STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK
CULVERT DESIGN

OFFICE OF DESIGN, DRAINAGE SECTION January 2004
TALLAHASSEE, FLORIDA

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Chapter 1

Introduction

1.1 Background

The 1987 Florida Department of Transportation Drainage Manual was published as a three volume set: Volume I – Policy; Volumes 2A and 2B – Procedures; Volume 3 – Theory. On October 1, 1992, Volume I – Policy was revised to Volume 1 – Standards. With that revision, Volumes 2A, 2B, and 3 were designated as general reference documents. The Volume I – Standards was revised in January 1997 and was renamed to simply the “Drainage Manual.” No revisions have been made, nor will be made to Volumes 2A, 2B, and 3 of the 1987 Drainage Manual.

This handbook is one of several the Central Office Drainage section is developing to replace Volumes 2A, 2B, and 3 of the 1987 Drainage Manual. In this form, the current Drainage Manual will be maintained as a “standards” document, while the handbooks will cover general guidance on FDOT drainage design practice, analysis and computational methods, design aids, and other reference material.

1.2 Purpose

This handbook is intended to be a reference for designers of FDOT projects, and to provide guidelines for the hydraulic design of cross drains, including culverts and bridge-culverts. These guidelines were developed to help the drainage engineer meet the standards addressed in Volume 1, Chapter 4 of the Drainage Manual and incorporate pertinent sections of the 1987 Drainage Manual.

The guidance and values in this handbook are suggested or preferred approaches or values, not requirements or standards. The values provided in the Drainage Manual are the minimum standards. This handbook does not replace the standards and in cases of discrepancy, the Drainage Manual standards shall govern. This handbook neither replaces the need for professional engineering judgment nor precludes the use of information not presented in the handbook. Situations exist where the guidance provided in this handbook will not apply. THE INAPPROPRIATE USE OF AND ADHERENCE TO THE GUIDELINES CONTAINED HEREIN DOES NOT EXEMPT THE ENGINEER FROM THE PROFESSIONAL RESPONSIBILITY OF DEVELOPING AN APPROPRIATE DESIGN.

1.3 Distribution

This handbook is available for downloading from the Drainage Internet site.

1.4 Revisions

Any comments or suggestions concerning the handbook can be made by mailing them to:

Florida Department of Transportation
Office of Design - Drainage Section
Mail Station 32
605 Suwannee Street
Tallahassee, Florida 32399-0450

Chapter 2

General

2.1 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
  Backwater is discussed in Chapter 4 of this handbook as well as in Chapter 4 of the Drainage Manual.

- Tailwater
  Tailwater is discussed in Chapter 5 of this handbook as well as in Chapter 4 of the Drainage Manual.

- Scour
  Scour is discussed in Chapters 2.2 and 7.2 of this handbook as well as in Chapter 4 of the Drainage Manual.

A risk analysis may be required to evaluate damage to structures and/or embankments caused by backwater and/or scour. Refer to Appendix A, Risk Evaluations.

2.2 Scour Estimates

Scour estimates for Bridge Culvert foundations should not be designed using the methods in FHWA'S HEC-18. Instead, the outlet velocity and degradation of the stream should be considered as discussed in Chapter 7 of this handbook.

Bridge culverts with no bottom slab and toewall should not be used unless the following approval/evaluation is made:

a) Prior approval from the District Drainage Engineer.

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

2.3 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 frequency", is discussed in Chapter 3 of this handbook.

- Base Flood

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

- Greatest Flood

  The Greatest Flood (500-year frequency flood) 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 one percent flood. 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 (Flood Emergency Management Agency) and other agencies compute the 500-year frequency flood.

- Overtopping Flood

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

  This information on this 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.

  The drainage engineer should 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 1 shows how the overtopping flood is determined.

Example 1 - Computing Overtopping Flood

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

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

<u>Solution:</u>

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

Note: Graphical estimation methods are explained in FHWA-IP-80-1 publication, “Hydrology for Transportation Engineers”, page 314.

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

3. Knowing what the overtopping stage is, the overtopping discharge can be conservatively approximated. The overtopping discharge was found to be 64 ft^3/s or cfs.

Note: For stages above overtopping significant relief is expected due to the overtopping flow. The stage versus discharge relationship usually flattens out after overtopping.

Step 2: 1. To determine the overtopping frequency, plot frequency versus discharge on log - normal probability paper for the 25 and 100 year floods as shown in Figure 2.

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

3. Knowing what the overtopping discharge is from Step 1 (3), the probability of the overtopping flood being exceeded in any year can be determined. It was determined to be 0.65 percent. This corresponds to a frequency of 154 years (i.e., 100/0.65).

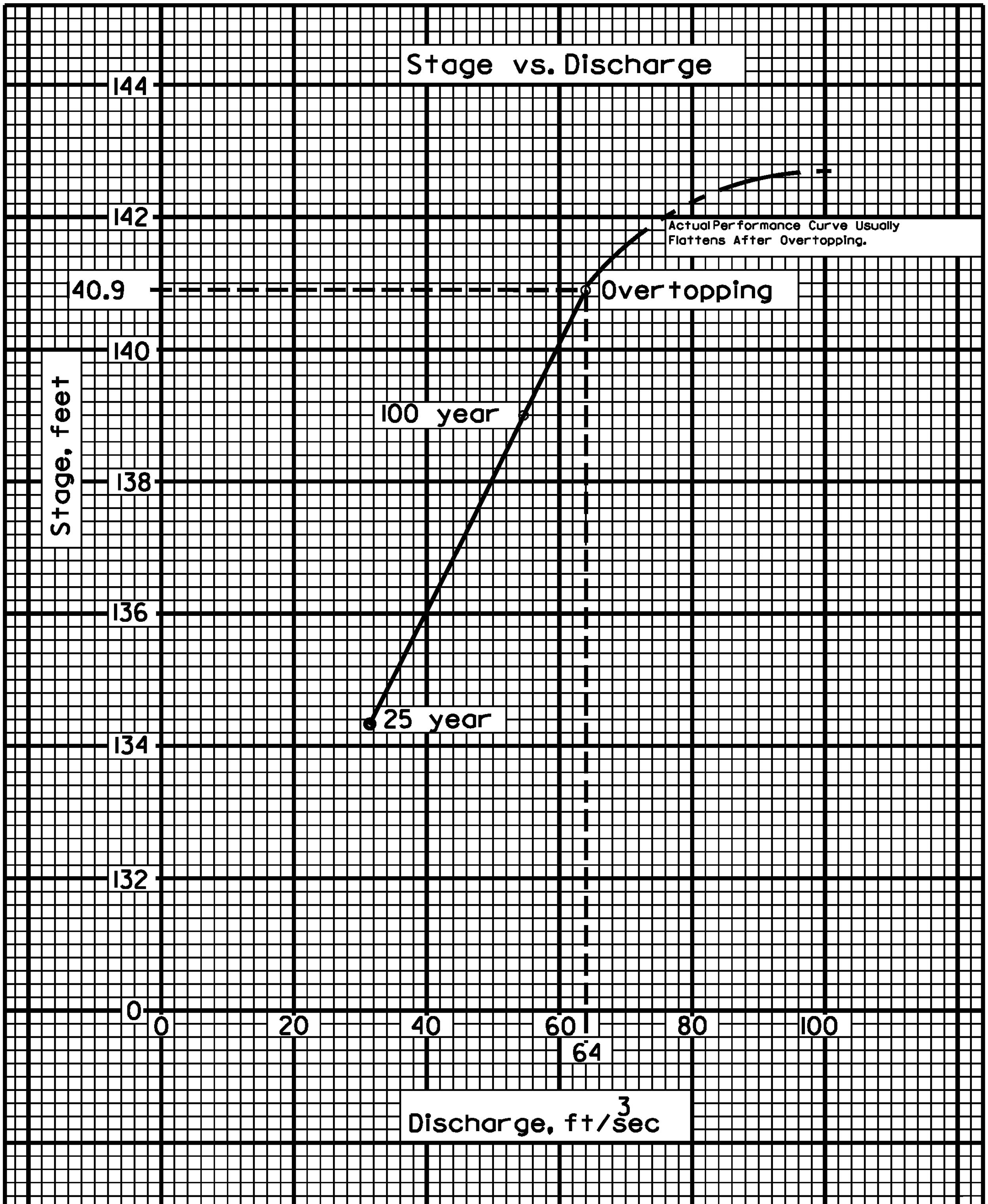

Figure 1
Example 1- Computing Overtopping Flood (Cont.)

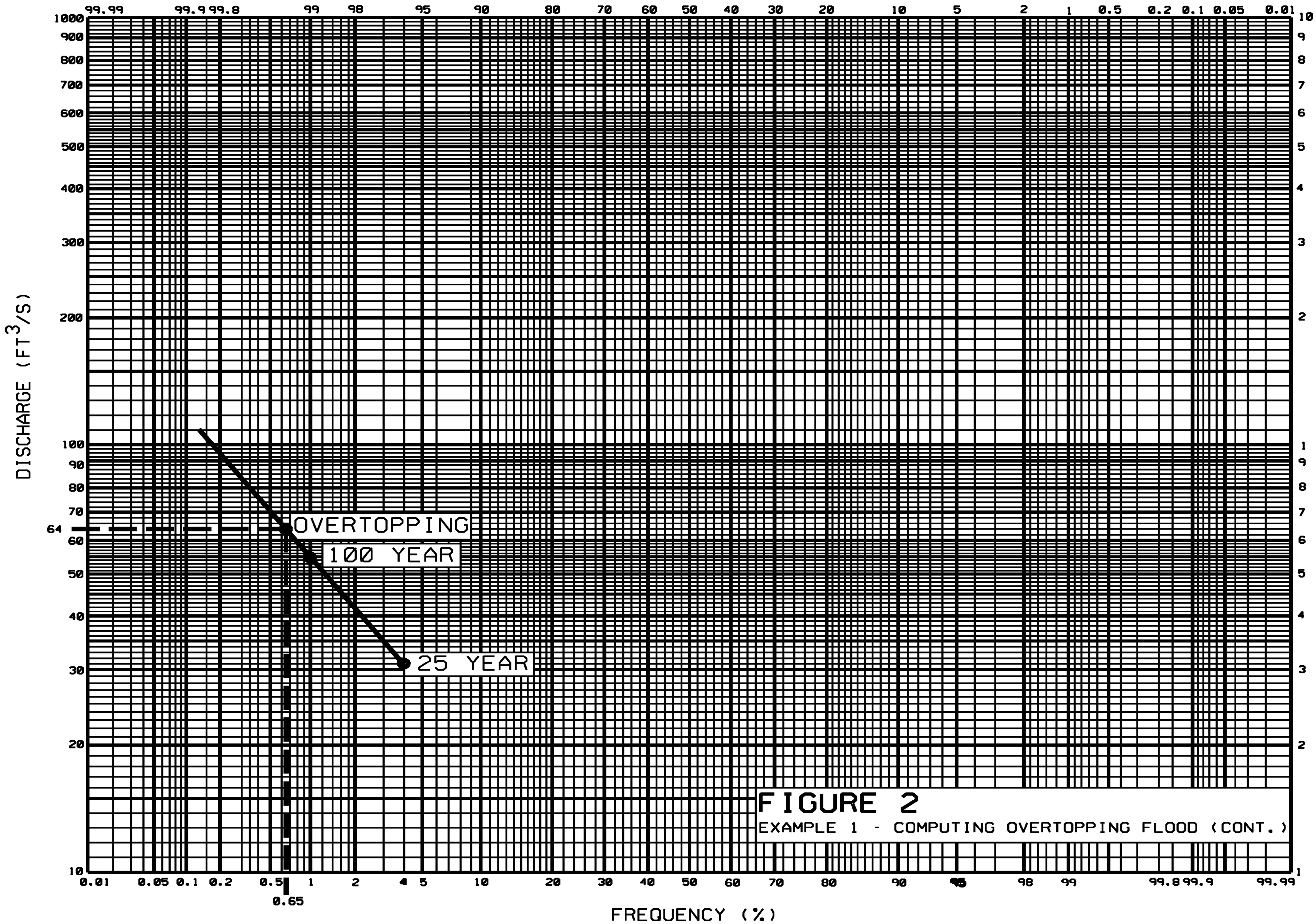

FIGURE 2
EXAMPLE 1 - COMPUTING OVERTOPPING FLOOD (CONT.)

Hydraulic Flood Data Sheet

For culverts other than bridge culverts, hydraulic data should be included in a Hydraulic Flood Data Sheet, such as shown in Figure 3. Because this sheet is shown in the plans, the drainage engineer should show peak stages and discharges for the events in the same units (SI or English) used to develop the roadway project. This sheet should be included for those conditions discussed in the Department's Plans Preparation Manual (PPM).

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 is shown for informational purposes only, to indicate the flood discharges and water surface elevations which may be anticipated in any given year. This data was 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 is sensitive to changes, particularly of antecedent conditions, urbanization, channelization and land use. Users of this 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 F.D.O.T. to be utilized to assure a standard level of hydraulic performance.

Base Flood: — The flood having a 1% chance of being exceeded in any given year. (100-year frequency)

Overtopping Flood: — The flood where flow occurs over the highway, over a watershed divide or through emergency relief structures.

Greatest Flood: — The most severe flood which can be predicted where overtopping is not practicable, normally one with a 0.2% chance of being exceeded in any given year. (500-year frequency)

Figure 3 – Hydraulic Flood Data Sheet

The drainage engineer should fill out the hydraulic flood data sheet according to the Federal Aid Policy Guide (23 CFR 650A) shown in Appendix A of the Drainage Manual. In general the following applies.

a. If the overtopping flood is less than the standard design frequency, a risk assessment must be performed to define the design flood as the overtopping flood. The information for the design (overtopping) flood, base flood, and overtopping flood must be filled out.

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

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

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

Example 2 shows how the Hydraulic Flood Data Sheet should be filled out when the overtopping flood is less than the greatest flood (500-year flood).

Example 3 shows how the Hydraulic Flood Data Sheet should be filled out when the overtopping flood occurs at a 10-year frequency.

Example 2 - Completing the Hydraulic Flood Data Sheet

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

Solution:

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

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. From Example 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.

Example 2 - Completing the Hydraulic Flood Data Sheet (Cont.)

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 is shown for informational purposes only, to indicate the flood discharges and water surface elevations which may be anticipated in any given year. This data was 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 is sensitive to changes, particularly of antecedent conditions, urbanization, channelization and land use. Users of this data are cautioned against the assumption of precision which can not be attained. Discharges are in cubic feet per second and stages are in feet.

Definitions

Design Flood: The flood selected by F.D.O.T. to be utilized to assure a standard level of hydraulic performance.

Base Flood: The flood having a 1% chance of being exceeded in any given year. (100-year Frequency)

Overtopping Flood: The flood where flow occurs over the highway, over a watershed divide or thru emergency relief structures.

Greatest Flood: The most severe flood which, can be predicted where overtopping is not practicable, normally one with a 0.2 % chance of being exceeded in any given year. (500-Year Frequency)

Figure 4 - Hydraulic Flood Data Sheet

Example 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 the on criteria from Section 4.3 of the Drainage Manual.

- The structure <u>overtops</u> during a 10-year frequency.

- A risk assessment has been performed 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

<u>Solution:</u>

Since the overtopping flood is less than the standard design frequency and a risk assessment was performed to define the design flood as the overtopping flood, the information for the design (overtopping) flood, base flood, and overtopping flood must be filled out. Put these values in the corresponding column as shown in Figure 5.

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

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

Example 3 - Completing the Hydraulic Flood Data Sheet (Cont.)

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 is shown for informational purposes only, to indicate the flood discharges and water surface elevations which may be anticipated in any given year. This data was 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 is sensitive to changes, particularly of antecedent conditions, urbanization, channelization and land use. Users of this data are cautioned against the assumption of precision which can not be attained. Discharges are in cubic feet per second and stages are in feet.

Definitions

Design Flood: The flood selected by F.D.O.T. to be utilized to assure a standard level of hydraulic performance.

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Overtopping Flood: The flood where flow occurs over the highway, over a watershed divide or thru emergency relief structures.

Greatest Flood: The most severe flood which can be predicted where overtopping is not practicable, normally one with a 0.2 % chance of being exceeded in any given year. (500-year Frequency)

Figure 5 - Hydraulic Flood Data Sheet

Chapter 3

Design Frequency

Design frequency is a frequency that can be accommodated without violating an adopted design criteria. Once the design frequency is determined, a discharge for the selected frequency can also be determined. This discharge is also known as the "design discharge". By definition, the design discharge does not overtop the road. Once the design discharge is determined, a headwater can be determined. This headwater is also known as the "design discharge headwater". The design discharge headwater may be at an elevation lower than the road's profile grade in order 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 traditionally 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. Selection of 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 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, a greater return interval design flood should be evaluated by use of a risk analysis. Risk analysis procedures are provided in FHWA's HEC 17 and discussed briefly in Appendix A Risk Evaluations. In addition, design standards of other agencies that have control or jurisdiction over the waterway or facility concerned should be incorporated or addressed in the design.

Chapter 4

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.

- Backwater Consistent with the Flood Insurance Study requirements.

- Backwater Effects on Land Use

  Backwater effects are important considerations in the design/analyses of cross drains in rural and urban areas.

  In rural areas, the concern is with increased flood stages. The degree and duration of an increased flood stage could affect present and future land uses. Even agricultural land use has to be evaluated for increased risks due to flooding. As an example, crops may be impacted by inundation.

  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 will have stream or watershed management regulations or may be part of the National Flood Insurance Program. These may dictate the limits on the changes, which can be made to flow characteristics of a watershed.

  A risk evaluation may be required in order to determine damage to surrounding property. Refer to <u>Appendix A Risk Evaluations</u>.

  <u>Obtaining Flood Rights:</u>

  The Department does not encourage obtaining flood rights; however, it is recognized that in some instances it may be necessary. The drainage engineer should evaluate all possible alternatives before recommending to the Department that flood rights should be obtained.

  On occasion, water from heavy rainfall events or non-permitted drainage connections will exceed the capacity of the highway drainage system, overflowing the system and flowing onto land that the Department does not own. When areas where this may occur can be determined in advance, and when such flooding occurs under a limited set of conditions and is temporary in nature, the Department may acquire a temporary flooding easement. This gives the Department <u>flood rights</u>, allowing temporary use of private property to ease flooding. The flood easement may or may not define conditions under which flooding may occur and the elevation water would be expected to reach under those conditions. Emphasis is placed on public safety and cost when negotiating for the easement.

Flood rights are usually purchased on land in a natural state, which already floods under certain weather conditions from non-highway sources. An example of this type of land is a land-locked natural basin, such as those found in northern Florida.

The Department may purchase either a temporary or permanent water storage easement to provide a retention or detention storage area for discharging water from the closed highway drainage system. This storage area may allow the water to be transported to waterways of the state or to evaporate or percolate into the soil over time, and may be in response to certain temporary conditions or can become part of the drainage system design.

Alternatives to obtaining flood rights for upstream flooding include:

a. Prior approval from the property owner.
b. Purchase of the property.
c. Upsizing the structure as long as there is no increased flooding to the downstream owner.

As stated in Section 4.4.2 of the Drainage Manual, "The acquisition of flood rights shall be based on a risk analysis to select the least total cost expected design". Risk Analyses are discussed briefly in Appendix A and extensively in HEC-17 (USDOT, FHWA, 1981).

Chapter 5

Tailwater

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

Additional guidelines for tailwater elevations are referenced in Chapter 6.

Chapter 6

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". The publication can be obtained through:

National Technical Information Service
Springfield, Virginia 22161
(703) 487-4650

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

- Allowable Headwater

  The allowable headwater elevation is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. The criteria of Section 4.4.3 of the Drainage Manual applies, but other situations which may limit the allowable headwater are:

  a. Non-damaging or permissible upstream flooding elevations (e.g., existing buildings or Flood Insurance Regulations) should be identified. Headwater should be kept below these elevations.

  b. State Regulatory Constraints (e.g. Water Management District)

  c. Other site-specific design considerations should be addressed as required.

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

- Inlet Control

  Nomographs:

  Inlet nomographs shown in FHWA HDS-5 have been developed to provide graphical solutions of 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, the approach velocity is ignored and the water surface and energy line at the entrance are assumed to be coincident. For this reason, the headwater depths obtained by using the nomographs can be higher than will occur in some installations.

The headwater elevation for inlet control is determined 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 headloss equations for various culvert materials, cross sections, and inlet combinations.

Culvert Entrance Loss Coefficients:

Culvert Entrance Loss Coefficients (k_e) for the end treatments are presented in the Applications Guide for Pipe End Treatments in Appendix B. For other types of end treatments, refer to FHWA HDS-5.

Critical Depth:

The critical depth for various sizes and types of culverts may be determined using FHWA HDS-5.

Equivalent Hydraulic Elevation:

For culvert 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:

$$h_o = \frac{D + d_c}{2} \qquad \textbf{(Equation 1)}$$

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 read from the figures of FHWA HDS-5 is greater than D, then h_o will equal D.

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

Tailwater:

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

a. If an upstream culvert outlet is located 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 which discharge to an open channel, the tailwater may be equal to the normal depth of flow in that channel. Normal depth may be calculated using a trial and error solution of the Manning equation. The known inputs are channel roughness, slope, and geometry.

   For bridge culverts, which discharge to an open channel, the tailwater may have to be determined by performing a standard backwater calculation. This analysis should be considered if the open channel does not have constant channel roughness, slope, and geometry or if there is a control structure downstream which 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 the Florida Department of Environmental Protection, usually establishes tailwater conditions.

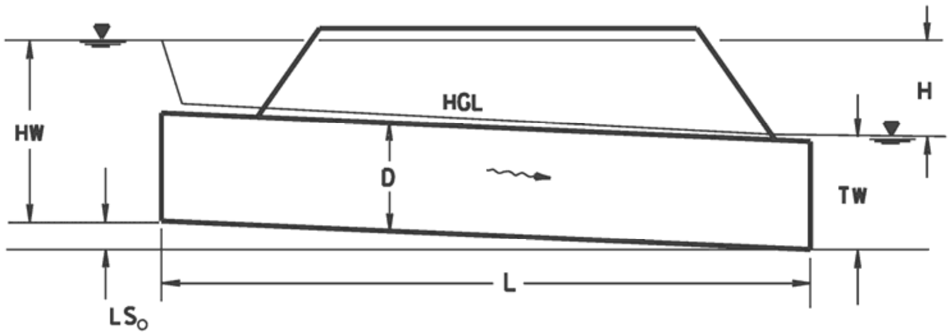
Design Tailwater:

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:

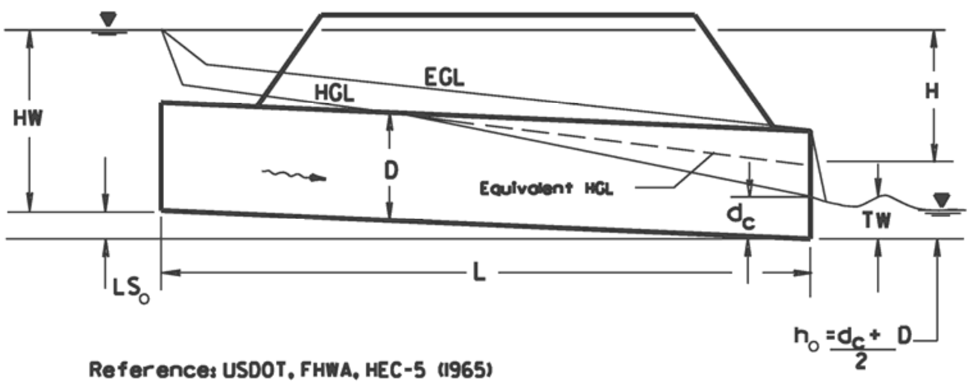
a. For the submerged outlet condition shown in Figure 6, TW is greater than h_o and, thus, TW becomes DTW.

b. For the unsubmerged outlet shown in Figure 7, TW is less than h_o, so the h_o elevation becomes DTW.



Reference: USDOT,FHWA,HEC-5 (1965)

Figure 6- Tailwater for Submerged Outlet Conditions



Reference: USDOT, FHWA, HEC-5 (1965)

Figure 7- Tailwater for Unsubmerged Outlet Conditions

Headwater Depth:

Having established the total head loss (H) and the design tailwater depth (DTW), the headwater depth (HW) can be computed as:

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

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 and may be used directly in Equation 2.

The headwater elevation for outlet control is determined 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, backwater calculations may be necessary to determine the outlet velocity. These calculations begin at the culvert entrance and proceed downstream to the exit. The flow velocity is obtained from the flow and the cross sectional area at the exit:

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

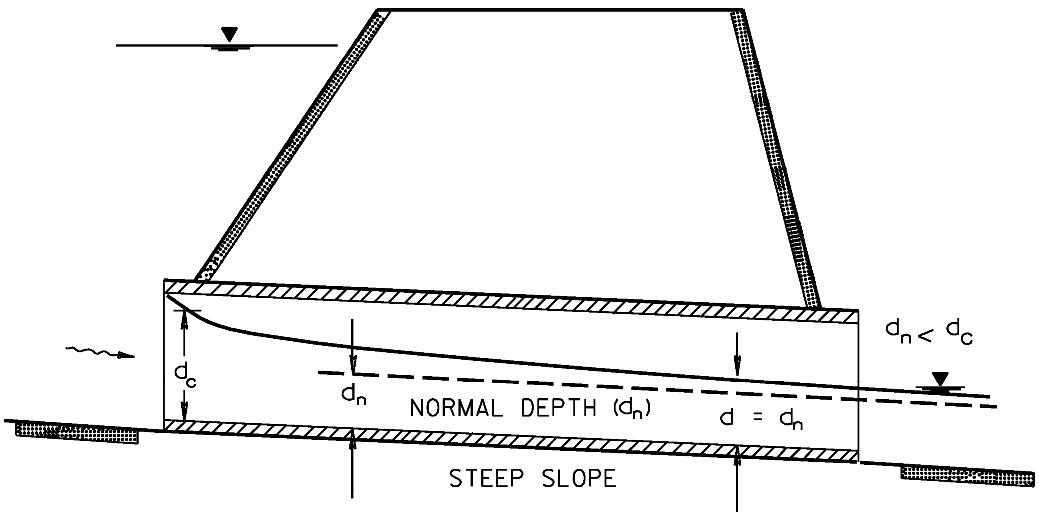
where:

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.

An approximation may be used to avoid backwater calculations in determining outlet velocity. Since the water surface profile converges toward normal depth as calculations proceed downstream, the normal depth can be assumed and used to define the area of flow at the outlet. The normal depth obtained can then be used to determine the outlet velocity (See Figure 8). The velocity obtained may be higher than the actual velocity at the outlet.

Normal depth may be calculated using a trial and error solution of the Manning equation. The known inputs are barrel resistance, slope, and geometry. The area of flow prism is then determined based on the culvert barrel geometry and depth equal to normal depth. Normal depth and area of flow may also be determined using the charts for various pipe cross section shapes in Appendix B.

Example 4 illustrates computing outlet velocity for inlet control.



$$V_{outlet} = \frac{Q}{A_p}; \; A_p = \text{AREA OF FLOW PRISM BASED ON BARREL GEOMETRY AND DEPTH EQUAL TO NORMAL DEPTH}$$

Figure 8 - Outlet Velocity For Inlet Control

Example 4 - Computing Outlet Velocity

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

Q_{design} = 18 ft^3/s
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:

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

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

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

Example 4 - Computing Outlet Velocity for Inlet Control (Cont.)

Step 3: Using Figure 9 determine the area of flow for the design discharge using the following relationship:

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

1. Enter on Figure 9, 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 flow curve.
4. Project vertically down from the area of flow curve and read from the horizontal axis a value of 0.66 or 66% of value for full section.
5. A relationship can be made 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}:

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

End of Example 4

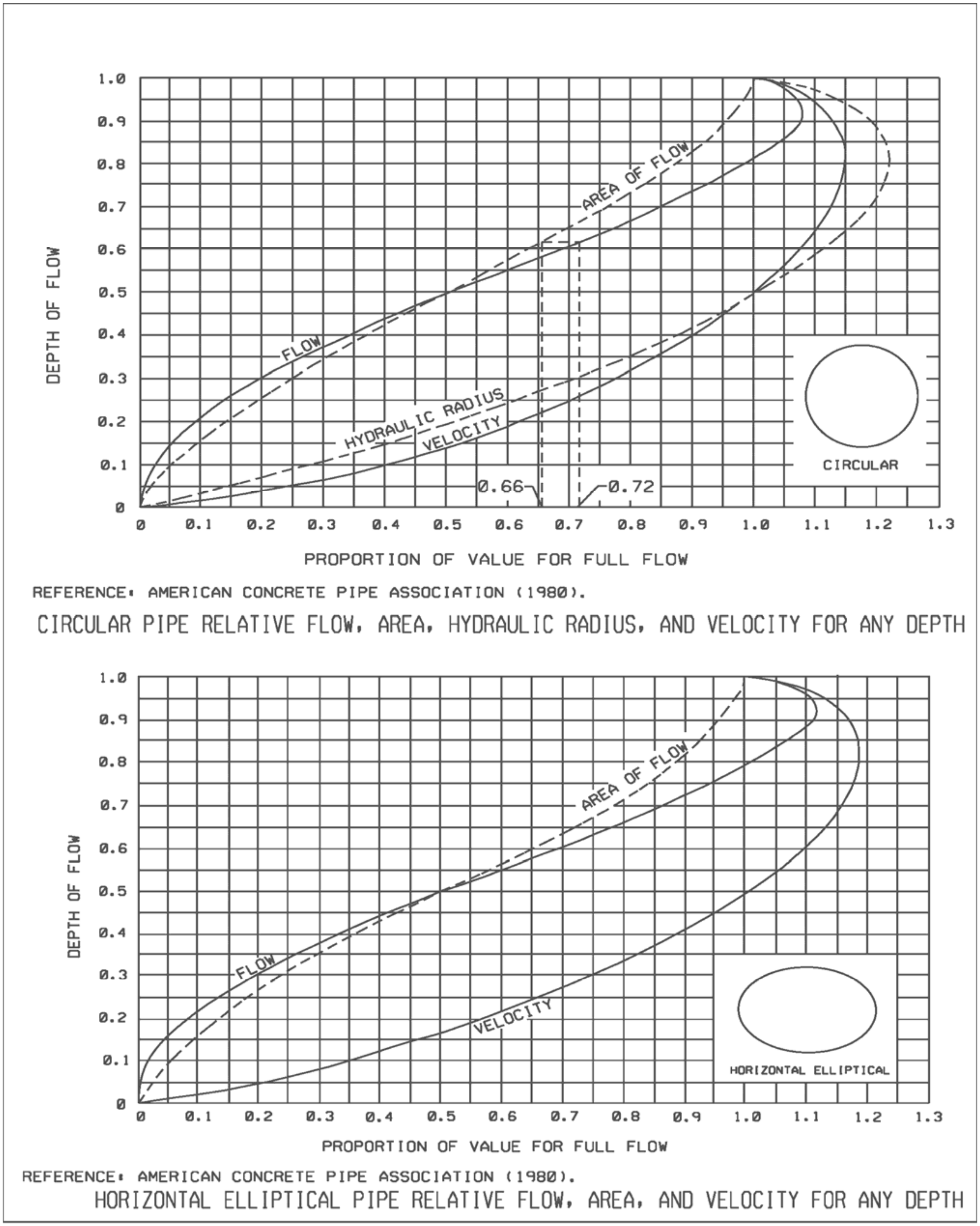


Figure 9 – Example 4

Outlet Control:

In outlet control, the cross sectional area of the flow (A_p) is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the culvert (See Figure 10).

Critical depth is used when the tailwater is less than critical depth and the tailwater depth is used 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.

$$V = \frac{Q}{A_p} \qquad \textbf{(Equation 4)}$$

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.

The area of flow prism based on barrel geometry and d, can be determined using the charts for various pipe cross section shapes in Appendix B.

Example 5 illustrates computing outlet velocity for outlet control.

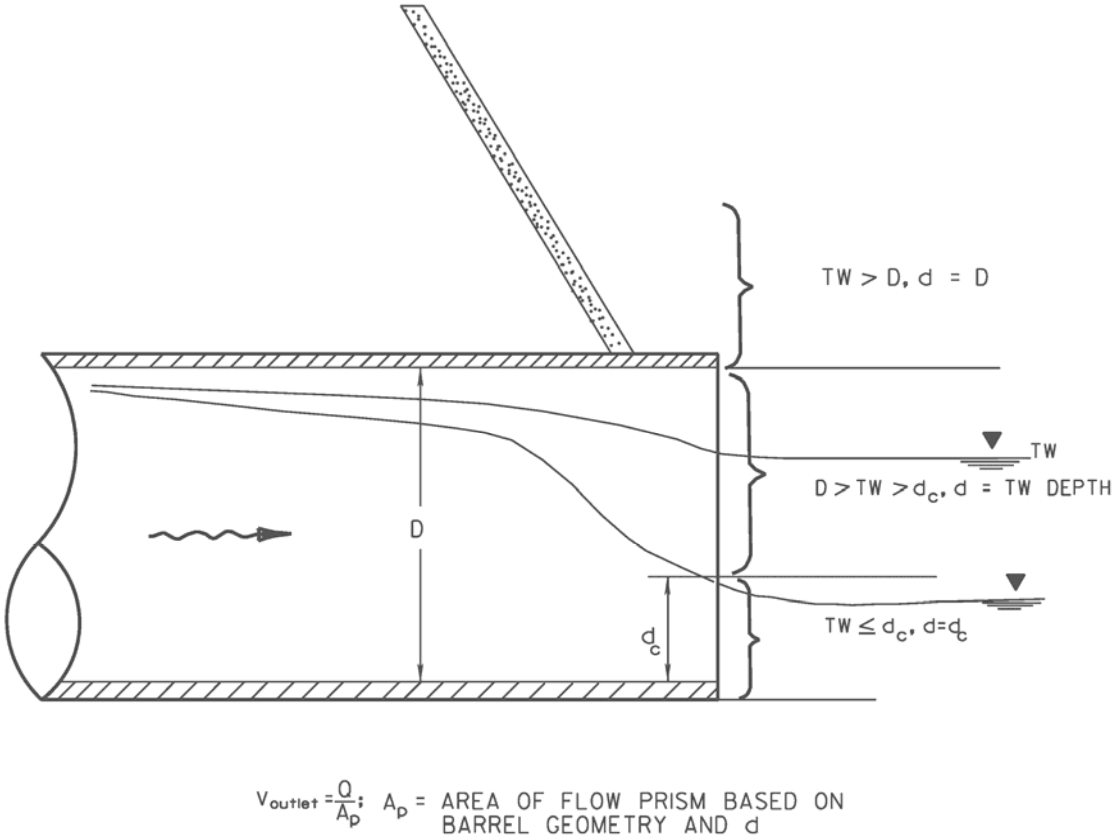


Figure 10 - Outlet Velocity For Outlet Control

Example 5 - Computing Outlet Velocity

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

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 11 determine the depth of flow to full depth flow (TW/D) or 2 ft./3 ft. = 0.67 or 67% of the full-full depth.

1. Enter on Figure 11, 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% for full section.

4. A relationship can be made between the full flow area and the normal depth area (A_{Design}):

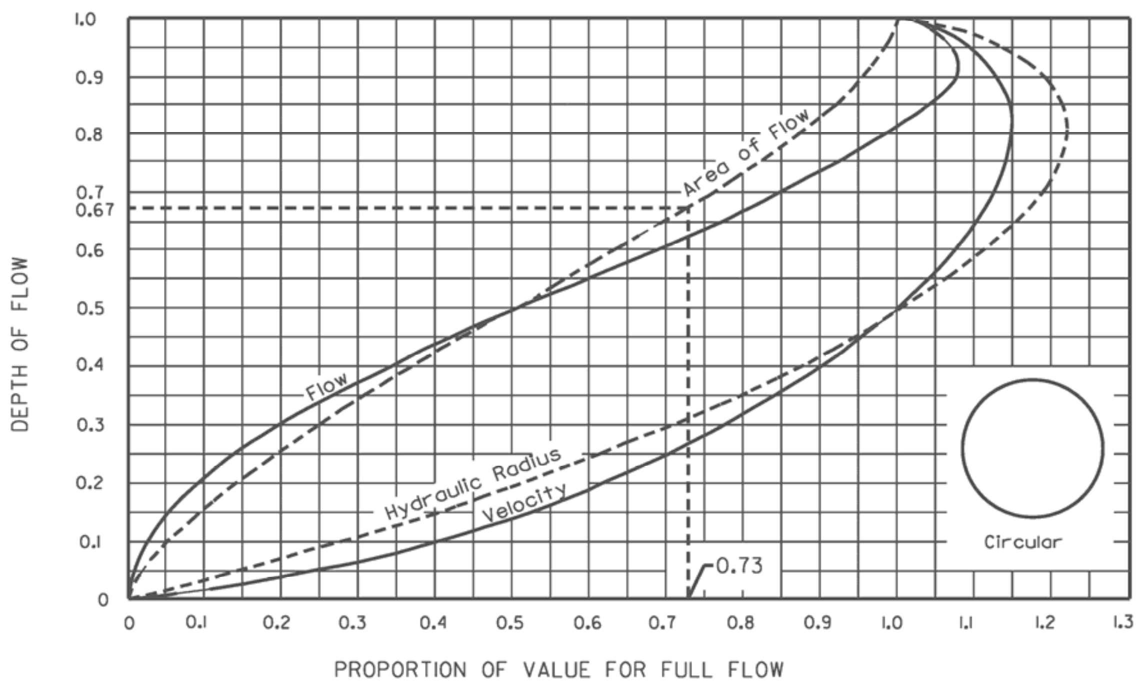
$$\frac{A_{Design}}{A_{Full}} = 0.73\ ;\ A_{Design} = 0.73 \times 7.07\ \ ft^2 = 5.61\ \ ft^2$$

Example 5 - Computing Outlet Velocity for Outlet Control (Cont.)

Step 4: Determine the outlet velocity using Q $_{Design}$ and A $_{Design}$:

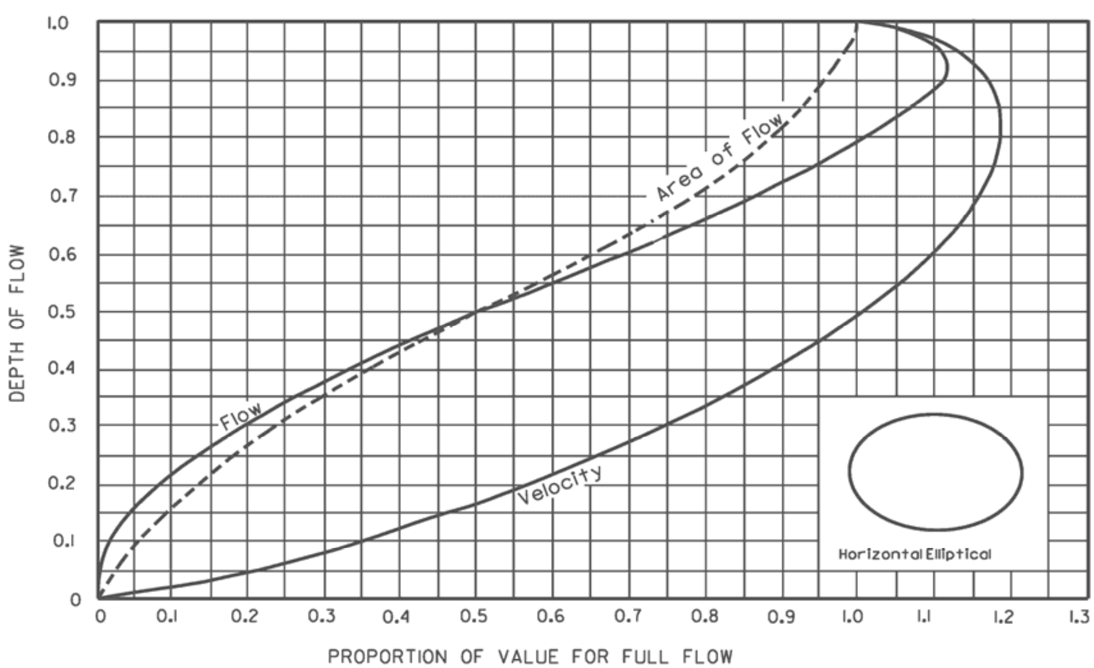
$$V_{Design} = Q_{Design} \,/\, A_{Design} = \frac{18\ ft^3/s}{5.61\ ft^2} = 3.2\ ft/s$$

End of Example 5



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 11 – Example 5

Culvert Capacity Calculations

a. Worksheet for manual calculations

A worksheet for doing culvert capacity calculations is presented in FHWA's HDS-5.

b. Computer Programs

FHWA's HY-8 Computer Program is only one of several programs which is capable of culvert capacity calculations. The computer program has been accepted for use by the Florida Department of Transportation and is available through the University of Florida McTrans Center. The District Drainage Engineer should approve other computer programs before using them in the design of a department project.

Chapter 7

Specific Standards Relating To All Cross Drains Except Bridges

7.1 Culvert Materials

Refer to Chapter 6 of the Drainage Manual when optional materials are considered.

Culvert linings are expected to last for the design service life (DSL) of the culvert.

When the vertical distance from invert to roadway is limited, arch culverts may be appropriate. When the rise of a culvert exceeds 4 feet, box culverts should be considered since they may offer cost advantages.

7.2 Scour Estimates

Scour prediction at culvert outlets is dependent upon the following characteristics:

a. Channel bed and bank material.
b. Velocity and depth of flow in the channel and at the culvert outlet.
c. Velocity distribution.
d. Amount of sediment and other debris in the flow.
e. 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.

Scour developed at the outlet of similar existing culverts is always a good guide in estimating potential scour at the outlet of proposed 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 which will cause scour do not occur at all flow rates. At locations where scour is expected 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, use of simple outlet treatment such as aprons of concrete, or riprap will provide adequate protection against scour. At other locations, use of 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, consideration should be given to energy dissipation devices such as those shown in the Department's Design Standards.

Chapter 8

Recommended Design Procedure

The following procedures will normally result in acceptable cost effective designs; however, their use does not exempt the engineer from developing an appropriate design. Further, the engineer is responsible for identifying which standards are not applicable to a particular design, and to obtain variances as necessary to achieve proper design.

The design procedures below do not account for structures within regulatory floodways; therefore, some deviation from these procedures may be necessary to satisfy regulatory agencies. The engineer must evaluate and determine the level of effort needed to produce an acceptable design.

Design procedures for 3 categories of cross drains are provided below. The 3 categories are:

1. Culvert Extensions (including side drain pipes)
2. Small Cross Drains (up thru 48")
3. Large Cross Drains (over 48" and less than 20' bridge)

8.1 Culvert Extensions

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

- Conduct Field Review to evaluate condition/adequacy of existing culvert. Review for condition, signs of scour, sedimentation. Check the available right of way to see if there’s room to transition ditches to meet the culvert extension. A review checklist (see the following possible format) can be used 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 R/W* *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, existing culvert may be extended. 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, the crown of the pipe at the outlet may be assumed.

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

  d. Documentation as required in the Drainage Manual.

- General Concerns

  There should be enough right of way 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 Design Standards 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 Standards, a sharper transition may be designed, but the need for channel lining to prevent erosion of the ditch side slopes should be evaluated.

  Example 6 illustrates this method.

Example 6: Culvert Extension

Existing: 2 lane rural road
ADT = 2000
36-inch diameter round concrete pipe (R.C.P)
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 a discussion with him 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 field review to evaluate condition/adequacy of existing culvert. Review for condition, signs of scour, sedimentation.

  We performed a field review with Mr. Smith on November 21, 1993. From our review, 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. It is recommended the existing 36 inch R.C.P., be extended 4 feet in both directions with 36 inch R.C.P. and straight endwalls (Index 250). The proposed flow line elevations are as follows:

  Flow line (Upstream) = 100.1 feet
  Flow line (Downstream) = 99.7 feet

Example 6: Culvert Extension (Cont.)

a. Estimate discharges as follows:

Area of 36 inch R.C.P. = $(\pi D^2)/4 = (\pi(36 \text{ inch}/12)^2)/4 = 7.07 \text{ ft}^2$

Q(25) = AV = 7.07 ft^2 x 6 ft/s = 42 ft^3/s
Q(100) = 1.4 x Q(25) = 59 ft^3/s
Q(500) = 1.7 x Q(100) = 100 ft^3/s

Since this roadway has an ADT > 1500, 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 1 is appropriate. For this example the Q (50) is 50 ft^3/s.

b. Estimate tailwater as discussed in the Open Channel Handbook 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) = 2.7 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, an analysis for the other frequencies would also have been computed. The analysis is for the proposed conditions. Figure 12 summarizes the following calculations.

HYDROLOGIC AND CHANNEL INFORMATION

D = Diameter or Height
B = Span

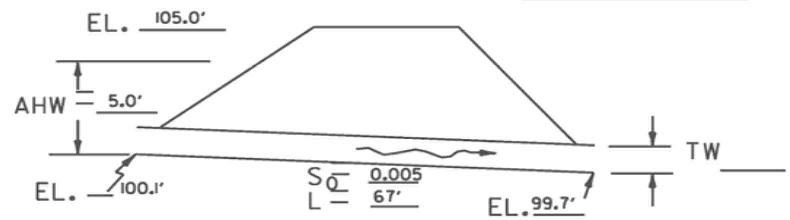
Q_1 = ______ TW_1 = ______

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	$\frac{Q}{B}$	$\frac{HW}{D}$	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 12 - Culvert Capacity Worksheet for Example 6

Example 6: Culvert Extension (Cont.)

- Inlet Control

Nomographs:

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

The headwater elevation is determined 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 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 B, based on the structure having a standard endwall treatment.

Critical Depth (d_c):

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

Equivalent Hydraulic Elevation (h_o):

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

Example 6: Culvert Extension (Cont.)

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, the headwater depth (HW) can be computed as:

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

The headwater elevation is determined 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 culvert for this type of problem does not need to be computed since the discharges were estimated using a 25-year velocity of 6 fps.

d. Documentation as required in the Drainage Manual.

End of Example 6

- 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 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. Documentation as required in the Drainage Manual.

- General

  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 7 illustrates this procedure.

Example 7: Culvert Extension

Existing: 2 lane rural road
ADT = 2000
2 foot x 2 foot concrete box culvert cross drain
Length of pipe is 50 feet
Straight endwalls

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. From a discussion with him we found that there has been has been history of overtopping of the roadway.

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

  We performed a field review with Mr. Smith on November 21, 1993. From our review, 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 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)

Example 7: Culvert Extension (Cont.)

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

$Q(50) = 35\ ft^3/s$
$Q(100) = 52\ ft^3/s$
$Q(500) = 88\ ft^3/s$

b. Determine tailwater as discussed in the Open Channel Handbook 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 a TW (50 year) = 2.5 ft.

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

   For this example only a hydraulic analysis for the 50-year frequency will be computed. The other frequencies would need to be analyzed for an actual project. The analysis is for the existing conditions. Figure 13 summarizes the following calculations.

   - Inlet Control

     Nomographs:

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

     The headwater elevation is determined by, taking the culvert invert at the entrance and adding the headwater depth:

     HW Elevation = 100.0 ft + 4.8 ft = 104.8 ft

Example 7: Culvert Extension (Cont.)

- Outlet Control

Nomographs:

Using Chart 15 in 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 B, 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.

Example 7: Culvert Extension (Cont.)

Headwater Depth (HW):

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

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

The headwater elevation is determined 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 chapter.

HYDROLOGIC AND CHANNEL INFORMATION

D = Diameter or Height
B = Span

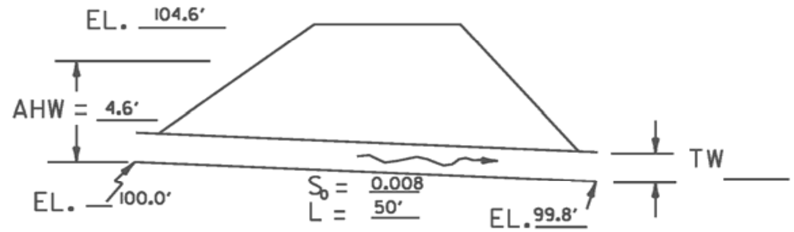
Q_1 = ________ TW_1 = ________

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 13 - Worksheet for Example 7

Example 7: Culvert Extension (Cont.)

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

   Review of Figure 13 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 8.2 of this handbook could be used. Example 8 illustrates this using the information from this example.

e. Documentation as required in the Drainage Manual.

End of Example 7

8.2 Small Cross Drains (Area of opening up through 48" diameter round culvert or equivalent)

- 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

- 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 4 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.

- If the trial selected size does not satisfy all design standards, by trial and error, determine the most economical culvert size that satisfies all standards, or if appropriate, obtain a variance.

- Documentation as required in the Drainage Manual.

Example 8 illustrates this procedure.

Example 8: Design of Small Cross Drain

Referring back to Example 7, it was determined that the 2-foot x 2-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 2-foot x 2-foot concrete box culvert was 50 feet. However, since the structure will have to be extended 4 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 7:

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

- 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

Example 8: Design of Small Cross Drain (Cont.)

- Conduct hydraulic analysis using FHWA HDS #5 procedures.

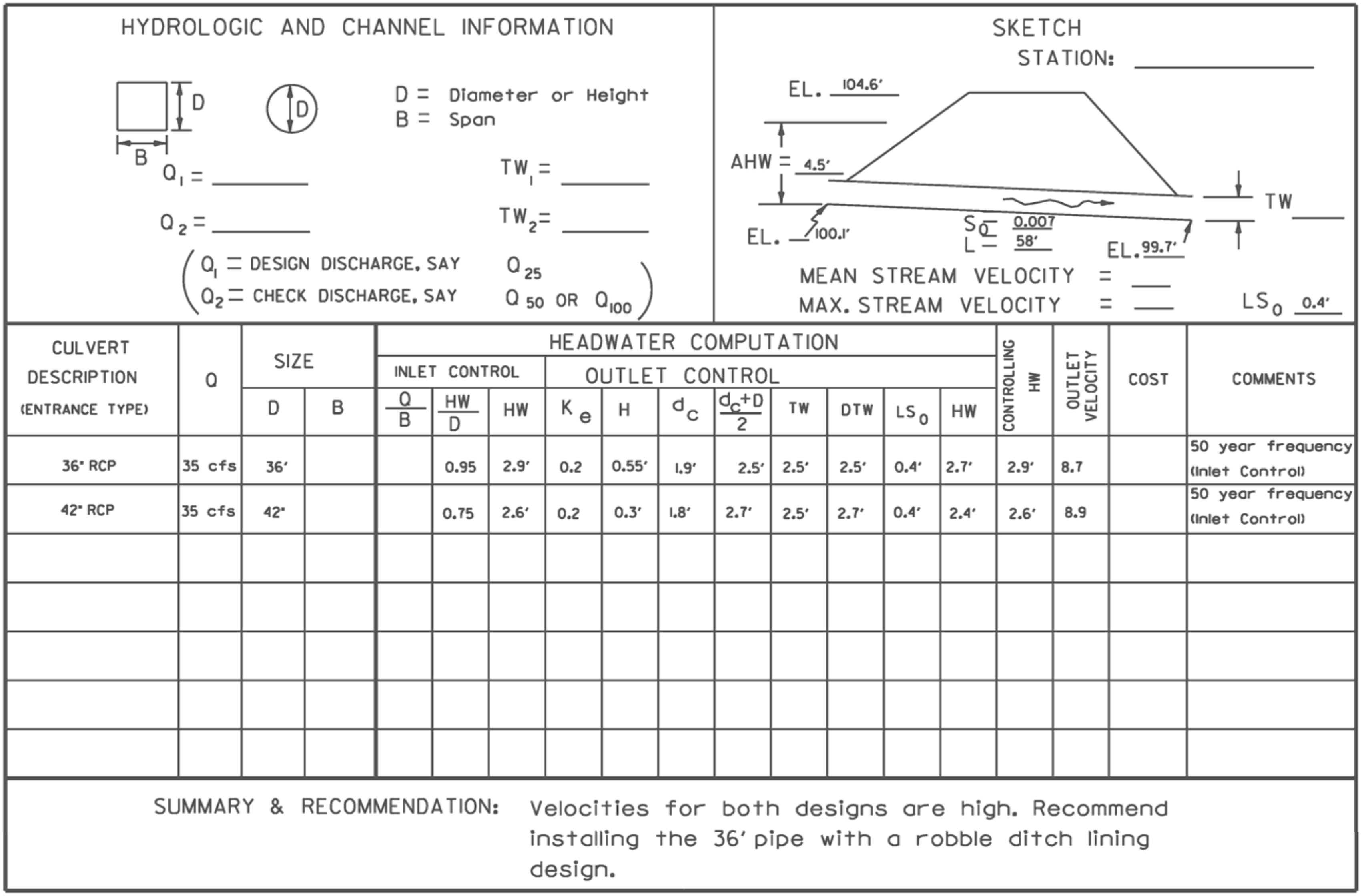
  The hydraulic analysis would be similar to what was done in Example 6 and 7. A worksheet of the calculations for the 50-year frequency is shown in Figure 14. The other frequencies would need to be analyzed for an actual project. The analysis shown in Figure 14 is for the proposed conditions.

- Check hydraulic results against design standards.

  Review of the worksheet in Figure 14 indicates that the roadway is 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 6 feet per second.

- If design does not meet standards or if there can be a more economical culvert used that satisfies the standards then perform new computations for that design.

- Documentation 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 D	SIZE B	INLET CONTROL Q/B	INLET CONTROL HW/D	INLET CONTROL HW	OUTLET CONTROL K_e	OUTLET CONTROL H	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL TW	OUTLET CONTROL DTW	OUTLET CONTROL LS_0	OUTLET CONTROL HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
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 14 - Worksheet for Example 8

8.3 Large Cross Drains (Area of opening greater than a 48 inch diameter pipe and less than a 20 feet 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)

- Remaining steps are as identified in Section 8.2 for small cross drains.

Appendix A

Risk Evaluations

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A.1 Risk Evaluation

All designs with flood plain encroachments should include an evaluation of the inherent flood-related risks to the highway facility and to the surrounding property. In the traditional design process, the level of risk is seldom quantified, but is instead implied through the application of predetermined design standards. For example, the design frequency, backwater limitations, and limiting velocity are parameters for which design standards can be set.

Two other approaches, however, are available that quantify risk on projects that involve highway facilities designed to encroach within the limits of a flood plain. These are risk assessment and economic analysis.

Consideration of capital costs and the risks should include, as appropriate, a risk analysis or risk assessment which includes:

The overtopping flood or the base flood, whichever is greater, or the greatest flood which must pass through the highway drainage structure(s), where overtopping is not practicable.

A.1.1 Risk Assessment

A risk assessment is a subjective analysis of the risks engendered by various design alternatives, without detailed quantification of flood risks and losses. It may consist of developing the construction costs for each alternative, and subjectively comparing the risks associated with each alternative. A risk assessment is usually more appropriate for small structures, or for structures whose size is highly influenced by non-hydraulic constraints. There are no well-defined procedures or criteria for performing risk assessments. However, an attempt should be made to screen projects and determine the level of analysis required. Some of the items to consider:

- Backwater

    a. Is the overtopping flood greater than the 100-year flood?

    b. Is the overtopping flood greater than the 500-year flood?

    c. Is there potential for major flood damage for the overtopping flood?

    d. Could flood damage occur even if the roadway crossing wasn't there?

e. Could flood damage be significantly increased by the backwater caused by the proposed structure?

f. Could flood damage occur to offsite property owners?

- Traffic Related Losses

  a. If the design flood is exceeded and the roadway is overtopped, is there a detour available?

- Roadway and/or Structure Repair Costs

  a. Is the overtopping flood greater or less than a 100-year flood?

  b. Is the embankment constructed from erosion resistant material such as a clay type soil?

  c. Does the embankment have good erosion resistant vegetation cover?

  d. How long will the duration of overtopping be?

  e. Will the cost of protecting the roadway and/or structure from damage exceed the cost of providing a relief structure?

  f. Is there damage potential to the structure caused by scour, debris, or other means during the lesser of the overtopping flood or the 100-year flood?

If the risk assessment indicates the risks warrant additional study, a detailed analysis of alternative designs (Economic Analysis) is necessary in order to determine the design with the least total expected cost (LTEC) to the public.

A.1.2 Economic Analysis

An economic analysis (sometimes called risk analysis) encompasses a complete evaluation of all quantifiable flood losses and the costs associated with them for each structure alternative. This can include damage to structures, embankments, surrounding property, traffic-related losses, and scour or stream channel change.

The level of expense and effort required for an economic analysis is considerably higher than for a risk assessment, and selection of the process to be used should be based on the size of the project and the potential risk involved.

Addition items for economic consideration include the following:

- Sections of metal pipe may be banded together offsite and laid in large sections to minimize disruption to traffic.
- Metal pipe may be cantilevered, eliminating the need for end treatment, and survive erosion at an outfall.
- When replacing an existing bridge with a culvert, consider pulling metal culverts under the existing bridge to reduce MOT efforts.
- Use the minimum number of culverts to reduce costs and minimize debris problems; i.e., use a single 72” pipe rather than 2 - 60” pipes.
- Keep box culvert dimensions at 12 ft. or less to avoid the need for a special design (structure standards are available up to 12 ft.).
- Maximize box culvert heights, targeting the difference between the waterway flow line and the DHW elevation before adding increased span width. Increases in span width increase reinforcing steel to support the span.
- MOT considerations may override economics and should be considered when selecting between a bridge, box culvert, and a pipe culvert.

Further details of the economic analysis process and procedures for using it have been documented in HEC-17 (USDOT, FHWA, 1981). The full-scale detailed risk analysis described in HEC-17 would not be necessary for normal stream crossings, but would apply to unusual, complex, or high-cost encroachments involving substantial flood losses.

An example of a simple risk analysis follows in Example A-1.

Example A-1 Sample Risk Analysis

Alternates considered:

Alternate 1: Extend existing double 10' x 4' CBC (60' total length) with no change to road. Overtops at about a 17 year frequency; flooding at the site has not caused any accidents.

Alternate 2: New quad 10' x 5' CBC (60' total length). Raise road to meet FDOT 50 year HW criteria and closely match existing 100 year HW. Overtops at frequencies greater than 50 year.

Alternate 3: Bridge

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)			
Annual Risks Costs $			
Total Costs			

Calculations for Alternate 1:

Capital Costs (Quantities from the Department's Culvert Design Program)

Extend 20' right	Concrete –	43.1 CY	Steel –	6622 lbs
Extend 8' left		23.5 CY		3283 lbs
Total quantity	Concrete	66.6 CY	Steel	9905 lbs

Unit prices $477 / CY $0.53 / lb

Total capital cost = $37,018 = $31,768 + $5,250

To convert to annual capital cost – use capital recovery factor (CRF) based on a discount rate of 7% and a 20-year design life.

$$CRF = \frac{i}{1-(1+i)^{-n}}$$

where n = 20 and I = 0.07

Annual capital costs = $37,018 x 0.0944 = $3,494

Additional Economic Costs

The following is an estimation of the additional losses associated with extending the existing culvert and allowing the road to overtop. The losses usually consist of embankment (and pavement), backwater, and traffic.

There are not expected to be any embankment losses. The existing road and culvert overtop and there is no history of embankment or pavement loss.

There will not be any additional backwater losses compared to Alternate 2. Both Alternate 1 and Alternate 2 have essentially the same backwater characteristics.

There may be additional traffic losses associated with Alternate 1 when compared with Alternate 2, which would raise the road to reduce overtopping potential. Traffic related costs consist of running time costs, lost time costs, and accident costs. Running time costs were estimated, lost time costs were ignored (detour length added only 1 mile to the travel distance), and accident costs were estimated but were found to be insignificant.

Assume traffic would have to be detoured:

1 day for 25-year storm (roadway tops at about a 17 year event)
2 days for 50-year storm
3 days for 100-year storm
4 days for 200-year storm.

The additional detour distance = 0.5 mile on 2-lane undivided and 0.5 mile on 4-lane divided

<u>Additional running costs</u> = Cost per mile x ADT x additional detour length (miles)
Assume cost per mile = $0.35 / mile

$\$_{25\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 1 day = $9,538.

$\$_{50\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 2 days = $19,075

$\$_{100\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 3 days = $28,615

$\$_{200\ yr}$ = $0.35 x 27250 vpd x 1.0 mi x 4 days = $38, 150

<u>Additional accident costs</u>: these are additional costs due to increased travel distance due to detour.

Additional detour length is 0.5 mi on 2-lane undivided and 0.5 mi on 4-lane divided.

Accident cost = crash rate x vehicle miles x cost per crash

Vehicle miles = ADT x additional detour distance x number of days of detour

Get the crash rate and the cost per crash from the safety office.

Crash rate =	1.9 crashes / million vehicle miles for urban 2-lane, undivided 0.8 crashes / million vehicle miles for urban 4-lane, divided
Cost per crash =	\$28,000 for urban 2-lane, undivided \$26,000 for urban 4-lane divided

$\$_{25}$ = (\$28,000 x [27,250 x 0.5 x 1] x 1.9) + (\$26,000 x [27,250 x 0.5 x 1] x 0.8)
$\$_{25}$ = 1008.25

Using same method, with 50 year detour = 2 days, 100 year detour = 3 days, and 100 year detour = 4 days:

$\$_{50}$ = 2016.50
$\$_{100}$ = 3024.75
$\$_{200}$ = 4033.00

Traffic losses in the following table are the sum of increased running costs and increased accident losses.

Summary of Economic Losses

Frequency (yr)	Losses ($)			Total Losses ($)
	Embankment & Pavement	Backwater	Traffic	
5	0	0	0	0
10	0	0	0	0
15	0	0	0	0
25	0	0	9538 + 1008.25 = 10,546.25	10, 546.25
50	0	0	21,091.50	21,091.50
100	0	0	31,639.75	31,639.75
200	0	0	42,183.00	42,183.00

Summary of Annual Risk Costs

Freq. (yr)	Exceed. Prob.	Losses ($)	Average Loss ($)	Delta Prob.	Annual Risk Costs ($)
5	0.2	0			
10	0.1	0			
15	0.07	0			
			5,273.13	0.03	158.19
25	0.04	10,546.25			
			15,818.88	0.02	316.38
50	0.02	21,091.50			
			26,365.63	0.01	263.66
100	0.01	31,639.75			
			36,911.38	0.005	184.56
200	0.005	42,183.00			
			42,183.00	0.005	210.92
	0	42,183.00			
Total Annual Risk Costs					1,133.71

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)	3,494		
Annual Risks Costs $	1,134		
Total Costs	4,628		

Alternate 2: Replace with quad 10' x 5' CBC

Capital Costs include:

Concrete (from box culvert program) = 219.7 cy @ $477/cy = 104,797
Steel (from box culvert program) = 42,251# @ $0.53/# = 22,393

Rebuild 400' of Roadway

Structural Course (2' x 24') = 1067 sy @ $3.40/sy = 3,628
Base group 9 = 1067 sy @ $6.16/sy = 6,573
Neglect earthwork costs

Total Capital Costs = $137,391

Annual Capital Cost = Total x CRF = $12,970

This alternate would overtop at frequencies greater than 50 year and would, therefore, have some annual risk costs. These were not calculated because the annual cost alone is greater than the total cost for alternate 1. If the capital costs for alternate 2 were less than the total cost for alternate 1, it would be necessary to calculate the other costs associated with this alternate.

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)	3,494	12,970	
Annual Risks Costs $	1,134	>0	
Total Costs	4,628	>12,970	

Alternate 3: 57' long x 44' wide flat slab bridge

Capital Costs:

57' x 44' x $40 / sf = 2508 sf x $40/sf = $100,320

Annual cost using CRF = 0.0944 = $9,470.

Costs not estimated:

Roadway fill and new base and asphalt. At a minimum 900' of roadway would have to be rebuilt to raise grade to meet the bridge. (Bridge would be raised to meet FDOT drift clearance requirements.)

Standard 1:2 front slopes encroach into roadside ditches. Since the upstream roadside ditch conveys substantial flow, it may not be possible or wise to reduce it's capacity. Vertical walls and/or additional right of way may be necessary.

Miscellaneous factors include driveway connections within the raised roadway section, and the aesthetics of the raised road and bridge.

	Alternate 1	Alternate 2	Alternate 3
Annual Capital Costs $ (i.e. Construction Costs)	3,494	12,970	9,740
Annual Risks Costs $	1,134	>0	>0
Total Costs	4,628	>12,970	>9,740

Alternate 1 is the most economical alternate and the most desirable when considering other impacts.

Appendix B

Reference Material

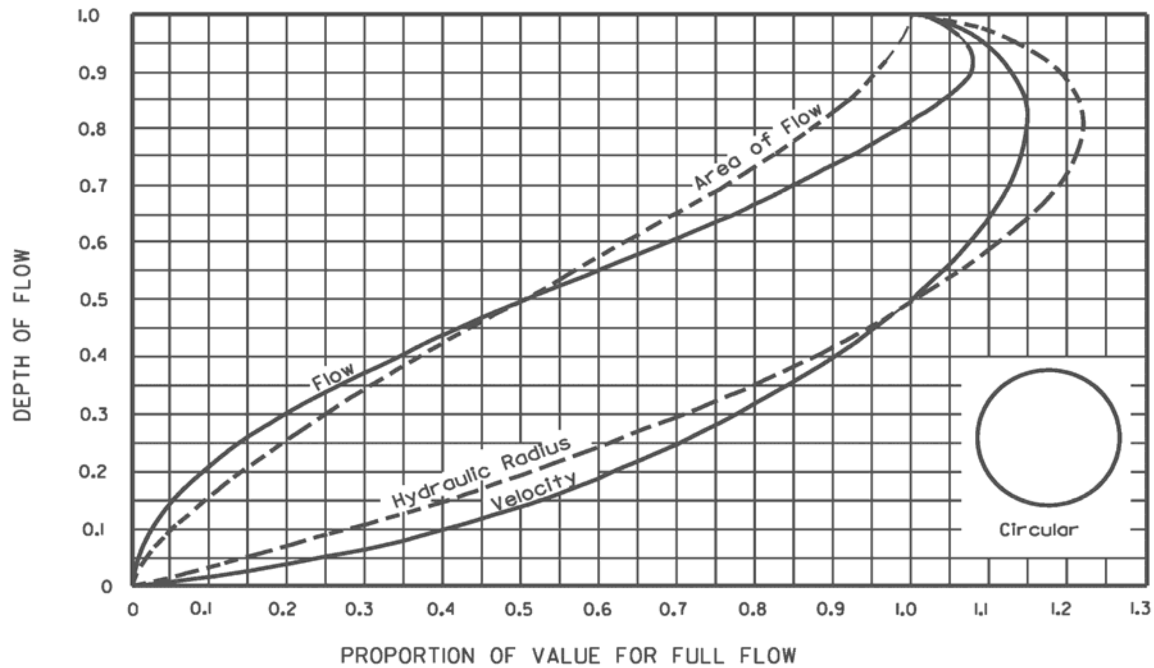

FIGURE B.1

Reference: American Concrete Pipe Association (1980).

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

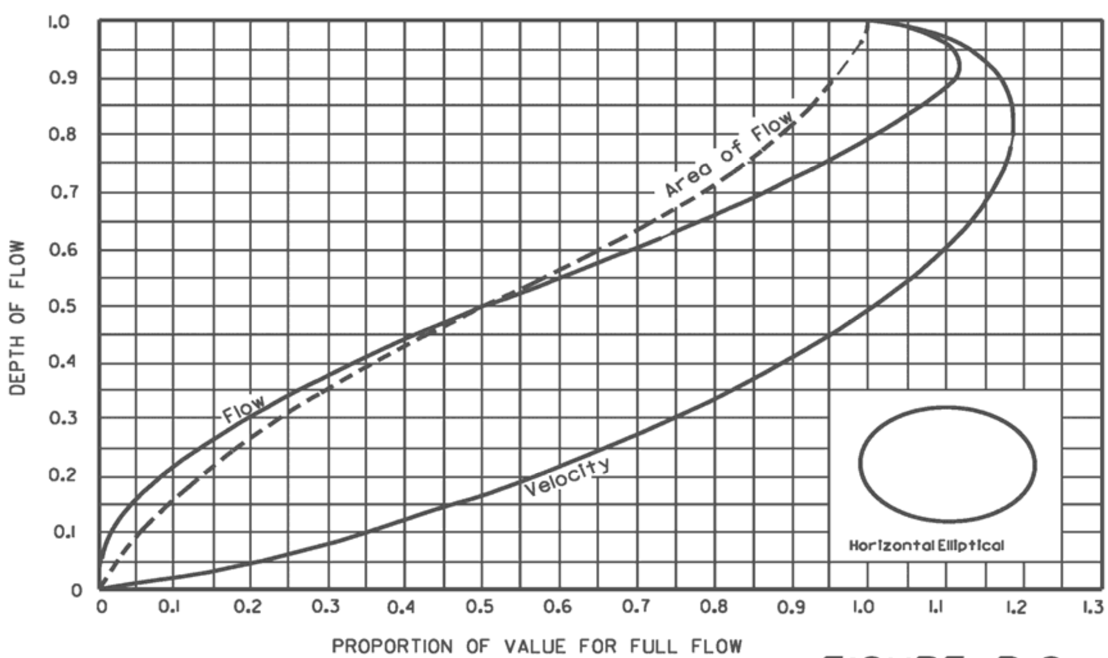

FIGURE B.2

Reference: American Concrete Pipe Association (1980).

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

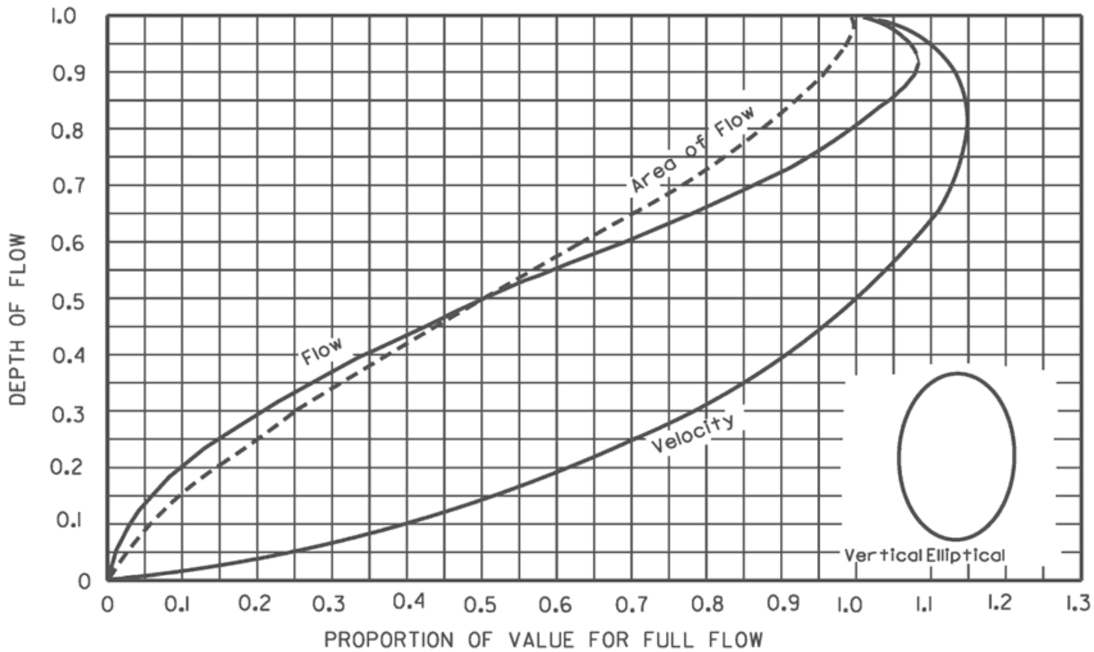

FIGURE B.3

Reference: American Concrete Pipe Association (1980).

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

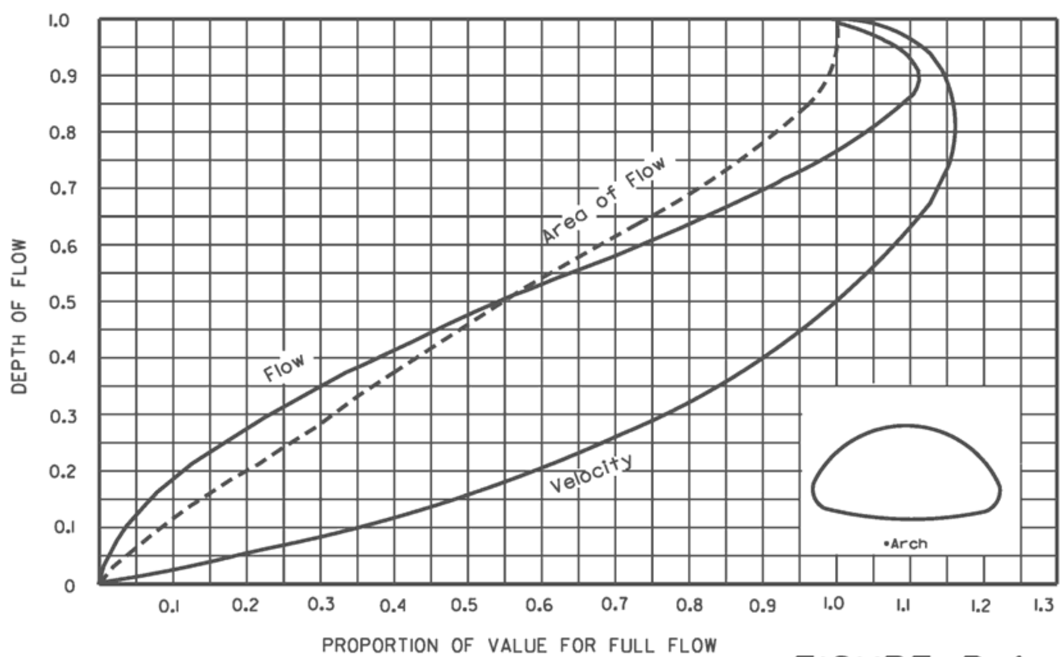

FIGURE B.4

Reference: American Concrete Pipe Association (1980).

Pipe-Arch Relative Flow, Area, and Velocity for Any Depth

APPLICATION GUIDELINES FOR PIPE END TREATMENTS (See page B-6 for notes)								
Index No.	Description		Application			Inlet End		
	Type	Pipe Size	Cross Drain	Side Drain	Median	Application	Hydraulic Performance	K_e
250	Straight Concrete	Single & Multiple 15" - 54"	(a) Yes	No	Limited	Yes	Excellent	0.2
251	Straight Concrete	Single & Double 60"	Yes	No	Limited	Yes	Excellent	0.2
252	Straight Concrete	Single & Double 66"	Yes	No	Limited	Yes	Excellent	0.2
253	Straight Concrete	Single & Double 72"	Yes	No	Limited	Yes	Excellent	0.2
255	Straight Concrete	Single 84"	Yes	No	Limited	Yes	Excellent	0.2
258	Straight Sand Cement	Single & Multiple 18" - 84"	(b) Limited	No	Limited	Yes	Very Good	0.3
260	U Type Concrete With Grate	Single 15" Thru 30"	Limited	No	Yes	Yes	Fair	0.7
261	U Type Concrete	Single 15" Thru 30"	Limited	No	Yes	Limited	Good	0.5 - 0.7
264	Concrete Energy Dissipater	Single 30" thru 72"	Limited	No	No	No	NA	NA
266	Winged Concrete	Single 12" Thru 48"	Yes	No	Yes	Yes	Very Good	0.3
268	U Type Sand Cement	Single & Multiple 15" - 60"	(b) Limited	No	Limited	Yes	Good	0.5
270	Flared End Section Concrete	Single 12" Thru 72"	Yes	No	Yes	Yes	Good	0.5
272	Cross Drain Mitered End Section	Single & Multiple 15' Thru 72"	Yes	No	Yes	Yes	Fair	0.7
273	Side Drain Mitered End Section	Single & Multiple 15' Thru 60"	No	Yes	No	Yes	Fair	0.7 w/o 1.0 w/ grate

APPLICATION GUIDELINES FOR PIPE END TREATMENTS (See page B-6 for notes)							
Index No.	Description		Outlet End		Safety		Economic Rating
	Type	Pipe Size	Applicable	Erosion Tolerant	Permitted Location	Traffic-Safe Grate Available	
250	Straight Concrete	Single & Multiple 15" - 54"	Limited	Good	Outside CZ	No	Fair
251	Straight Concrete	Single & Double 60"	Limited	Good	Outside CZ	No	Fair
252	Straight Concrete	Single & Double 66"	Limited	Good	Outside CZ	No	Fair
253	Straight Concrete	Single & Double 72"	Limited	Good	Outside CZ	No	Fair
255	Straight Concrete	Single 84"	Limited	Good	Outside CZ	No	Fair
258	Straight Sand Cement	Single & Multiple 18" - 84"	Yes	Good	Outside CZ	No	Good
260	U Type Concrete With Grate	Single 15" Thru 30"	Yes	Very Good	Inside CZ	Required	Good
261	U Type Concrete	Single 15" Thru 30"	Yes	Good	Grate Required Inside CZ	Yes	Fair
264	Concrete Energy Dissipater	Single 30" thru 72"	Yes	Excellent	Outside CZ	No	NA
266	Winged Concrete	Single 12" Thru 48"	Yes	Good	Outside CZ	No	Good
268	U Type Sand Cement	Single & Multiple 15" - 60"	Yes	Very Good	Outside CZ	No	Very Good
270	Flared End Section Concrete	Single 12" Thru 72"	Yes	(c) Very Good	(c) Outside CZ	No	Very Good
272	Cross Drain Mitered End Section	Single & Multiple 15' Thru 72"	Yes	Good	(d) Outside CZ	No	Very Good
273	Side Drain Mitered End Section	Single & Multiple 15' Thru 60"	Yes	Good	(e) Inside CZ	Yes	Good

a) For back of sidewalk location see Index 282.
b) For temporary construction or use on a minor facility.
c) Construction of optional toewall and concrete jacket may be necessary. Flared end section sizes 12" and 15" may be located as close as 8' beyond the outside edge of the shoulder.
d) Mitered end section sizes 15", 18", and 24" may be located as close as 8' beyond the outside edge of the shoulder.
e) Mitered end section size 30" and larger require use of grate. Grate may be deleted if pipe is located outside CZ and is offset from approach ditch alignment.

NOTES:

1. All end treatments must be selected to satisfy hydraulic suitability with proper consideration given to safety and economics.
2. CZ denotes clear zone, formerly CRA denoting clear recovery area.
3. Grates should not be placed on outlet ends unless positive debris protection is provided at inlet end.
4. Additional notes concerning application restrictions may be shown on individual indexes.
5. Economic ratings are based on statewide average costs.
6. End treatments with a K_e of 0.5 or greater should be used only in areas of low design velocities and negligible debris.
7. Pipe sizes are circular, Class III B wall, concrete pipe. Elliptical pipe and corrugated pipe are to be checked for fit in accordance with Index No. 201; metal pipe sizes should be reviewed using 2 ⅔" x ½" corrugation up to 30" and 3" x 1" corrugation for larger sizes.