CHAPTER 4: CULVERT

4. CULVERT

4.1 GENERAL

4.1.1 Cross Drain Design

Section 4.2 of the *Drainage Manual* states, "All cross drains shall be designed to have sufficient hydraulic capacity to convey the selected design frequency flood without damage to the structure and approach embankments, with due consideration to the effects of greater floods." This requires evaluation of the following:

Backwater
Refer to Section 4.3 of this design guide and Section 4.4 of the *Drainage Manual.*

Tailwater
Refer to Section 4.4 of this design guide and Section 4.5 of the *Drainage Manual*.

Scour
Refer to Sections 4.1.2 and 4.6.2 of this design guide and Section 4.9.2 of the *Drainage Manual*.

You may need to perform a risk analysis to evaluate damage to structures and/or embankments caused by backwater and/or scour. Refer to Appendix G, Risk Evaluations.

4.1.2 Scour Estimate

When producing scour estimates for bridge culvert foundation designs, it is best not to use the methods in FHWA'S HEC-18. Instead, consider the outlet velocity and degradation of the stream, discussed in Section 4.6.2 of this document.

To use bridge culverts with no bottom slab and toe wall, you need to get the following approval/evaluation:

a) Prior approval from the District Drainage Engineer.

b) An analysis of the degradation that could take place through the bridge culvert. This would require you to recommend the toe wall depths of the bridge culvert and the need for scour protection for the design-year frequency, 100-year frequency, and 500-year frequency.

4.1.3 Flood Definition

Design Flood

The “design flood” is defined as the flood or storm surge associated with the probability of exceedance (frequency) selected for the design of a highway encroachment. This frequency, known also as the "design-year frequency," is discussed in Section 4.2.

Base Flood

The “base flood” (100-year frequency flood event) is defined as the flood or storm surge having a 1-percent chance of being exceeded in any given year. The base flood is the standard in Federal Emergency Management Agency (FEMA) flood insurance studies and many agencies have adopted it to comply with regulatory requirements.

Greatest Flood

The “greatest flood” (500-year frequency flood event) is defined as the flood or storm surge having a 0.2-percent chance of being exceeded in any given year. This event is used to define the possible consequences of a flood occurrence significantly greater than the 1-percent flood event. While it is seldom possible to compute the discharge for the 500-year frequency flood with the same accuracy that you would compute the discharge for the base flood, it serves to draw attention to the fact that floods greater than the base flood can occur. In some cases, FEMA and other agencies compute the 500-year frequency flood.

Overtopping Flood

The “overtopping flood” is described by the probability of exceedance and water surface elevation at which water begins to flow over the highway, a watershed divide, or through structure(s) providing for emergency relief.

The overtopping flood is of particular interest because it will indicate one of the following:

1. When a highway will be inundated

2. The limit (stage) at which the highway, ditch, or some other control point will act as a significant flood relief for the structure of interest

Carefully compare roadside ditch elevations with respect to the water surface elevation for the structure being designed or analyzed. There may be instances where the ditch elevation will provide significant relief to the structure for a certain flood. This ditch elevation will define the overtopping flood stage.

Example 4.1-1 shows how the overtopping flood is determined.

Example 4.1-1—Computing the Overtopping Flood

Given the information below, determine the discharge and frequency for the overtopping flood.

Q (25)	= 31 ft^3/sec	Stage (25)	= 134.3 ft.
Q (100)	= 55 ft^3/sec	Stage (100)	= 139.0 ft.
Q (Overtopping)	= ?	Stage (Overtopping)	= 140.9 ft.

Solution

Step 1:

a. To determine the overtopping discharge, plot stage versus discharge on algebraic scale graph paper for the 25-year and 100-year floods, as shown on Figure 4.1-1.

Note: Graphical estimation methods are explained in *Hydraulic Design Series No. 2 (HDS-2)*, Publication No. FHWA-NHI-02-001, October 2002.

b. Draw the best-fit line through these points.

c. Knowing what the overtopping stage is, you can conservatively approximate the overtopping discharge. The overtopping discharge was found to be 64 ft^3/sec.

Note: For stages above overtopping, the overtopping flow can provide significant relief. The stage versus discharge relationship usually flattens out after overtopping.

Step 2:

a. To determine the overtopping frequency, plot frequency versus discharge on log-normal probability paper for the 25-year and 100-year floods, as shown in Figure 4.1-2.

b. Draw the best-fit line through these points.

c. Knowing the overtopping discharge from Step 1c, you can determine the probability of the overtopping flood being exceeded in any year. In this case, the probability is 0.65 percent. This corresponds to a frequency of 154 years (i.e., 100/0.65).

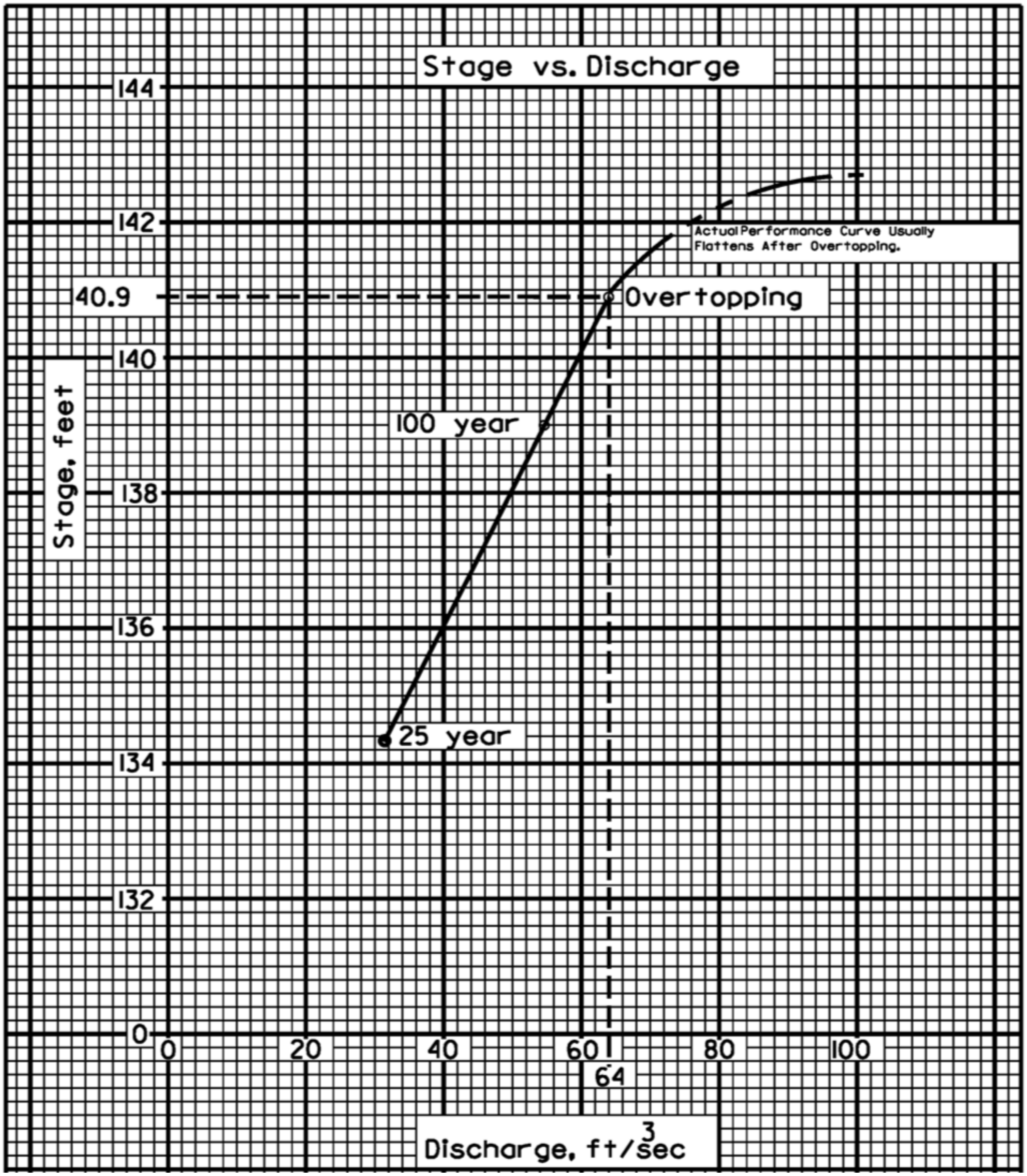


Figure 4.1-1: Example I - Computing Overtopping Flood (cont.)

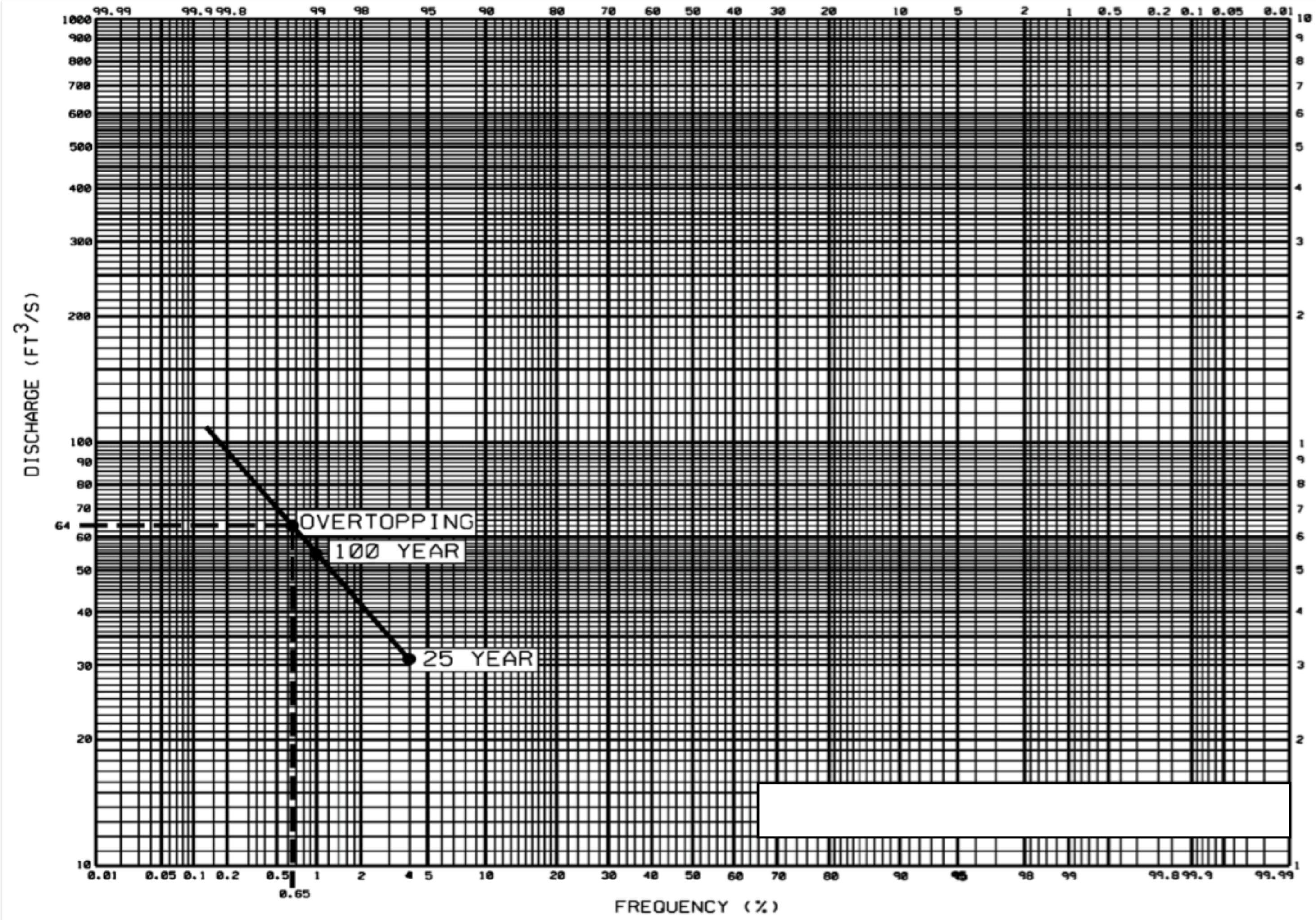


Figure 4.1-2: Example I - Computing Overtopping Flood (cont.)

Flood Data Summary Box

For culverts other than bridge culverts, include hydraulic data in a Flood Data Summary Box similar to the example shown in Figure 4.1-3. Include these data for those conditions discussed in *FDM 305*.

STRUCTURE NO.	STATION	DESIGN FLOOD		BASE FLOOD		OVERTOPPING FLOOD				GREATEST FLOOD			
		___% PROB. ____ YR FREQ		___% PROB. ____ YR FREQ									
		DISCHARGE	STAGE	DISCHARGE	STAGE	DISCHARGE	STAGE	PROB %	FREQ. YR	DISCHARGE	STAGE	PROB %	FREQ YR

Note: The hydraulic data are shown for informational purposes only, to indicate the flood discharges and water surface elevations that may be anticipated in any given year. These data were generated using highly variable factors determined by a study of the watershed. Many judgments and assumptions are required to establish these factors. The resultant hydraulic data are sensitive to changes, particularly of antecedent conditions, urbanization, channelization, and land use. Users of these data are cautioned against the assumption of precision, which cannot be attained. Discharges are in cubic feet per second and stages are in feet.

Definitions:

Design Flood	The flood selected by FDOT to be utilized to assure a standard level of hydraulic performance
Base Flood:	The flood having a 1-percent chance of being exceeded in any given year (100-year frequency)
Overtopping Flood:	The flood that causes water to flow over the highway, over a watershed divide, or through emergency relief structures
Greatest Flood:	The most severe flood that can be predicted, where overtopping is not practicable; normally, one with a 0.2-percent chance of being exceeded in any given year (500-year frequency)

Figure 4.1-3: Flood Data Summary Box

Fill out the hydraulic flood data sheet according to the Federal Aid Policy Guide (23 CFR 650A). You can find this policy guide at http://www.fhwa.dot.gov/legsregs/directives/fapg/cfr0650a.htm. In general, the following applies.

a. If the overtopping flood is less than the standard design frequency, perform a risk assessment to define the design flood as the overtopping flood. Fill out the information for the design flood, base flood, and overtopping flood.

b. If the overtopping flood is between the standard design frequency and the base flood (100-year flood), then fill out the information for the design flood, base flood, and overtopping flood.

c. If the overtopping flood is between the base flood (100-year flood) and the greatest flood (500-year flood), then fill out the information for the design flood, base flood, and overtopping flood.

d. If the overtopping flood is larger than the greatest flood (500-year flood), then fill out the information for the design flood, base flood, and greatest flood.

Example 4.1-2 shows you how to complete the Flood Data Summary Box when the overtopping flood is less than the greatest flood (500-year flood).

Example 4.1-3 shows you how to complete the Flood Data Summary Box when the overtopping flood occurs at a 10-year frequency.

Example 4.1-2—Completing the Flood Data Summary Box

Referring back to Example 4.1-1, assume the design flood is the 25-year frequency. Fill out the Flood Data Summary Box.

Solution
Since the overtopping flood is between the base flood (100-year flood) and the greatest flood (500-year flood), then fill out the information for the design flood, base flood, and overtopping flood.

Q (25) = 31 ft^3/sec
Stage (25) = 134.3 ft.

Q (100) = 44 ft^3/sec
Stage (100) = 136.4 ft.

Q (Overtopping) = 64 ft^3/sec
Stage (Overtopping) = 140.9 ft.

Put these values in the corresponding column, as shown in Figure 4.1-4. From Example 4.1-1, the overtopping flood was found to have a 0.65 percent chance of being exceeded in any year, or a frequency of 154 years.

STRUCTURE NO.	STATION	DESIGN FLOOD 4% PROB. 25-YR FREQ		BASE FLOOD 1% PROB. 100-YR FREQ		OVERTOPPING FLOOD				GREATEST FLOOD			
		DISCHARGE	STAGE	DISCHARGE	STAGE	DISCHARGE	STAGE	PROB %	FREQ. YR	DISCHARGE	STAGE	PROB %	FREQ YR
S-1	30+50	31	134.3	44	136.4	64	140.9	0.65	154				

Note: The hydraulic data are shown for informational purposes only, to indicate the flood discharges and water surface elevations that may be anticipated in any given year. These data were generated using highly variable factors determined by a study of the watershed. Many judgments and assumptions are required to establish these factors. The resultant hydraulic data are sensitive to changes, particularly of antecedent conditions, urbanization, channelization, and land use. Users of these data are cautioned against the assumption of precision, which cannot be attained. Discharges are in cubic feet per second and stages are in feet.

Definitions:

Design Flood:	The flood selected by FDOT to be utilized to assure a standard level of hydraulic performance
Base Flood:	The flood having a 1-percent chance of being exceeded in any given year (100-year frequency)
Overtopping Flood:	The flood that causes water to flow over the highway, over a watershed divide, or through emergency relief structures
Greatest Flood:	The most severe flood that can be predicted, where overtopping is not practicable; normally, one with a 0.2-percent chance of being exceeded in any given year (500-year frequency)

Figure 4.1-4: Flood Data Summary Box

Example 4.1-3—Completing the Hydraulic Flood Data Sheet

Given the information below, fill out the Hydraulic Flood Data Sheet.

The standard frequency for Structure 1 is 50 years, based on the criteria from Section 4.3 of the *Drainage Manual*.

The structure overtops during a 10-year frequency flood.

Perform a risk assessment to define the design flood as the overtopping flood.

Q (Overtopping) = 20 ft^3/sec
Stage (Overtopping) = 45 ft.

Q (100) = 37 ft^3/sec
Stage (100) = 50.5 ft.

Solution
Since the overtopping flood is less than the standard design frequency and you performed a risk assessment to define the design flood as the overtopping flood, fill out the information for the design (overtopping) flood, base flood, and overtopping flood. Put these values in the corresponding columns, as shown in Figure 4.1-5.

Q (Overtopping) = 20 ft^3/sec
Stage (Overtopping) = 45 ft.

Q (100) = 37 ft^3/sec
Stage (100) = 50.5 ft.

Example 4.1-3—Completing the Flood Data Summary Box

STRUCTURE NO.	STATION	DESIGN FLOOD 10% PROB. 10-YR FREQ		BASE FLOOD 1% PROB. 100-YR FREQ		OVERTOPPING FLOOD				GREATEST FLOOD			
		DISCHARGE	STAGE	DISCHARGE	STAGE	DISCHARGE	STAGE	PROB %	FREQ. YR	DISCHARGE	STAGE	PROB %	FREQ YR
S-1	30+50	20	45	37	50.5	20	45	10	10				

Note: The hydraulic data are shown for informational purposes only, to indicate the flood discharges and water surface elevations that may be anticipated in any given year. These data were generated using highly variable factors determined by a study of the watershed. Many judgments and assumptions are required to establish these factors. The resultant hydraulic data are sensitive to changes, particularly of antecedent conditions, urbanization, channelization, and land use. Users of these data are cautioned against the assumption of precision, which cannot be attained. Discharges are in cubic feet per second and stages are in feet.

Definitions:

Design Flood:	The flood selected by FDOT to be utilized to assure a standard level of hydraulic performance
Base Flood:	The flood having a 1-percent chance of being exceeded in any given year (100-year frequency)
Overtopping Flood:	The flood that causes water to flow over the highway, over a watershed divide, or through emergency relief structures
Greatest Flood:	The most severe flood that can be predicted, where overtopping is not practicable; normally, one with a 0.2-percent chance of being exceeded in any given year (500-year frequency)

Figure 4.1-5: Flood Data Summary Box

4.2 DESIGN FREQUENCY

“Design frequency” means a frequency that accommodates an adopted design criterion. After you determine the design frequency, you can then determine a discharge for the selected frequency. This discharge is known as the "design discharge." By definition, the design discharge does not overtop the road. After you determine the design discharge, you can determine a headwater. This headwater also is known as the "design discharge headwater.” The design discharge headwater may be at an elevation lower than the road's profile grade to meet other design criteria, such as protection of property, accommodating land use needs, lowering velocities, reducing scour, or complying with regulatory mandates.

To provide an acceptable standard level of service against flooding, the Department typically employs widely used pre-established design frequencies, which are based on the importance of the transportation facility to the system and allowable risk for that facility. Selecting the appropriate design storm from these standards is a matter of professional judgment since it is rarely either possible or practical to provide for the greatest possible flood. The design flood frequency standards for cross drains listed in Section 4.3 of the *Drainage Manual* provide an engineering consensus on reasonable values. The actual design must consider the consequences of greater events, such as the 100-year flood for culverts and bridges and even the 500-year flood for bridges.

Under certain conditions, it may be appropriate to establish a level of risk allowable for a site and to design to that level. When the risks associated with a particular project are significant for floods of greater magnitude than the standard design flood, evaluate a greater return interval design flood by using a risk analysis. Risk analysis procedures are provided in FHWA’s HEC 17 and discussed briefly in Appendix G, Risk Evaluations. In addition, consider incorporating or addressing design standards of other agencies that have control or jurisdiction over the waterway or facility of concern in the design.

4.3 BACKWATER

Backwater is defined as the increase of water surface elevation induced upstream from a bridge, culvert, dike, dam, another stream at a higher stage, or other similar structures; or conditions that obstruct or constrict a channel relative to the elevation occurring under natural channel and floodplain conditions.

4.3.1 Backwater Consistent with the Flood Insurance Study Requirements

Backwater Effects on Land Use

Backwater effects are important to consider in the design/analysis of cross drains in rural and urban areas.

In rural areas, the concern centers on increased flood stages. The degree and duration of an increased flood stage could affect present and future land uses. You certainly must evaluate agricultural land use for increased risks due to flooding. As an example, inundation may impact crops or livestock.

In urban areas, the effects of increased flood stages or increased velocities become an important consideration. In addition to the impact on future land use, the existing property may suffer extensive physical damage. Many urban areas have stream or watershed management regulations or are part of the National Flood Insurance Program (NFIP). These regulations may dictate limits on changes that can be made to flow characteristics of a watershed.

You may need to perform a risk evaluation to determine damage to surrounding property. Refer to Appendix G, Risk Evaluations.

Obtaining Flood Rights

The Department does not encourage obtaining flood rights; however, it is recognized that, in some instances, it may be necessary. Evaluate all possible alternatives before recommending that the Department obtains flood rights.

Alternatives to obtaining flood rights for upstream flooding include:

- Prior approval from the property owner
- Purchase of the property
- Upsizing the structure as long as there is no increased flooding to the downstream owner

Consider performing a risk analysis for situations where you are evaluating acquiring flood rights. Appendix G briefly discusses risk analysis, whereas the topic is extensively covered in HEC 17 (USDOT, FHWA, 2016).

Further discussion about obtaining flood rights is included in Appendix B of the *Drainage Manual.*

4.4 TAILWATER

Section 4.5 of the *Drainage Manual* states: "For the sizing of cross drains and the determination of headwater and backwater elevations, use the highest tailwater elevation that can reasonably be expected to occur coincident with the design storm event."

Additional guidelines for tailwater elevations are provided in Section 4.5. For cross drains subject to tidal conditions, include in the tailwater determination a sea-level rise analysis, as described in Section 3.4.1 of the *Drainage Manual*.

4.5 HYDRAULIC ANALYSIS

During a storm event, a culvert may operate under inlet control, outlet control, or both. Different variables and equations determine the culvert capacity for each type of control. For more detailed information on theory, refer to Federal Highway Administration Hydraulic Design Series No. 5 (HDS-5), *Hydraulic Design of Highway Culverts*. You can find the publication on FHWA's website at:
http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7&id=13.

Guidelines that pertain to the hydraulic analysis of bridge culverts and other culverts are presented below.

- **Allowable Headwater**

  You can determine the allowable headwater elevation by evaluating land use upstream of the culvert and the proposed or existing roadway elevation. The criteria in Section 4.4 of the *Drainage Manual* apply, but other factors that may limit the allowable headwater are:

  - Identify non-damaging or permissible upstream flooding elevations (e.g., existing buildings or flood insurance regulations). Keep headwater below these elevations.
  - Identify state regulatory constraints (e.g., Water Management District).
  - Address other site-specific design considerations, as required.

In general, the constraint that gives the lowest allowable headwater elevation should establish the basis for hydraulic calculations.

- **Inlet Control**

Nomographs

FHWA has developed inlet nomographs, shown in FHWA HDS-5, to provide graphical solutions to headwater equations for various culvert materials, cross sections, and inlet combinations. Because of the low velocities in most entrance pools and the difficulty in determining the velocity head for all flows, ignore the approach velocity and assume the water surface and energy line at the entrance are coincident. The headwater depths obtained by using the nomographs can be higher than will occur in some instances because of this factor.

You can determine the headwater elevation for inlet control by taking the culvert invert elevation at the entrance and adding the headwater depth.

- **Outlet Control**
  Nomographs

Outlet control nomographs have been developed and are shown in FHWA HDS-5 to provide graphical solutions to the head loss equations for various culvert materials, cross sections, and inlet combinations.

Culvert Entrance Loss Coefficients

Appendix F, *Applications Guide for Pipe End Treatments* presents culvert entrance loss coefficients (k_e) for the end treatments. For other types of end treatments, refer to FHWA HDS-5.

Critical Depth

Use FHWA HDS-5 or other suitable methods to determine the critical depth for various sizes and types of culverts.

Equivalent Hydraulic Elevation

For culverts flowing partially full, the distance from the invert of the culvert outlet to the equivalent hydraulic grade line is termed the equivalent hydraulic elevation and is expressed as:

$$h_o = \frac{D + d_c}{2}$$ **(Equation 4.5-1)**

where:
h_o = Equivalent hydraulic elevation, in feet, for an unsubmerged outlet condition
D = Depth of the culvert, in feet
d_c = Critical depth at the culvert outlet, in feet

If the value for d_c from the figures of FHWA HDS-5 is greater than D, then h_o will equal D.

The equivalent hydraulic elevation is valid as long as the headwater is not less than 0.75D. For headwaters lower than 0.75D, perform backwater calculations to obtain headwater elevations.

Tailwater

Tailwater (TW) is the depth of water measured from the invert of the culvert at the outlet to the water surface elevation due to downstream conditions. Evaluate the hydraulic conditions downstream of the culvert site to determine a tailwater depth for the discharge and frequency under consideration. Determine tailwater as follows:

a. If an upstream culvert outlet is near the inlet of a downstream culvert, the headwater elevation of the downstream culvert may define the tailwater depth for the upstream culvert.

b. For culverts that discharge to an open channel, the tailwater may be equal to the normal depth of flow in that channel. Calculate normal depth using a trial-and-error solution of the Manning's equation. The known inputs are channel roughness, slope, and geometry.

   For bridge culverts that discharge to an open channel, you may have to determine the tailwater by performing a standard backwater calculation. Consider this analysis if the open channel does not have constant channel roughness, slope, and geometry or if there is a control structure downstream that could cause backwater.

c. If the culvert discharges to a lake, pond, or other major water body, the expected high-water elevation of the particular water body may establish the culvert tailwater. However, it is probably not appropriate to use a 25-year lake stage for a cross drain that uses a 25-year design frequency, due to the difference in time relationship between occurrences. Usually, the mean annual stage would be appropriate.

d. If tidal conditions occur at the outlet, the mean high water, as determined by sources such as the National Oceanic and Atmospheric Administration (NOAA), usually establishes the initial basis for tailwater conditions. Adjust the mean high water for sea level rise as described in Section 3.4.1 of the *Drainage Manual*.

Design Tailwater

The tailwater condition that prevails during the design event is called the design tailwater (DTW). The design tailwater may be a function of either downstream or culvert outlet conditions.

Two tailwater conditions can affect the selection of a design tailwater:

a. For the submerged outlet condition shown in Figure 4.5-1, TW is greater than h_o and, thus, TW becomes DTW.

b. For the unsubmerged outlet shown in Figure 4.5-2, TW is less than h_o, so the h_o elevation becomes DTW.

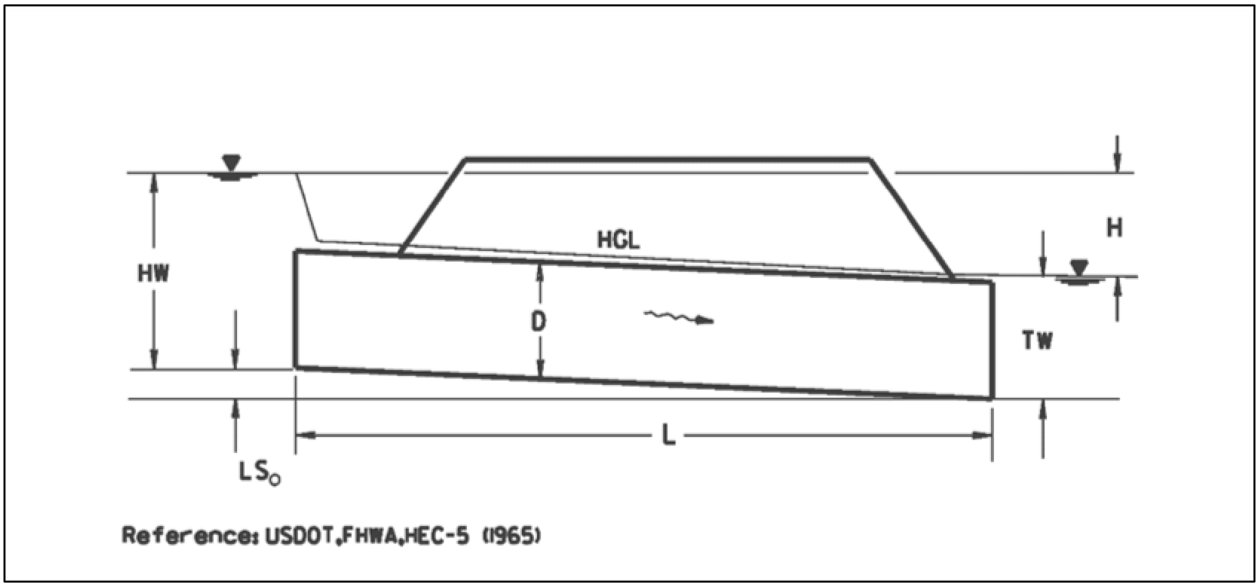


Figure 4.5-1: Tailwater for Submerged Outlet Conditions

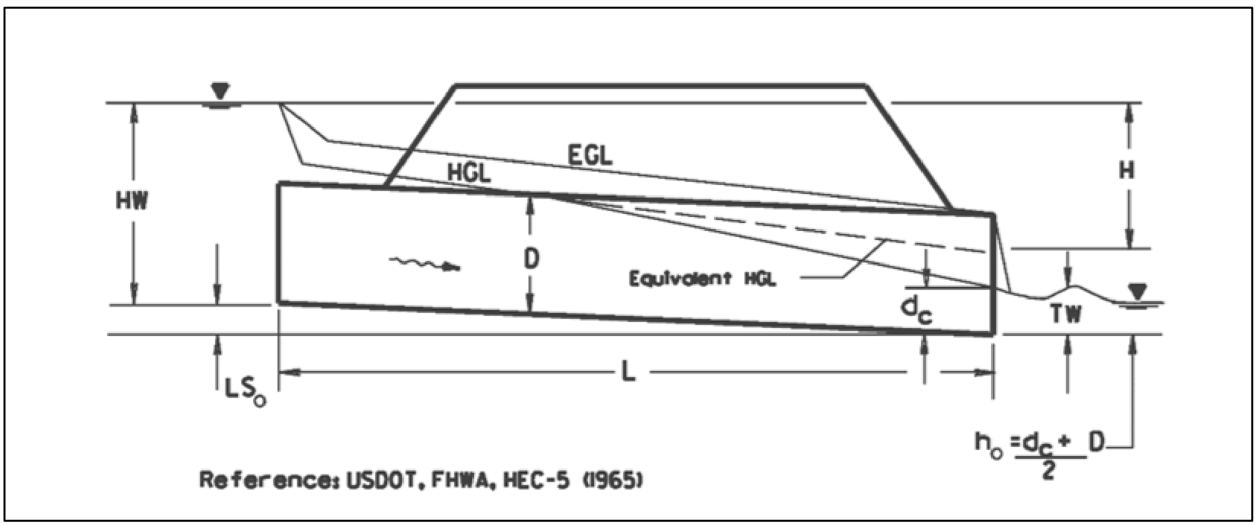


Figure 4.5-2: Tailwater for Unsubmerged Outlet Conditions

Headwater Depth
Having established the total head loss (H) and the design tailwater depth (DTW), compute the headwater depth (HW), as follows:

$$HW = H + DTW - LS_o \qquad \textbf{(Equation 4.5-2)}$$

where:

HW = Headwater depth for outlet control, in feet
H = Total head, in feet
DTW = Design tailwater depth, in feet
L = Length of culvert barrel, in feet
S_o = Barrel slope, in feet/feet

The difference in elevation between the culvert inlet and the culvert outlet is equal to LS_o. You may use it directly in Equation 4.5-2.

Determine the headwater elevation for outlet control by taking the culvert invert elevation at the entrance and adding the headwater depth.

- Controlling Headwater Depth or Elevation

  The controlling headwater depth or elevation is defined as the greatest headwater depth or elevation between the inlet and outlet control conditions.

- Outlet Velocity
  Inlet Control

  In inlet control, you may need to make backwater calculations to determine the outlet velocity. These calculations begin at the culvert entrance and proceed downstream to the exit. Obtain the flow velocity from the flow and the cross sectional area at the exit:

$$V = \frac{Q}{A} \qquad \textbf{(Equation 4.5-3)}$$

where:

V = Average velocity in the culvert, in feet per second
Q = Flow rate, in cubic feet per second
A = Cross sectional area of the flow, in square feet

To avoid backwater calculations in determining outlet velocity, you may use an approximation. Since the water surface profile converges toward normal depth as calculations proceed downstream, you can assume the normal depth and use it to define the area of flow at the outlet. Then you can use the normal depth obtained to determine the outlet velocity (see Figure 4.5-3). The velocity obtained may be higher than the actual velocity at the outlet.

Calculate normal depth using a trial-and-error solution of the Manning equation. The known inputs are barrel resistance, slope, and geometry. Then, determine the area of flow prism based on the culvert barrel geometry and depth equal to normal depth. You also can determine normal depth and area of flow using the charts for various pipe cross section shapes in Appendix E.

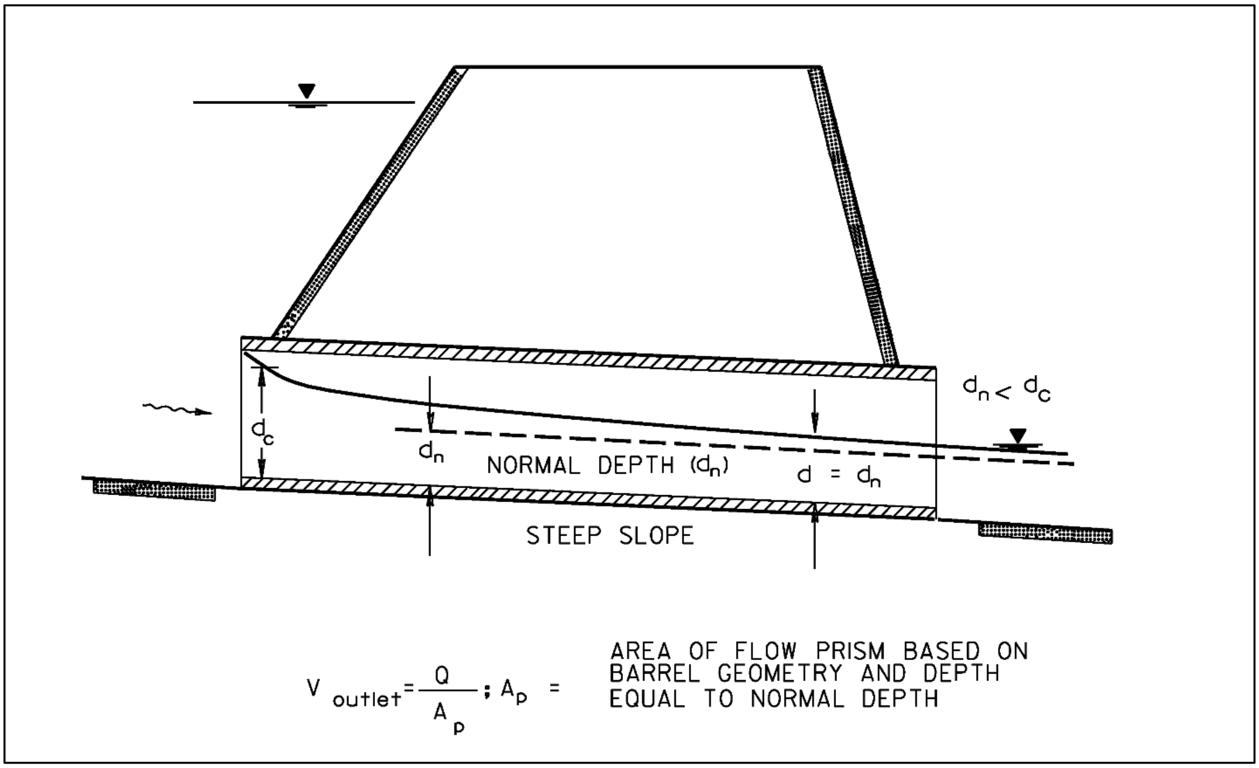


Figure 4.5-3: Outlet Velocity for Inlet Control

Example 4.5-1 illustrates computing outlet velocity for inlet control.

Example 4.5-1—Computing Outlet Velocity for Inlet Control

Given the information below, determine the outlet velocity for inlet control.

where:
Q_{design} = 18 ft^3/sec
Diameter of Pipe (D) = 24 in.
Slope of Pipe (S) = 0.01 ft./ft.
Roughness Coefficient (n) = 0.012

Solution

Step 1: Determine area, wetted perimeter, and hydraulic radius of the pipe flowing full.

$$Area\,(A) = \frac{(\pi D^2)}{4} = \frac{\pi \times (24\ inches\ /12)^2}{4} = 3.14\ ft^2$$

Wetted perimeter (WP) = πD = π * (24 in./12) = 6.28 ft.

Hydraulic radius (R) = A/WP = 3.14 ft^2/6.28 ft. = 0.5 ft.

Step 2: Using Manning's Equation, determine the discharge and velocity of the pipe flowing full.

$$Q_{Full} = \frac{1.49}{n} A\, R^{2/3}\, S^{1/2}$$

$$Q_{Full} = \frac{1.49}{0.012}(3.14\ ft^2)(0.5\ ft.)^{2/3}(0.01\ ft./ft.)^{1/2} = 24.56\ ft^3/s\ (say\ 25\ ft^3/s)$$

$$V_{Full} = Q_{Full} / A_{Full} = 25\ ft^3/s\ /\ 3.14\ ft^2 = 7.96\ ft/s\ (say\ 8.0\ ft/s)$$

Step 3: Using Figure 4.5-4, determine the area of flow for the design discharge using the following relationship:

$$\frac{Q_{Design}}{Q_{Full}} = \frac{18\ ft^3/s}{25\ ft^3/s}$$ = 0.72 or 72 % of value for section

1. Enter on Figure 4.5-4 the value of 0.72 on the horizontal axis.
2. Project vertically up until the flow curve is met.
3. Project horizontally from the flow curve to the area of the flow curve.
4. Project vertically down from the area of the flow curve and read from the horizontal axis a value of 0.66 or 66 percent of value for full section.
5. You can make a relationship between the full flow area and the normal depth area (A_{Design}):

$$\frac{A_{Design}}{A_{Full}} = 0.66\ ;\ A_{Design} = 0.66 \times 3.14\ ft^2 = 2.07\ ft^2$$

Step 4: Determine the outlet velocity using Q_{Design} and A_{Design}:

$$V_{Design} = \frac{Q_{Design}}{A_{Design}} = \frac{18\ ft^3/s}{2.07\ ft^2} = 8.70\ ft/s$$

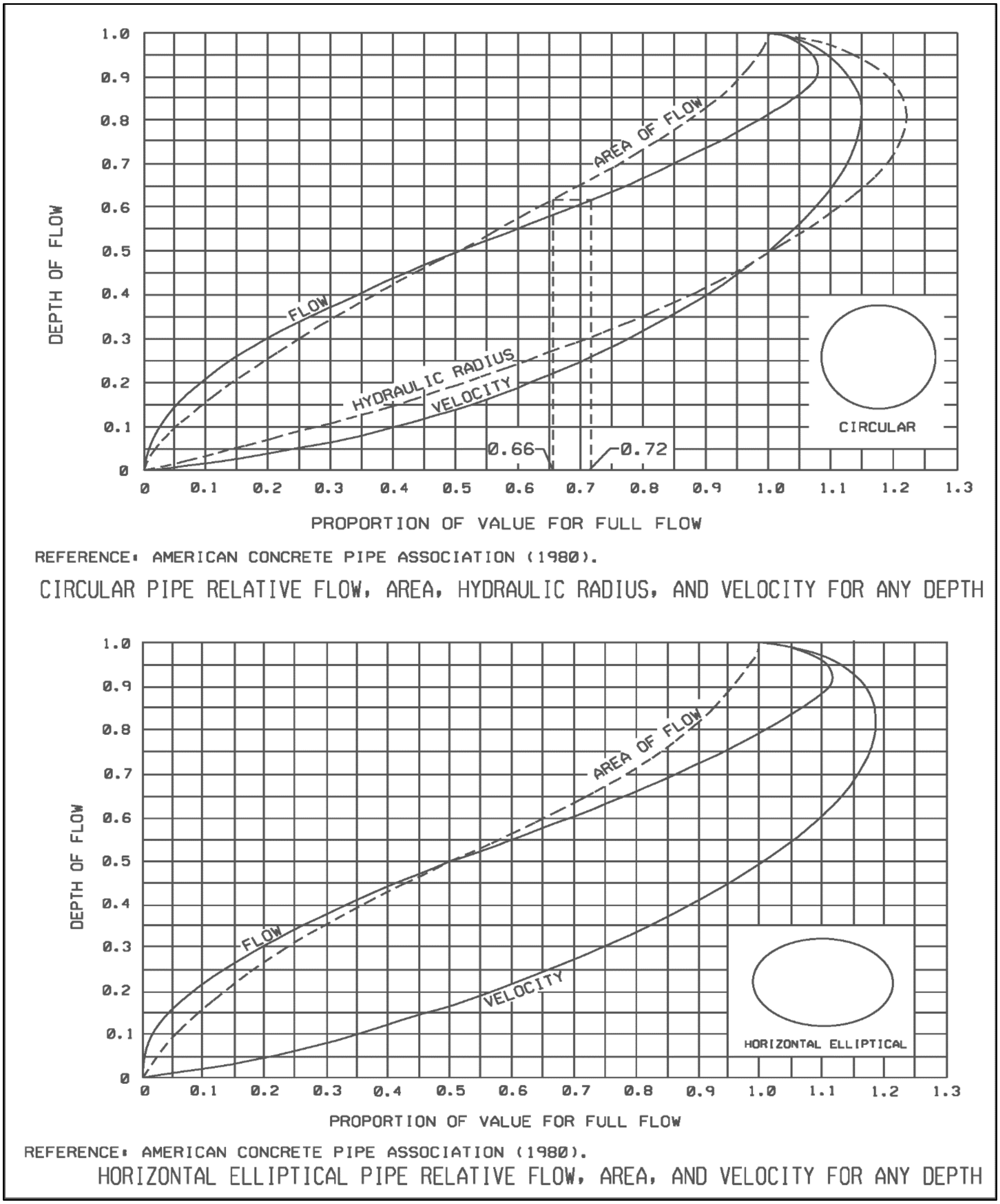


Figure 4.5-4: Example 4.5-1

Outlet Control:

In outlet control, the cross sectional area of the flow (Ap) is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the culvert (see Figure 4.5-5).

Use critical depth when the tailwater is less than critical depth; use the tailwater depth when tailwater is greater than critical depth but below the top of the barrel. The total barrel area is used when the tailwater exceeds the top of the barrel.

$$V = \frac{Q}{A_p}$$ **(Equation 4.5-4)**

where:

V = Average velocity in the culvert, in feet per second
Q = Flow rate, in cubic feet per second
Ap = Cross sectional area of the flow defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the culvert, in square feet

You can determine the area of flow prism based on barrel geometry and depth of flow (d) using the charts for various pipe cross section shapes in Appendix E.

Example 4.5-2 illustrates computing outlet velocity for outlet control.

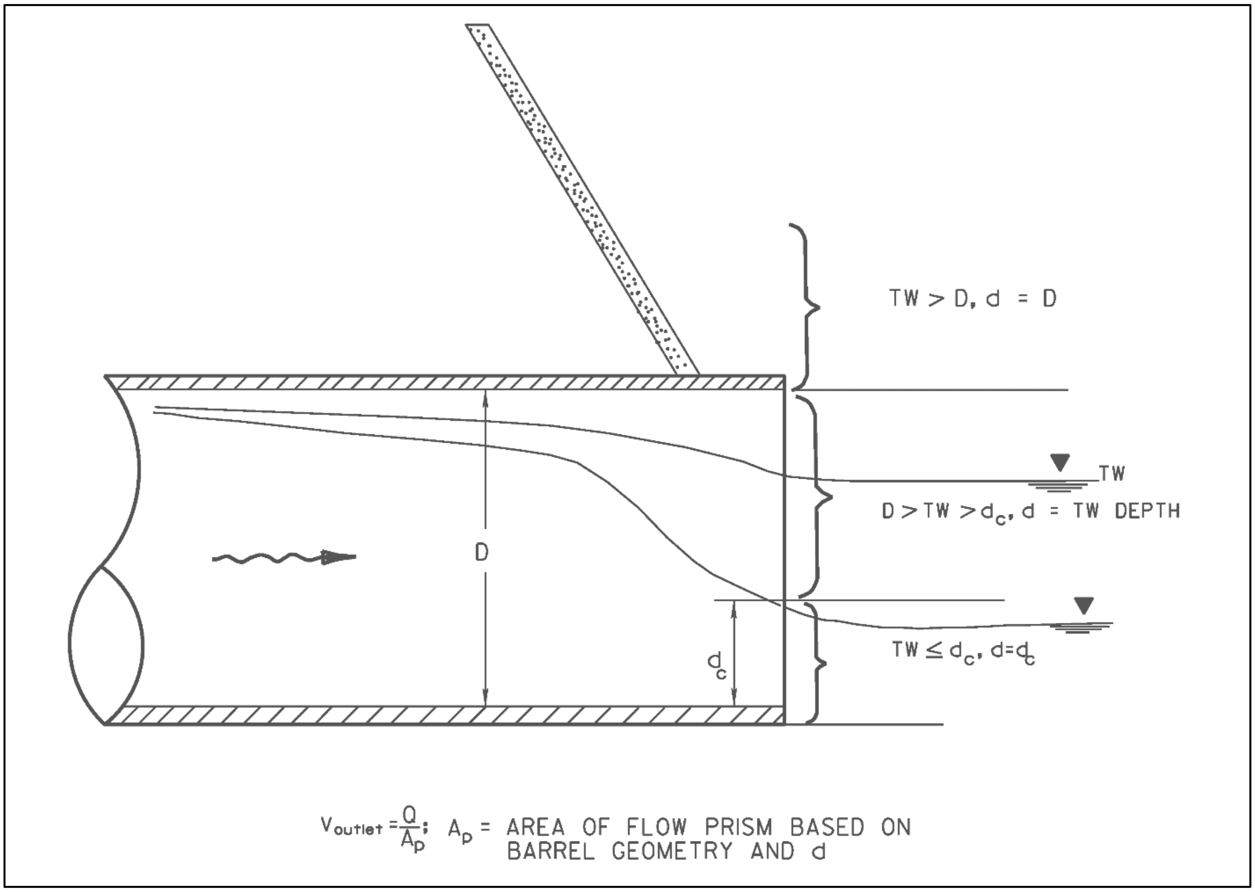


Figure 4.5-5: Outlet Velocity for Outlet Control

Example 4.5-2—Computing Outlet Velocity

Given the information below, determine the outlet velocity for outlet control.

where:
Q_{Design} = 18 ft^3/s
Diameter of Pipe = 36 in.
Slope of Pipe = 0.01 ft./ft.
Roughness Coefficient (n) = 0.012
Critical Depth (d_c) = 1.4 ft. (determined from FHWA HDS-5, for Q_{Design} = 18 ft^3/s)
Tailwater (TW) = 2.0 ft.

Solution

Step 1: Determine the area of the pipe flowing full:

$$Area\,(A) = \frac{\pi D^2}{4} = \frac{\pi \times (36\ inches\ /12)^2}{4} = 7.07\ ft^2$$

Step 2: Since D > TW > d_c, then d = TW Depth or d = 2.0 ft

Step 3: Using Figure 4.5-6 determine the depth of flow to full depth flow (TW/D) or 2 ft./3 ft. = 0.67, or 67 percent of the full depth.

1. Enter on Figure 4.5-6 this value of 0.67 on the horizontal axis.

2. Project horizontally to the area of flow curve.

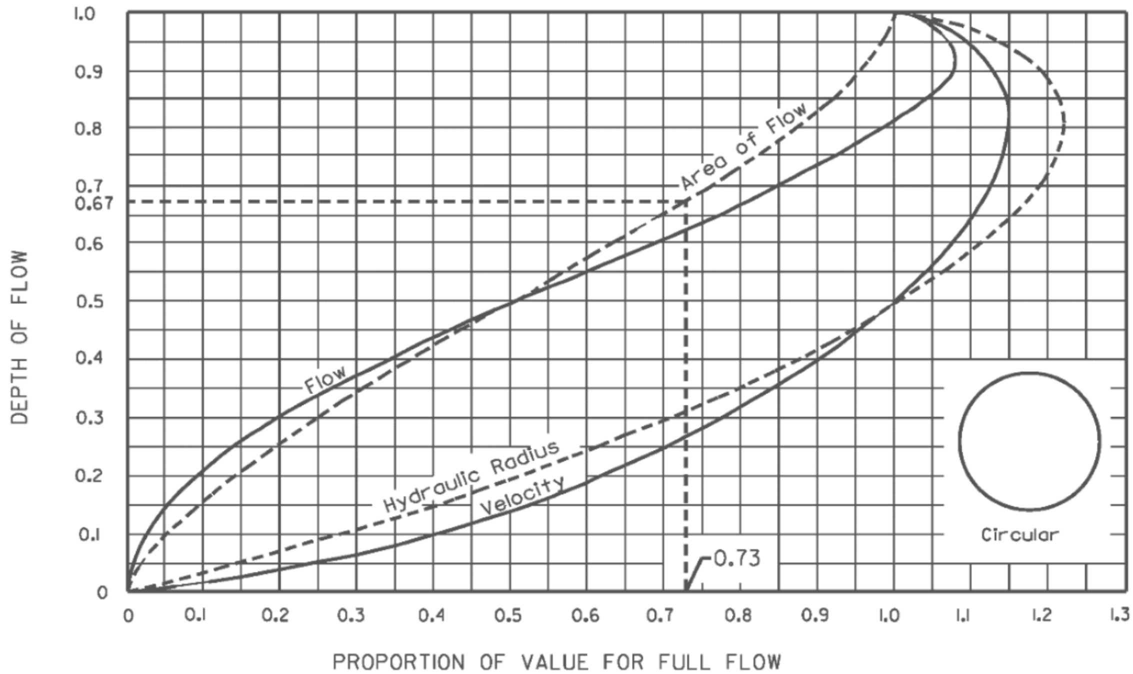
3. Project vertically down from the area of flow curve and read from the horizontal axis a value of 0.73, or 73 percent for full section.

4. Use this relationship to determine the normal depth area (A_{Design}):

$$\frac{A_{Design}}{A_{Full}} = 0.73\ ;\ A_{Design} = 0.73 \times 7.07\ ft^2 = 5.61\ ft^2$$

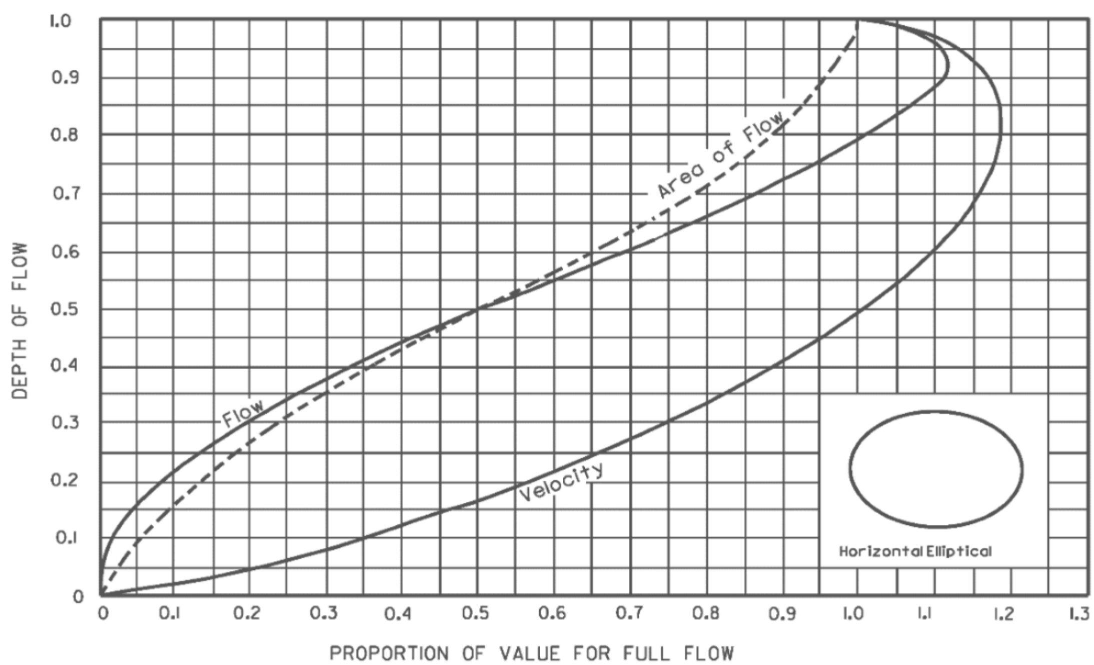
Step 4: Determine the outlet velocity using Q_{Design} and A_{Design}:

$$V_{Design} = Q_{Design}\ /\ A_{Design} = \frac{18\ ft^3/s}{5.61\ ft^2} = 3.2\ ft/s$$



Reference: American Concrete Pipe Association (1980).

Circular Pipe Relative Flow, Area, Hydraulic Radius, and Velocity for Any Depth



Reference: American Concrete Pipe Association (1980).

Horizontal Elliptical Pipe Relative Flow, Area, and Velocity for Any Depth

Figure 4.5-6: Example 4.5-2

Culvert Capacity Calculations

a. Worksheet for manual calculations

FHWA's HDS-5 presents a worksheet for doing culvert capacity calculations.

b. Computer programs

FHWA's HY-8 computer program is only one of several programs that are capable of culvert capacity calculations. The Department has accepted the computer program for use and it is available through FHWA's website (http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/). Before you use other computer programs in the design of a Department project, the District Drainage Engineer should approve them.

4.6 SPECIFIC STANDARDS RELATING TO ALL CROSS DRAINS EXCEPT BRIDGES

4.6.1 Culvert Materials

Chapter 6 of the *Drainage Manual* provides standards for suitable optional culvert materials.

When the vertical distance from invert to roadway is limited, arch culverts may be appropriate. When the rise of a culvert exceeds four feet, consider the use of box culverts since they may offer cost advantages.

4.6.2 Scour Estimates

Scour prediction at culvert outlets depends on the following characteristics:

- Channel bed and bank material
- Velocity and depth of flow in the channel and at the culvert outlet
- Velocity distribution
- Amount of sediment and other debris in the flow
- Culvert end section and treatment

A method for estimating the dimensions of a scour hole at a culvert outlet is available in HEC 14, Chapter 5, linked below:

http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14ch05.cfm.

A good guide in estimating potential scour at the outlet of proposed culverts is to look for scour developed at the outlet of similar existing culverts. Scour does not develop at all suspected locations because the susceptibility of the stream to scour is difficult to assess and the flow conditions that will cause scour do not occur at all flow rates. At locations where you expect scour to develop only during relatively rare flood events, the most economical solution may be to repair or retrofit the damage after it occurs.

At many locations, using simple outlet treatments—such as aprons of concrete or riprap—will provide adequate protection against scour. At other locations, using a rougher culvert material may be sufficient to prevent damage from scour.

When the outlet velocity is greater than or equal to 12 ft/sec, consider energy dissipation devices, such as those shown in the *Standard Plans*.

4.7 RECOMMENDED DESIGN PROCEDURE

The following procedures normally will result in acceptable, cost-effective designs. However, you are not exempt from developing an appropriate design. You are responsible for identifying which standards are not applicable to a particular design and for obtaining variances as necessary to achieve proper design.

The design procedures below do not account for structures within regulatory floodways; therefore, you may need to deviate from these procedures to satisfy regulatory agencies. Evaluate and determine the level of effort needed to produce an acceptable design.

Design procedures for three categories of cross drains are provided, including:

1. Culvert extensions (including side drain pipes)
2. Small cross drains (up to 48 inches round or equivalent other shape)
3. Large cross drains (more than 48 inches, but less than a 20-foot bridge)

4.7.1 Culvert Extensions

- **Contact the appropriate FDOT Maintenance Office** to determine if there is any history of problems associated with the existing culvert (e.g., flooding, scour, etc.).

- **Conduct a field review** to evaluate the condition/adequacy of the existing culvert. Review for condition, signs of scour, and sedimentation. Check the available right of way to see if there is room to transition ditches to meet the culvert extension. You can use a review checklist (see the following suggested format) to document the field review.

Review Checklist

Date: ______________________________

Project: ______________________________

Location: ________________________ Size/Type ___________________________

Road surface/Leaking joints? ____

Recent development in basin? ____

Overtopping? *Roadway* *Basin divide* *In roadway ditch*

____

Concerns with culvert extension? *Limited right of way* *Wetlands*

Normal high water marks: ____

Tailwater: *Ditch* *Piped outfall* *Overland flow* *Swamp*

____

Erosion/Sedimentation: ____

Misc. Comments: ____

____

____

____

____

- **Method 1: No Known Historical Problems**

  If there are no signs of undesirable scour at inlet and outlet ends, no excessive sedimentation, and no history of problems, you may extend the existing culvert. The hydrologic and hydraulic analysis would follow the procedure shown below:

  a. Estimate discharges as follows:

     i. 25 yr. Q = AV where — A = Existing Culvert Area; V = 6 feet per second (Confirm this value with the District Drainage Engineer; some districts use a lower velocity)

     ii. 100 yr. Q = 1.4 x (25 yr Q)

     iii. 500 yr. Q = 1.7 x (100 yr Q)

  b. Estimate tailwater. If the outlet is in a free-flowing condition, assume the crown of the pipe at the outlet is the tailwater.

  c. Conduct hydraulic analysis to compute stages using FHWA HDS 5 techniques.

  d. Document as required in the *Drainage Manual*.

  General Concerns
  Make sure enough right of way exists beyond the ends of the extended culvert to tie in the roadside ditches and provide for outlet treatment if necessary. There is a detail in the *Standard Plans* for ditch transitions at culvert locations. If right of way is inadequate, consider adjusting the ditch cross-section. If there is not enough room for the transition shown in the *Standard Plans*, you may design a sharper transition, but evaluate the need for channel lining to prevent erosion of the ditch side slopes. Example 4.7-1 illustrates this method.

Example 4.7-1—Culvert Extension

Existing: Two-lane rural road
ADT = 2,000 vehicles
36-inch diameter round concrete pipe (RCP)
Length of pipe is 59 feet
Straight end walls

Elevations are as follows:
Allowable headwater (Edge of travel lane) = 105.0 feet
Flow line (upstream) = 100.0 feet
Flow line (downstream) = 99.8 feet

- **Contact the appropriate FDOT Maintenance Office** to determine if there is any history of problems associated with the existing culvert (e.g., flooding, scour, etc.).

  Spoke with Mr. Steve Smith from the FDOT Maintenance Office on November 18, 1993. From our discussion, we found that there has been no history of problems in overtopping of the roadway and no complaints of flooding from upstream property owners have been found.

- **Conduct a field review** to evaluate condition/adequacy of existing culvert. Review for condition, signs of scour, and sedimentation.

  We performed a field review with Mr. Smith on November 21, 1993. From our review, we determined the culvert was in good condition, with no signs of sedimentation or scour.

- **No Known Historical Problems**
  Since there were no known historical problems, use Method 1. Recommend that the existing 36-inch RCP be extended four feet in both directions with 36-inch RCP and straight endwalls (*Standard Plans, Index 430-030*). The proposed flow line elevations are as follows:

  Flow line (upstream) = 100.1 feet
  Flow line (downstream) = 99.7 feet

  a. Estimate discharges as follows:

  Area of 36-inch RCP = $(\pi D^2)/4 = (\pi(36 \text{ inch}/12)^2)/4 = 7.07 \text{ ft}^2$

  $Q(25) = AV = 7.07 \text{ ft}^2 \times 6 \text{ ft/sec} = 42 \text{ ft}^3/\text{sec}$
  $Q(100) = 1.4 \times Q(25) = 59 \text{ ft}^3/\text{sec}$
  $Q(500) = 1.7 \times Q(100) = 100 \text{ ft}^3/\text{sec}$

Since this roadway has an ADT > 1,500, the design frequency is 50 years (determined from the *Drainage Manual*). To determine the 50-year discharge, a procedure similar to that used in Example 4.1-1 is appropriate. For this example, the Q(50) is 50 ft^3/sec.

b. Estimate tailwater as discussed in Chapter 3 (Open Channel) or if outlet is in a free-flowing condition, assume the crown of the pipe at the outlet is the tailwater.

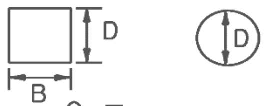
   For this example, the 50-year tailwater elevation to be used will be:

   TW (50 year) = 2.7 ft.

c. Conduct hydraulic analysis using the procedures in FHWA HDS 5.

   For this example, only the hydraulic analysis for the 50-year frequency will be computed. However, you also would need to compute an analysis for the other frequencies. The analysis is for the proposed conditions. Figure 4.7-1 summarizes the following calculations.

HYDROLOGIC AND CHANNEL INFORMATION



D = Diameter or Height
B = Span

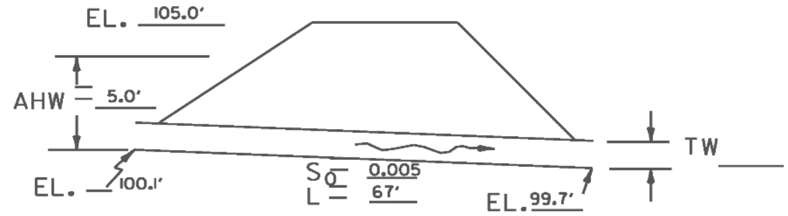
Q_1 = ________ TW_1 = ________

Q_2 = ________ TW_2 = ________

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ________



MEAN STREAM VELOCITY = ___

MAX. STREAM VELOCITY = ___

LS_o 0.4'

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE		HEADWATER COMPUTATION											CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
				INLET CONTROL			OUTLET CONTROL											
		D	B	Q/B	HW/D	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	DTW	LS_o	HW				
36" RCP	50 cfs	36"			1.27'	3.8'	0.2	1.55'	2.3'	2.65'	2.7'	2.7'	0.4'	3.9'	3.9'			50 year frequency

SUMMARY & RECOMMENDATION:

Figure 4.7-1: Culvert Capacity Worksheet for Example 4.7-1

- Inlet Control

  Nomographs:

  Using Chart 1 in FHWA HDS 5, HW/D = 1.27
  Therefore,
  HW = 1.27 x D = 1.27 x 3 ft. = 3.81 ft., say 3.8 ft.

  Determine the headwater elevation by taking the culvert invert at the entrance and adding the headwater depth:

  HW Elevation = 100.1 ft. + 3.8 ft. = 103.9 ft.

- Outlet Control

  Nomographs:

  Using Chart 5 in FHWA HDS 5 with a pipe length of 67 feet (existing 59 ft. + 8 ft. of extension) and an entrance loss coefficient of 0.2 feet (as determined below), the headwater (H) for the 50-year discharge is 1.55.

  Culvert Entrance Loss Coefficients (K_e):

  Culvert entrance loss is 0.2 as determined from the *Application Guidelines for Pipe End Treatment*, Appendix F, based on the structure having a standard end wall treatment.

  Critical Depth (d_c):

  Using Chart 4 in FHWA HDS 5, the critical depth was found to be 2.3 feet.

  Equivalent Hydraulic Elevation (h_o):

$$h_o = \frac{D + d_c}{2} = \frac{3\ ft + 2.3\ ft}{2} = 2.65\ ft.$$

Design Tailwater (DTW):

Since the TW > h_o, then the DTW = TW = 2.7 ft.

Headwater Depth (HW):

Having established the total head loss (H) and the design tailwater depth (DTW) as described above, compute the headwater depth (HW), as follows:

HW = H + DTW - LS_o
HW = 1.55 ft. + 2.70 ft. - (0.4 ft.)
HW = 3.85 ft., say 3.9 ft.

Determine the headwater elevation by taking the culvert invert at the entrance and adding the headwater depth:

HW Elevation = 100.1 ft. + 3.9 ft. = 104.0 ft.

- Controlling Headwater (HW) Depth or Elevation

  Since the HW depth or elevation for outlet control (HW Elevation = 104.0 feet) is greater than that of inlet control (HW Elevation = 103.9 feet), then the controlling HW Elevation is 104.0 feet.

- Outlet Velocity

  Outlet velocity for a culvert for this type of problem does not need to be computed since the discharges were estimated using a 25-year velocity of 6 ft/sec.

d. Document as required in the *Drainage Manual*.

End of Example 4.7-1

- **Method 2: Known Historical Problems or If the Analysis Yields Unrealistic Results**
  If scour, sedimentation, or other known historical problems exist, or if Method 1 yields unrealistic results, conduct complete hydrologic and hydraulic analysis and evaluate alternatives.

  a. Conduct a complete hydrologic analysis using one of the following methods, as appropriate (see Section 4.7 of the *Drainage Manual*):

     - Frequency analysis of observed data
     - Regional or local regression equation
     - Rational Equation (up to 600 acres)

  b. Determine tailwater conditions.

  c. Conduct hydraulic analysis using procedures in FHWA HDS 5.

  d. Assess cause of problem and investigate/evaluate alternative solutions. Final recommended design should address the problem with consideration to design standards.

  e. Document as required in the *Drainage Manual*.

  General Considerations
  The ditch transition concerns in the previous section also apply here. In addition, any problems such as scour, sedimentation, etc., should be limited to within the right of way or not extend any further outside the right of way than they currently extend. Example 4.7-2 illustrates this procedure.

Example 4.7-2—Culvert Extension

Existing: Two-lane rural road
ADT = 2,000
2 foot x 2 foot concrete box culvert cross drain
Length of pipe is 50 feet
Straight end walls

Elevations are as follows:
Allowable headwater (edge of travel lane) = 104.6 feet
Flow line (upstream) = 100.0 feet
Flow line (downstream) = 99.8 feet

- **Contact appropriate FDOT Maintenance Office** to determine if there is any history of problems associated with the existing culvert (e.g., flooding, scour, etc.).

  Spoke with Mr. Steve Smith from the FDOT Maintenance Office on November 18, 1993. We found that there has been history of overtopping of the roadway.

- **Conduct a field review** to evaluate condition/adequacy of existing culvert. Review for condition, signs of scour, and sedimentation.

  We performed a field review with Mr. Smith on November 21, 1993. From our review, we discovered that the area around the outlet end of the culvert showed signs of scouring.

- **Known Historical Problem**
  Since the area around the outlet end of the culvert showed signs of scouring, analyze the structure using Method 2.

  a. Conduct a complete hydrologic analysis using one of the following methods, as appropriate (see Section 4.7 of the *Drainage Manual*):

     - Frequency analysis of observed data
     - Regional or local regression equation
     - Rational Equation (up to 600 acres)

From the field review and hydrologic calculations, the following design information is known:

Q(50) = 35 ft^3/sec
Q(100) = 52 ft^3/sec
Q(500) = 88 ft^3/sec

b. Determine tailwater as discussed in Chapter 3 (Open Channel) or if outlet is in a free-flowing condition, the crown of the pipe at the outlet may be assumed.

For this example, the 50-year tailwater elevation to be used will be TW (50 year) = 2.5 ft.

c. Conduct hydraulic analysis using the procedures in FHWA HDS 5.

For this example, only a hydraulic analysis for the 50-year frequency will be computed. The other frequencies also would need to be analyzed for an actual project. The analysis is for the existing conditions. Figure 4.7-2 summarizes the following calculations.

- Inlet Control

Nomographs

Using Chart 8 in FHWA HDS 5, Q/B = (35 ft^3/sec)/2 ft. = 17.5 ft^3/sec. Therefore, HW/D = 2.4 and HW = 2.4 ft. x D = 2.4 ft. x 2 ft = 4.8 ft.

Determine the headwater elevation by taking the culvert invert at the entrance and adding the headwater depth:

HW Elevation = 100.0 ft + 4.8 ft = 104.8 ft

- Outlet Control

Nomographs

Using Chart 15 in FHWA HDS 5, the headwater (H) for the 50-year discharge is 2.2 feet based on the pipe length of 50 feet and entrance loss coefficient of 0.2, as determined below.

Culvert Entrance Loss Coefficients (K_e)

Culvert entrance loss is 0.2 as determined from the *Application Guidelines for Pipe End Treatments*, Appendix F, based on the structure having a straight end wall treatment.

Critical Depth (d_c)

Using Chart 14 in FHWA HDS 5, the critical depth was found to be 2 feet.

Equivalent Hydraulic Elevation (h_o)

$$h_o = \frac{D + d_c}{2} = \frac{2\ ft. + 2\ ft.}{2} = 2\ ft.$$

Design Tailwater (DTW)

Since the TW > h_o, then the DTW = TW = 2.5 ft.

Headwater Depth (HW):

Having established the total head loss (H) and the design tailwater depth (DTW) as described above, compute the headwater depth (HW), as follows:

$HW = H + DTW - LS_o$
$HW = 2.2 \text{ ft.} + 2.5 \text{ ft.} - (0.2 \text{ ft.})$
$HW = 4.5 \text{ ft.}$

Determine the headwater elevation by taking the culvert invert at the entrance and adding the headwater depth:

HW Elevation = 100.0 ft. + 4.5 ft. = 104.5 ft.

- Controlling Headwater (HW) Depth or Elevation

  Since the HW depth or elevation for inlet control (HW elevation = 104.8 feet) is greater than that of outlet control (HW elevation = 104.5 feet), then the controlling HW elevation is 104.8 feet.

- Outlet Velocity

  Since the existing structure was found to be inlet control, the outlet velocity was determined as discussed earlier in this section.

HYDROLOGIC AND CHANNEL INFORMATION

D = Diameter or Height
B = Span

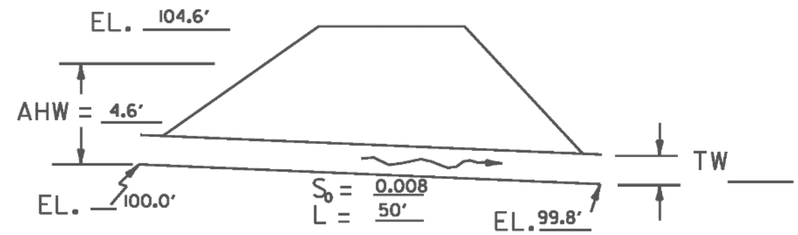
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ______



MEAN STREAM VELOCITY = ___
MAX. STREAM VELOCITY = ___
LS_0 0.2'

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE		HEADWATER COMPUTATION											CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
				INLET CONTROL			OUTLET CONTROL											
		D	B	$\frac{Q}{B}$	$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	DTW	LS_0	HW				
2'x2'cbc	36 cfs	2'	2'	17.5	2.4	4.8'	0.5	2.2'	2'	2'	2.5'	2.5'	0.2'	4.4'	4.8'	8.8		OVERTOPPING OCCURS FOR 50 YEAR DESIGN FREQUENCY

SUMMARY & RECOMMENDATION: Since minimum design frequency for this crossing is exceeded, redesign structure as shown in example 8

Figure 4.7-2: Worksheet for Example 4.7-2

d. Assess the cause of the problem and investigate/evaluate alternative solutions. Final recommended design should address the problem with consideration to design standards.

   Review of Figure 4.7-2 indicates that the roadway is overtopped for a 50-year design frequency. Therefore, recommend replacing the structure. It is anticipated that a cross drain no larger than a 48-inch diameter would be appropriate for this location. The procedure in Section 4.7.2, following, could be used. Example 4.7-3 illustrates this using the information from this example.

e. Document as required in the *Drainage Manual.*

End of Example 4.7-2

4.7.2 Small Cross Drains

This information applies to cross drains having an area of opening up through a 48-inch-diameter round culvert or the equivalent.

- **Conduct hydrologic analysis**
  Estimate discharges for design year frequency, base flood, and greatest flood. Use one of the following procedures as appropriate (see Section 4.7 of the *Drainage Manual*):

  - Rational Equation (up to 600 acres)
  - Regional or Local Regression Equation

- **Select trial culvert size** based on the following:

$$A = Q/V$$

Where:
A = Culvert area (square feet)
Q = Design discharge (e.g., 50 year)
V = Average velocity (feet per second); use an average velocity of four feet per second

- **Estimate tailwater.** If the outlet is in a free-flowing condition, the crown of the pipe at the outlet may be assumed.

- **Conduct hydraulic analysis** using techniques provided in FHWA HDS 5. Compute headwater conditions for the selected size for the design flood, base flood, and greatest flood or overtopping flood as appropriate.

- **Check hydraulic results** against design standards for backwater, minimum size, and scour. If these standards are satisfied, the trial culvert size is acceptable.

- **Determine the most economical culvert size** that satisfies all standards. If the trial selected size does not satisfy all design standards, obtain a variance.

- **Document** as required in the *Drainage Manual.*

Example 4.7-3 illustrates this procedure.

Example 4.7-3—Design of Small Cross Drain

Referring back to Example 4.7-2, you determined that the two-foot x two-foot concrete box culvert should be replaced. A design frequency of 50 years was determined as the minimum for this roadway. The existing length of the two-foot x two-foot concrete box culvert was 50 feet. However, since the structure will have to be extended four feet on each side, the design length of the proposed structure will be 58 feet.

Proposed Elevations are as follows:
Allowable headwater (edge of travel lane) = 104.6 ft
Flow line (upstream) = 100.1 ft
Flow line (downstream) = 99.7 ft

- **Conduct hydrologic analysis**
  Estimate discharges for design-year frequency, base flood, and greatest flood. Use one of following procedures as appropriate (see Section 4.7 of the *Drainage Manual*):

  - Rational Equation (up to 600 acres)
  - Regional or Local Regression Equation

  Use the same discharges from Example 4.7-2:

  Q(50) = 35 ft^3/sec
  Q(100) = 52 ft^3/sec
  Q(500) = 88 ft^3/sec

- **Select trial culvert size**

$$A = \frac{Q}{V} = \frac{35\ ft^3/s}{4\ ft/s} = 8.8\,ft^2$$

  D = 3.3 ft., so try D = 36-inch pipe and 42-inch pipe

- **Conduct hydraulic analysis** using FHWA HDS 5 procedures.

  The hydraulic analysis would be similar to what was done in Example 4.7-1 and Example 4.7-2. A worksheet of the calculations for the 50-year frequency is shown in Figure 4.7-3. The other frequencies also would need to be analyzed for an actual project. The analysis shown in Figure 4.7-3 is for the proposed conditions.

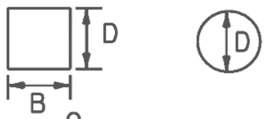
- **Check hydraulic results** against design standards.

  Review of the worksheet in Figure 4.7-3 indicates that the roadway will not overtop for the 50-year frequency for either culvert size. There is very little difference between the 36-inch and 42 inch pipe as far as controlling headwater. Therefore, either pipe size would be adequate. However, it is recommended that the 36-inch pipe be installed since it would be slightly less in cost than the 42-inch pipe. In addition, it would be recommended that a rubble ditch lining design be installed at the outlet end due to velocities exceeding six feet per second.

- If design does not meet standards or if you can use more economical culvert size that satisfies the standards, then perform new computations for that design.

Document as required in the *Drainage Manual*.

HYDROLOGIC AND CHANNEL INFORMATION



D = Diameter or Height
B = Span

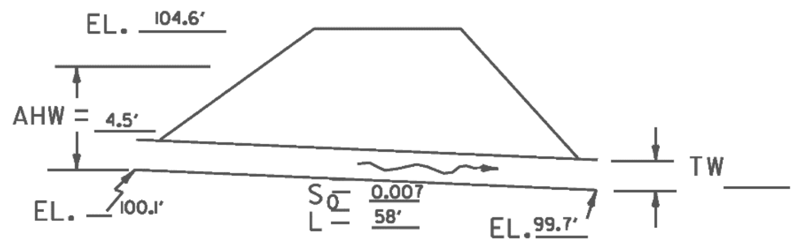
Q_1 = ________ TW_1 = ________

Q_2 = ________ TW_2 = ________

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ____________



MEAN STREAM VELOCITY = ___
MAX. STREAM VELOCITY = ___ LS_0 0.4'

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE		HEADWATER COMPUTATION											CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
				INLET CONTROL			OUTLET CONTROL											
		D	B	$\frac{Q}{B}$	$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	DTW	LS_0	HW				
36" RCP	35 cfs	36'			0.95	2.9'	0.2	0.55'	1.9'	2.5'	2.5'	2.5'	0.4'	2.7'	2.9'	8.7		50 year frequency (Inlet Control)
42" RCP	35 cfs	42"			0.75	2.6'	0.2	0.3'	1.8'	2.7'	2.5'	2.7'	0.4'	2.4'	2.6'	8.9		50 year frequency (Inlet Control)

SUMMARY & RECOMMENDATION: Velocities for both designs are high. Recommend installing the 36' pipe with a robble ditch lining design.

Figure 4.7-3: Worksheet for Example 4.7-3

4.7.3 Large Cross Drains

This information applies to cross drains having an area of opening greater than a 48-inch diameter pipe and less than a 20-foot bridge. The procedure for large cross drains is similar to that for small cross drains except that a greater level of effort and detail is expected in developing the hydrologic estimates and the determination of tailwater conditions.

- **Conduct hydrologic analysis**
  Estimate discharges for design-year frequency, base flood, and greatest flood. Use one of following procedures as appropriate (see Section 4.7 of the *Drainage Manual*):

  - Frequency analysis of observed conditions
  - Regional or Local Regression Equation
  - Rational Equation (up to 600 acres)

The remaining steps are the same as those identified in Section 4.7.2 for small cross drains.