

CHAPTER 7. CULVERT AND BRIDGE HYDRAULIC DESIGN**CONTENTS**

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1.0 CULVERTS INTRODUCTION AND OVERVIEW

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.

The function of a culvert is to convey surface water under a roadway, railroad, walking path, 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 plastic pipes for driveways to large multiple side-by-side concrete box sections that can be used in lieu of a bridge.

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 design criteria are dictated by the City of Bella Vista. 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?

- 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.

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 \(table cb1\)](#) 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 should 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.

2.0 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
- A = Flow Area (ft²)
- R = Hydraulic Radius (ft)
- S = Channel Slope (ft/ft)

$$Q = v_1 A_1 = v_2 A_2 \quad \text{(Equation CB-2)}$$

where:

- Q = Flow rate or discharge (ft³/sec)
- v = Velocity (ft/sec)
- A = Flow Area (ft²)

$$\frac{v^2}{2g} + \frac{p}{\gamma} + z + \text{losses} = \text{constant} \quad \text{(Equation CB-3)}$$

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)

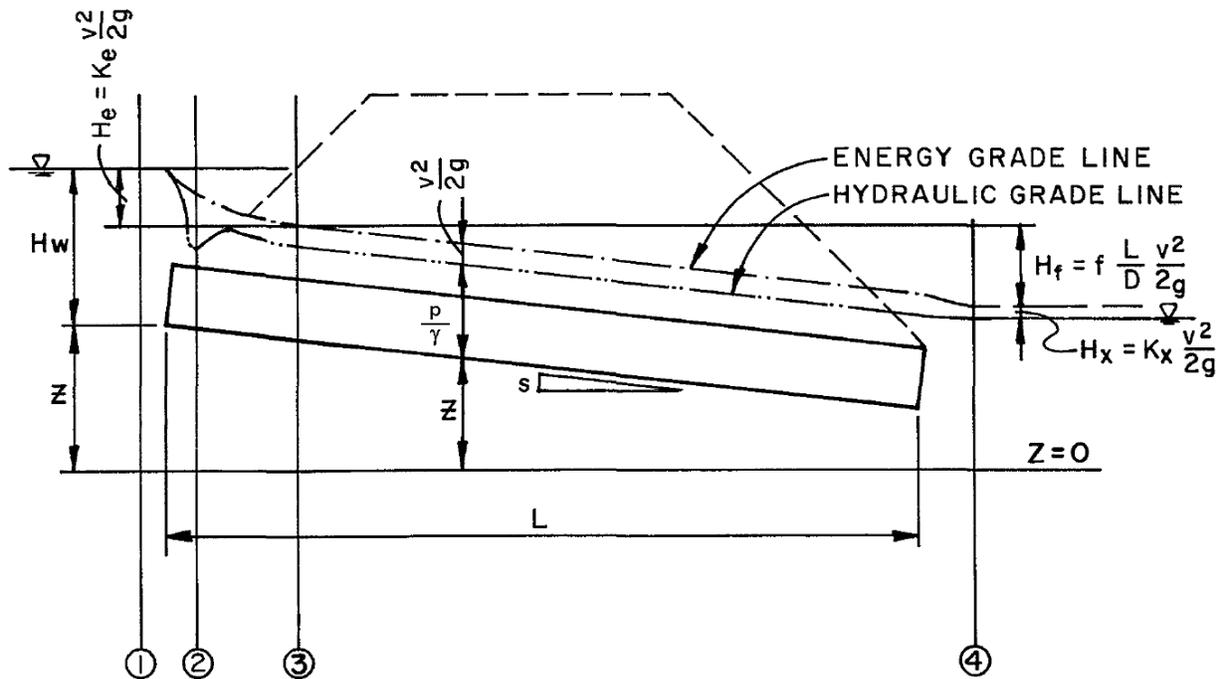
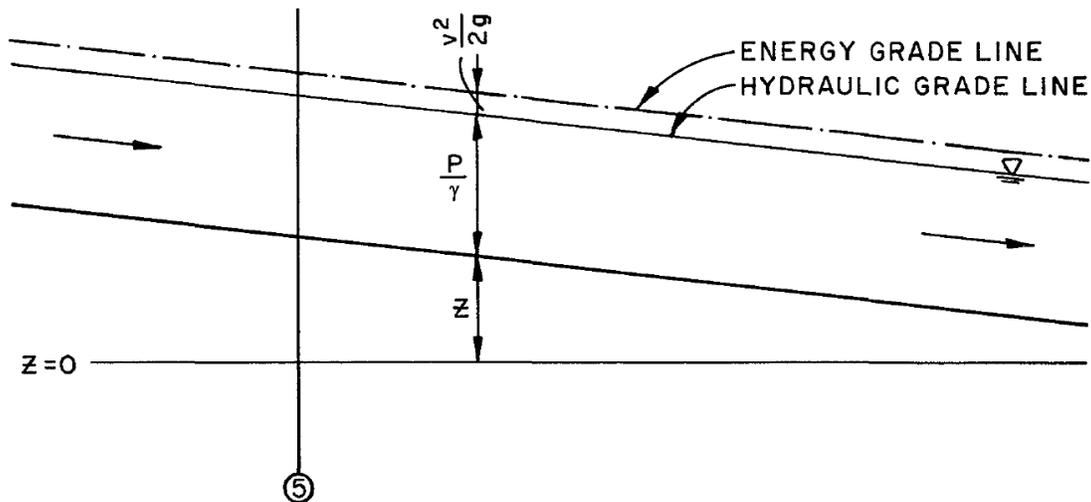


Figure CB-2: Definition of Terms for Open Channel Flow
(UDFCD USDCM 2001)



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

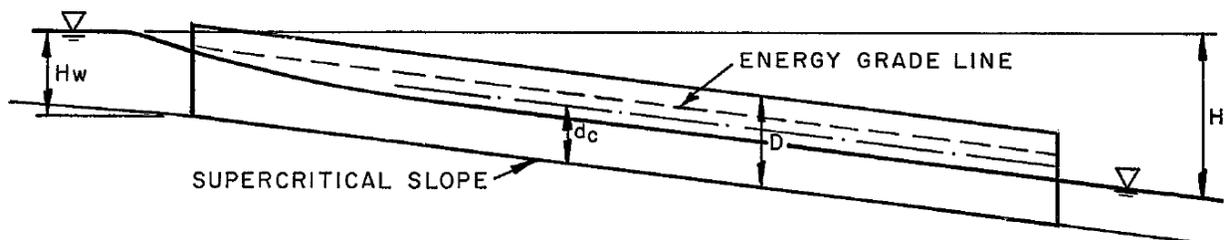
A culvert operates with inlet control when the flow capacity is controlled at the entrance by these factors:

- 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.

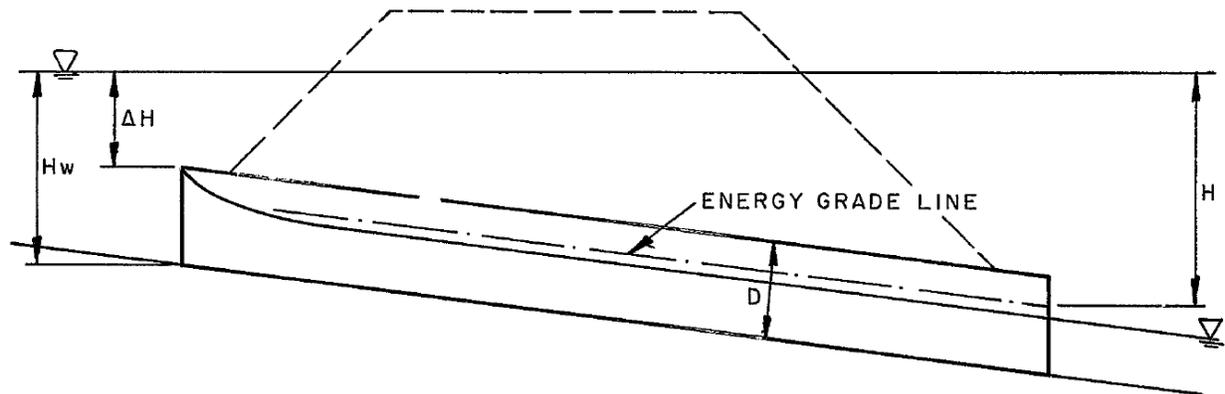
Inlet control for culverts may occur in two ways. The least common occurs when the headwater depth is not sufficient to submerge the top of the culvert and, concurrently, the culvert invert slope is supercritical as shown in [Figure CB-3](#).

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 as in [Figure CB-4](#) below 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)

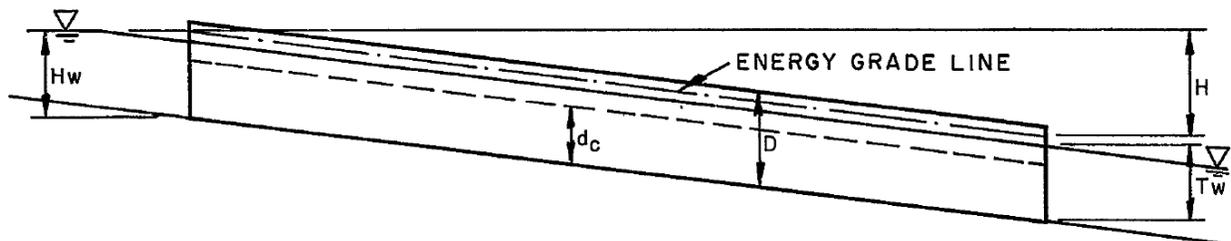
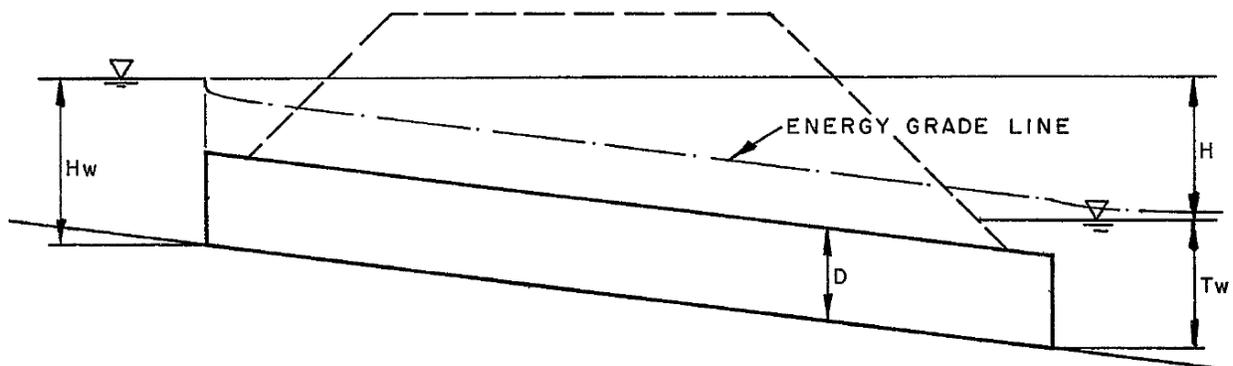


Figure CB-6: Outlet Control - Full Conduit
(UDFCD USDCM 2001)



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: Contract 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 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.

3.0 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 below:

Table CB-2: Design Storm Frequencies and Minimum Freeboard

Description	Design Storm Frequency	Minimum Freeboard
Trails	2	12 inches
Alley / Residential	10	12 inches
Sub-Collector / Collector	25	12 inches
Minor Arterial & Major Arterial	50	12 inches
Bridges (Alley/Residential & Collector Roadways)	50	12 inches *
Bridges (Arterial & Critical Service Access Roadways/Drives)	100	12 inches *

* 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.

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 and tailwater elevations along with an 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 square or rectangular box; or circular, elliptical, or arch pipe.

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

The following outline provides guidelines for the appropriate use of various culvert shapes and materials.

- Reinforced Concrete Pipe (RCP)
 - Shall be used under all public roadways;
 - RCP shall conform to AASHTO M170 for circular pipe and AASHTO M206 for arch pipe;
 - All pipe having a diameter of 36-inches or greater must be RCP;
 - When pipe cover is:
 - Less than 2-ft - RCP must meet ASTM Class IV specifications; and/or
 - 2-foot or larger - RCP must meet ASTM Class III specifications
- Reinforced Concrete Box (RCB)
 - Shall be structurally designed to accommodate the earth and live loads to be imposed;
 - Refer to the Arkansas State Highway and Transportation Department's Reinforced Concrete Box Culvert Standard Drawings; and
 - Within public right of way, shall be capable of withstanding a minimum HL-93 loading.
- Corrugated Metal Pipe (CMP) [including Smooth Lined (SLCMP)]
 - Can only be used in areas outside of street right-of-way;
 - Shall conform to AASHTO M36 and M218;
 - Shall have minimum 2-foot cover; and
 - Shall be properly bedded and backfilled with granular material.
- Corrugated Polyethylene Pipe (CPP) [including Smooth Lined (SLCPP) such as HDPE]
 - Can only be used in areas outside of street right-of-way;
 - Shall conform to AASHTO M294, Type S specification; and
 - Shall have minimum 2-foot cover; and
 - Shall be properly bedded and backfilled with granular material.

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-2](#):

Table CB-3: Maximum Allowable Discharge Velocities

Downstream Condition	Maximum Allowable Discharge Velocity
Grass	4 fps
Riprap	12 fps
Concrete	18 fps
Flow transition mat (or TRM)	Manufacturer's Specs.

3.5 Minimum Slope

To minimize sediment deposition in the culvert, the culvert slope must be equal to or exceed the slope required for a minimum velocity of 3 fps when the pipe is flowing full as recommended in FHWA [HEC-22](#). The slope should be checked for each design. 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.

4.0 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 outlet) requires a balance between hydraulic efficiency; purpose; and topography at the proposed culvert site. Where there is sufficient allowable headwater, the choice of inlets may not be critical. When headwater depth is limited, erosion or sedimentation is more likely to occur so a more efficient inlet will be required to obtain the necessary discharge capacity for the culvert.

Although the primary purpose of a culvert is to convey flows, it may also be used to restrict flow. An example of this is where the area upstream from the culvert is designed to be used for detention storage so a reduced peak discharge rate is controlled by an inlet with limited capacity.

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-3](#).

Table CB-4: Inlet Coefficients for Outlet Control

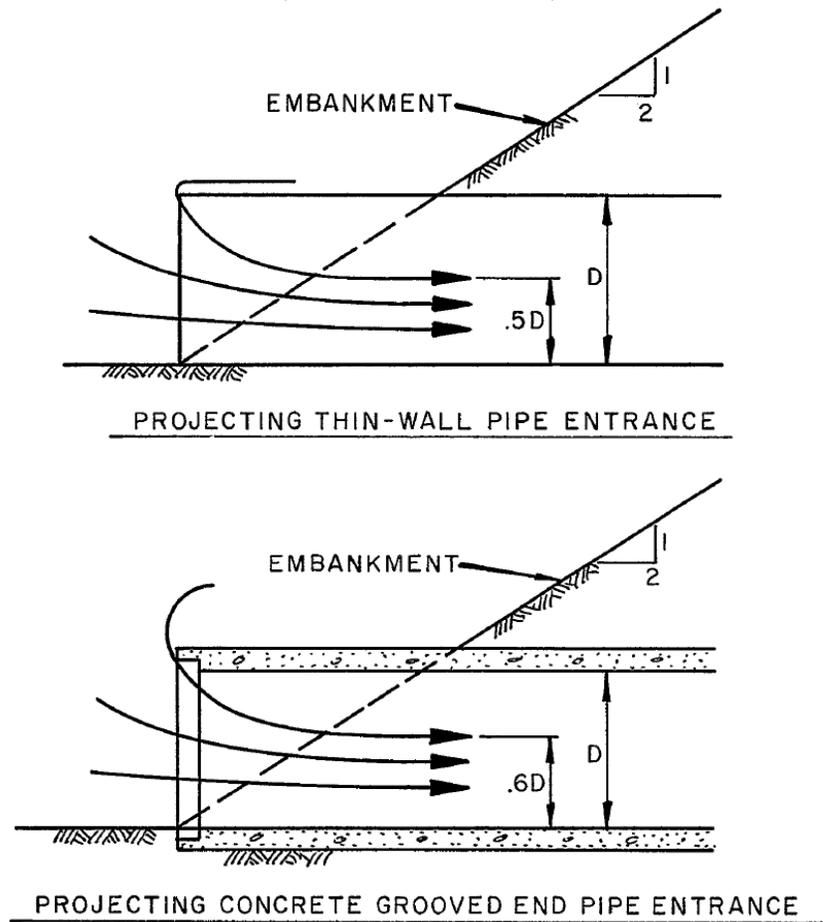
Circular Culvert (including elliptical and arch)	Coefficient, K_e
Square End Projection	0.2
Square End with Headwall	0.5
Grooved End Projection	0.2
Grooved End with Headwall	0.2
1.1:1 Beveled Edge	0.2
1.5:1 Beveled Edge	0.2
Box Culvert (including square and rectangular)	Coefficient, K_e
Square Edge with 90- to 15-Degree Headwall	0.5
1.5:1 Bevel with 90-Degree Headwall	0.2
1:1 Bevel with Headwall	0.2
Square Edge with 30- to 78-Degree Flared Wingwall	0.4
Square Edge with 90- to 15-Degree Flared Wingwall	0.5
Square Edge with 0-Degree Flared Wingwall	0.7
1.5:1 Bevel with 18- to 34-Degree Flared Wingwall	0.2
1.5:1 Bevel with 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.

Projecting inlets of corrugated metal 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. 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. Due to difficulty of maintaining projecting inlets, they should not be used except for special circumstances and will require approval by city staff on a case-by-case basis. For the special circumstances when projecting inlets are allowed, the following design guidelines shall apply.

Figure CB-7: Common Projecting Culvert Inlets
(UDFCD USDCM 2001)



4.1.1 Corrugated Metal Pipe

The 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-3](#)).

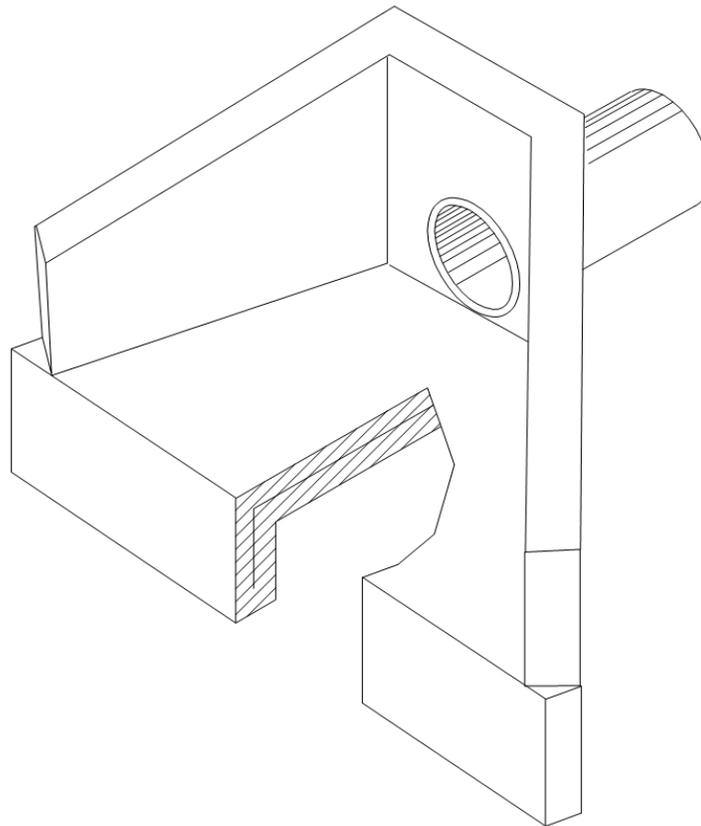
4.1.2 Concrete Pipe

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 0.25. For concrete pipe that has been cut, the entrance is square-edged so the entrance coefficient is 0.5 (see [Table CB-3](#)).

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-3](#). 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 with an entrance coefficient of 0.4. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced to 0.15 for rounding the edge to an edge radius equal to 0.15 times the culvert diameter; or to 0.10 for rounding the edge to an edge 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, there is little increase in hydraulic efficiency 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. The entrance coefficient is equal to 0.4 for concrete pipe that has been cut.

4.2.3 Wingwalls

Wingwalls are used where the side slopes of the channel adjacent are unstable or 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. 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.

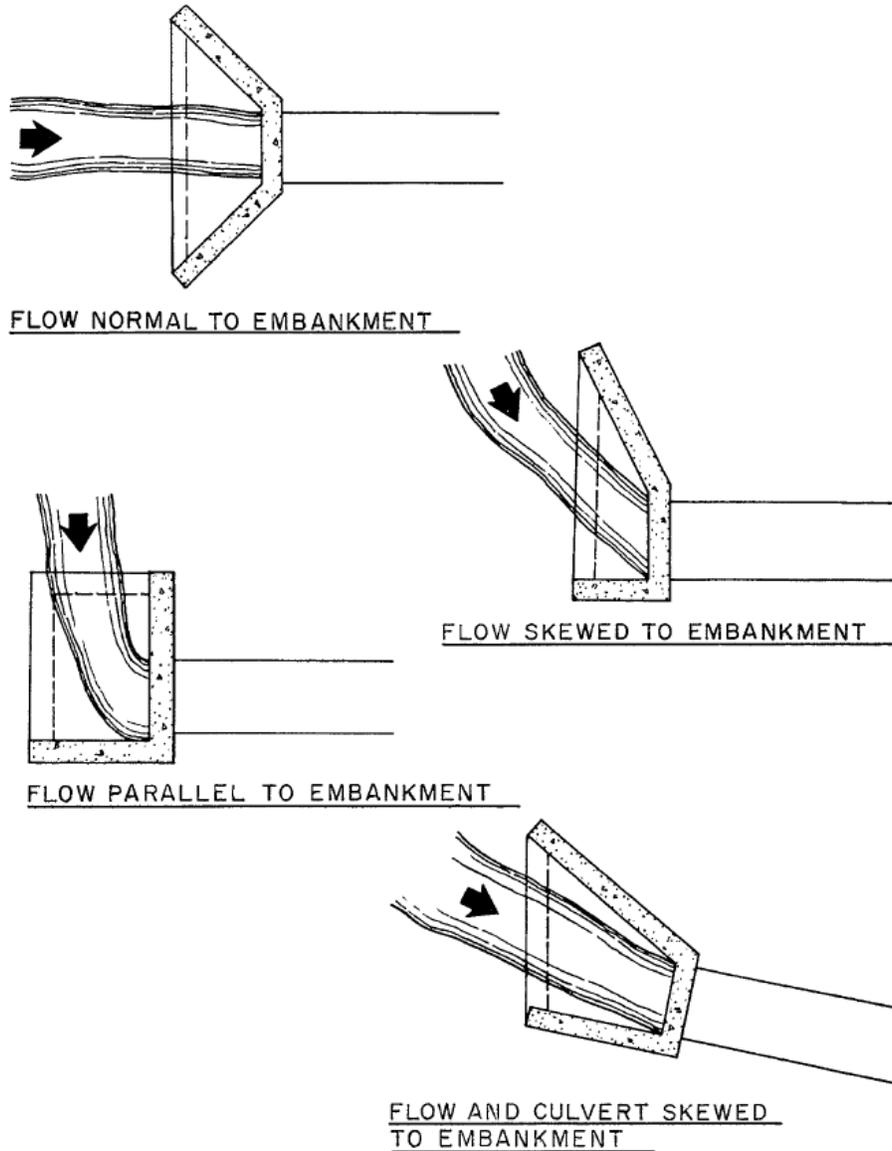
4.2.4 Aprons

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. Also 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 will also be required on the apron.

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 both outlets and inlets and is 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 more specifically 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; and
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 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. Where the full contraction to the culvert diameter takes place in the throat of the end section, the entrance coefficient, K_e , is equal to 0.25.

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. In practice if the embankment is not paved, the entrance usually does not conform to the side slopes so results in a projecting entrance with K_e equal to 0.9. If the embankment is paved, a sloping headwall is obtained with K_e equal to 0.60. Beveling the edges will result in a K_e equal to 0.50.

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-edged 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 which improves 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 situations 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.

5.0 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; or
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; and
- 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 will form just inside the inlet. As a result a buoyant effect is created once the inlet is submerged. The buoyancy forces increase as the headwater depth increase. These forces, along with vortexes and eddy currents, can cause scour; undermine culvert inlets; and erode embankment slopes. This makes the inlet vulnerable to failure, especially with deep headwater.

The large unequal pressures resulting from inlet constriction are accentuated when the capacity of the culvert is impaired by debris or damage. These pressures are essentially buoyancy forces that can cause entrance failures - particularly on corrugated metal pipe with mitered; skewed; or projecting ends. The failure potential of the pipe 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 required (unless otherwise approved by the City) to counteract the hydrostatic uplift and to prevent failure due to buoyancy.

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 can lead to a large bending moment being 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.

6.0 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; 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 locating and 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 can protect the channel while redistributing or spreading its 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](#) (2006).

Three forms of energy dissipators and erosion control are approved by the City of Bella Vista: Riprap (requiring City approval), Drop Structures and Flow Transition Mats.

6.2.1 Riprap as Outlet Protection

Riprap is generally 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 must be grouted in place. Riprap shall not be the first choice for energy dissipation/erosion control. 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 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-11](#).

6.2.1.1 Length of Protection

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

$$L_p = \left(\frac{1.7 * Q}{D_o^{3/2}} \right) + 8 * D_o \quad \text{if culvert is flowing less than half full} \quad \text{(Equation CB-7)}$$

$$L_p = \left(\frac{3.0 * Q}{D_o^{3/2}} \right) \quad \text{if culvert is flowing at least half full} \quad \text{(Equation CB-8)}$$

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 $20 * D_o$.

6.2.1.2 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):

$$W = 3 * D_o + 0.4 * L_p \quad \text{if } TW \text{ is equal or above the elevation of center of culvert} \quad \text{(Equation CB-9)}$$

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

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 $3 \cdot D_o$. See [Figure CB-10](#) for additional details on outlets that don't have a well-defined channel downstream.

6.2.1.3 Thickness and Stone Size/Gradation

The riprap blanket thickness shall be at least double the median stone diameter (d_{50}) for the first 50% of its length. Immediately after the culvert discharge, the blanket thickness can decrease to not less than one-and-a-half-times the median stone diameter. Furthermore, the riprap blanket shall extend up the side slopes at least 12-inches above the design tailwater elevation, but no lower than 67% 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 the median stone diameter. A geo-textile 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-11](#) for additional details on riprap extents.

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

$$d_{50} = \left(\frac{0.02}{TW} \right) * \left(\frac{Q}{D_o} \right)^{4/3} \quad \text{(Equation CB-14)}$$

where:

d_{50} = Median stone size (ft)

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

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-14](#) 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-10: Configuration of Conduit Outlet Protection for Undefined Channel Downstream (NJDOT, SCS 2008)

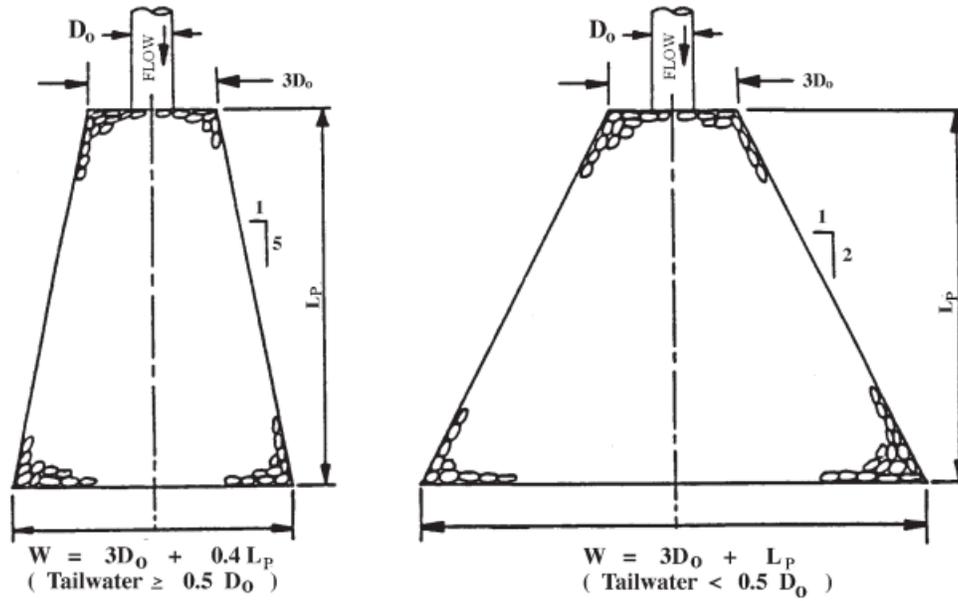
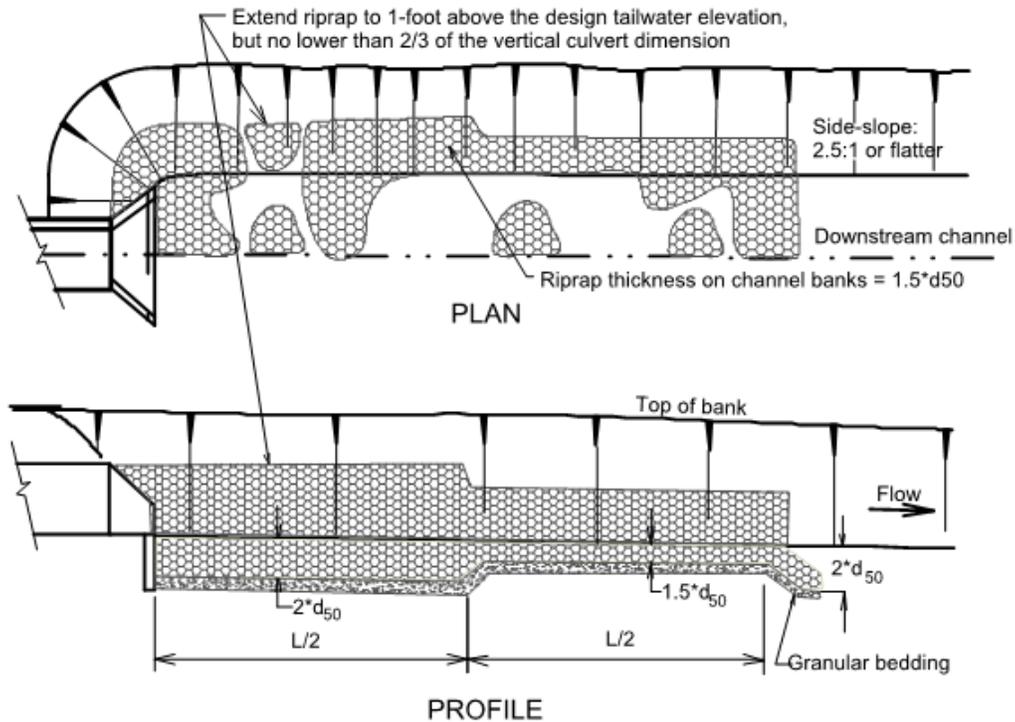


Figure CB-11: Culvert and Pipe Outlet Erosion Protection (UDFCD USDCM 2002)



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 sections. Minimum depth of cutoff wall equals 2 times d_{50} or 3-feet, whichever is deeper.
3. Provide joint fasteners for flared-end sections.

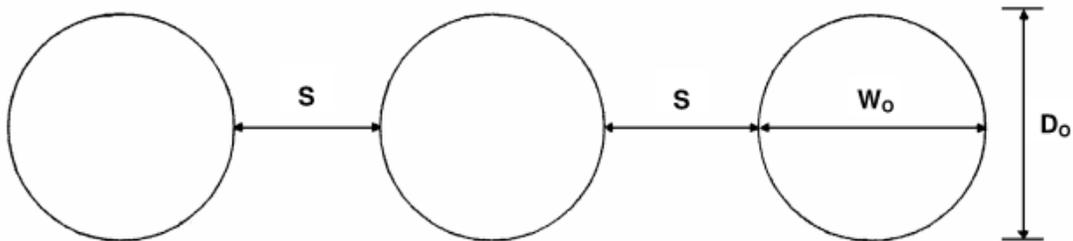
6.2.1.4 Multiple Culverts Outlets

When more than one culvert outlet is positioned at the same location, use the guidelines below to size the riprap protection apron. See [Figure CB-12](#) 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-12: Guidance for Outlet Protection for Multiple Culverts
(NJDOT, SESCO 2008)

All Culverts Same Diameter Discharging Same Q



For S less than 25% W_o , size the riprap & length for one pipe; Width shall accommodate all culverts.
For S equal or more than 25% W_o , size the riprap & length for one pipe; Increase values by 25%.

6.2.2 Drop Structures

Drop structures are commonly used for flow control and energy dissipation. Changing the channel slope from steep to mild and/or by placing drop structures at intervals along the channel reach can change a continuous steep slope into a series of gentle slopes and vertical drops. By slowing down and transferring high velocities to prevent erosion, drop structures control the slope of the channel to prevent erosion 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.

6.2.3 Flow Transition Mats

A flow transition mat (FTM) is a long-term non-biodegradable bio-technical alternative for hard armor such as riprap. It is usually a mechanically-anchored matting designed with voids throughout the structure to enable vegetative growth and cover the material while also providing mechanical protection in areas where design discharges exert velocities and shear stresses that exceed the limits of natural vegetation. FTMs are used to extend the performance limits of natural vegetation by retaining soil particles and vegetative seeds; promoting conditions for accelerated vegetative growth; and reinforcing the vegetative cover. The EPA has documented FTMs as useful BMPs for stormwater runoff. The EPA's Storm Water Technology Fact Sheet for FTMs 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.

Many different manufacturers produce FTMs, each with its own patented methods and material combinations. Some of the manufacturers/distributors of FTMs include North American Green, Propex, ScourStop, Tensar, and many others. FTMs 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 shows that it is adequately suited for the situation in the field.

Figure CB-13: Typical Flow Transition Mat Application
(Scourstop.com)



7.0 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 drainage expeditiously and effectively. For possible culvert locations, the designer should identify all live stream crossings; springs; low areas; gullies; and impoundment areas created by new embankments and/or excavations. 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 when velocities are smaller than the design flow. The deposits may be removed during larger storms when it has the relative transport capacity of flow in the stream and in the culvert. Removal will also be dependent on the compaction and composition of the deposits; flow duration; pond depth above the culvert; and other factors.

Culvert location in both plan and profile is of particular importance to the maintenance of sediment-free culverts. Deposits most often occur in culverts when the sediment transport capacity of the flow in 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

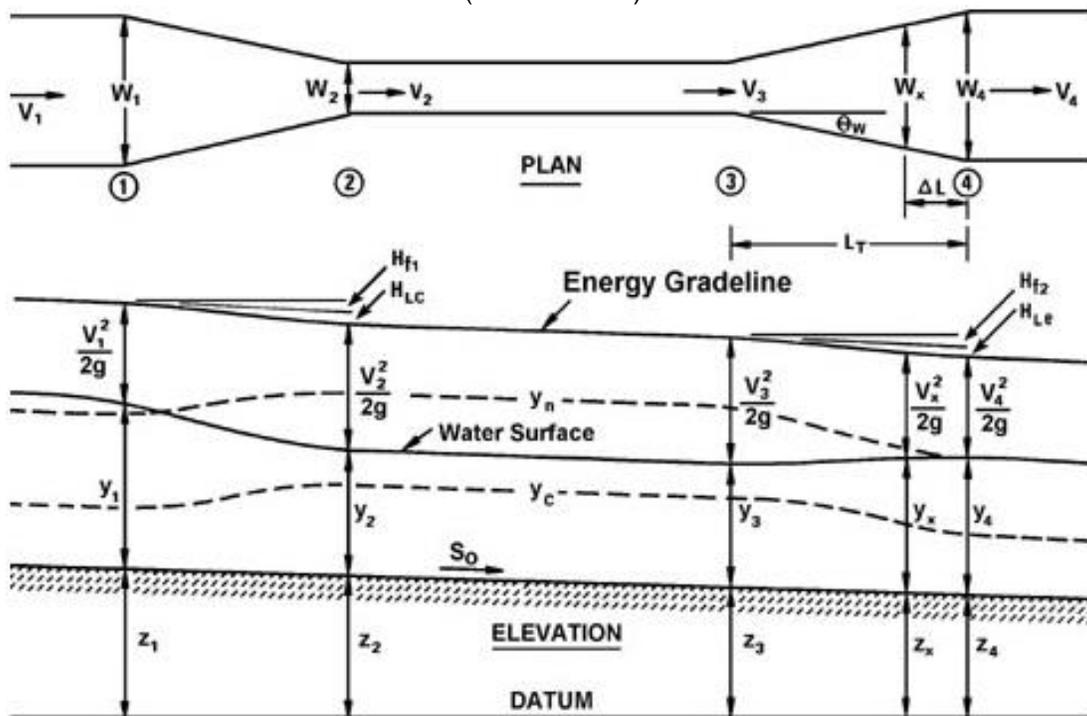
Concepts of conservation of energy and open channel hydraulics must be included in the design when transitioning from pipe flow to open channels; between different rigid channels; and from slow flow to supercritical flow. Primarily, a transition is necessary to change the shape or cross-section of flowing water.

Normally, the designer will have as an objective the avoidance of excessive energy losses; cross waves; and turbulence. It is also necessary to protect 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 12.5 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](#) (2006).

Figure CB-14: 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. 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 over 30-inches and less than 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; extending 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 (1996).

7.7 Cover, Fill Heights and Bedding for Culverts

Pipe cover shall be in accordance with Section 3.3.2 of this manual. Minimum cover less than these values shall be fully justified in writing and approved by the City prior to approval of plans. Maximum fill heights and bedding descriptions for pipes are shown on [Tables CB-4](#) and [CB-5](#).

Box culverts shall be structurally designed to accommodate earth and live loads 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-4: Maximum Heights of Fill over RCP Culverts
(AHTD Standard Drawing PCC-1)

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

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

Table CB-5: Pipe Bedding Installation Types
(AHTD 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 AHTD Standard Specifications for Highway Construction, Latest Edition. Materials shall not include organic materials or stones larger than 3-inches.

8.0 BRIDGES INTRODUCTION AND OVERVIEW

Bridges are important and expensive 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, the sizing of the bridge opening(s) is of paramount importance. Open channels with improperly designed bridges will either have excessive scour; deposition; or not be able to carry the design flow.

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 while being consistent with good bridge design and economics. However, with respect to supercritical flow in a lined channel, the bridge shall not affect the flow at all. That is, there shall be no projections in the design water prism that could create a hydraulic jump or flow instability that results in reflecting and/or 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/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 a local, collector, arterial, critical service bridge is 12-inches for a 100-year. In no case shall any local/collector bridge be overtopped in the 100-year event - no matter the allowable freeboard. Any AHTD requirements for freeboard shall be adhered to on all state and interstate highways. Refer to AHTD'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-15](#).

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-15](#). 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.

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

$$h_1^* = K^* \left(\frac{V_{n2}^2}{2g} \right) + \alpha 1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad \text{(Equation CB-15)}$$

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-15](#), it is necessary to obtain the approximate value of h_1^* by using the first part of the equation:

$$h_1^* = K^* \left(\frac{V_{n2}^2}{2g} \right) \quad \text{(Equation CB-16)}$$

The value of A_1 in the second part of [Equation CB-15](#), 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-16](#) 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.

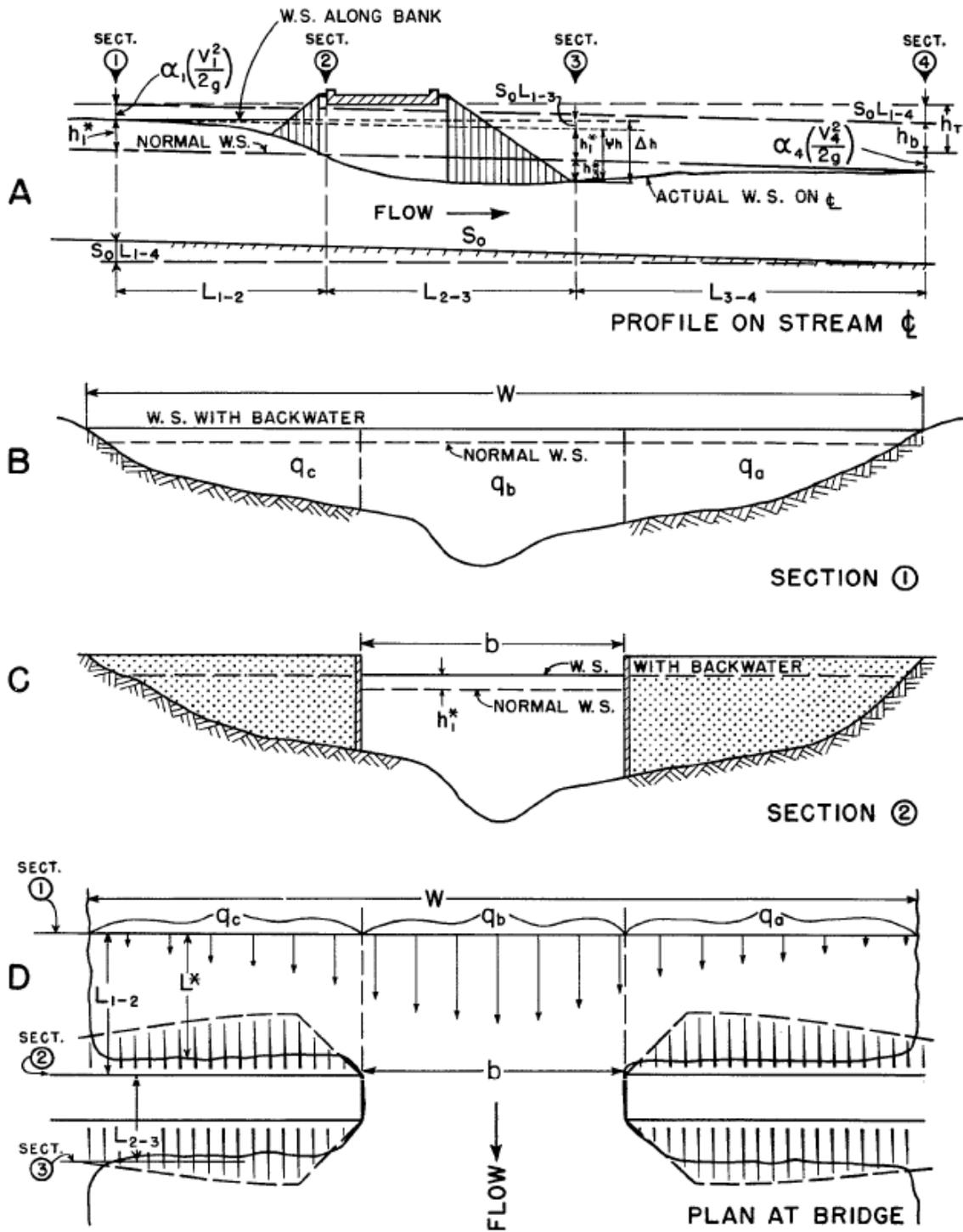
To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$, where M = bridge opening ratio = b/W ([Figure CB-15](#))

$V_{n2} < 7$ ft/sec

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

Figure CB-15: Normal Bridge Crossing Designation
(FHWA HDS-1)



If values meet all three conditions, the backwater obtained from [Equation CB-16](#) 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-15](#) in its entirety. The use of the guides is further demonstrated in the examples given in FHWA (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-16](#) 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-17](#) 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 CD-16](#) and [CB-17](#) 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.

Figure CB-16: Base Curves for Wingwall Abutments
 (Source: UDFCD USDCM 2001)

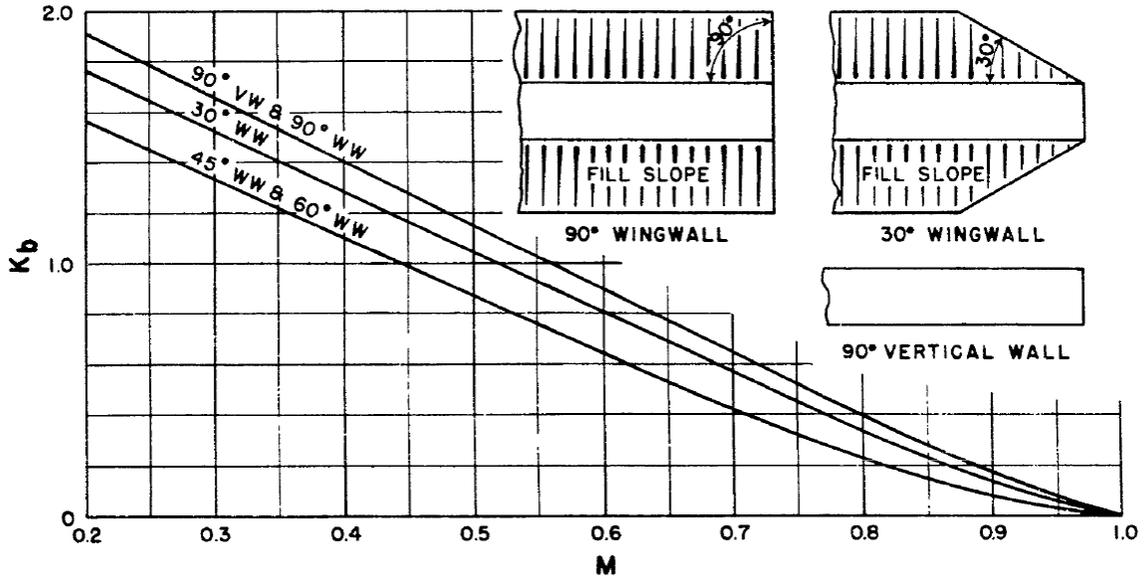
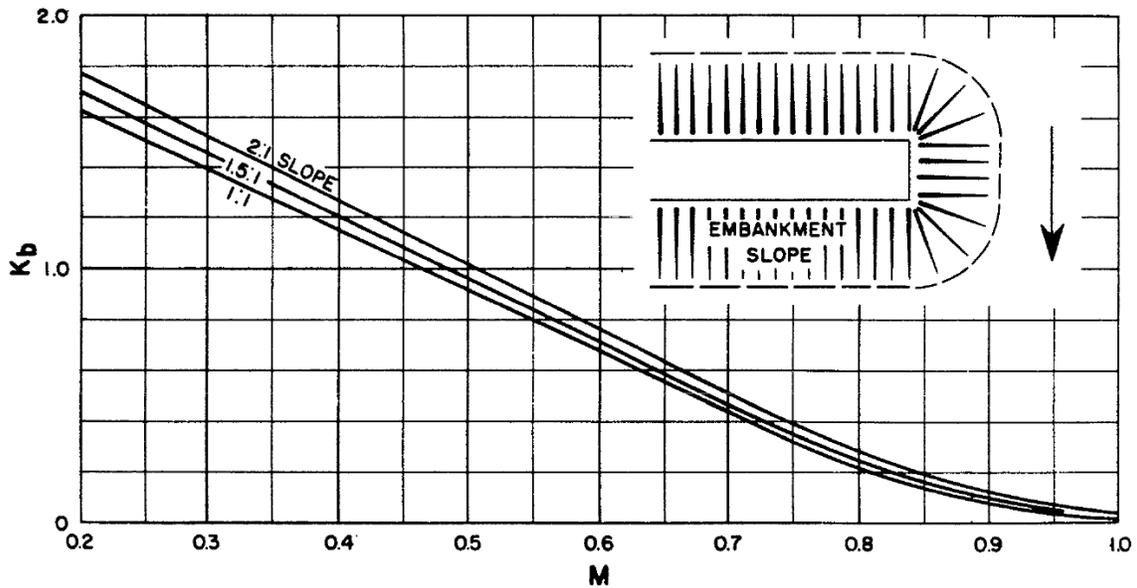


Figure CB-17: Base Curves for Spill-through Abutments
 (Source: UDFCD USDCM 2001)



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-18](#). The procedure is to enter Chart A, [Figure CB-18](#), with the proper value of J and read ΔK and obtain the correction factor σ from Chart B, [Figure CB-18](#), for opening ratios other than unity. The incremental backwater coefficient is then

$$\Delta K_p = \Delta K \sigma \quad \text{(Equation CB-17)}$$

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:

$$K^* = K_b \text{ (Figures CB-16 or CB-17) } + \Delta K_p \text{ (Figure CB-18)} \quad \text{(Equation CB-18)}$$

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 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 representative cross section of 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 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 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-16](#).
15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in FHWA's Hydraulic Design Series No. 1, Hydraulics of Bridge Waterways (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:

$$Q_m = C_b A_b \sqrt{2gH_{br}} \quad \text{(Equation CB-14)}$$

or

$$H_{br} = 0.04 \left(\frac{Q_m}{A_b} \right)^2 \quad \text{(Equation CB-15)}$$

in which:

Q_m = Major storm discharge (ft³/s)

C_b = Bridge opening coefficient (0.6 assumed in Equation CB-14)

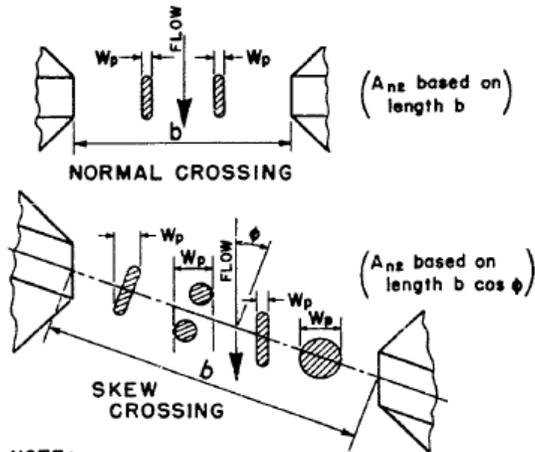
A_b = Area of the bridge opening (ft²)

g = Acceleration of gravity (32.2 ft/s²)

H_{br} = Vertical distance from the bridge opening center point to the upstream water surface at a distance about 10 times H 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.

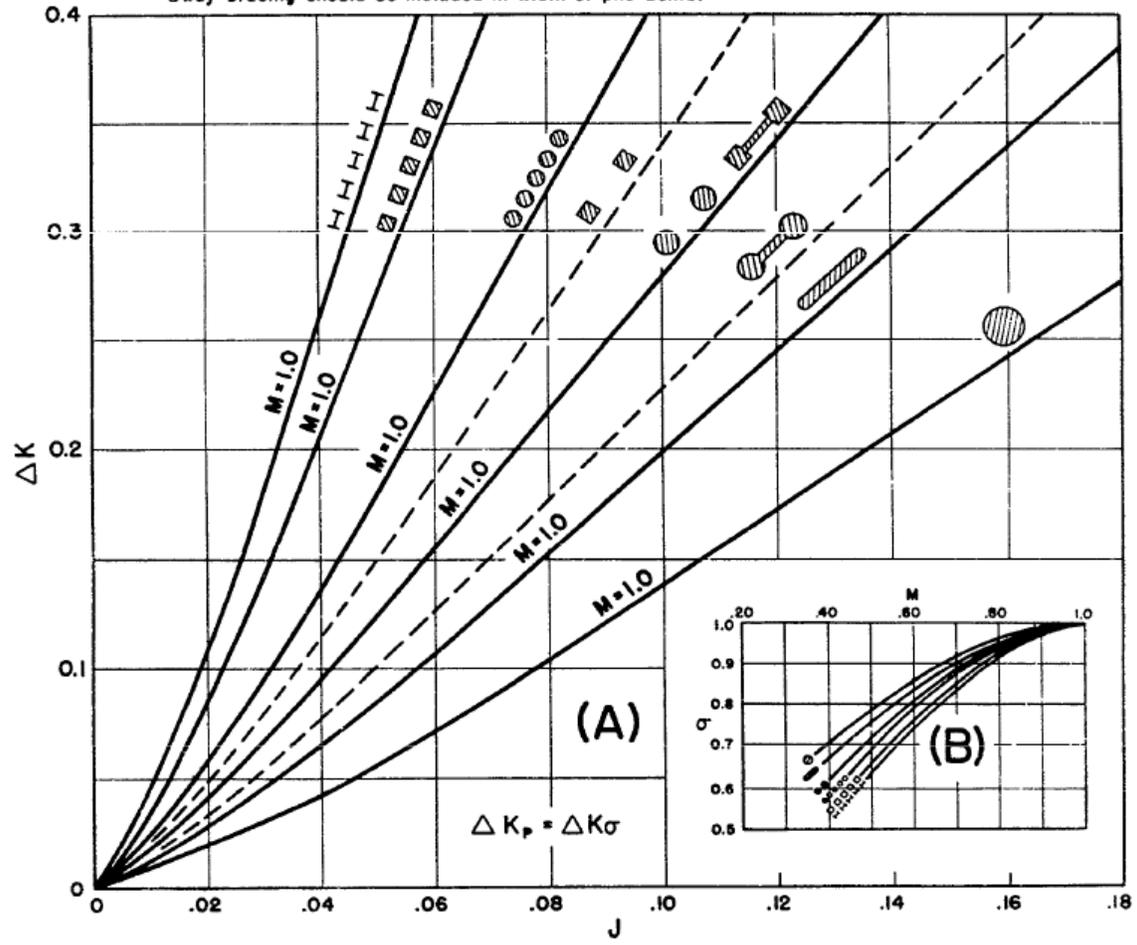
These expressions are valid when the water surface is above the top of the bridge opening.

Figure CB-18: Incremental Backwater Coefficient for Pier
(FHWA HDS-1)



- 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.



9.0 REFERENCES

- American Association of State Highway and Transportation Officials (AASHTO), Highway Subcommittee on Bridges and Structures. 2007. LRFD Bridge Design Specifications, 4th Ed. Washington, DC: AASHTO
- American Association of State Highway and Transportation Officials (AASHTO) Task Force for Roadside Safety, Highway Subcommittee on Design. 1996. *Roadside Design Guide*. Washington, DC: AASHTO.
- . 1987. *Hydraulic Analysis. Location and Design of Bridges*. Washington, DC: AASHTO.
- American Concrete Pipe Association. 2000. Concrete Pipe Design Manual. Irving, TX: American Concrete Pipe Association.
- Arkansas State Highway and Transportation Department (AHTD). 1982. Drainage Manual. Little Rock, AR: AHTD
- . 2000. Standard Drawing PCC-1, *Concrete Pipe Culvert Fill Heights and Bedding*. Little Rock, AR: Arkansas State Highway and Transportation Department.
- Chow, V.T. 1959. Open Channel Hydraulics. New York: McGraw-Hill Book Company, Inc.
- Ginsberg, A. 1987. HY8 Culvert Analysis Microcomputer Program Applications Guide. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- King, H.W. and E.F. Brater 1976. Handbook of Hydraulics. New York: McGraw-Hill Book Company.
- New Jersey Department of Transportation (NJDOT). 2008. *Soil Erosion and Sediment Control Standards*. Trenton, New Jersey: NJDOT
- Rhode Island Department of Environmental Management. 1989. *Soil Erosion and Sediment Control Handbook*. Providence, RI: RI DEM
- Richardson, E.V. 1974. *Highways in the River Environment*. Washington, DC: U. S. Department of Transportation, Federal Highway Administration.
- Scourstop.com. 2008. ScourStop. 1 Sep. 2009 <<http://scourstop.com/index.php>>.
- Urban Drainage and Flood Control District. 2001. Urban Storm Drainage Criteria Manual. Denver, CO: Urban Drainage and Flood Control District.
- U.S. Army Corp of Engineers (USACE). 1952. *Hydrologic and Hydraulic Analysis, Computation of Backwater Curves in River Channels*. Engineering Manual. Washington, DC: Department of the Army.
- U.S. Bureau of Reclamation. 1960. Design of Small Dams. Denver, CO: Bureau of Reclamation.

- U.S. Federal Highway Administration (FHWA). 1960. *Hydraulics of Bridge Waterways*. Hydraulic Design Series No.1. Washington, DC: FHWA.
- . 1965. *Hydraulic Charts for the Selection of Highway Culverts*. Hydraulic Engineering Circular No. 5. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- . 1971. *Debris Control Structures*. Hydraulic Engineering Circular No. 9. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- . 1972b. *Hydraulic Design of Improved Inlets for Culverts*. Hydraulic Engineering Circular No. 13. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- . 1978. *Hydraulics of Bridge Waterways*. Hydraulic Design Series No. 1. Washington, DC: Department of Transportation.
- . 1983. *Hydraulic Design of Energy Dissipators for Culverts and Channels*. Hydraulic Engineering Circular No.14. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- . 1985. *Hydraulic Design of Highway Culverts*. Hydraulic Design Series No. 5. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- . 1988. *Highways in the River Environment*. Washington, DC: U. S. Department of Transportation, Federal Highway Administration.
- . 1996. *Federal Lands Highway Project Development and Design Manual*. 1996 Metric Revision. FHWA-DF-88-003. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- . 2000. *Hydraulic Design of Energy Dissipators for Culverts and Channels*. Hydraulic Engineering Circular No. 14M. Washington, DC: U.S. Department of Transportation, Federal Highway Administration.
- Virginia Department of Transportation (VDOT). 2002. *Drainage Manual*. Richmond, Virginia: VDOT