Chapter 8 - Culverts

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Chapter 8 - Culverts

8.1 Introduction

Culverts are usually defined as short conduits used to convey flow through a highway fill. The flow types that occur in culverts are many and varied. In this chapter, culvert design considerations are presented from the planning stage through the design stage. Design should consider hydraulic and structural capacity, erosion and debris control, environmental impacts, safety concerns, and legal aspects. Design concepts are covered and design procedures are summarized along with sample problems. Sources of additional information on culvert design are provided.

For additional details on culvert design, refer also to Chapter 15, Drainage Design Instructions (DDM 1).*

The FHWA web site for specific publications, http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm may be used for additional information on the following topics:

HDS-5, Hydraulic Design Of Highway Culverts

Note: An errata sheet for this publication is available at the above web site

HEC-9, Debris Control Structures,

HEC-13, Hydraulic Design of Improved Entrances for Culverts,

HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels,

HEC-20, Stream Stability At Highway Structures,

Note: An errata sheet for this publication is available at the above web site

HDS-6, River Engineering for Highway Encroachments – Highways in the River Environment,

8.1.1 Objective

The objective of this chapter is to provide the user with the information needed to select, plan, and design highway culverts based on VDOT methods. Using the information provided, the user will be able to design conventional culverts. The chapter also

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provides references, which will enable the user to apply special culvert designs for unusual circumstances.

8.2 Design Policy

8.2.1 Federal and VDOT Policy

The following policies will guide the selection, planning, and design of highway culverts:

- All culverts should be hydraulically designed
- The overtopping flood selected should be consistent with the class of highway and commensurate with the risks at the site
- Survey information should include topographic features, channel characteristics, aquatic life, highwater information, existing structures and other related site-specific information
- Culvert location in both plan and profile should be investigated to consider sediment build-up in the barrels, upstream, or downstream of the culvert
- The cost savings of multiple use culverts (utilities, stock and wildlife passage, land access and fish passage) should be weighed against the advantages of separate facilities
- Culverts should be designed to accommodate debris or proper provisions should be made for debris maintenance
- Material selection should consider service life, which includes abrasion and corrosion
- Culverts should be located and designed to present a minimum (reasonable) hazard to traffic and people
- The detail of documentation for each culvert site should be commensurate with the risk and importance of the structure.
- Where practicable, some means should be provided for personnel and equipment access to facilitate maintenance
- Culverts should be regularly inspected and maintained
- The impacts of the 100 yr storm should be evaluated on all culverts, regardless of the drainage area size[*]

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8.3 Design Criteria

Criteria for the planning and design of culverts are discussed in this section. These criteria should be considered for all culvert designs.

8.3.1 Site Criteria

The following criteria relate to site conditions, which affect the selection of a particular culvert type, geometry, and debris protection.

8.3.1.1 Structure Type Selection

Often, a choice must be made between a culvert and a bridge at a given site. In making that decision, the following criteria should be considered.

Culverts are used:

- Where bridges are not hydraulically required or feasible
- Where debris and ice are tolerable
- Where a culvert is more economical than a bridge
- Where environmentally acceptable

Bridges are used:

- Where culverts cannot be used
- Where a bridge is more economical than a culvert
- To satisfy land use requirements
- To mitigate environmental impacts caused by a culvert
- To avoid floodway encroachments
- To accommodate ice and large debris
- To avoid cost and impact of channel diversions necessary for culvert construction

8.3.1.2 Topography

The culvert length and slope should be chosen to fit the existing topography, to the degree practicable:

- The culvert invert should be aligned with the channel bottom and the skew angle of the stream, except in instances where countersinking one or more culvert barrels is needed to satisfy environmental requirements

8.3.1.3 Debris Control

Debris control should be designed using the FHWA's Hydraulic Engineering Circular No. 9, "Debris-Control Structures" and should consider:

- Where experience or physical evidence indicates the watercourse will transport a heavy volume of debris
- Culverts located in mountainous or steep regions of the state

- Culverts that are under high fills
- Where clean-out access is limited. (However, access must be available to clean out and otherwise maintain the debris control device)

8.3.2 Hydraulic Criteria

These criteria relate to the hydraulic design of culverts based on flood flows, upstream and downstream water surface elevations, allowable velocities, and flow routing.

8.3.2.1 Flood Frequency

Culverts should be designed to accommodate the following minimum flood frequencies where the primary concern is the maintenance of traffic flow and the convenience of the highway user:

Roadway	*Flood Frequency (Annual Risk)*
Interstate	50-year (2%)
Primary & Arterial	25-year (4%)
Secondary	10-year (10%)

The above requirements are minimum, and deviation requires approval from VDOT. Culverts should be designed to pass floods greater than those noted above where warranted by potential damage to adjacent property, loss of human life, injury, or heavy financial loss.

Future development of contributing watersheds and floodplains that have been zoned or delineated should be considered in determining the design flood. For the Interstate System, development during a period 20 years in the future should be considered. Adopted regional plans and approved zoning will be considered in determining the design discharge on all systems.

In compliance with the National Flood Insurance Program (NFIP) it is necessary to consider the 100-year frequency flood at all locations where construction will encroach on a floodplain. The 100-year floodplains and the 100-year water surface profiles are delineated and established for the Federal Emergency Management Agency (FEMA), the administrator of NFIP. The NFIP prohibits any construction in floodplains, when combined with all other existing and anticipated uses, which would increase the water surface elevation of the 100-year flood more than one foot at any given point.[*] It should be noted that it is VDOT's policy not to allow any increase in the level of the 100-year flood where actual flood elevations have been established and published. This

[*] *This value of 1.0 foot is to be considered maximum and local conditions may require a lesser value for the particular site'. Also see The Governor of Virginia's Policy Memorandum 3-78 (1)*

does not necessarily require that the culvert be sized to pass the 100-year flood, provided the capacity of the culvert plus flow bypassing the culvert is sufficient to accommodate the 100-year flood without raising the water surface elevation excessively. An example of this may be where the culvert is designed to keep the roadway free from inundation during the 10-year design storm and the road grade is held down such that the culvert capacity plus flow over the roadway is sufficient to pass the 100-year flood without raising the upstream water surface elevation. In instances where a delineated NFIP floodplain is involved (example: Zone AE), the same procedure used to establish the floodplain and water surface elevations (usually a step-backwater computer model such as HEC-RAS, HEC-2, ~~WSPRO,~~ etc.) should be used to design the proposed culvert. In instances where actual 100-year flood elevations have not been established (example: Zone A, Zone X, etc.), the procedure presented in this chapter may be employed to perform the hydraulic analysis. It will still be necessary to evaluate both existing and proposed conditions however to insure that the existing 100-year flood level won't be increased by more than 1.0 ft.

8.3.2.2 Allowable Headwater

The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood, measured from the culvert inlet invert. The headwater depth or elevation may also be limited by giving due consideration to inlet and outlet velocities and the following upstream water surface elevation controls:

- Not higher than an elevation that is 18 inches below the outer edge of the shoulder at its lowest point in the grade
- Upstream property damage
- Elevations established to delineate NFIP or other floodplain zoning
- HW/D is at least 1.0 and not to exceed 1.5 where HW is the headwater depth from the culvert inlet invert and D is the height of the barrel
- Low point in the road grade which is not necessarily at the culvert location
- Elevation of terrain or ditches that will permit flow to divert around the culvert

8.3.2.3 Review Headwater

The review headwater is the flood depth that:

- Does not increase the existing 100-year flood elevation in the NFIP mapped floodplains, or in the vicinity of insurable buildings
- Has a level of inundation that is tolerable to upstream property and roadway for the frequency of the check storm

8.3.2.4 Tailwater Relationship – Channel

When the tailwater relationship is developed for the receiving channel, the designer should:

- Evaluate the hydraulic conditions of the downstream channel to determine tailwater depths for a range of discharges which include the check storm (see Chapter 6, Hydrology)
- For minor drainage installations with a 100-year discharge of less than 500 cfs, calculate the tailwater using a single cross section analysis
- For sensitive locations and for major drainage installations with a 100-year discharge equal to or greater than 500 cfs, calculate the tailwater depth using step-backwater methods (such as HEC-RAS, etc.) or other step methods as appropriate. (Step-backwater methods yield the most accurate tailwaters)
- When using step-backwater methods to define barrel losses for subcritical flow in the culvert barrel, use critical depth at the culvert outlet if it is greater than the channel depth
- When using full flow nomographs to define barrel losses, use a calculated tailwater based on critical depth (d_c) and the height of the barrel (D) when that term [$TW = (d_c + D)/2$] is greater than the depth of flow in the outlet channel
- Use the headwater elevation of a downstream culvert if it is greater than the channel depth

8.3.2.5 Tailwater Relationship - Confluence or Large Water Body

When the tailwater relationship is developed from the confluence of a large body of water, the designer should:

- Use the highwater elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent)
- If events are statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth. Guidelines are provided in Joint Probability Analysis, Chapter 6, Appendix 6I.
- If tidal conditions are present at the site, use the mean high tide

8.3.2.6 Maximum Outlet Velocity

Our culvert outlet protection procedure[*] has emphasis on the existing soil type to: (1) insure protection of the downstream channel or swale where material or lining in the downstream channel or swale may be unstable (erodible) under the anticipated velocities exiting the culvert, and (2) insure protection of the culvert end by providing measures to prevent the formation of a scour hole at the culvert outlet.

The type of material in the swale/channel at culvert outlets will need to be determined based on observations or field borings secured in accordance with the guidance found in Drainage Design Memorandum 1 (DDM1), Drainage Design Instructions, located in Chapter 15 of the VDOT Drainage Manual. The allowable velocity for natural material can be found in the table shown in Appendix 7D-2 of the VDOT Drainage Manual. The

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guidelines and procedures presented herein shall be implemented on all VDOT projects and those which will ultimately come under Department jurisdiction .

Highlights of these procedures/details are:

1. Maintains current rip rap sizes for outlet velocities 8 fps and greater[*]

2. Establishes new riprap size for outlet velocities up to 8 fps

3. Allows the use of EC-3 Type B for velocities less than 6 fps

4. Maintains current apron dimensions for culvert installations with a total hydraulic opening of less than 7 square feet.

5. Increases apron length to five times the height of the culvert for culvert installations with a total hydraulic opening of 7 square feet or greater.

6. Evaluates need for outlet protection based on 2 year culvert outlet velocity and allowable velocity of material in outlet channel or swale

7. Evaluates type of outlet protection required based on culvert outlet velocity for design discharge

The objectives of these details/procedures are to:

1. Minimize impacts to right of way of easement areas at smaller culvert sites

2. Minimize length of stream impacts

3. Minimize need for outlet protection where channel/swale material will be stable for culvert outlet velocities

4. Provide alternative to riprap at sites with low outlet velocities

5. Satisfy DCR Minimum Standard 11

OUTLET PROTECTION DETAILS

- Dimensions Of Outlet Protection Apron:
    - Type A Installation – Minimum 3H Length & Minimum 3S Width
    - Type B Installation – Minimum 5H Length & Minimum 3S Width
        - Where: S = Span of Culvert

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H = Height of Culvert

- For a multiple culvert line installations the largest S and H, dimensions of the individual culvert lines should be used in determining the minimum apron length dimensions.

- Outlet Protection Material*

  - Standard EC-3 Type B
  - Class A1 – Class AI Dry Riprap
  - Class I – Class I Dry Riprap
  - Class II – Class II Dry Riprap

NEW OUTLET PROTECTION PROCEDURE

The following procedure shall be used to analyze the need for outlet protection on:

- All cross drain culverts
- All storm drain outlet pipes
- All entrance and crossover pipes with a diameter of 24” (or equivalent hydraulic opening) or greater

Step 1 - Determine if Culvert Outlet Protection is required for protection of swale or channel.

A. *Compute culvert outlet velocity for 2 year design flood.*

B. *Compare 2 year design flood culvert outlet velocity to allowable velocity for outlet swale/channel material or lining.*

   - *Swale/channel material type based on field borings/observations or proposed lining.*
   - *Allowable velocity for natural swale/channel material based on VDOT Drainage Manual Chapter 7 - Appendix 7D-2.*

C. *If two year design storm culvert outlet velocity is equal to or less than allowable velocity for swale/channel material, no Culvert Outlet Protection is required for swale/channel protection.*

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- *Go to Step 2.*

D. If two year design flood culvert outlet velocity is greater than allowable velocity for swale/channel material, Culvert Outlet Protection is required.

- *Go to Step 3[*].*

Step 2 - Determine Culvert Outlet Protection required for culvert end protection

A. Compute culvert outlet velocity for culvert design flood.

B. If culvert outlet velocity for culvert design flood is less than 6 fps, Culvert Outlet Protection is not required for culvert end protection.

- *Stop*

C. If culvert outlet velocity for design storm is 6 fps or greater, Culvert Outlet Protection is required for culvert end protection.

- *Go to Step 3.*

Step 3 – Determine Class of Culvert Outlet Protection to use.

A. When EC-1 Culvert Outlet Protection is required by either Step 1 or Step 2, EC-3 Type B or the Class of EC-1 to be specified shall be based on the culvert design storm outlet velocity with the following velocity limitations.

- *EC-3 Type B – maximum outlet velocity is 6 fps.*
- *EC-1 Class A1 – maximum outlet velocity is 8 fps.*
- *EC-1 Class I – maximum outlet velocity is 14 fps.*
- *EC-1 Class II – maximum outlet velocity is 19 fps.*
- *Use Special Design Culvert Outlet Protection for outlet velocity greater than 19 fps.*

- *Go to Step 4*

Step 4 - Determine Type of EC-1 Installation to use.

A. When Culvert Outlet Protection is required by either Step 1 or Step 2, specify the Type of Installation to use based on the total hydraulic opening of the culvert installation.

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- *Use Type A Installation for culvert installations with a total hydraulic opening of less than 7 square feet[*].*

- *Use Type B Installation for culvert installations with a total hydraulic opening of 7 square feet or greater.*

PLAN DESCRIPTION

- ____ Sq. Yds. (Tons) Standard EC-1 Class ____ Required Type _____ Installation

- ____ Sq. Yds. Standard EC-3 Type B Culvert Outlet Protection Required

Road and Bridge Standard drawings 114.01 and 114.03 and Road and Bridge Specification Sections 414 and 603 have been revised to incorporate these protection measure details.

8.3.2.7 Minimum Velocity

The minimum velocity in a culvert barrel should be adequate to prevent siltation at low flow rates. When the streambed material size is unknown, use three (3) feet per second.

8.3.2.8 Storage Routing - Temporary or Permanent

"It is VDOT practice to design culverts without recognizing or calculating the available upstream storage." This statement reflects the fact that the Department does not permit the consideration of any upstream floodplain storage, with the resultant attenuation of peak discharges, in the design of any culverts, bridges, or other drainage structures for either its own facilities or those that would ultimately come under its jurisdiction. The Department will permit such consideration where it can be clearly shown that the drainage structure and roadway embankment in question and the upstream floodplain area have been designed to function as an impoundment for such facilities as ponds, lakes, detention/retention basins, etc. Another exception would be where approved FEMA delineated floodplain studies are in effect which indicate that the peak discharges have been reduced due to consideration of upstream flood storage. VDOT's State Hydraulics Engineer must approve any exception to the above.

In the event of an approved exception, the following concerns should be addressed:

- Limit the total area of flooding
- Limit the average time that bankfull stage is exceeded for the design flood to 48 hours in rural areas or 6 hours in urban areas
- Ensure that the storage area will remain available for the life of the culvert through the purchase of additional right-of-way or easements
- Consider environmental impacts

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- Consider sediment deposition
- Consider the impacts of the delayed peak coinciding with downstream peak discharges

8.3.2.9 Roadway Overtopping

Roadway overtopping should not be allowed for discharges equal to or less than the design discharge for new culvert installations. Overtopping can occur and is sometimes encouraged, for higher discharges. Roadway overtopping may occur:

- When evaluating existing culvert installations for current design flows
- When performing check flood calculations
- When the design flood has less than a 100-year return period, and the roadway is held down so that the total flow through the culvert and over the roadway is sufficient to pass the 100-year storm without raising the 100-year water surface elevation upstream of the culvert
- Due to a flood event which exceeds the design storm and check storm

If roadway overtopping is indicated, it is necessary to consider the risk to highway users of loss of life, injury, and property damage. The highway embankment may be at risk based on:

- The depth of flow across the roadway
- The velocity of flow across the roadway
- The duration of roadway overtopping
- The resistance of the embankment to scour

Storage routing may be useful in evaluating the impacts of roadway overtopping.

8.3.3 Geometric Criteria

Design criteria related to the culvert geometry, including the inlet structure, the barrel, and the outlet structure are summarized in this section.

8.3.3.1 Culvert Size and Shape

The culvert size and shape selected should be based on engineering and economic criteria related to site conditions.

- For the Interstate System, the minimum size of main line culverts will generally be 24 inches due to maintenance considerations
- For other systems, 15 inches will generally be the minimum culvert diameter, except that hydraulically adequate 12-inch diameter culverts may be used if the culvert length is less than 50 feet or if it is located under an entrance
- Use arch or elliptical shapes only if required by hydraulic limitations, site characteristics such as cover, structural criteria, or environmental criteria

8.3.3.2 Multiple Barrels
Multiple barrel culverts should be designed to utilize the natural dominant channel with minimal or preferably no widening of the channel so as to avoid conveyance loss through sediment deposition in some of the barrels. An example of this concept would be a situation wherein a quadruple 10’x10’ box culvert is to be placed in a 15 ft. wide* channel. The gross waterway opening (i.e. 400 sq. ft.) and/or the configuration of that opening must be reduced for computational purposes commensurate with the amount of over bank area which will be displaced, on the premise that the natural stream channel and overbank configuration will reestablish through the culvert over time. It may even be necessary to place temporary timber/rock weirs to assist this process. Multiple barrels should be avoided where:

- The approach flow is high velocity, particularly if supercritical. (These sites require either a single barrel or special inlet treatment to avoid adverse hydraulic jump effects.)
- Fish passage is required unless special treatment is provided to ensure adequate low flow. When fish passage is required, <u>all</u> barrels are laid 6 inches below the streambed and a low flow diversion should be used to maintain the necessary depth in the appropriate barrel(s).
- A high potential exists for debris clogging the culvert inlet

8.3.3.3 Culvert Skew
The culvert skew should not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without the approval of VDOT.

8.3.3.4 End Treatment (Inlet or Outlet)
The culvert inlet type should be selected from the following categories based on the considerations given and the inlet entrance loss coefficient, K_e. Appendix 8D-2 provides recommended values of K_e. Consideration should also be given to safety since some end treatments can be hazardous to errant vehicles. All culverts 48 inches in diameter and larger should employ VDOT's standard headwalls, where available, or a comparable special design end treatment where a standard treatment does not apply. End treatments shall be employed in accordance with Chapter 15, DDMI drainage instructions. The following sections present pros and cons for each type of end treatment.

8.3.3.4.1 Projecting Inlets or Outlets
Projecting inlets or outlets extend beyond the roadway embankment. These structures:

- Are susceptible to damage during roadway maintenance and from errant vehicles
- Have low construction cost
- Have poor hydraulic efficiency for thin materials such as corrugated metal

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- Should not be used for culverts 48 inches in diameter and larger
- Are subject to buoyancy

8.3.3.4.2 Prefabricated End Sections
Prefabricated end sections are available for both corrugated metal and concrete pipes. These sections:

- Should not be used for culverts 48 inches in diameter or larger
- Retard embankment erosion and incur less damage from maintenance
- May improve projecting pipe entrances by increasing hydraulic efficiency, reducing the accident hazard, and improving their appearance
- Are hydraulically equivalent to a headwall, but can be equivalent to a beveled or side- tapered entrance if a flared, enclosed transition takes place before the barrel
- Are susceptible to buoyancy and may need concrete anchor blocks to resist hydrostatic uplift forces

8.3.3.4.3 Headwalls with Bevels
Headwalls with bevels are the standard VDOT design. These headwalls:

- Increase culvert efficiency
- Provide embankment stability and embankment erosion protection
- Provide protection from buoyancy
- Shorten the required structure length
- Reduce maintenance damage

8.3.3.4.4 Improved Inlets
Improved inlets are special designs which:

- Should be considered for exceptionally long culverts which will operate in inlet control or widening projects with increased flow to eliminate replacing existing culvert barrel(s)
- Can increase the hydraulic performance of the culvert, but may also increase total culvert cost
- If slope-tapered, should not be considered where fish passage is required
- Can increase outlet velocity

8.3.3.4.5 Wingwalls
Wingwalls are generally used in conjunction with headwalls and:

- Are used to retain the roadway embankment to avoid a projecting culvert barrel
- Are used where the side slopes of the channel are unstable
- Are used where the culvert is skewed to the normal channel flow
- Provide the best hydraulic efficiency if the flare angle is between 30° and 60°
- Are governed by VDOT height of embankment guidelines

8.3.3.4.6 Aprons
Aprons are special designs that can be used at culvert inlets and outlets and:

- Are used to reduce scour from high headwater depths or from high approach velocities in the channel
- Should extend at least one pipe diameter upstream
- Should not protrude above the normal streambed elevation

8.3.3.4.7 Cut-off Walls
Cut-off walls may be used at the entrance or the outlet of a culvert, and:

- Are used to prevent piping along the culvert barrel and undermining at the culvert ends
- Are an integral part of all of VDOT's standard endwalls
- Should be included (a minimum of 1.5 ft. in depth) when other than VDOT standard endwalls are employed

8.3.3.4.8 Trash Racks or Debris Deflectors
Trash racks or debris deflectors may be necessary at sites where large amounts of detritus are produced. Such structures:

- May create clogging problems
- Require maintenance
- Should only be used where there is an established need

8.3.4 Safety Considerations

Each site should be inspected periodically to determine if safety problems exist for traffic or for the structural safety of the culvert and embankment.

Culvert headwalls and endwalls should be located outside the clear zone distance of the highway. The clear zone distance from the edge of pavement is a function of the design speed of the roadway. The typical clear zone distance for a high-speed highway is 30-feet. The designer is referred to the VDOT Road Design Manual for further information and to AASHTO for additional guidance. An exception to this clear zone requirement occurs if traffic is separated from the walls by guardrail that is required due to obstacles other than the walls.

Where feasible, grate drop inlets or load-carrying grates may be substituted for culvert headwalls or endwalls, and thereby reducing safety hazards. However, in making this substitution, consideration must be given to the possibility of creating a greater safety hazard by increasing the potential for flooding if the grates clog. The drainage designer should continuously coordinate roadway related issues and information with the roadway design team.

8.3.5 Allowable Pipe Materials

Refer to Road and Bridge Standards PB-1.*

8.3.6 Other Design Considerations

8.3.6.1 Buoyancy Protection

When water is displaced by embankment material or by a culvert, a buoyant or upward force exists. If the buoyant force is greater than the weight of the object displacing the water, flotation will occur. Pipe flotation (or hydrostatic uplift) can be a problem where the following conditions exist:

- Lightweight pipe is used (i.e., corrugated metal or plastic)
- Pipe is on a steep grade (usually inlet control)
- There is little or no weight on the end of the pipe (i.e., flat embankment slopes, minimal cover and/or no endwalls)
- High headwater depths (HW/D> 1.0)

8.3.6.2 Relief Opening

Where multiple-use culverts or culverts serving as relief openings have their outlet set above the normal stream flow line, special precautions should be provided to prevent headcuts, erosion from undermining the culvert outlet, or damage to downstream properties due to concentrated flow.

8.3.6.3 Land Use Culverts

Land use culverts are installations where storm drainage requirements are combined with other land based uses, such as farm or pedestrian crossings. For such installations:

- The land use is temporarily forfeited during the design flood, but is available during lesser floods
- Two or more barrels may be required, with one situated to be dry during floods less than the selected design flood
- The outlet of the higher land use barrel may need protection from headcutting
- The culvert should be sized so as to ensure that it can serve its intended land use function up to and including a 2-year flood
- The height and width constraints should satisfy the hydraulic or land use requirements, whichever use requires the larger culvert

8.3.6.4 Erosion and Sediment Control

Temporary erosion and sediment control measures should be included in the construction plans. These measures include the use of the following: sediment basins

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and traps, silt barriers, dewatering basins, filter cloth, temporary silt fence and rock check dams. These measures should be utilized as necessary during construction to minimize pollution of streams and damage to wetlands. For more information, see Chapter 10, Erosion and Sediment Control.

8.3.6.5 Environmental Considerations

In addition to controlling erosion, siltation and debris at the culvert site, care must be exercised in selecting the location of the culvert. Where compatible with good hydraulic engineering a site should be selected that will permit the culvert to be constructed in the "dry" or that will cause the least impact to the stream or wetlands. This selection must consider the entire site involvement, preferably eliminating or at least minimizing the need for entrance and exit channels.

8.3.6.5.1 U.S. Army Corp of Engineers Jurisdictional Stream Bed[*]

Where there is a U.S. Army Corps of Engineers jurisdictional stream bed, as determined by the Environmental Division, both up and downstream inverts of the proposed culvert will be set lower than the normal flow line of the stream in order to provide for the re-establishment of the streambed and low flow depth in the culvert that will facilitate fish passage.

Where the culvert is a multiple barrel or multiple cell structure, and* all barrels or cells are to be lowered below stream grade a low flow diversion should be used to maintain low flow in the appropriate barrel(s). The grade of a culvert located to facilitate fish passage should never be steeper than the grade of the natural stream in the site area. Preferably, the culvert barrel should be flattened as necessary to limit the velocity of flow in the culvert. The Corps of Engineers' culvert countersinking requirements are described in detail in section 8.3.7.

8.3.7 Counter Sinking and Low Flow Considerations

8.3.7.1 DEFINITIONS

8.3.7.1.1 Stream Bed

The substrate along the length of a stream, which lies below the ordinary high water elevation. The substrate may consist of organic matter, bedrock or inorganic particles that range in size from clay to boulders, or a combination of materials. Areas contiguous to the stream bed, but above the ordinary high water elevation, are not considered part of the stream bed.

[*] Rev 9/09

8.3.7.1.2 Culvert

A culvert is generally defined as an enclosed structure that is used to convey surface waters from one side of an embankment to the other. For the purposes of this Manual there is no distinction between temporary and permanent culvert installations.*

8.3.7.2 Policy

8.3.7.2.1

The District Environmental staff will determine if the culvert impacts a jurisdictional stream bed (US Army Corps of Engineers) and will notify the appropriate project authority and the Hydraulic Engineer when the below requirements must be incorporated into the design.

8.3.7.2.2

Culverts constructed in jurisdictional stream beds are required to have the upstream and downstream inverts set (countersunk) below the natural stream bed elevation to stimulate natural stream bed establishment within the culvert and to meet the requirements of the environmental permitting process. The countersinking requirement does not apply to floodplain culverts or extensions or maintenance of existing structures where the existing structure will remain in service.

8.3.7.2.3

When performing the hydraulic analysis for any culvert installation that is to be countersunk, the analysis shall either:

1) Consider the hydraulic opening as being that above the countersunk portion of the culvert, or

2) Determine the required hydraulic opening (size) based on no countersinking; then specify the next larger size structure (3” or 6” greater height as appropriate) with the additional opening installed below the steam bed.

8.3.7.2.4

When performing a hydraulic analysis for any multiple barrel culvert crossing, it is appropriate to consider the natural channel and flood plain configuration as projecting through the crossing, the same as if it were a bridge spanning a flood plain. For the purpose of determining the hydraulic capacity of the crossing, any culvert area that is outside the natural channel area and below the flood plain elevation will be considered obstructed and, therefore, not available for hydraulic conveyance.

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8.3.7.2.5
Culverts will be adequately sized to allow for the passage of ordinary high water with the countersinking, invert and flood plain restrictions taken into account.

8.3.7.2.6
If the culvert is greater than 24" (or equivalent) in diameter, or rise in the case of noncircular* shapes, the inlet and outlet ends shall be countersunk a minimum of 6" below the natural stream bed. If the culvert is 24" (or equivalent) or less in diameter, or rise in the case of noncircular shapes, the inlet and outlet ends shall be countersunk a minimum of 3" below the natural stream bed.

8.3.7.3 MULTIPLE BARREL CULVERTS

8.3.7.3.1
When multiple barrel culverts are used, the 6" countersink requirement may only be needed for one barrel. The Hydraulic Engineer should determine whether it is appropriate and/or feasible to countersink one barrel or all of the barrels considering the following:

8.3.7.3.1.1
Width of Normal Stream - The width of the culvert barrel(s) receiving the low flow should approximate the width of the normal stream to avoid accelerating velocities (at normal flow) through the culvert.

8.3.7.3.1.2
Width of Floodplain - Narrow and constricted floodplains may necessitate all barrels being at the lowest possible elevation. Wide floodplains with significant over bank areas may permit one barrel to be countersunk and the remaining barrels to be either at the floodplain elevation or at an elevation slightly higher than the natural stream bed.

8.3.7.3.1.3
Pipe Culverts – Pipe Culverts may be designed to have barrels at different invert elevations. However, special provisions are needed to ensure proper bedding and backfill. Special Design Endwalls will be required. These considerations may negate any potential cost savings associated with not countersinking all barrels a like amount.

8.3.7.3.1.4
Box Culverts - Precast box culverts may be designed to have barrels at different invert elevations. In doing so, the installation is usually configured with the top of all barrels at the same elevation. This will require the same special considerations for bedding, backfill and endwall design as noted in Section 3.1.3. Cast in place box culverts usually have all barrels of the same size and elevation in order to construct the box culvert using standard details.

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8.3.7.3.2

Multiple barrel culverts that are constructed with all barrels countersunk shall provide measures for directing the low flow through one or more barrels that approximate the width of the normal stream. (See Road and Bridge Standards EC-13) .*

8.3.7.3.2.1

If the normal stream width is approximately equal to the total span of all barrels, low flow diversion measures normally should not be needed. If the Hydraulic Engineer elects not to utilize a low flow diversion structure, the District Environmental Manager shall be notified of the decision and be provided justification in order to advise the environmental review agencies during the permitting process.

8.3.7.3.2.2

When low flow diversion measures are needed, they shall be constructed to permit the stream to continue the natural meander or moving process normally associated with flood flows. The low flow diversion structures shall be constructed of rip rap, or other similar material. The rip rap material used should be small enough to allow movement during flood events (i.e., Class I Dry Rip Rap).

8.3.7.3.2.3

Other methods of achieving the desired low flow conditions may also be employed. These shall be reviewed and approved by the District Environmental Manager.

8.3.7.4 SPECIAL CULVERT INSTALLATIONS

8.3.7.4.1

Culverts on Bedrock: If the bedrock prevents countersinking, evaluate the use of a three-sided structure to cross the waterway or evaluate alternative locations for the new culvert that will allow for countersinking. If none of these alternative measures are practicable, the Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life. Options that must be considered include partial countersinking (such as less than 3” of countersinking, or countersinking of only one end of the culvert), constructing stone step pools and low rock weirs downstream of the culvert, or other measures that provide for the movement of aquatic life.

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NOTE: Blasting of bedrock stream bottoms through the use of explosives is not acceptable as a means of providing for countersinking of pipes on bedrock.

8.3.7.4.2

Culverts on Steep Terrain: Culverts on steep terrain (slope of 5% or greater) may* generate flow velocities that cause excessive scour at the outlet and may prevent the establishment of a natural bed of material through the culvert. Should this situation present itself, the Hydraulic Engineer shall coordinate the evaluation of alternatives to countersinking. These include partial countersinking of the inlet end and implementation of measures to minimize any disruption of the movement of aquatic life, constructing a stone step/pool structure, using river rock/native stone rather than riprap or constructing low rock weirs to create a pool or pools. Stone structures should be designed with sufficient-sized stone to prevent erosion or washout and should include keying-in as appropriate. These structures should be designed both to allow for aquatic life passage and to minimize scour at the outlet. The Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life.

8.3.7.4.3

Culverts at the Confluence of Two Streams: The outlet end of culverts that discharge a tributary directly into another stream must be countersunk below the natural stream bed at the discharge point. If this measure is not practicable, the Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life.

8.3.7.4.4

Other unusual circumstances that prohibit countersinking shall be evaluated on a case-by-case basis. The Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life.

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8.3.7.4.5

Proposed culverts that do not include countersinking are subject to environmental agency review and approval and may require additional documentation or evaluation of other alternative measures.

8.3.7.4.6

If it has been determined that environmental permits will be required, the standard form* LD-294 must be employed to address pertinent hydraulic-related environmental concerns as well as any FEMA designated floodplain involvement. This form may be obtained at http://vdotforms.vdot.virginia.gov . This form, as well as a copy of an excerpt from the FEMA map panel covering the crossing site, must be provided to the District Environmental Manager to be included in the permit assembly package.

* Rev 9/09

8.4 Design Concepts

8.4.1 General

The design of a culvert system for a highway crossing should consider: roadway requirements, planning and location, hydrology, ditches and channels, and erosion and sediment control. Each of these chapters should be consulted as appropriate. The discussion in this section is focused on alternative analyses and design methods.

For economy and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during design flood flows. At many sites, either a bridge or a culvert will fulfill the structural and hydraulic requirements; therefore, the structure choice should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental considerations.

8.4.2 Design Methods

The designer should choose whether:

- To assume a constant discharge or to route a hydrograph
- To use hand methods (nomographs or equations) or computer software solutions, such as FHWA's HY8

The FHWA's Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts," is the primary reference on culvert design.

8.4.2.1 Hydrologic Methods

Hydrologic methods are either steady state (constant discharge over time) or unsteady (flow varies with time, as in a hydrograph). See Chapter 6 for recommended methods.

8.4.2.1.1 Constant Discharge

The constant discharge method:

- Is the typical method used for most culvert designs
- Is usually assumed to be the peak discharge
- Will yield a conservatively sized structure where temporary storage is available but is not considered

8.4.2.1.2 Hydrograph and Storage Routing

Hydrograph and storage routing method:

- Is used when unusual circumstances exist
- Considers the storage capacity behind a highway embankment which attenuates a flood hydrograph and reduces the peak discharge
- May reduce the required culvert size, given adequate storage

- Is checked by routing the design hydrographs through the culvert to determine the outflow hydrograph and upstream water surface elevation
- Procedures are in Chapter 11, Stormwater Management, and in HDS-5, Section V

8.4.2.2 Computational Methods

Computational methods include manual* methods (LD-269) and computer solutions. Manual methods usually employ design nomographs, provided in Appendix 8C. However, the design equations may also be applied. Computer solutions are usually employed for larger installations; however, they can be used for all situations.

8.4.2.2.1 Manual Methods

Manual methods using design equations and nomographs through design form LD-269 (Appendix 8B-1):

- Require a trial and error solution that is straightforward and easy using design nomographs
- Provide reliable designs for many applications
- Require additional computations for tailwater, outlet velocity, hydrographs, routing and roadway overtopping
- Nomographs for a variety of barrel shapes are included in Appendix 8C

8.4.2.2.2 Computer Solution

One example of culvert analysis software is HY8, FHWA's Culvert Analysis Microcomputer Program, which:

- Is an interactive program
- Uses the theoretical basis for the nomographs
- Can compute tailwater, improved inlets, road overtopping, hydrographs, routing and multiple independent barrels, and irregular shaped conduits
- Calculates backwater profiles in the culvert barrel(s)
- Develops and plots tailwater rating curves
- Develops and plots performance curves

8.4.3 Culvert Hydraulics

An exact theoretical analysis of culvert flow is extremely complex because the following are required:

- Analysis of non-uniform flow with regions of both gradually varying and rapidly varying flow
- Determination of how the flow type changes as the flow rate and tailwater elevations change

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- Application of backwater and drawdown calculations and energy and momentum balances
- Incorporation of the results of hydraulic model studies
- Determination of whether hydraulic jumps occur and whether they are inside or downstream of the culvert barrel
- Analysis of flows under subatmospheric pressure in the culvert barrel

The design procedures described in this chapter incorporate the following concepts:

8.4.3.1 Control Section

- The control section is the location where there is a unique relationship between the flow rate and the upstream water surface elevation
- The control section may be located at or near the culvert inlet (inlet control) or the culvert outlet (outlet control)
- Inlet control is governed by the inlet geometry
- Outlet control is governed by the culvert inlet geometry, as well as the barrel characteristics, and tailwater elevation(s)
- Tailwater control may be located downstream of the culvert

8.4.3.2 Minimum Performance

Minimum performance is determined by analyzing both inlet and outlet control and using the highest resultant headwater. The culvert may operate more efficiently than minimum performance at times (more flow for a given headwater level), but it will not operate at a lower performance level than the one calculated using this concept.

8.4.3.3 Inlet Control

For inlet control, the control section is at, or near, the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater elevation, a hydraulic jump may occur downstream of the inlet.

8.4.3.3.1 Headwater Factors - Inlet Control

The following factors are considered when calculating the inlet control headwater.

- **Headwater depth** is measured from the inlet invert of the inlet control section to the surface of the upstream pool
- **Inlet area** is the cross-sectional area of the face of the culvert. The inlet face area is the same as the barrel area, except for tapered improved inlets
- **Inlet edge** configuration describes the entrance geometry. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge
- **Inlet shape** is usually the same as the shape of the culvert barrel except for some improved inlets. Typical shapes are rectangular, circular, elliptical, and arch. Carefully check for additional control sections for special culvert designs.

8.4.3.3.2 Flow Conditions – Inlet Control

Three regions of inlet control flow are shown in Figure 8-1. They are unsubmerged, transition, and submerged. Generally, as the flow rate increases, inlet control flow passes through an unsubmerged condition (water surface below the crown of the control section), transition (between partly full and full flow), and submerged (water surface above the crown of the control section). The transition region is poorly defined and tends to be unstable. Its curve is usually drawn tangent to the unsubmerged and submerged performance curves.

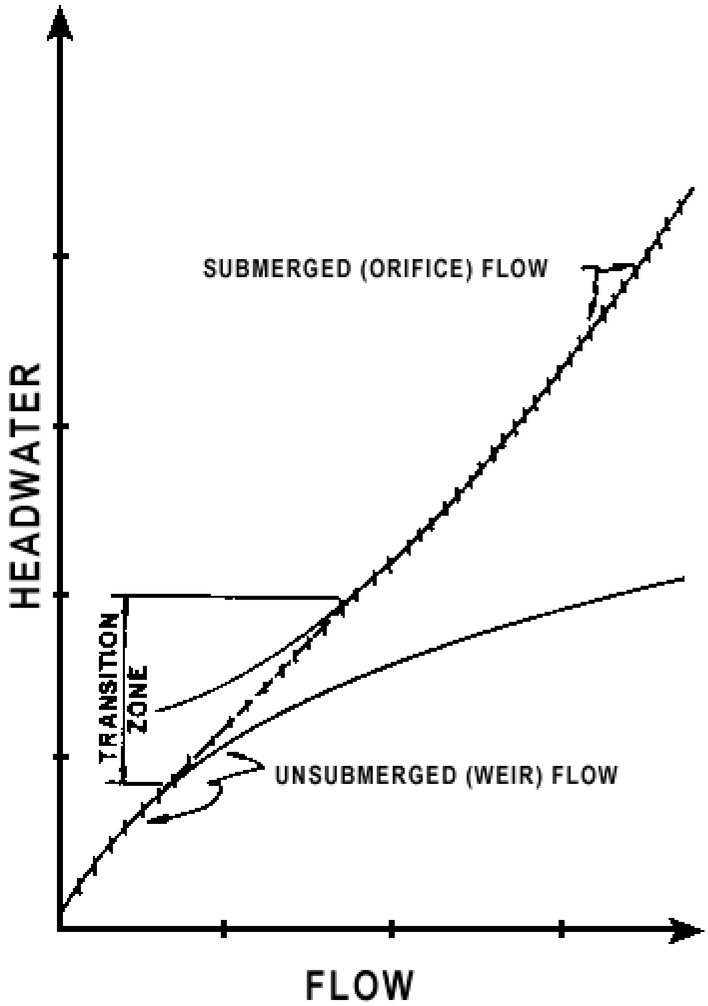

Figure 8-1. Performance Curves - Unsubmerged, Transition, and Submerged

Four types of inlet control flow profiles within culverts are shown in Figure 8-2.

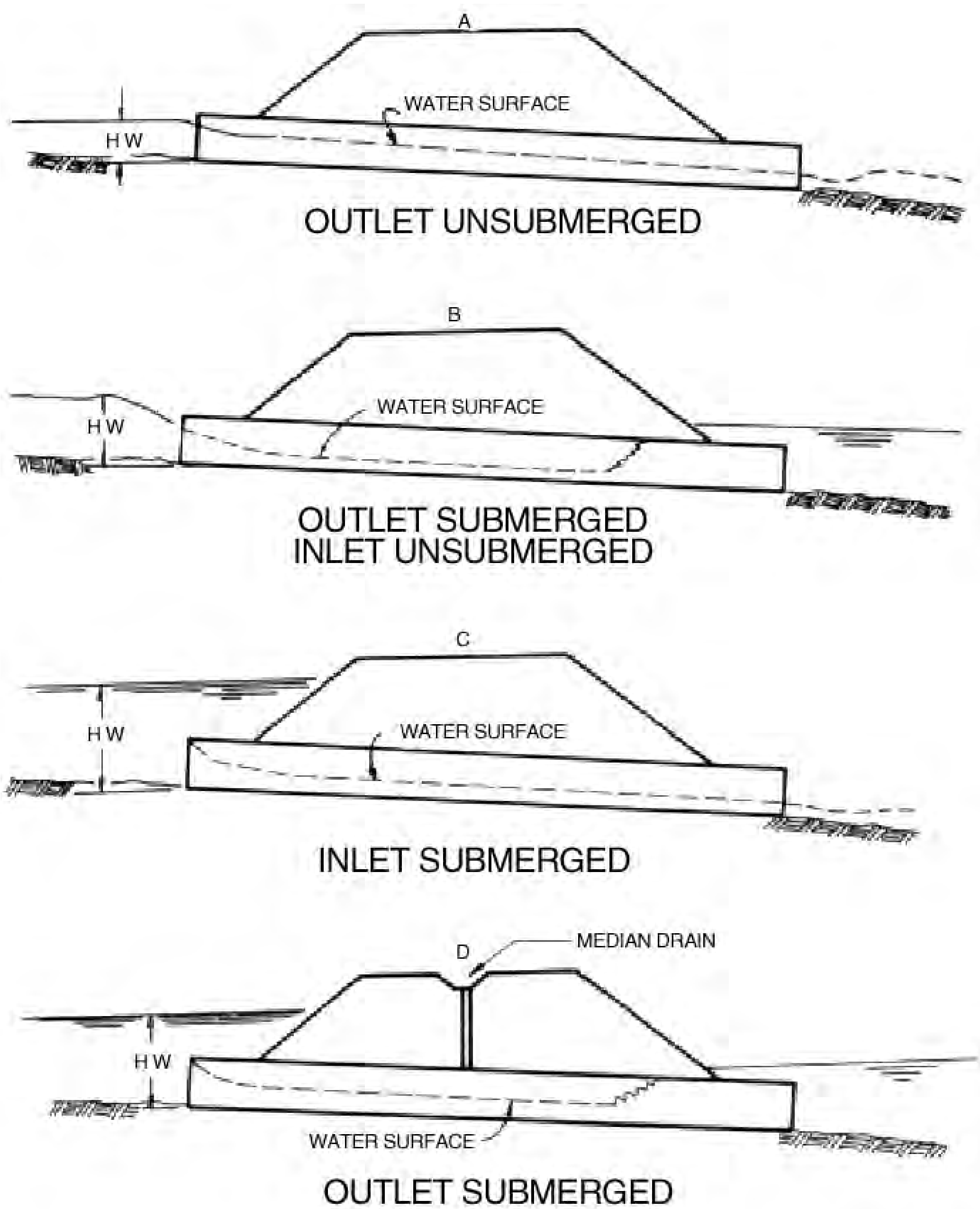

Figure 8-2. Types of Inlet Control Flow

8.4.3.3.2.1 Unsubmerged - Inlet Control
For headwaters below the inlet crown, the entrance operates as a weir, as shown in Figure 8-2, diagrams A and B. As shown, the outlet of the culvert may be unsubmerged or submerged.

- A weir is a flow control section where the upstream water surface elevation can be predicted for a given flow rate
- The relationship between flow and water surface elevation must be determined by model tests of the weir geometry or by measuring prototype discharges
- Such tests are then used to develop equations. Appendix A of HDS-5 contains the equations, which were developed from model test data.

8.4.3.3.2.2 Submerged - Inlet Control
For headwaters above the inlet crown, the culvert operates as an orifice as shown in Figure 8-2, diagram C.

- An orifice is a submerged opening flowing freely on the downstream side, which functions as a control section
- The relationship between flow and headwater can be defined based on results from model tests. Appendix A of HDS-5 contains flow equations, which were developed from model test data.

8.4.3.3.2.3 Transition Zone - Inlet Control
The transition zone is located between the unsubmerged and the submerged flow conditions where the relationship between flow and headwater depth is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves.

8.4.3.3.2.4 Special Condition - Inlet Control
Figure 8-2, diagram D illustrates a special case of inlet control, where both the entrance and the outlet are submerged. To maintain this condition, a source of air must be supplied to the barrel; otherwise the barrel will tend to surge and alternate between full flow and partly full flow.

8.4.3.3.2.5 Inlet Control Nomographs
The inlet control flow versus headwater curves, which are established using the above procedure, are the basis for constructing the inlet control design nomographs in Appendix 8C. Note that in the inlet control nomographs, headwater (HW) is measured from the inlet invert to the total upstream energy grade line, including the approach velocity head.

8.4.3.4 Outlet Control
Culverts operating in outlet control have subcritical or full flow in their barrels. The control of the flow is at the downstream end of the culvert (the outlet) or further downstream. The tailwater depth is assumed to be a function of either critical depth at

the culvert outlet or the downstream channel depth, whichever is higher. In outlet control, the type of flow is dependent on the entire culvert, including the inlet configuration, the barrel, and the tailwater.

Five types of outlet control flow profiles within culverts are depicted in Figure 8-3. Note that both the inlet crown and the outlet crown may be submerged or unsubmerged.

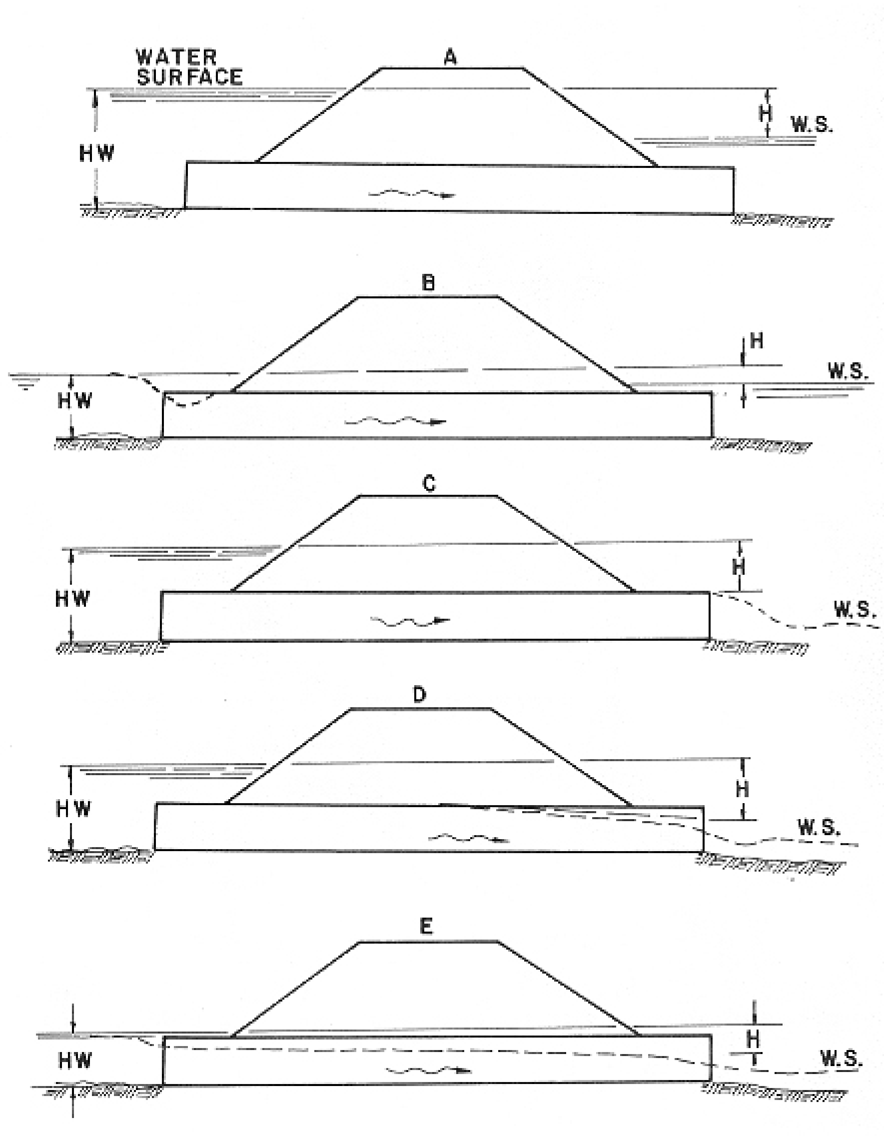

Figure 8-3. Types of Outlet Control Flow

Figure 8-3; diagram A represents full flow throughout the culvert barrel. Both the entrance and the outlet are submerged.

Figure 8-3; diagram B shows the barrel inlet flowing partly full, but the rest of the barrel under full flow. The entrance is unsubmerged due to the inlet contraction, and the outlet is submerged.

Figure 8-3, diagram C represents full flow in the culvert barrel. The entrance is submerged and the outlet is unsubmerged.

Figure 8-3, diagram D represents full flow in the upper section of the barrel and partly full flow (subcritical) in the lower section of the barrel. The entrance is submerged and the outlet is unsubmerged.

Figure 8-3, diagram E depicts partly full flow (subcritical) over the length of the barrel. Both the entrance and the outlet are unsubmerged.

8.4.3.4.1 Headwater Factors - Outlet Control

The following factors are considered when calculating the headwater from outlet control.

- Barrel Roughness is a function of the barrel material and geometry. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as Manning's n-value. Typical Manning's n-values are presented in Appendix 8D-1.
- Barrel Area is the full flow cross-section measured perpendicular to the flow.
- Barrel Length is the total culvert length from the entrance crown to the exit crown of the culvert. Because the design height of the barrel and the embankment slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.
- Barrel Slope is the actual slope of the culvert barrel, and is often the same as the natural stream slope. However, when the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope.
- Tailwater Elevation is based on the downstream water surface elevation. Backwater calculations from a downstream control, single section approximation, downstream lake levels, tidal elevations, or field observations are used to define the tailwater elevation. Tailwater elevations are normally calculated for different flood frequencies.

8.4.3.4.2 Flow Condition - Outlet Control

Full flow in the culvert barrel is assumed for the analysis of outlet control hydraulics. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool. The outlet control headwater can be computed using the following equations:

8.4.3.4.2.1 Losses

The total headloss through the culvert is defined by Equation 8.1.

$$H_L = H_e+H_f+H_o+H_b+H_j+H_g+H_v \qquad (8.1)$$

Where:

H_L = Total energy loss, ft
H_e = Entrance loss, ft

H_f = Friction losses, ft
H_o = Exit loss, ft (equals velocity head if K_e = 1.0)
H_b = Bend losses, ft (see HDS-5)
H_j = Losses at junctions, ft (see HDS-5)
H_g = Losses at grates, ft (see HDS-5)
H_v = Velocity head, ft

8.4.3.4.2.2 Velocity

Velocity is computed using the continuity equation.

$$V = \frac{Q}{A} \tag{8.2}$$

Where:

V = Average full barrel velocity, fps
Q = Flow rate, cfs
A = Cross sectional area of flow with the barrel full, sq. ft.

8.4.3.4.2.3 Velocity Head

The velocity head represents the kinetic energy of full flow in the culvert barrel. It is used in calculating the losses in the culvert (inlet, barrel, outlet, etc.).

$$H_v = \frac{V^2}{2g} \tag{8.3}$$

Where:

g = Acceleration due to gravity, 32.2 ft/s^2

8.4.3.4.2.4 Entrance Loss

The losses at the culvert entrance are a function of the velocity head. The more efficient the inlet, the lower the K_e value.

$$H_e = K_e\left(\frac{V^2}{2g}\right) \tag{8.4a}$$

Where:

K_e = Entrance loss coefficient, see Appendix 8D-2

8.4.3.4.2.5 Friction Loss

Friction loss in the culvert barrel is due to wall friction. It is a function of barrel roughness, size, shape, and velocity head, and is calculated using Manning's Equation.

$$H_f = \frac{29n^2L}{R^{1.33}}\left(\frac{V^2}{2g}\right) \qquad (8.4b)$$

Where:

n = Manning's roughness coefficient, see Appendix 8D-1
L = Length of the culvert barrel, ft
R = Hydraulic radius of the full culvert barrel = $\left(\frac{A}{P}\right)$, ft
A = Cross section area of pipe, sq. ft.
P = Wetted perimeter of the barrel, ft

8.4.3.4.2.6 Exit Loss

The exit loss is a function of the velocity head in the barrel and the velocity head in the downstream channel. The latter is often neglected.

$$H_o = 1.0\left[\frac{V^2}{2g} - \frac{V_d^2}{2g}\right] \qquad (8.4c)$$

Where:

V_d = Channel velocity downstream of the culvert, fps (if downstream velocity is neglected, use Equation 8.4d).

$$H_o = H_v\left(\frac{V^2}{2g}\right) \qquad (8.4d)$$

8.4.3.4.2.7 Other Losses

Other possible losses in the culvert include junctions, bends, grates, etc. If present, these losses are functions of the velocity head multiplied by a loss coefficient. The loss coefficients are found in HDS-5.

8.4.3.4.2.8 Barrel Losses

The various culvert losses are totaled to obtain the total headloss in the barrel. Losses for bends, junctions, grates, etc., should be added to Equation 8.5.

$$H = H_E + H_o + H_f$$

$$H = \left[1 + K_e + \frac{29n^2L}{R^{1.33}}\right]\left(\frac{V^2}{2g}\right) \qquad (8.5)$$

8.4.3.4.2.9 Energy Grade Line - Outlet Control

The energy grade line represents the total energy at any point along the culvert barrel. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure 8-4, the following relationship results:

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L \tag{8.6}$$

Where:

HW_o = Headwater depth above the outlet invert, ft
V_u = Approach velocity, fps
TW = Tailwater depth above the outlet invert, ft
V_d = Downstream velocity, fps
H_L = Sum of all losses (Equation 8.1)

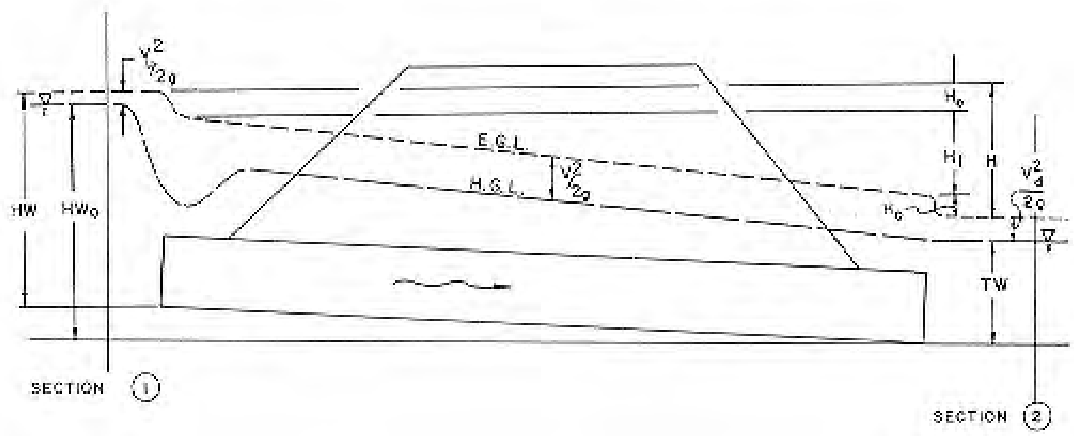

Figure 8-4. Full Flow Energy and Hydraulic Grade Lines

8.4.3.4.2.10 Hydraulic Grade Line - Outlet Control

The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are straight, parallel lines separated by the velocity head except at the inlet and the outlet.

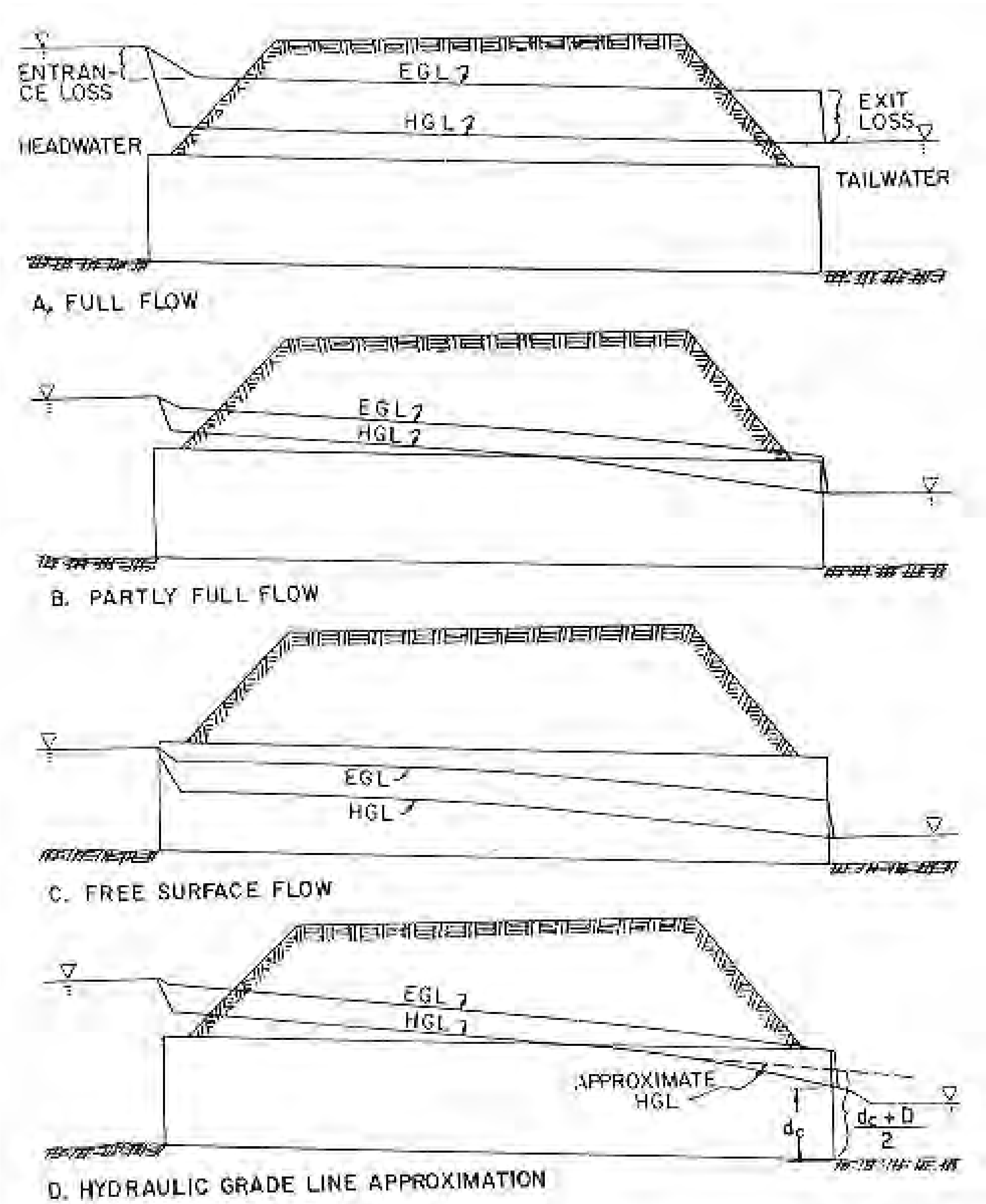

Figure 8-5. Outlet Control Energy and Hydraulic Grade Lines

8.4.3.4.2.11 Outlet Control Nomographs (Full-flow)

The outlet control nomographs were developed assuming that the culvert barrel is:

- Flowing full (See Figure 8-5, diagrams A and B)
- $d_c \geq D$, (See Figure 8-5, diagram C)
- V_u is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert
- V_d is small and its velocity head can be neglected

For these conditions, Equation 8.6 becomes:

$$HW = TW + H - S_o L \tag{8.7}$$

Where

HW= Depth from the inlet invert to the energy grade line, ft
H = Headloss read from the nomograph (Equation 8.5), ft
S_o = Slope of culvert barrel, ft/ft
L = Length of culvert barrel, ft

8.4.3.4.2.12 Outlet Control (Partly Full-flow)

Equations 8.1 through 8.7 were developed for full barrel flow. The equations also apply to the flow situations which are effectively full flow conditions, if TW < d_c (Figure 8-5, diagrams C and D), backwater calculations may be required which begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel (Figure 8-5, diagram D), the full flow hydraulic grade line extends from that point upstream to the culvert entrance.

8.4.3.4.2.13 Outlet Control Nomographs (Partly Full-flow) - Approximate Method

Based on numerous backwater calculations performed by the FHWA staff, it was found that the full flow hydraulic grade line, extended from the upstream end of the barrel to the outlet, pierces the plane of the culvert outlet at a point about one-half way between critical depth and the top of the barrel, or $(d_c+D)/2$ above the outlet invert. TW based on the downstream channel depth should be used if it is higher than $(d_c+D)/2$.

The following equation should be used for headwater (HW):

$$HW = h_o + H - S_o L \tag{8.8}$$

Where:

h_o = The larger of TW or $\left(\frac{d_c+D}{2}\right)$, ft

Adequate results are obtained down to about HW = 0.75D. For lower headwaters, backwater calculations are required.

8.4.3.5 Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually have outlet velocities that are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. If outlet erosion protection is necessary, the flow depth and the Froude number may also be needed.

8.4.3.5.1 Inlet Control

The velocity is calculated using Equation 8.2 with the flow area (A) equal to the cross section of the flow prism at the culvert outlet. First, the outlet depth must be determined. Either of the following methods may be used.

- Calculate the water surface profile through the culvert. Begin the computation at d_c at the entrance and proceed downstream to the exit. Determine the depth and flow prism area at the exit
- Assume normal depth and velocity in the culvert barrel. This approximation may be used since the water surface profile approaches normal depth if the culvert is long enough. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depths may be obtained from design aids in Appendix 8C.

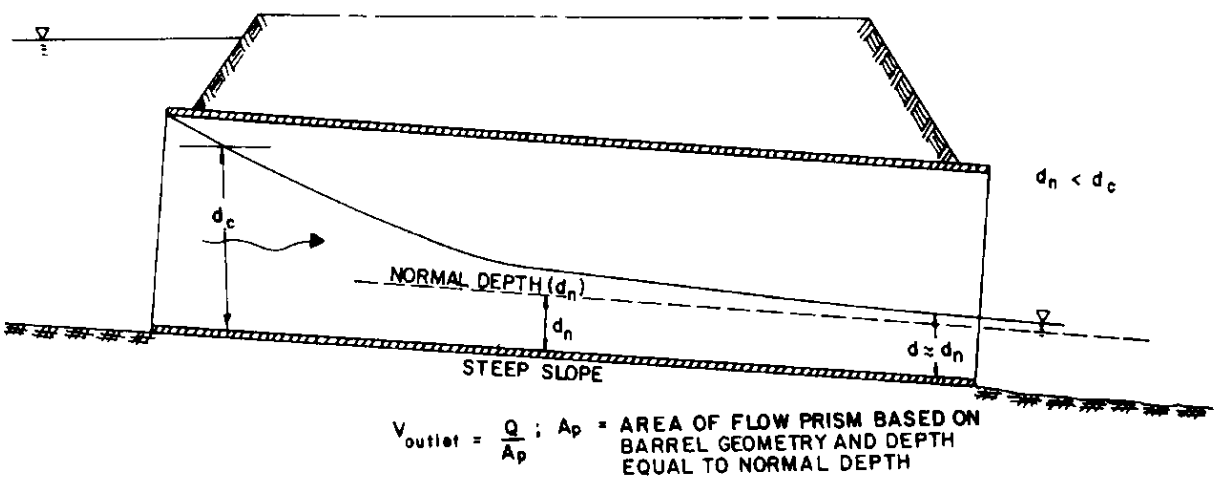

Figure 8-6. Outlet Velocity - Inlet Control

8.4.3.5.2 Outlet Control

The cross sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater (downstream channel) depth, or the height of the conduit.

- Critical depth is used when the tailwater is less than critical depth
- 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 level exceeds the top of the barrel

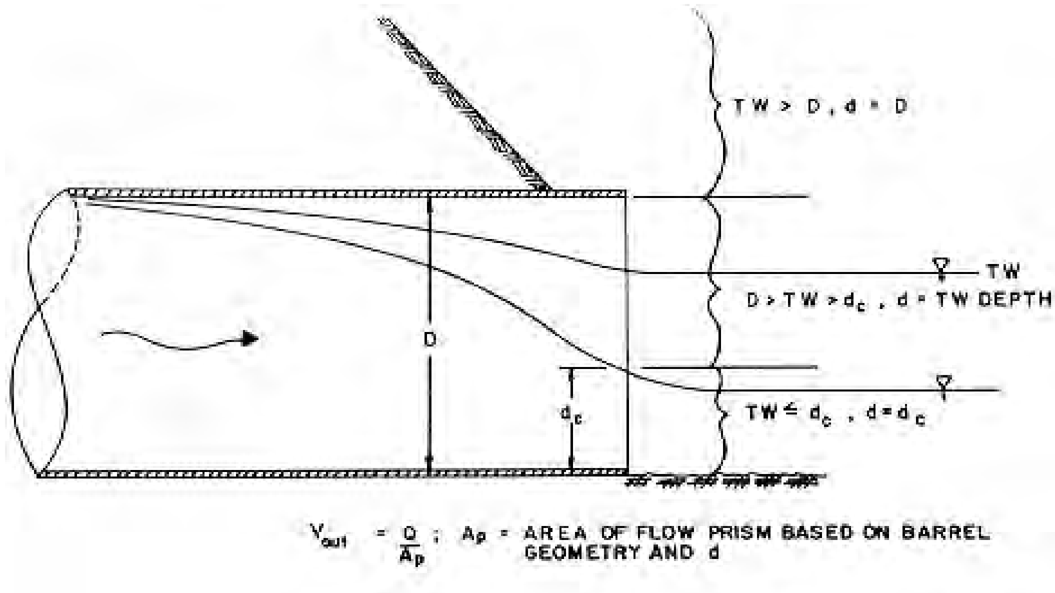

Figure 8-7. Outlet Velocity - Outlet Control

8.4.3.6 Roadway Overtopping

Roadway overtopping will begin when the culvert headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad crested weir. Flow coefficients for flow overtopping roadway embankments are found in the FHWA's HDS No. 1, Hydraulics of Bridge Waterways. Curves for discharge coefficients are also included in Appendix 8C-60.

8.4.3.6.1 Length of Roadway Crest

The length of the roadway (weir) crest is difficult to determine when the crest is defined by a roadway sag vertical curve. It is recommended that the sag vertical curve be subdivided into a series of segments. The flow over each segment is then calculated for a given headwater. The flows for each segment are then added together to determine the total flow. Alternatively, the entire length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth (area/length) of the upstream pool above the roadway. The computer program HY8 allows input of the actual road surface x and y coordinates.

8.4.3.6.2 Total Flow

The flow over the roadway is calculated for a given upstream water surface elevation using Equation 8.9.

$$Q_r = C_d L HW_r^{1.5} \quad (8.9)$$

Where:

Q_r = Overtopping flow rate, cfs
C_d = Overtopping discharge coefficient (weir coefficient) = $k_t\, C_r$
k_t = Submergence coefficient
C_r = Discharge coefficient
L = Length of the roadway crest, ft
HW_r = Headwater depth, measured above the roadway crest, ft

- Roadway overflow plus culvert flow must equal the total design flow
- A trial-and-error process is necessary to determine the flow passing through the culvert and the amount flowing across the roadway for various headwater elevations
- Performance curves for the culvert and the road overflow may be summed to yield an overall performance curve

Computer programs such as HY8 are recommended for design when evaluating roadway overtopping.

8.4.3.6.3 Performance Curves

Performance curves are plots of flow rate versus headwater depth or water surface elevation. The culvert performance curve is made up of the controlling portions of the individual performance curves for each of the following control sections as shown in Figure 8-8:

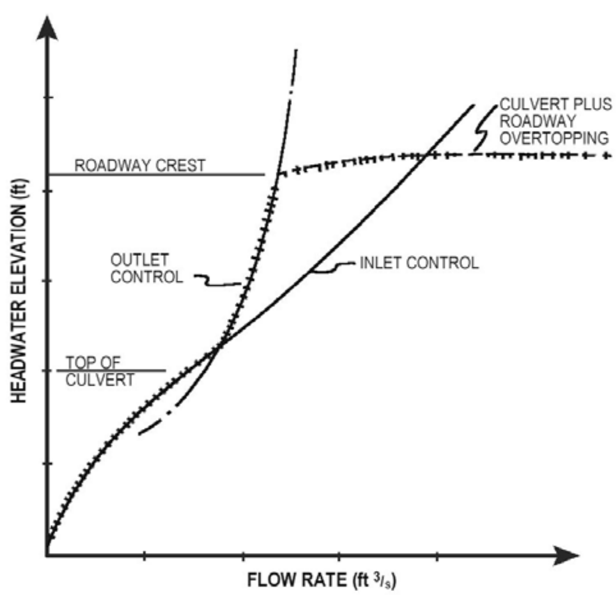

Figure 8-8. Overall Culvert Performance Curve

- Inlet control performance curve is developed using the inlet control nomographs in Appendix 8C
- Outlet control performance curve is developed using Equations 8.1 through 8.7, the outlet control nomographs in Appendix 8C, or backwater calculations
- Roadway overtopping performance curve is developed using Equation 8.9
- Overall performance curve is the sum of the flow through the culvert and the flow across the roadway and can be determined by performing the following steps

Step 1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. <u>Both inlet and outlet control headwaters should be calculated.</u>

Step 2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

Step 3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will occur. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and Equation 8.9 to calculate flow rates across the roadway.

Step 4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve, as shown in Figure 8-8.

8.4.4 Special Design Considerations

8.4.4.1 General

The following sections describe and discuss special culvert design considerations. References are provided for the detailed design methods.

8.4.4.2 Tapered Inlets

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet with additional depression at the upstream end also improves performance by increasing the head applied to the throat section.

- Tapered inlets are not recommended for use on short culverts or culverts flowing in outlet control because the simple beveled edge is of equal hydraulic benefit
- Design criteria and methods have been developed for two basic tapered inlet designs: the side-tapered inlet and the slope-tapered inlet
- Tapered inlet design charts from FHWA's HDS-5 for both rectangular box culverts and circular pipe culverts are included in Appendix 8C.

Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets), and the throat. The headwater depth for each control section is referenced to the invert of that section.

8.4.4.2.1 Side-Tapered Inlets

The side-tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the sidewalls (Figure 8-9). The face section is about the same height, as the barrel height and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than 10 percent (1.1D). The intersection of the tapered sidewalls and the barrel is defined as the throat section. There are two possible control sections, the face and the throat. HW_f, shown in Figure 8-9, is the headwater depth measured from the face section invert and HW_t is the headwater depth measured from the throat section invert. The throat of a side-tapered inlet is a very efficient control section. The flow contraction is nearly eliminated at the throat.

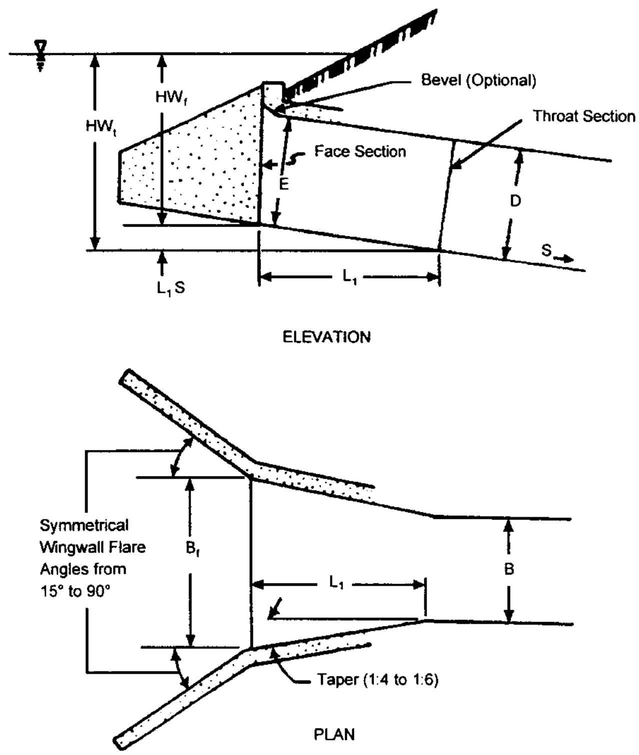

Figure 8-9. Side-Tapered Inlet

The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters.

8.4.4.2.2 Slope-Tapered Inlets

The slope-tapered inlet, like the side-tapered inlet, has an enlarged face section with tapered sidewalls meeting the culvert barrel walls at the throat section as shown in Figure 8-10). In addition, a vertical FALL is incorporated into the inlet between the face and throat sections. This FALL concentrates more head on the throat section. At the location where the steeper slope of the inlet intersects the flatter slope of the barrel, a third section, designated the bend section, is formed. Therefore, a slope-tapered inlet has three possible control sections, the face, the bend, and the throat.

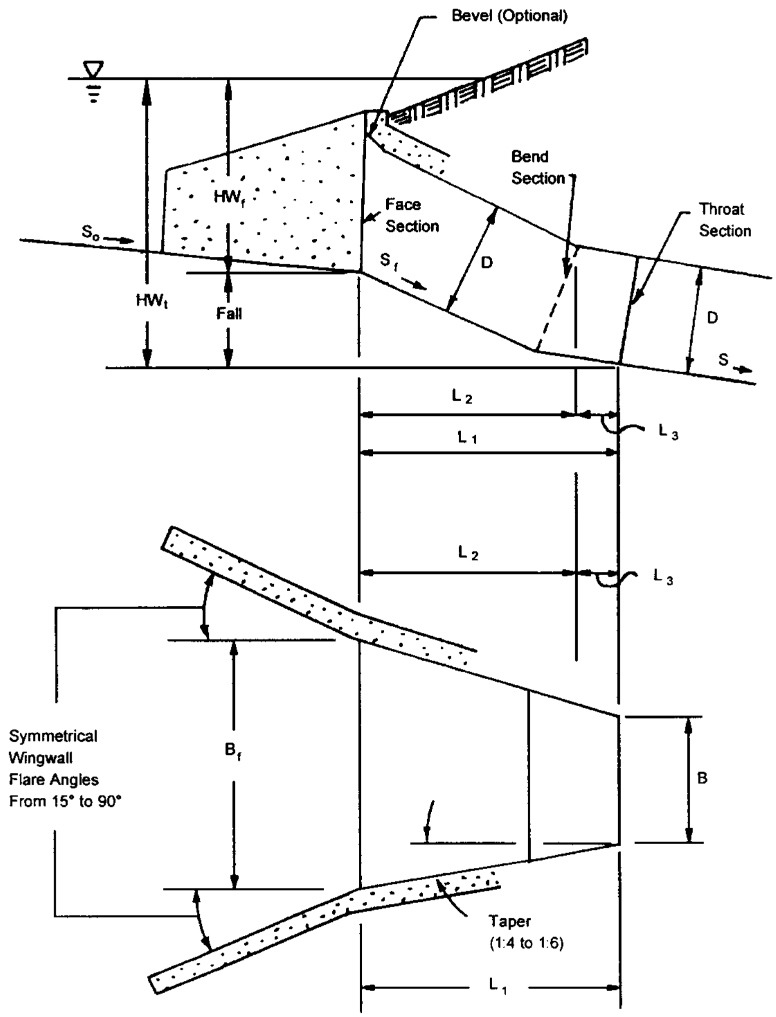

Figure 8-10. Slope-Tapered Inlet

The slope-tapered inlet combines an efficient throat section with additional head exerted on the throat. The face section does not benefit from the FALL between the face and throat; therefore, the face sections of these inlets are larger than the face sections of equivalent depressed side-tapered inlets. The required face size can be reduced by the use of bevels or other favorable edge configurations. The slope-tapered inlet is the most complex inlet improvement recommended in this drainage manual. Construction difficulties are inherent, but the benefits in increased performance can be significant. With proper design, a slope-tapered inlet passes more flow at a given headwater elevation than any other configuration. Slope-tapered inlets can be applied to both box culverts and circular pipe culverts. The slope-tapered inlet throat should be the primary control section in a slope-tapered inlet design.

HDS-5, Hydraulic Design of Highway Culverts, contains complete design methodology and design charts and forms for culverts with improved inlets. Most of the design charts have been included in Appendix 8C.

8.4.4.3 Buoyancy Protection

The buoyancy of a pipeline depends upon the weight of the pipe, the weight of the volume of water displaced by the pipe, the weight of the liquid load carried by the pipe and the weight of the backfill over the pipe. Lighter weight pipe materials are generally more susceptible to uplift forces than heavier materials.

If the summation of the weight of the pipe, weight of the water in the pipe (based on normal depth) and the weight of the fill over the pipe is less than the hydrostatic uplift (buoyant) forces acting upon the pipe, additional weight must be added to the pipe in order to stabilize it for the design conditions. The normal depth for determining buoyancy protection should be either the Q_{100} headwater depth or[*] the depth of overtopping, whichever is less.

A concrete endwall will usually provide sufficient weight to counteract potential buoyant forces. However, in low fill situations it is usually more desirable and economical to use end sections in lieu of endwalls or, in the case of secondary roadways, pipes are often installed projecting beyond the embankment slopes with no end treatment. In these situations, a concrete anchor block (counterweight) must be designed for each installation where it is determined that flotation may be a potential problem. See Figure 8-17.

In Section 8.5.3, a procedure is outlined (with example) showing how to analyze a pipe installation for flotation potential and, where it is determined that there is a potential problem, how to determine the amount of counterweight needed.

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8.5 Design Procedures and Examples

8.5.1 Documentation Requirements

These items establish a minimum documentation requirement for culvert design. The following items used in the design or analysis should be included in the documentation file:

- Allowable headwater elevation and basis for its selection
- Cross section(s) used in the design highwater determinations
- Roughness coefficient assignments (n-values)
- Observed highwater, dates and discharges
- Stage discharge curve at outlet for undisturbed, existing and proposed conditions to include the depth and velocity measurements or estimates and locations for the design, and check floods
- Documentation showing the calculated backwater elevations, outlet velocities, scour for the design storm, check storm and any available historical floods, including any necessary scour countermeasures or foundation adjustments*
- Type of culvert entrance condition
- Culvert outlet appurtenances and energy dissipation calculations and designs
- Copies of all computer analyses, such as HY8 output, and VDOT Form LD-269 (Appendix 8B-1) if the culvert was designed manually
- Roadway geometry (plan and profile)
- Potential flood hazard to adjacent properties

*3-sided drainage structures, bottomless culverts, arches, etc., are to be considered as vertical wall bridge abutments for the purpose of determination of scour and/or the need for scour countermeasures. See Chapter 12 – Bridge and Structure Hydraulics for further details.[*]

All hydrologic and hydraulic design computations and all maps, studies, reports, comments and interviews pertinent to the culvert design must be concisely and legibly recorded in a form that can be easily filed and microfilmed for ready recall. This data must remain easily accessible for reference when filing for permits and when future changes or designs are made.

In most cases, the design flood, the overtopping flood, and the 100-year flood should be shown in the culvert design documentation. The use of the VDOT culvert form (LD-269) in Appendix 8B-1 provides a convenient form on which to record the pertinent data.

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8.5.2 VDOT Culvert Design Procedure

The following design procedure provides a convenient and organized method for designing culverts for a constant discharge, considering inlet and outlet control. The procedure does not address the effect of storage, which is discussed in Chapter 11, Stormwater Management.

- The designer should be familiar with all of the equations in Section 8.4 before using these procedures
- Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or overly costly structure
- The culvert calculation form has been provided in Appendix 8B-1 to guide the user. It contains blocks for the project description, designer's identification, hydrologic data, culvert dimensions and elevations, roadway controls and property elevations, trial culvert description, inlet and outlet control HW, culvert barrel selected, and comments.

Step 1 Assemble site data and project file

a. The minimum site data are:

- USGS, site and location map
- Embankment cross section
- Roadway profile
- Photographs
- Field visit (sediment, debris)
- Design data at nearby structures
- Existing utilities

b. Studies by other agencies including:

- Small dams — NRCS, USCOE, TVA, BLM
- Canals — NRCS, USCOE, TVA, USBR
- Floodplain — NRCS, USCOE, TVA, FEMA, USGS, NOAA
- Storm drain - local or private

c. Environmental constraints including:

- Commitments contained in review documents
- Commitments contained in permits or permit applications
- Fish migration
- Wildlife passage
- Wetlands resources

d. Design criteria:

- Review Section 8.3 for applicable criteria

- Prepare risk assessment or analysis, if needed

Step 2 Determine hydrology

- See Chapter 6, Hydrology
- Minimum data are drainage area map and a discharge-frequency plot

Step 3 Design downstream channel

- See Chapter 7, Ditches and Channels
- Minimum data are geometry and the rating curve for the channel that provides tailwater elevations for various flood frequencies

Step 4 Summarize data on design form

- Enter data from steps 1-3

Step 5 Select design alternative

- See Section 8.3.3, Geometric Criteria
- Choose culvert material, shape, and entrance type
- Consider flow line, cover, and utilities

Step 6 Select design discharge (Q_d)

- See Section 8.3.2 Hydraulic Criteria
- Determine flood frequency from criteria
- Determine Q from discharge-frequency plot (Step 2)
- Divide Q by the number of barrels
- Select trial size

Step 7 Determine inlet control headwater depth (HW_i)

- Use the appropriate inlet control nomographs in Appendix 8C

Step 8 Determine outlet control headwater depth at inlet (HW_{oi}):

a. Calculate the tailwater depth (TW) using the design flow rate and normal depth (single section), using a water surface profile, or obtain it from other sources

b. Calculate critical depth (d_c) using the appropriate chart in Appendix 8C

   - Locate flow rate and read d_c
   - d_c cannot exceed D
   - If d_c>0.9D, consult Handbook of Hydraulics (King and Brater) for a more accurate d_c, if needed, since curves are truncated where they converge

c. Calculate $\left(\frac{d_c+D}{2}\right)$

d. Determine h_o

h_o = the larger of TW or $\left(\frac{d_c+D}{2}\right)$

e. Determine K_e

f. Entrance loss coefficient from Appendix 8D-2

g. Determine losses through the culvert barrel (H)

- Use the nomographs in Appendix 8C or Equation 8.5 or 8.6 if outside range of nomograph scales

h. Calculate outlet control headwater (HW_{oi})

- Use Equation 8.8, if V_u and V_d are neglected:

$$HW_{oi} = h_o + H - S_o L$$

- Add other losses (bends, grates, etc.) to right side of equation.
- Use Equation 8.1, 8.4c and 8.6 to include V_u and V_d.
- If HW_{oi} is less than 1.2D and control is outlet control:
  - The barrel may flow partly full
  - The approximate method of using the greater of tailwater or $\left(\frac{d_c+D}{2}\right)$ may not be applicable
  - Backwater calculations should be used to check the result
  - If the headwater depth falls below 0.75D, the approximate nomograph should not be used

Step 9 Determine controlling headwater (HW_c)

a. Compare HW_i and HW_{oi}, and use the higher

b. Compare HW to allowable HW criteria (cover, $\left(\frac{HW}{D}\right)$, shoulder)

Step 10 Compute discharge over the roadway (Q_r) if applicable (See Section 8.4.3.6)

Step 11 Compute total discharge (Q_t)

$$Q_t = Q_d + Q_r$$

Step 12 Calculate outlet velocity (V_o) and normal depth (d_n)

If **inlet control** is the controlling headwater:

a. Calculate flow depth at culvert exit

- Use normal depth (d_n), or
- Use water surface profile

b. Calculate flow area (A).

c. Calculate exit velocity, $V_o = \frac{Q}{A}$

If **outlet control** is the controlling headwater:

a. Calculate flow depth at culvert exit

- Use (d_c) if d_c > TW
- Use (TW) if d_c < TW < D
- Use (D) if D < TW

b. Calculate flow area (A)

c. Calculate exit velocity, $V_o = \frac{Q}{A}$

Step 13 Review results

Compare alternative design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:

- The barrel must have adequate cover
- The length should be close to the approximate length
- The headwalls and wingwalls must fit the site
- The allowable headwater should not be exceeded and $\left(\frac{HW}{D}\right)$ should be at least 1.0 and not exceed 1.5
- The allowable overtopping flood frequency should not be exceeded

Step 14 Select and analyze check storm discharge

Step 15 Related designs

Consider the following options (See Sections 8.3.6 and 8.4.4 and Chapter 11, Stormwater Management):

- Tapered inlets if culvert is extremely long, in inlet control, and has limited available headwater
- Flood routing if a large upstream headwater pool exits
- Energy dissipators or standard EC-1, as needed. Special design energy dissipators may be required. Appendix 8E-1 contains procedures and discussion for a riprap basin
- Weirs, if needed to maintain low flow through multiple barrel culverts

8.5.2.1 Culvert Design Sample Problems

The following example problem follows the Design Procedure Steps described in Section 8.5.2.

Step 1. Assemble Site Data and Project File

a. Site survey project file contains:

   - USGS, site, and location maps
   - Roadway profile, and
   - Embankment cross-section

b. Site visit notes indicate:

   - No sediment or debris problems
   - No nearby structures
   - Studies by other agencies - none

c. Environmental, risk assessment shows:

   - No buildings near floodplain
   - No sensitive floodplain values
   - No FEMA involvement
   - Convenient detours exist

d. Design criteria:

   - 50-year frequency for design, and
   - 100-year frequency for check storm
   - Allowable headwater depth for design flood = 8.5 ft.
   - 100-year floodplain depth = 10.0 ft.

Step 2. Determine Hydrology

USGS Regression equations yield:

Q_{50} = 400 cfs
Q_{100} = 500 cfs

Step 3. Account for tailwater

Slope = 0.05 ft./ft.
Length = 100 ft.

The predetermined depths and velocities for the downstream channel are:

Q (cfs)	TW (ft)	V (ft/s)
400	2.8	18
500	3.1	19

Step 4. Summarize data on design form

(See Figure 8-11)

Step 5. Select trial design structure

Shape: Box
Size: 7.0 ft (B) by 6.0 ft (D)
Material: Concrete
Entrance: Wingwalls with 30^{o}-75^{o} flare

Step 6. Select design discharge

$Q_d = Q_{50} = 400$ cfs

Step 7. Determine inlet control headwater depth (HW_i)

Use inlet control nomograph - Appendix 8C-8

a. D = 6.0 ft

b. $\frac{Q}{B} = \frac{400}{7} = 57 \text{ cfs}$

c. $\frac{HW}{D} = 1.30$

d. $HW_i = \left(\frac{HW}{D}\right)D = (1.30)(6.0) = 7.80 \text{ ft.}$ (Neglect the approach velocity.)

Step 8. Determine outlet control headwater depth at inlet (HW_{oi}):

a. TW = 2.8 ft. for Q_{50} = 400 cfs

b. d_c = 4.6 ft. from Appendix 8C-14

c. $\left(\frac{d_c + D}{2}\right) = \left(\frac{4.6 + 6.0}{2}\right) = 5.3$ ft.

d. h_o = the larger of TW or $\left(\frac{d_c + D}{2}\right)$ = 5.3 ft.

e. K_e = 0.4 from Appendix 8D-2 (for 30^o-75^o wingwalls)

f. Determine (H) - use Appendix 8C-15

- K_e scale = 0.4
- Culvert length, L = 100 ft.
- n = 0.012 (same as on Appendix 8C-15)
- Area = 42 sq. ft.
- H = 2.3 ft.

g. $HW_{oi} = h_o + H - LS_o$

$= 2.3 + 5.3 - 100(0.05)$

$= 2.6$ ft.

Step 9. Determine controlling headwater (HW_c)

HW_i = 7.80 ft.

HW_{oi} = 2.6 ft.

HW_c = The greater of HW_i or HW_{oi}

$HW_c = HW_i$ = 7.80 ft.

The culvert is in inlet control

Step 10. Compute discharge over the roadway (Q_r)

Not applicable

Step 11. Compute total discharge (Q_r)

Q_t = 400 cfs

Step 12. Determine outlet velocity (V_o)

- Use Appendix 8C-83
- Enter 4.8 (400 x 0.012) on the horizontal, "Qn"* scale

* Rev 9/09

- Read vertically to the “Slope” curve of 0.05
- Read horizontally to the “Vn" scale and find a value of 0.37. Then[*] divide this by the “n” value (0.012) and find a velocity of 30.8 fps.

Step 13. Repeat steps 5-10 for check flood (100-yr.):

- Compare design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:
- 100-year floodplain depth = 10.0 ft.>9.6 ft.
- Overtopping flood frequency > 50-yr.

Step 14. Design Considerations (None)

Step 15. Complete any additional necessary documentation

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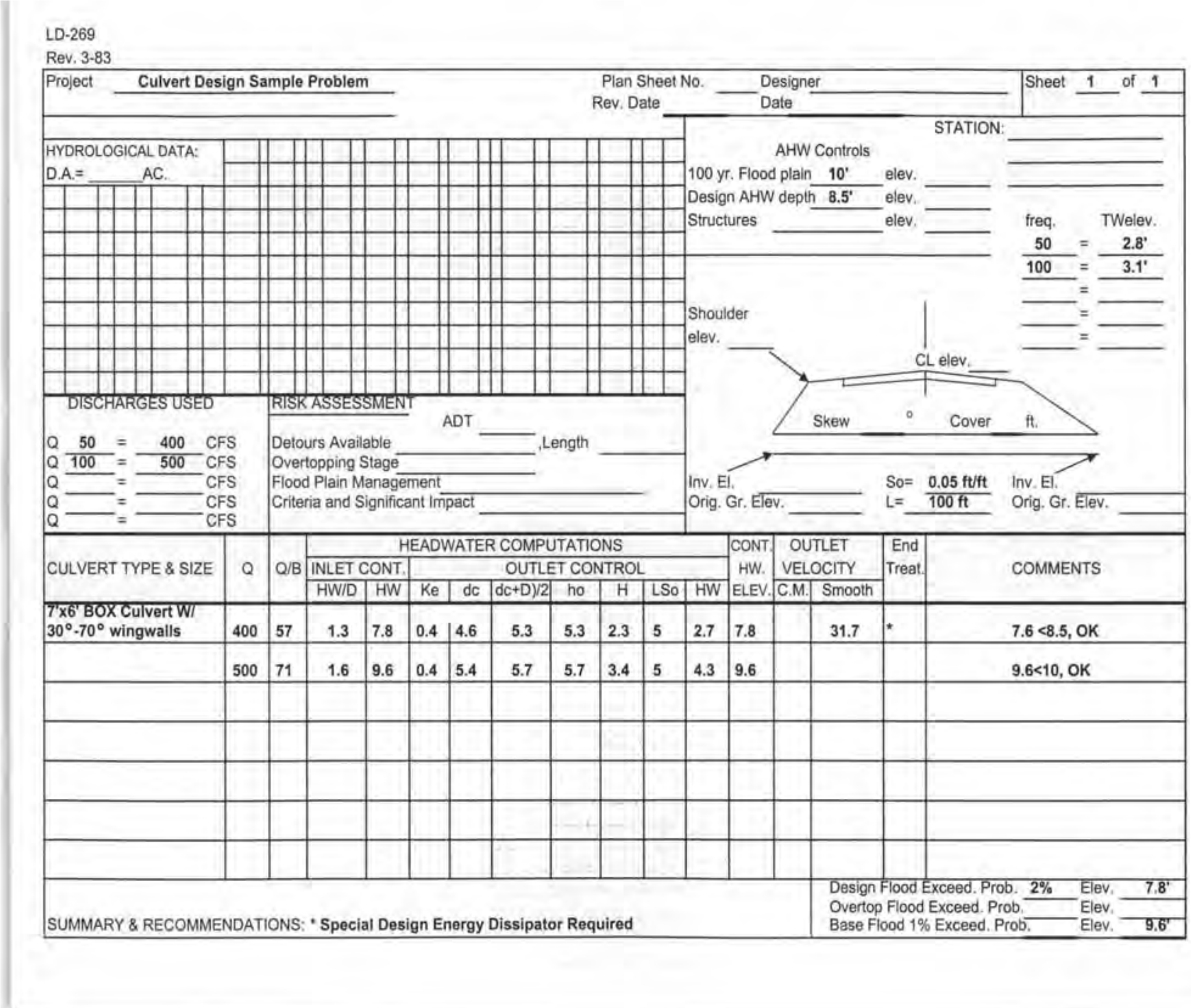

LD-269
Rev. 3-83

Project: Culvert Design Sample Problem
Plan Sheet No. ____ Designer ____ Sheet 1 of 1
Rev. Date ____ Date ____

STATION: ____

HYDROLOGICAL DATA:
D.A.= ______AC.

AHW Controls
100 yr. Flood plain 10' elev. ____
Design AHW depth 8.5' elev. ____
Structures ____ elev. ____

freq.		TWelev.
50	=	2.8'
100	=	3.1'
	=	
	=	
	=	

Shoulder elev. ____ CL elev. ____
Skew ____ ° Cover ____ ft.
Inv. El. ____ Orig. Gr. Elev. ____
So= 0.05 ft/ft L= 100 ft
Inv. El. ____ Orig. Gr. Elev. ____

DISCHARGES USED
Q 50 = 400 CFS
Q 100 = 500 CFS
Q ____ = ____ CFS
Q ____ = ____ CFS
Q ____ = ____ CFS

RISK ASSESSMENT
ADT ____
Detours Available ____, Length ____
Overtopping Stage ____
Flood Plain Management ____
Criteria and Significant Impact ____

CULVERT TYPE & SIZE	Q	Q/B	HEADWATER COMPUTATIONS: INLET CONT. HW/D	HW	OUTLET CONTROL: Ke	dc	(dc+D)/2	ho	H	LSo	HW	CONT. HW. ELEV.	OUTLET VELOCITY C.M.	Smooth	End Treat.	COMMENTS
7'x6' BOX Culvert W/ 30°-70° wingwalls	400	57	1.3	7.8	0.4	4.6	5.3	5.3	2.3	5	2.7	7.8		31.7	*	7.6 <8.5, OK
	500	71	1.6	9.6	0.4	5.4	5.7	5.7	3.4	5	4.3	9.6				9.6<10, OK

SUMMARY & RECOMMENDATIONS: * Special Design Energy Dissipator Required

Design Flood Exceed. Prob. 2% Elev. 7.8'
Overtop Flood Exceed. Prob. ____ Elev. ____
Base Flood 1% Exceed. Prob. ____ Elev. 9.6'

Figure 8-11. Completed Culvert Design Form, Sample Problem

8.5.3 Buoyancy Protection Procedure

8.5.3.1 Hydrostatic Uplift and Resistance

- Resistance = Weight of pipe + Weight of water (in pipe) + Weight of fill (over pipe), lbs/ft.
- Hydrostatic Uplift (Buoyant) Force = Weight of water displaced by the pipe, lbs. per ft.
- The following average values can be used in the analysis:

Weight of Pipe - See manufacturer's weight tables for type and size of pipe specified.

Weight of Fill (Dry) - 100 lbs. per cubic foot

Weight of Fill (Saturated) - 37.6 lbs. per cubic foot

Weight of Water - 62.4 lbs. per cubic foot

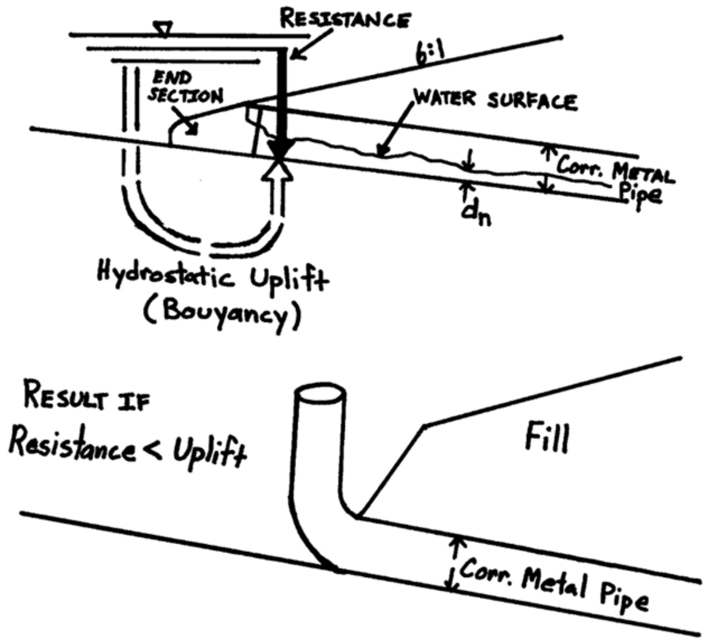

Figure 8-12. Hydrostatic Uplift Forces and Effects on Pipe

If Resistance < Hydrostatic Uplift:

Increase weight on end of pipe by adding concrete endwall or concrete anchor blocks.

8.5.3.2 Buoyancy Protection Sample Problem

Given:

48-inch Corrugated Metal Pipe, 12 gage
Fully Coated with Paved Invert
Std. ES-2 End Section
Q = 96 cfs

Computed Values:

HW/D= 1.25
HW= 5 ft.
d_n= 2.5 ft.
d_c= 2.8 ft.

Assumed Values:

Weight of Fill (Dry) = 100 lbs. per cu. ft.
Weight of Fill (Saturated) = 37.6 lbs. per cu. ft.
Weight of Water = 62.4 lbs. per cu. ft.
Weight of Pipe = 84 lbs. per L.F.
(Handling weight of corrugated steel pipe available in Appendix 8F-1 and 8F-2)

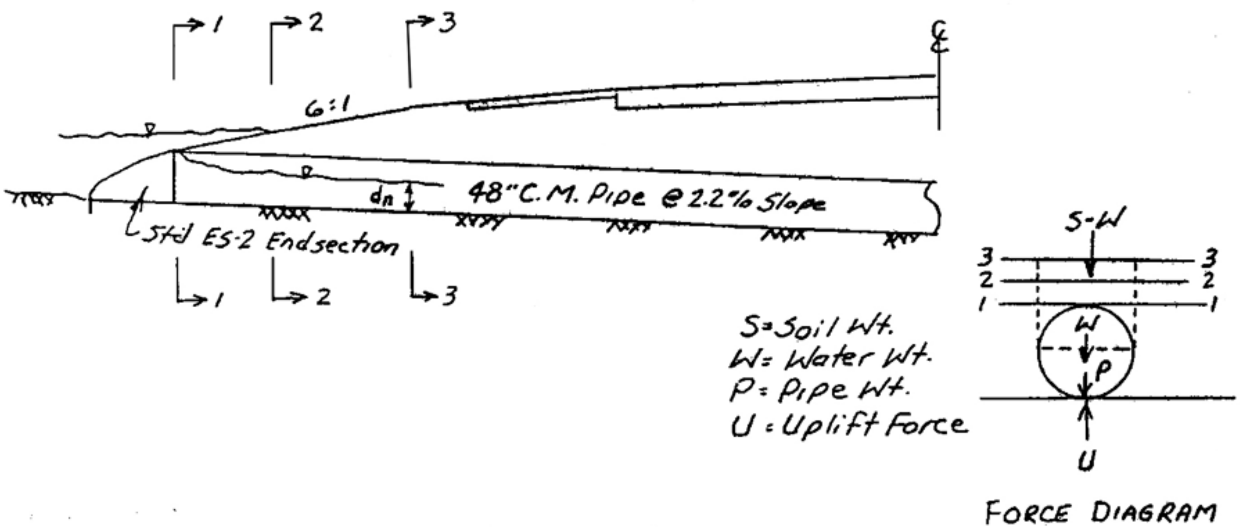

Figure 8-13. Buoyant Forces Acting on Pipe

Step 1: Compute buoyant force acting on pipe
(At any section along length of pipe)

Buoyant force (lbs./ft.) = Weight of water displaced by pipe (lbs./cu. ft.)

Buoyant force = L(A)(γ)

Where:

L = Unit length of pipe, ft
A = Cross sectional area of pipe, sq. ft.
γ = Unit weight of water = 62.4 lbs/ft^3

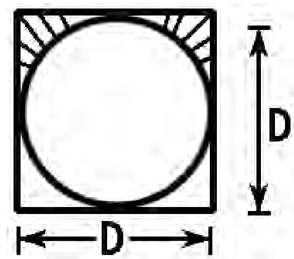

Figure 8-14. Weight of Fill, Section 1

$$= 1\left(\frac{\pi D^2}{4}\right)(62.4)$$

$$= 1\left(\frac{\pi (4)^2}{4}\right)(62.4)$$

$$= 784 \text{ lbs/ft.}$$

Step 2: Compute total surcharge at section 1

(Located at inlet end of pipe)

Surcharge (lbs./ft.) = Wt. of Fill + Wt. of Water + Wt. of Pipe

Compute weight of fill material (Hatched Area, Figure 8-14)

Weight of Fill = Area x 1 ft. x 37.6 lbs./cu. ft. (saturated fill)

$$\text{Area} = \left[\frac{D^2 - \frac{\pi D^2}{4}}{2}\right] = \left[\frac{4^2 - \frac{\pi(4^2)}{4}}{2}\right] = 1.7 \text{ sq.ft.}$$

Weight of Fill = 1.7 (1)(37.6)

Weight of Fill = 64 lbs/ft

Compute weight of water (inside pipe)

Weight of Water = Area of flow x 1 ft. x 62.4 lbs./cu. ft.

Assume depth of water, $d = d_n = 2.5$ ft.

$$\frac{d}{D} = \frac{2.5}{4} = 0.625$$

$$\frac{\text{Area}}{D^2} = 0.516 \text{ (From Appendix 8F-5)}$$

Area of flow = 0.516 x 4 ft.^{2} = 8.26 ft^2

Weight of Water = 8.26 ft.^{2} x 1 ft x 62.4 lbs/ft$^{3.}$
Weight of Water = 515 lbs/ft.

- Determine weight of pipe

  84 lbs/ft.

- Compute total surcharge at section 1

  Surcharge = Wt. of Fill + Wt. of Water + Wt. of Pipe
  Surcharge = 64 + 515 + 84
  = 663 lbs/ft.

- Section 1 Summary

  Surcharge (663 lbs./ft.) < Buoyant Force (784 lbs./ft.)

  Therefore, pipe is unstable at Section 1

Step 3: Compute total surcharge at Section 2

(Section 2 is located where headwater elevation intercepts the fill slope 6 ft from the inlet end of the pipe)

Surcharge (lbs./ft.) = Wt. of Fill + Wt. of Water + Wt. of Pipe

- *Compute weight of fill material (Hatched Area, Figure 8-15)*

  Weight of Fill = Area ft^2 x 1 ft. x 37.6 lbs./ft^3 (saturated fill)

$$\text{Area} = \left[\frac{D^2 - \frac{\pi D^2}{4}}{2}\right] + 1(D) = \left[\frac{4^2 - \frac{\pi 4^2}{4}}{2}\right] + 1(4) = 5.7 \text{ sq. ft.}$$

  Weight of Fill = 5.7 ft.^{2} x 1 ft. x 37.6 lbs./ft^3

  Weight of Fill = 214 lbs/ft.

- *Compute weight of water (Inside Pipe)*

  Assume depth of water, $d = d_n = 2.5$ ft.

  Weight of Water = 515 lbs./ft.
  (Same as Section 1)

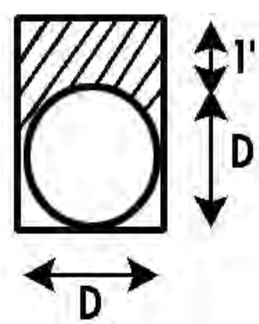

Figure 8-15. Weight of Fill Material, Section 2

- *Determine weight of pipe*

  84 lbs./ft. (Same as Section 1)

- *Compute total surcharge at Section 1*

  Surcharge (lbs./ft.) = Wt. Fill + Wt. Water + Wt. Pipe

  Surcharge = 214 + 515 + 84 = 813 lbs./ft.

- *Section 2 Summary*

  Surcharge (813 lbs./ft.) > Buoyant Force (784 lbs./ft.)

  Therefore, pipe is stable at Section 2

Step 4: Determine minimum weight required to counteract buoyant force

a. Plot a graph of length along the pipe (from inlet end) versus total surcharge buoyancy (weight). Let the horizontal axis represent the length along the pipe (ft.) and the vertical axis represent the surcharge/buoyancy (lbs./ft) as shown in Figure 8-16.

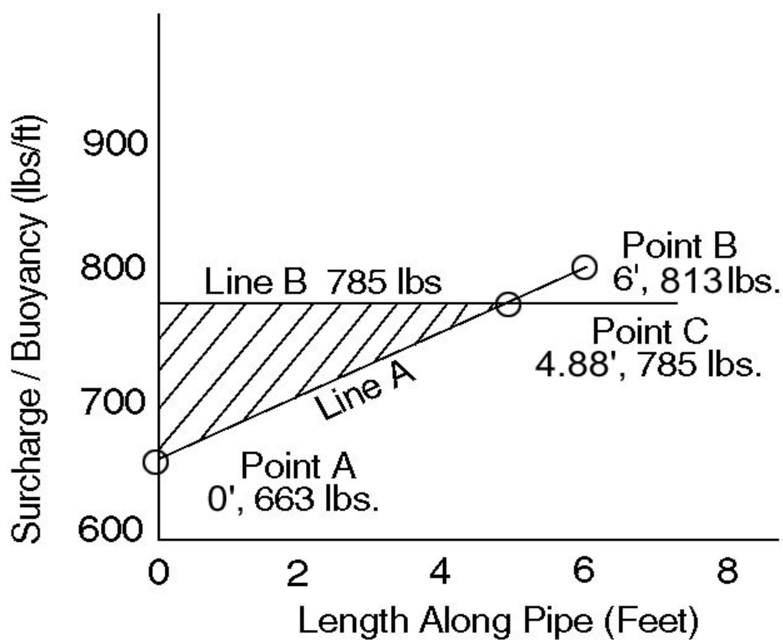

Figure 8-16. Surcharge/Buoyancy along Length of Pipe

b. Plot the values of length along pipe and total surcharge for Section 1 (from Step 2) and Section 2 (from Step 2) on the graph (Points A and B) and connect them with a straight line (Line A).

c. Plot horizontal line (Line B) on the graph representing the buoyant force computed in Step 1.

d. The area of the triangle formed by Line A, Line B and the vertical axis of the graph (hatched area) represents the minimum weight required to balance the uplift (buoyant) force.

e. Determine minimum required weight (area of triangle).

   1) Using ratio and proportion analysis, determine length along horizontal axis where Line A and Line B intersect (Point C).

      Find intersection (Point C) at 4.88 ft.

   2) Weight (Area) = (Vertical side x Horizontal side)/2.

   3) Weight = (122 lbs./ft. x 4.88 ft.) / 2 = 298 lbs.

f. Determine minimum weight of required anchor block. Set minimum weight of anchor block equal to the greater of:

   1) The required additional weight (Step 4e) plus 100 lbs. or

   2) 1.5 times the required additional weight (Step 4e).

g. Determine size of required anchor block.

   1) Use minimum size anchor block if its weight is equal to or greater than minimum weight required (Step f).

   2) If minimum weight required (Step f) is greater than weight of minimum size anchor block, increase dimensions of minimum size anchor block to provide weight equal to or greater than minimum required weight (Step f).

   Typical counterweight details are shown in Figure 8-17. Dimensions for the weight of minimum size counterweight can be found in Appendix 8F-3.

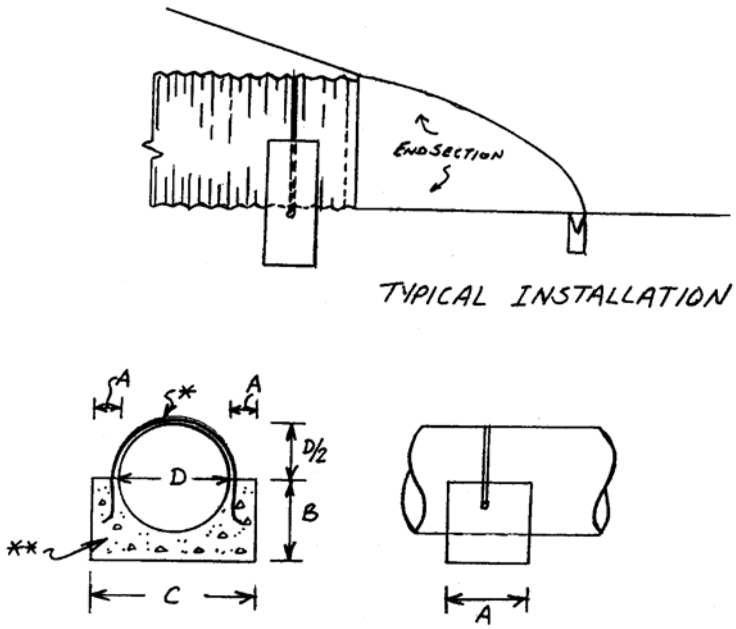

Figure 8-17. Counterweight Details for Pipes Subject to Uplift Forces

*1/2-inch diameter steel rod to be field bent as necessary and embedded in fresh concrete
** Class A-3 concrete

Dimensions:
A=Variable as needed, 6-inch minimum
B=D/2+12 inch
C=Variable as needed, D+12 inch minimum
D=Pipe diameter

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