0211 ft/ft	Pipe Manning's n-value	n = 0.0120	Pipe Diameter	D = 24.00 inches	Design discharge	Q = 20.00 cfs	Full-flow Capacity (Calculated)		Full-flow area	Af = 3.14 sq ft	Full-flow wetted perimeter	Pf = 6.28 ft	Half Central Angle	Theta = 3.14 radians	Full-flow capacity	Qf = 35.70 cfs	Calculation of Normal Flow Condition		Half Central Angle (0<Theta<3.14)	Theta = 1.64 radians	Flow area	An = 1.71 sq ft	Top width	Tn = 2.00 ft	Wetted perimeter	Pn = 3.28 ft	Flow depth	Yn = 1.07 ft	Flow velocity	Vn = 11.68 fps	Discharge	Qn = 20.00 cfs	Percent Full Flow	Flow = 56.02% of full flow	Normal Depth Froude Number	Fr_n = 2.22 supercritical	Calculation of Critical Flow Condition		Half Central Angle (0<Theta-c<3.14)	Theta-c = 2.22 radians	Critical flow area	Ac = 2.70 sq ft	Critical top width	Tc = 1.59 ft	Critical flow depth	Yc = 1.61 ft	Critical flow velocity	Vc = 7.40 fps	Critical Depth Froude Number	Fr _c = 1.00
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Figure CB-22 – BC-Culvert Spreadsheet Culvert Tab

CULVERT STAGE-DISCHARGE SIZING (INLET vs. OUTLET CONTROL WITH TAILWATER EFFECTS)							
1							
2							
3	Project: Blue cells are for the user to enter data into						
4	Basin ID: Green cells are calculated values, filled from the VB macro code						
5							Clear Worksheet
6							Update Graph
7							
8							
9							
10							
11							
12							
13							
14							
15							
16	Design Information (Input):						
17	Circular Culvert: Barrel Diameter in Inches		D = <input type="text" value="28.00"/>		inches		
18	Inlet Edge Type (choose from pull-down list)		<input type="text" value="Square End with Headwall"/>		<input type="text" value="Square End with Headwall"/>		
19	OR:						
20	Box Culvert: Barrel Height (Rise) in Feet		Height (Rise) = <input type="text"/>		ft.		
21	Barrel Width (Span) in Feet		Width (Span) = <input type="text"/>		ft.		
22	Inlet Edge Type (choose from pull-down list)		<input type="text" value="1:1 Bevel w/ Headwall"/>		<input type="text" value="1:1 Bevel w/ Headwall"/>		
23							
24	Number of Barrels		No = <input type="text" value="1"/>				
25	Inlet Elevation at Culvert Invert		Inlet Elev = <input type="text" value="5283.5"/>		ft. elev.		
26	Outlet Elevation at Culvert Invert OR Slope of Culvert (ft v./ft h)		Slope = <input type="text" value="0.0211"/>		ft. vert. / ft. horiz.		
27	Culvert Length in Feet		L = <input type="text" value="95.00"/>		ft.		
28	Manning's Roughness		n = <input type="text" value="0.0130"/>				
29	Bend Loss Coefficient		K _b = <input type="text" value="0.00"/>				
30	Exit Loss Coefficient		K _e = <input type="text" value="1.00"/>				
31							
32							
33	Design Information (calculated):						
34	Entrance Loss Coefficient		K _e = <input type="text" value="0.50"/>				
35	Friction Loss Coefficient		K _f = <input type="text" value="0.96"/>				
36	Sum of All Loss Coefficients		K _Σ = <input type="text" value="2.46"/>				
37	Orifice Inlet Condition Coefficient		C _d = <input type="text" value="0.86"/>				
38	Minimum Energy Condition Coefficient		KE _{min} = <input type="text" value="-0.6355"/>				
39							
40	Calculations of Culvert Capacity (output):						
41							
42							
43							
44							
45	Water Surface Elevation <input type="text" value="Enter HW Elev"/> (ft., linked)	Tailwater Surface Elevation ft (input if known)	Culvert Inlet-Control Flowrate cfs (output)	Culvert Outlet-Control Flowrate cfs (output)	Controlling Culvert Flowrate cfs (output)	Inlet Equation Used: (output)	
46	5285.00	5284.50	9.53	13.07	9.53	regression equation	
47	5285.25	5284.50	12.21	16.00	12.21	regression equation	
48	5285.50	5284.50	15.12	18.47	15.12	regression equation	
49	5285.75	5284.50	18.06	20.65	18.06	regression equation	
50	5286.00	5284.50	20.88	22.62	20.88	regression equation	
51	5286.25	5284.50	23.50	24.44	23.50	regression equation	
52	5286.50	5284.50	25.90	26.12	25.90	regression equation	
53	5286.75	5284.50	28.10	27.71	27.71	regression equation	
54	5287.00	5284.50	30.14	29.20	29.20	regression equation	
55	5287.25	5284.50	32.03	30.63	30.63	regression equation	
56	5287.50	5284.50	33.80	31.99	31.99	regression equation	
57	5287.75	5284.50	35.47	33.29	33.29	regression equation	
58	5288.00	5284.50	37.06	34.55	34.55	regression equation	
59	5288.25	5284.50	38.58	35.76	35.76	regression equation	
60	5288.50	5284.50	40.03	36.94	36.94	regression equation	
61	5288.75	5284.50	41.43	38.07	38.07	regression equation	
62	5289.00	5284.50	42.78	39.18	39.18	regression equation	
63	5289.25	5284.50	44.09	40.25	40.25	regression equation	
64	5289.50	5284.50	45.36	41.29	41.29	regression equation	
65	5289.75	5284.50	46.60	42.31	42.31	regression equation	
66	5290.00	5284.50	47.81	43.31	43.31	regression equation	
67	5290.25	5284.50	48.99	44.28	44.28	regression equation	

10.

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