CHAPTER 9

CULVERTS

9.0 INTRODUCTION TO CULVERTS

A. General - Culverts. A culvert conveys surface water through a roadway embankment or away from the highway right-of-way. In addition to this hydraulic function, it also must carry construction traffic, highway traffic, and earth loads; therefore, culvert design involves both hydraulic and structural design. The hydraulic and structural designs must be such that risks to traffic, property damage, and failure from floods are consistent with good engineering practice and economics. This chapter describes the hydraulic aspects of culvert design, construction and operation of culverts, and makes references to structural aspects only when they are related to the hydraulic design.

Culverts, as distinguished from bridges, usually are covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed or concrete riprap channel serving as the bottom. For economic and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions.

At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements for the stream crossing. The designer should choose the appropriate structure based on the following (not necessarily in order):

- Construction and maintenance costs.
- Risk of failure.
- Risk of property damage.
- Traffic safety.
- Environmental and aesthetic considerations.
- Construction expedience.

For project sites where a the normal clear span is 24 feet or less and a box culvert is proposed to replace an existing short-span bridge, the *Joint Agency Guidance for The Analysis of Environmental Impacts and Other Issues for Short Span Structures* should be used. The guidance document is located in Appendix 9A. The document provides an overview of the various alternatives related to structure replacements and their associated construction and environmental impacts and other issues associated with construction of a box culvert versus bottomless structures, including bridges, rigid frames, and arches. This guidance is intended to form a basis for the development of an alternatives analysis suitable for submission with a permit application to the Department of Environmental Protection (PA DEP) to satisfy 25 PA Codes 105.13 and 93.4c.

Culverts are considered minor structures as compared with bridges, but they are of great importance for drainage and the integrity of the facility. Although the cost of individual culverts usually is relatively small, the total cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, culvert maintenance may account for a large share of the total cost of maintaining highway hydraulic features. The designer can achieve improved traffic service and reduced cost by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

The designer must consider analysis of the following items before starting the culvert design process:

- Site and roadway data.
- Design parameters, including shape, material and orientation.
- Hydrology (flood magnitude versus frequency relation).
- Channel analysis (stage versus discharge relation).

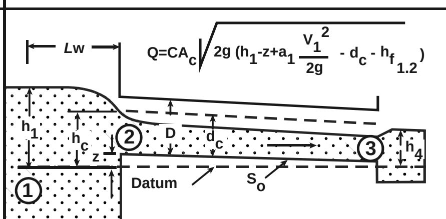
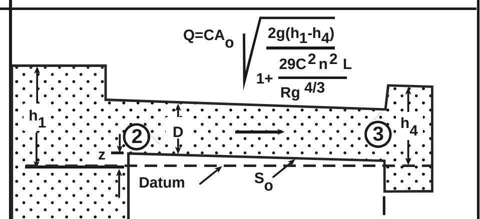
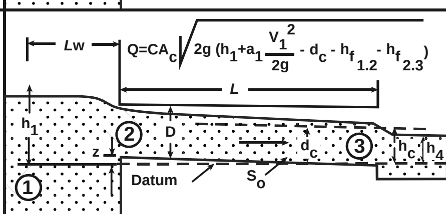
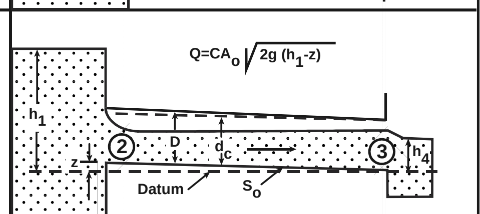
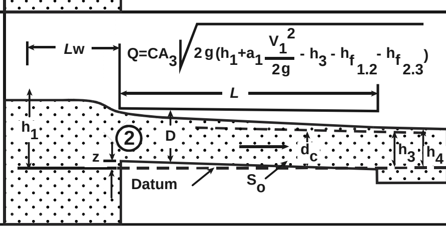
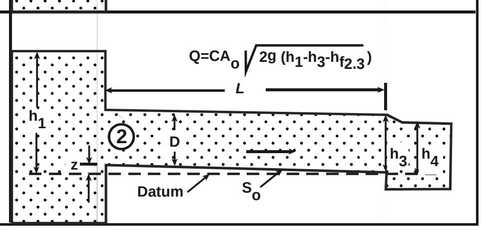
B. Concepts. Important terms utilized in culvert design are defined below.

1. Critical Depth. Critical depth is the depth at which the specific energy of a given flow rate is at a minimum. For a given discharge and channel cross section geometry there is only one critical depth.

2. Crown. The crown is the inside top of the culvert.

3. Flow Type. The USGS (1988) has established six culvert flow types which assist in determining the flow conditions at a particular culvert site. Figure 9.1 illustrates the six flow conditions.

Figure 9.1 USGS Flow Types (USGS, 1988)

TYPE	EXAMPLE	TYPE	EXAMPLE
1 CRITICAL DEPTH AT INLET $\frac{h_1 - z}{D} < 1.5$ $h_4 / h_c < 1.0$ $S_o > S_c$	 $Q = CA_c\sqrt{2g\left(h_1 - z + a_1\frac{V_1^2}{2g} - d_c - h_{f_{1.2}}\right)}$	4 SUBMERGED OUTLET $\frac{h_1 - z}{D} > 1.0$ $h_4 / D > 1.0$	 $Q = CA_o\sqrt{\frac{2g(h_1 - h_4)}{1 + \frac{29C^2n^2L}{Rg^{4/3}}}}$
2 CRITICAL DEPTH AT OUTLET $\frac{h_1 - z}{D} < 1.5$ $h_4 / h_c < 1.0$ $S_o < S_c$	 $Q = CA_c\sqrt{2g\left(h_1 + a_1\frac{V_1^2}{2g} - d_c - h_{f_{1.2}} - h_{f_{2.3}}\right)}$	5 RAPID FLOW AT INLET $\frac{h_1 - z}{D} \geq 1.5$ $h_4 / D \leq 1.0$	 $Q = CA_o\sqrt{2g(h_1 - z)}$
3 TRANQUIL FLOW THROUGHOUT $\frac{h_1 - z}{D} < 1.5$ $h_4 / D \leq 1.0$ $h_4 / h_c > 1.0$	 $Q = CA_3\sqrt{2g\left(h_1 + a_1\frac{V_1^2}{2g} - h_3 - h_{f_{1.2}} - h_{f_{2.3}}\right)}$	6 FULL FLOW FREE OUTFALL $\frac{h_1 - z}{D} \geq 1.5$ $h_4 / D \leq 1.0$	 $Q = CA_o\sqrt{2g(h_1 - h_3 - h_{f_{2.3}})}$

4. Free Outlet. A free outlet has a tailwater equal to or lower than critical depth in the culvert. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

5. Improved Inlet. An improved inlet has an entrance geometry which decreases the flow contraction at the inlet and thus increases the capacity of culverts. These inlets are referred to as either side- or slope-tapered (walls or bottom tapered).

6. Invert. The invert is the flowline of the culvert (inside bottom).

7. Normal Flow. Normal flow occurs in a channel reach when the discharge, velocity and depth of flow along a streamline do not change with longitudinal position. The water surface and channel bottom will be parallel. This type of flow can exist in a culvert operating on a constant slope provided the culvert is sufficiently long.

8. Slope:

    - A steep slope occurs when critical depth is greater than normal depth.
    - A mild slope occurs when critical depth is less than normal depth.

9. Submerged:

- A submerged outlet occurs when the tailwater elevation is higher than the crown of the culvert outfall.
- A submerged inlet occurs when the headwater submerges the inlet. This usually occurs when the headwater is greater than 1.2D, where D is the culvert diameter or barrel height.

C. General Culvert Considerations. A culvert is a structure that is usually a closed conduit or waterway that may be designed hydraulically to take advantage of submergence to increase hydraulic capacity. Culverts are constructed from a variety of materials and are available in different shapes and configurations. Culvert type selection includes the choice of material, shape and cross section, and the number of culvert barrels. Total culvert cost can vary considerably depending upon the culvert type selected. Roadway profile, terrain, foundation condition, fish passage requirements, shape of the existing channel, allowable headwater, channel characteristics, flood damage evaluations, construction and maintenance costs, and service life are some of the factors which influence culvert type selection.

1. Shapes. The shape of a culvert is not the most important consideration at most sites, so far as hydraulic performance is concerned. Rectangular, arch or circular shapes of equal hydraulic capacity generally are satisfactory. It often is necessary, however, for the culvert to have a low profile to accommodate terrain or limited fill height. Construction cost, the potential for clogging by debris, limitations on headwater elevation, fill height, and the hydraulic performance of the design alternatives enter into the selection of the culvert shape. Design and construction specifications and methods of determining maximum cover for some shapes and materials are included in PennDOT's Design Manual series.

Numerous cross sectional shapes are available. The most commonly used shapes include circular, box (rectangular), elliptical, pipe-arch, and arch. Shape selection is based on the cost of construction, upstream water surface elevation, roadway embankment height, and hydraulic performance, including limitations. Several commonly used culvert shapes are discussed in the following sections.

a. Circular. The most commonly used culvert shape is circular. This shape is preferred due to the available structural options for various fill heights. Various standard lengths of circular pipe in standard strength classes are available from local suppliers at reasonable cost. The need for cast-in-place construction generally is limited to culvert end treatments and appurtenances such as access holes.

b. Box or Rectangular. A culvert of rectangular cross section can be designed to pass large floods and to fit nearly any site condition. A rectangular culvert lends itself more readily than other shapes to low allowable headwater situations, since the height may be decreased and the total span increased to satisfy the requirement. The required total span can consist of one or multiple cells. Precast concrete and metal box sections often are used to overcome the increased construction time required for cast-in-place boxes.

c. Pipe Arch and Elliptical. Pipe arch and elliptical shapes are used in lieu of circular pipes where there is limited cover or overfill. Structural strength characteristics usually limit the height of fill over these shapes except when the major axis of the elliptical shape is laid in the vertical plane. When compared to circular sections, these shapes are more expensive for equal hydraulic capacity because of the additional structural material required.

d. Arches. Arch culverts are used in locations where vertical clearance above a waterway is a desirable feature, and where foundations are adequate for structural support. Such structures can be installed to maintain the natural stream bottom for fish passage, but the potential for failure from scour must be carefully evaluated. Structural plate metal arches are limited to use in low cover situations but have the advantage of rapid construction and low transportation and handling costs. This is especially advantageous in remote areas and in rugged terrain. Additionally, precast concrete arch systems can be fabricated in a variety of span lengths, and can be considered instead of short bridges in some instances.

e. Multiple Barrels. Culverts consisting of more than one barrel are useful in wide channels where the constriction or concentration of flow is to be kept to a minimum. A low roadway embankment offering limited cover may require the use of a series of small openings. The barrels may be separated by a considerable distance in order to maintain flood flow distribution. The practice of altering channel geometry to accommodate a wide culvert generally will result in deposition in the widened channel and in the culvert. Where overbank flood flows occur, relief culverts with inverts at the floodplain elevation should be considered to avoid the need for channel alteration.

In the case of box culverts, it may be more economical to use multiple structures rather than a wide single span. In some locations, multiple barrels have a tendency to catch debris which clogs the waterway. They are also susceptible to ice jams and the deposition of silt in one or more barrels. Alignment of the culvert face normal to the approach flow and installation of debris control structures can help to alleviate these problems. To avoid widening the natural channel, to provide overflow (flood) relief, to support environmental preservation, and to reduce sedimentation and debris problems, it is good practice to design one barrel of the multiple-barrel system to carry low flows.

2. Material. The selection of the material for a culvert is dependent upon several factors that can vary considerably depending on location; therefore, consideration should be given to the following variables:

- Structural strength, considering fill height, loading conditions, and foundation conditions.
- Hydraulic efficiency, considering inlet geometry, Manning's roughness, cross section area, shape, and tailwater.
- Installation, local construction practices, availability of pipe bedding material, and joint tightness requirements.
- Durability, considering water and soil environment (pH and resistivity), corrosion (metallic coating selection), and abrasion.
- Cost, considering availability of materials.

Commonly used culvert materials include:

- Concrete (reinforced and non-reinforced).
- Steel (smooth and corrugated).
- Aluminum (smooth and corrugated).
- Plastic (smooth and corrugated).

The most economical culvert is one which has the lowest total annual cost over the design life of the project; however, the initial cost should not be the only basis for culvert material selection. Replacement costs and traffic delay are usually the primary factors in selecting a material that has a long service life. If two or more culvert materials are equally acceptable for use at a site, including hydraulic performance and annual costs for a given life expectancy, consideration should be given to bidding the materials as alternates.

3. Inlets. A multitude of different inlet configurations is utilized on culvert barrels. These include both prefabricated and cast-in-place installations. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete head walls, pre-cast or prefabricated end sections, and culvert ends mitered to conform to the fill slope. Structural stability, aesthetics, erosion control, and fill retention are considerations in the selection of various inlet configurations.

The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the natural channel is often wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will lessen the energy loss and thus create a more hydraulically efficient inlet condition. Beveled edges are more efficient than square edges; side-tapered and slope-tapered inlets, commonly referred to as improved inlets, further reduce head loss due to flow contraction. Depressed inlets, such as slope-tapered inlets, increase the effective head on the flow control section, thereby increasing the culvert efficiency.

D. Economics. The hydraulic design of a culvert installation always includes an economic evaluation. A wide spectrum of flood flows with associated probabilities will occur at the culvert site during its service life. The benefits of constructing a large capacity culvert to accommodate all of these events with no detrimental flooding effects are normally outweighed by the initial construction costs. Thus, an economic analysis of the trade-offs may be performed with varying degrees of effort and thoroughness depending on the project scope.

- Benefits and costs. The purpose of a highway culvert is to convey water through a roadway embankment. The major benefits of the culvert are decreased traffic interruption time due to roadway flooding and increased driving safety. The major costs are associated with the construction of the roadway embankment and the culvert itself. Maintenance of the facility and flood damage potential also may be factored into the cost analysis.
- Analysis. Traditional economic evaluations for minor stream crossings have been somewhat simplistic. Culvert design flows are based on the importance of the roadway being served with little attention given to other economic and site factors. Inundation of the travelway dictates the level of traffic service provided by a waterway facility. The travelway overtopping flood level identifies the upper limit of serviceability, and it provides one of the important definitions of the term design flood. The minimum magnitudes of design floods are given in Chapter 7, *Hydrology*.
- Regulatory requirements also may apply. These may include:

  25 PA Code § 102 Erosion and Sediment Control
  25 PA Code § 105 Dam Safety and Waterway Management
  25 PA Code § 106 Floodplain Management
  25 PA Code § 111 Stormwater Management
  FAPG 23 CFR 650.115 (a)(1)(ii) Risk Assessment

9.1 DESIGN CONSIDERATIONS

The hydraulic design of a culvert consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. To meet this conveyance function adequately, the design must include consideration of the variables discussed in the following sections.

A. Design Frequency. Choose the appropriate design storm frequency according to Section 7.2 and Sections 10.2.C, 10.3.C, 10.6.E and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Table 10.6.1.

B. Headwater. Energy is required to accelerate flow through a culvert. This energy comes from an increase in depth versus the natural unobstructed water surface elevation on the upstream side of the culvert. The depth of the upstream water surface measured from the invert at the culvert entrance generally is referred to as headwater depth.

The headwater of a culvert is a function of several parameters, including the culvert geometric configuration. The culvert geometric configuration is based primarily on the allowable headwater. This geometric configuration consists of the following:

- The number of barrels.
- Barrel dimensions.
- Length.
- Slope.
- Entrance characteristics.
- Barrel roughness characteristics.

Potential damage to adjacent property or inconvenience to property owners should be of primary concern in the design of all culverts. In urban areas, the potential for damage to adjacent property is greater because of the high number and value of properties that can be affected. If roadway embankments are low, flooding of the roadway and delay to traffic are usually of primary concern, especially on highly traveled routes.

Culvert installations under high fills may present the designer an opportunity for use of a high headwater or ponding to attenuate flood peaks. If deep ponding is considered, the possibility of catastrophic failure (see DEP 25 PA Code §105) should be investigated because a breach in the highway fill could be quite similar to a dam failure.

The study of culvert headwater should include verification that watershed divides are higher than design headwater elevations. If the divides are not sufficiently high to contain the headwater, culverts of lesser depths or earthen training dikes may be used, in some instances, to avoid diversion across drainage divides. In flat terrain, drainage divides often are undefined or nonexistent, and culverts should be located and designed for the least disruption of the existing flow distribution.

C. Site Data. For purposes of this section, site information from any source is broadly classified as survey data. The survey should provide the designer with sufficient data for locating the culvert and identifying information on all features which may be affected by installation of the culvert. Examples of such information include elevations and locations of houses, commercial buildings, croplands, roadways and utilities.

Sources of data include aerial or field survey; interviews; water resource, fish and wildlife, and planning agencies; newspapers; and floodplain zoning studies. Complete and accurate survey information is necessary to design a culvert to best serve the requirements of a site. The individual in charge of the drainage survey should have a general knowledge of drainage design and coordinate the data collection with the hydraulics engineer. The amount of survey data gathered should be commensurate with the importance and cost of the proposed structure.

The extent of survey coverage required for culvert design varies with location. In areas with relatively flat slopes, backwater effects may propagate a considerable distance upstream and require an extensive survey. The extent of ponding behind culverts located in depressed regions or areas with steep slopes may be very small and require only a limited survey.

The items which should be known either through data collection or calculation include:

- Adjacent property descriptions.
- Allowable headwater level, $HW_{max.}$
- Allowable outlet velocity, $v_{max.}$
- Aquatic wildlife requirements.
- Basin area.
- Basin hydrologic characteristics.
- Channel geometry.
- Culvert shape and material.
- Culvert slope, $S_{o.}$
- Culvert hydraulic length, L.
- Debris characteristics.
- Design discharge, Q.
- Design tailwater, TW.
- Entrance conditions.
- Existing culvert description.
- Maximum allowable depth of barrel.
- Recreational requirements.
- Site geological report.
- Site history.
- Threatened and endangered species report *(impact).*

1. Field Review. Engineers designing drainage structures should be thoroughly familiar with the watershed site under consideration. Much can be learned from survey notes, but the most complete survey cannot adequately depict all watershed site considerations or substitute for a personal inspection. A site examination can be mutually beneficial to the designer and construction engineer by helping to improve the drainage design and reducing construction problems.

2. Drainage Area. Drainage area is an important factor in estimating the flood potential. For a detailed discussion on drainage area, refer to Chapter 7, *Hydrology*.

3. Channel Characteristics. The survey should describe the physical characteristics of the existing stream channel. For purposes of documentation and design analysis, sufficient channel cross sections (at least four), a stream bed profile and the horizontal alignment should be obtained to provide an accurate representation of the channel, including the floodplain area. These cross sections can be used to obtain the natural streambed width, side slopes, and floodplain width. Often, the proposed culvert is positioned at the same longitudinal slope as the streambed. The channel profile should extend far enough beyond the proposed culvert location to define the slope and locate any large stream bed irregularities, such as headcutting. The designer must also use this pre-construction data to predict the consequences of constricting the natural flood plain by installing an embankment across a flood plain.

General characteristics helpful in making design decisions should be noted. These include channel roughness, Manning's n values, the type of soil or rock in the stream bed, the bank conditions, type and extent of vegetal cover, permanent or intermittent wetlands, amount of drift and debris, ice conditions, and any other factors which could affect the sizing of the culvert and the durability of culvert materials. Photographs of the channel and the adjoining area can be valuable aids to the designer and serve as excellent documentation of existing conditions.

4. Aquatic Wildlife. Survey data should include information regarding necessary steps to accommodate aquatic wildlife. Information regarding the species present and required measures to accommodate their presence may be obtained from the resource agencies. A culvert designed for fish passage is discussed in more detail in Section 9.7.D.

5. Highwater Information. Highwater marks can be can be used to check results of flood estimating procedures, establish highway grade lines, and locate hydraulic controls. Often the highwater mark represents the energy of the stream and not the water surface. Even though the highwater marks are available it often is difficult to determine the flood discharge that created them.

When highwater information is obtained, the individuals contacted should be identified and the length of their familiarity with the site should be noted. In addition, the designer should ascertain whether irregularities such as channel blockage or downstream backwater altered the expected highwater. Other sources for such data might include commercial and school bus drivers, mail carriers, law enforcement officers, and highway and railroad maintenance personnel.

6. Existing Structures. The designer should place considerable importance on the hydraulic performance of existing structures. The performance of structures upstream or downstream from the culvert site can be helpful in the design.

Data on existing structures should be collected as available to include the following:

- Date of construction.
- Major flood events since construction, dates of occurrence, and water surface elevation.
- Performance during past floods based on interviews with the sources mentioned above.
- Scour indicated near the structure.
- Type of material in streambed and banks.
- Alignment and general description of structure, including:
    - Condition (abrasion, corrosion, or deterioration).
    - Dimensions.
    - Shape and material.
    - Flowline invert elevation.
    - Is the top of the foundation exposed, if so, what is the elevation.
- Highwater elevations with datum and dates of occurrence.
- Location and description of overflow areas.
- Photographs.
- Silt and drift accumulations.
- Evidence of headcutting in stream.
- Appurtenant structures such as:
    - Energy dissipaters.
    - Debris control structures.
    - Stream grade control devices.
    - As-built plan of structure.

7. Waterway Data. Changes in the flood plain due to roadway embankments can have significant effects on streamflow. In addition to the site data discussed in Section 9.1.C., information on the waterway must also be gathered. The waterway data includes hydraulic resistance, downstream water surface elevation (tailwater) information, and upstream storage capacity.

a. Cross Sections. A field survey provides one of the best ways of collecting stream cross section data. At least four cross sections and a channel profile should be taken to establish the stream slope, the configuration of natural channel, and the possible culvert profile. The natural streambed width, side slopes, and flood plain width may be obtained from these cross sections. The cross sectional data also will help verify the accuracy of existing topographic maps. If significant ponding is likely, additional sections may be necessary to determine the storage capacity upstream of the culvert. Likewise, additional downstream sections may be necessary to establish downstream water level (tailwater) conditions. To assess the impact of the culvert, water surface profiles may be computed beginning at some downstream cross section and carrying the computations to some point upstream. For more information on the location and extent of the data needed see Chapter 8, *Open Channels*.

Additional information on stream slope and upstream storage volume also should be obtained from the topographic maps.

b. Stream Slope. The longitudinal slope of the existing channel in the vicinity of the proposed culvert should be determined in order to establish the culvert vertical profile and to define flow characteristics in the natural stream. Often, the proposed culvert is positioned at the same longitudinal slope as the streambed. However, the culvert may have to be positioned on a different slope than the streambed, as discussed in Section 9.1.F.

c. Resistance. The hydraulic resistance of the natural channel must be evaluated in order to calculate pre-project flow conditions. This resistance usually is represented by an average Manning's "n" value. Various methods are available to evaluate resistance coefficients for natural streams including comparisons with photographs of streams with known resistance values or tabular methods based on stream characteristics. Some of these methods are discussed in:

- Chapter 8, *Open Channels*.
- HEC-RAS Hydraulic Reference Manual (U.S. Army Corps of Engineers, 2002).
- HDS-6, *River Engineering for Highway Encroachments* (FHWA, 2001b).

d. Tailwater. Tailwater is defined as the depth of water downstream of the culvert measured from the outlet invert. It is an important factor in determining culvert capacity under outlet control conditions. Tailwater may be caused by an obstruction in the downstream channel, by the hydraulic resistance of the channel, by a lake, or by a receiving stream with high water surface elevation under design assumptions. In any case, backwater calculations from the downstream control point can be performed to estimate tailwater. When hydraulic resistance of the channel controls the flow depth, normal depth approximations may be used instead of backwater calculations. Normal depth assumptions are that the channel bed slope, water surface slope, and energy grade line slope are equal and that the depth of flow does not change.

e. Upstream Storage. The stream storage capacity upstream from a culvert may have an impact upon its design. The designer can approximate upstream storage capacity from contour maps of the upstream area. However, it is preferable to obtain a number of cross sections upstream of the proposed culvert. These sections must be referenced horizontally as well as vertically. The length of the upstream reach required will depend on the expected headwater and the stream slope. The cross sections can be used to develop contour maps, or the cross sectional areas can be used to compute storage. The topographic information should extend upward from the channel bed to an elevation equal to, or greater than, the design headwater elevation in the area upstream of the culvert. Usually, it is necessary to compute backwater up to the cross section where the change in water surface elevation is equal to zero.

8. Roadway Data. The proposed or existing roadway profile affects the culvert cost, hydraulic efficiency, and alignment. Information from the roadway profile and the roadway cross sections can be obtained from preliminary roadway drawings or from standard details on roadway sections. When the culvert must be sized prior to the development of preliminary plans, a best estimate of the roadway section can be used, but the culvert design must be checked and confirmed after the roadway plans are completed.

a. Roadway Section. The roadway cross section normal to the centerline typically is available from highway plans. However, the cross section needed by the culvert designer is the section at the stream crossing where the culvert is to be located. This section may be skewed with reference to the roadway centerline. For a proposed culvert, the roadway plan, profile, and cross sectional data should be combined as necessary to obtain the desired section.

b. Culvert Length. Important dimensions and features of the culvert will become evident when the desired roadway cross section is measured or established. The dimensions are obtained by superimposing the estimated culvert barrel on the roadway cross section and the streambed profile. This superposition establishes the inlet and outlet invert elevations. If the culvert is to be depressed below the natural ground slope then the length needs to be estimated with this in mind. These elevations and the resulting culvert length are approximate since the final culvert barrel size still must be determined.

Because of the difference between the lengths of metal and non-metal end sections for pipe culverts, the length of connecting pipe can vary. To avoid showing different lengths of the connecting pipe on the plans, the designer should show on the plan only the length of the metal type pipe for all alternatives.

c. Roadway Horizontal Alignment and Vertical Profile. The roadway embankment represents the obstruction encountered by the flowing stream. The embankment can act much like a dam. The culvert is similar to the normal release structure, and the roadway crest acts as an emergency spillway in the event that the upstream pool (headwater) attains a sufficient elevation. The location of initial overtopping is dependent upon the roadway geometry. Generally, the location of overtopping should be designed to conform as closely as possible to the location of the majority of flood flow under existing conditions.

The vertical profile contained in highway plans generally follows the roadway center-line. These elevations may not represent the high point in the highway cross section. The culvert designer should determine the vertical profile which establishes roadway flooding and roadway overflow elevations. Necessary provisions must be provided to ensure that all overtopping flow is returned to the original channel.

The minimum diameter of a pipe culvert shall be 450 mm (18 in), except pipes under a 7.6 m (25 ft) or greater fill shall not be less than 600 mm (24 in). Culverts shall be provided in 150 mm (6 in) increments.

For the inverts of new pipe culverts, refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10 for the appropriate stream invert grade at both the inlet and outlet ends.

9. Allowable Headwater. The designer should base allowable headwater elevation on the following site characteristics:

- Regulatory requirements (e.g., DEP, FHWA, FEMA, USACOE, and USCG).
- Critical elevations on the highway itself, which should include:
  - Edge of pavement.
  - Elevation at which overtopping flow begins.
  - Low point of sub-base.
  - Top of headwall.
- Elevation above which adverse impact (i.e., flooding, standing water) from highwater might occur to adjacent developed property, such as buildings, agricultural crops, or other facilities.

The maximum allowable headwater (HW) is the depth of water, measured from the entrance invert that can be ponded during the design flood. The surrounding features, flow limitations, and roadway classification must be considered for each situation.

The surrounding features that may limit the allowable headwater include the following:

- Lowest elevation of the roadway adjacent to the ponding area.

- Flowline of the roadway ditch which passes water along the roadway to another drainage basin.
- Upstream property, such as buildings or farm crops, which will be damaged if inundated.
- Elevation established to delineate floodplain zoning.

Flow limitation factors that can affect the allowable headwater include the following:

- The debris which could plug the structure.
- Excessive ponding which would allow too much silting.
- High hydrostatic pressure which would cause seepage along the culvert backfill.

The HW/D ratio to be considered for design is the ratio of headwater depth to the diameter, height, or rise of a culvert entrance. The following are the maximum allowable HW/D ratios for the design of new culverts:

- HW/D = 2.0: Circular and elliptical (squash) pipe culverts with diameters (or equivalent diameters) of 750 mm (30 in) or less.
- HW/D = 1.5: Circular and elliptical pipe culverts with diameters greater than 750 mm (30 in) and less than or equal to 1800 mm (72 in), and other culverts with cross-sectional areas equal to or less than 2.8 m^2 (30 ft^2).
- HW/D = 1.2: Circular and elliptical pipe culverts with diameters greater than 1800 mm (72 in), and other culverts with cross-sectional areas greater than 2.8 m^2 (30 ft^2).

The headwater should be checked for the design flood, based on roadway classification, and for the 100-year flood to ensure compliance with floodplain management criteria. The maximum acceptable outlet velocity should be identified. The headwater should be set to produce acceptable velocities; otherwise, stabilization or energy dissipation should be provided where acceptable velocities are exceeded. For streams with debris issues, trash racks should be considered.

Occasional flowage easement shall normally be obtained for new flooding areas beyond the right-of-way line for the 100-year storm event for flow (Q_{100}). Q_{100} shall be used to be consistent with FEMA requirements and the Pennsylvania Floodplain Management Act. Except for the Interstate Highway, which cannot be inundated at the 50-year storm event for flow (Q_{50}), all classes of highways may be inundated at the design Q if a practicable alternative is not available.

In any event, the design discharge should not inundate the travel way for the highway design event. Additionally, where practicable for the 100-year event, the net increase in water surface at the upstream face of the culvert should not exceed 0.3 m (1 ft). If the project is located in an NFIP study area, NFIP procedures must be followed and NFIP criteria must be met. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10 should be consulted regarding increases to the 100-year flood profile in FEMA study areas.

D. Culvert Locations. Culvert location deals with the horizontal and vertical alignment of the culvert with respect to both the stream and the highway. The culvert location is important to the hydraulic performance of the culvert, stream and embankment stability, construction and maintenance costs, and the safety and integrity of the highway.

Horizontal and vertical alignment are important in maintaining a sediment free culvert. At normal depth, deposition may occur in culverts because the sediment transport capacity of flow within the culvert is less than in the stream. The following factors contribute to deposition in culverts:

- Point bars form on the inside of stream bends, and culvert inlets placed at bends in the stream will be subjected to deposition in the same manner. This effect is most pronounced in multiple barrel culverts with the barrel on the inside of the curve often becoming almost totally plugged with sediment deposits;
- Abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce deposition. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

Deposition usually occurs at flow rates smaller than the design flow rate. The deposits may be removed during larger floods, dependent upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert and other factors.

Most culvert locations approximate the natural stream bed. Other locations sometimes are chosen for economic reasons. Modified culvert slopes, or slopes other than that of the natural stream, can be used to arrest stream degradation, induce sedimentation, improve the hydraulic performance of the culvert, shorten the culvert, or reduce structural requirements. Modified slopes also can cause stream erosion and deposition; therefore, slope alterations should be given special attention to ensure that significant detrimental effects do not result from the change.

E. Stream Modification. Plan location deals basically with the route the flow will take in crossing the right of way. Regardless of the degree of sinuosity of the natural channel within the right of way, a crossing is generally accomplished by using a straight culvert either normal to or skewed with the roadway centerline.

Ideally, a culvert should be placed in the natural channel (Figure 9.2). This location usually provides good alignment of the natural flow with the culvert entrance and outlet, and little structural excavation and channel work are required. Where location in the natural channel would require an inordinately long culvert, some stream modification may be in order (Figure 9.3). Such modifications to reduce skew and shorten culverts should be designed carefully to avoid erosion and siltation problems.

Culvert locations normal to the roadway centerline are not suggested where severe or abrupt changes in channel alignment are required upstream or downstream of the culvert. Short radius bends are subject to erosion on the concave bank and deposition on the inside of the bend. Such changes upstream of the culvert result in poor alignment of the approach flow to the culvert, subject the highway fill to erosion, and increase the probability of deposition in the culvert barrel. Abrupt changes in channel alignment downstream of culverts may cause erosion on adjacent properties.

Figure 9.2 Culvert Located in a Natural Streambed

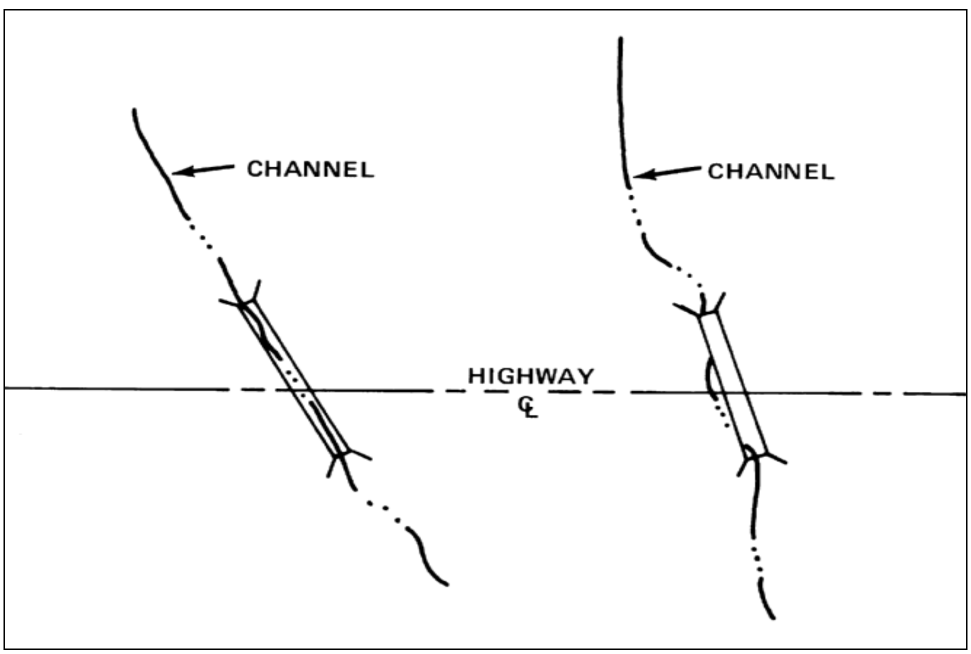
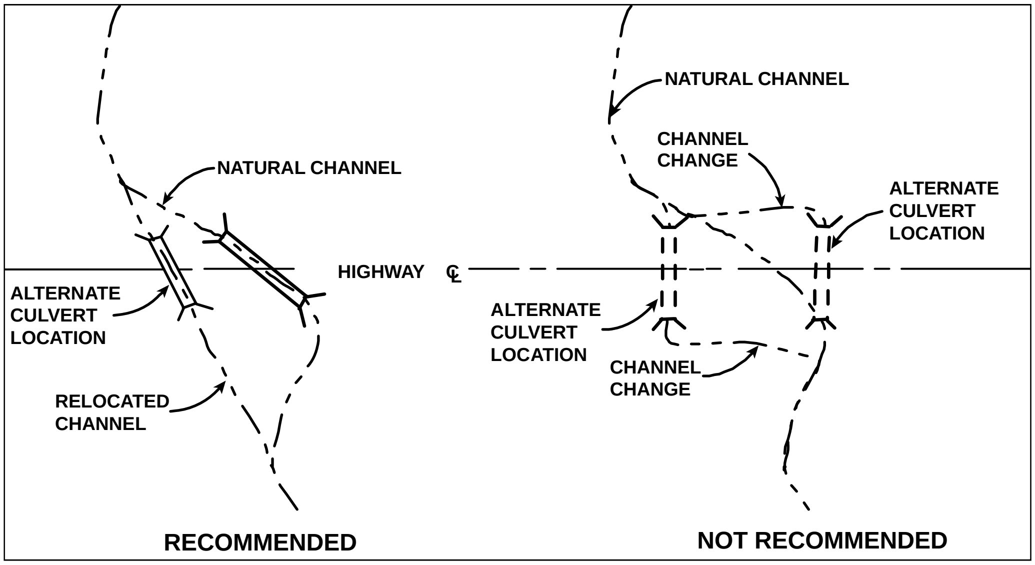


Figure 9.3 Stream Relocation Options



Culverts in live stream environments may necessitate temporary diversion channels to carry flow around the work site, as discussed in Section 9.1.G.

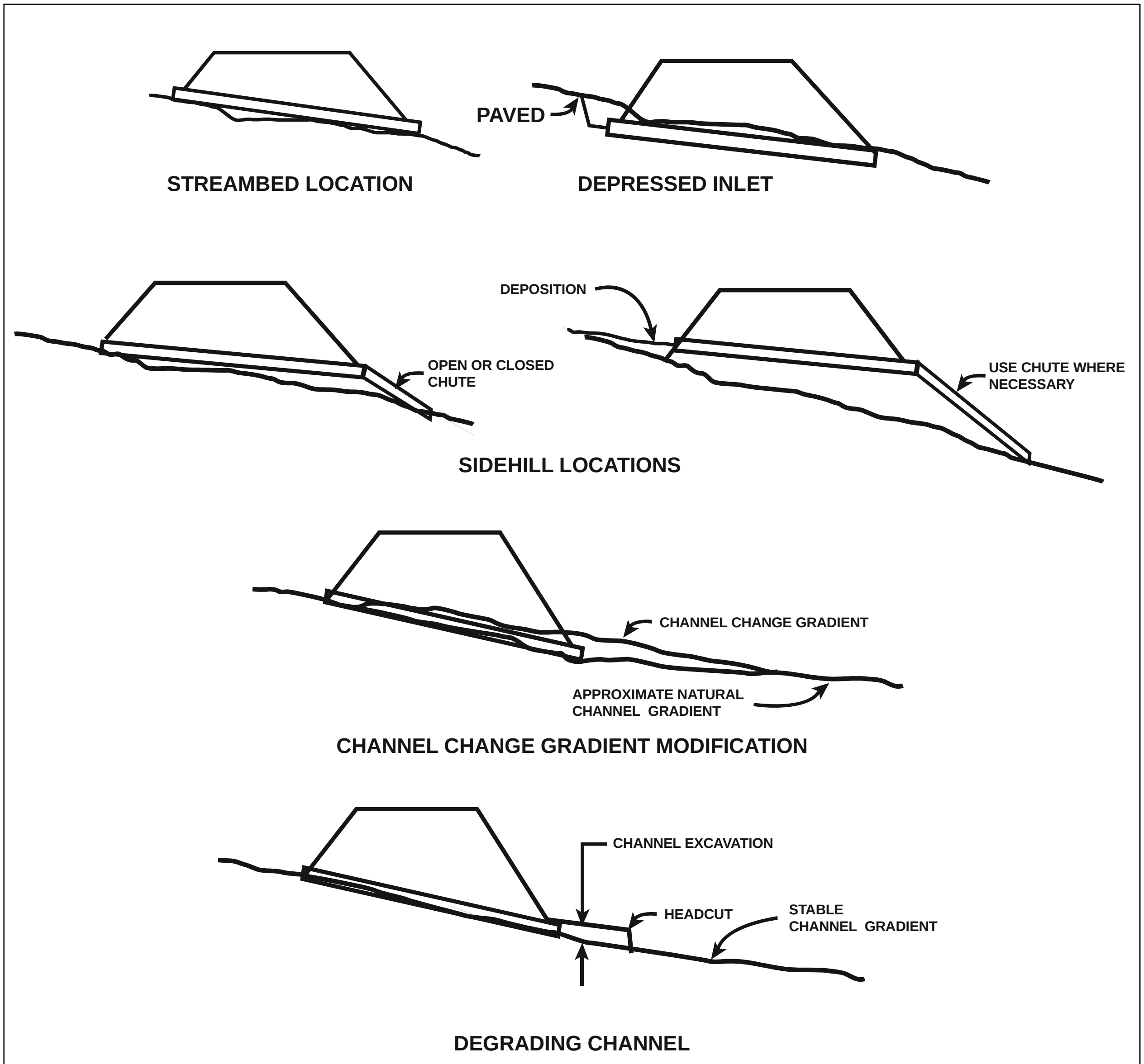
F. Culvert Profile. The establishment of the vertical profile of the culvert usually is a matter of placing the upstream and downstream flow line elevations of the culvert at the same elevations as the existing streambed or counter-sinking the entire culvert so that the upstream and downstream ends of the culvert are an equal distance below the natural streambed. In some instances, the upstream flowline may need to be lowered. Lowering the upstream flowline can provide improved hydraulic operation, but may create maintenance problems due to a higher potential for both sedimentation and scour.

The designer should avoid placing the downstream flowline of the culvert at a level higher than the roadway embankment toe of slope. Such a configuration results in an increase in the potential for erosion.

Channel changes often are shorter and steeper than the natural channel. A modified culvert slope can be used to achieve a flatter gradient in the channel so that degradation will not occur. Figure 9.4 illustrates some possible culvert profiles.

Where channel excavation is planned, culvert invert elevations can be established to accommodate drainage requirements if concurrent channel and highway construction is possible. If concurrent construction is not feasible, a joint or cooperative project should be investigated so that highway culverts can be designed and constructed to serve current highway drainage requirements as well as future needs for land drainage.

Figure 9.4 Culvert Placement Locations



G. Temporary Diversion Channels. Culvert construction in live stream environments frequently necessitates the installation of temporary diversion channels to carry the stream around the work site. The temporary diversion channels need protective linings to prevent erosion. At times it also may be necessary to develop a staged construction sequence which will permit a portion of the work to be done; stream flow is then diverted through the completed portion of the culvert while the remainder of the culvert installation is constructed. Additional information on temporary erosion and sediment control measures that can be used at a construction site may be found in Chapter 12, *Erosion and Sediment Pollution Control* and Volume III, Erosion and Sediment Control in Highway Construction of the *Highway Drainage Guidelines* (AASHTO, 2003).

H. Design and Allowable Outlet Velocity. At flood conditions, a culvert may restrict the available channel area, and flow velocities in the culvert may be higher than in the channel. These increased velocities can cause streambed scour, bank erosion in the vicinity of the culvert outlet, and can hamper fish passage. Minor problems occasionally can be avoided by increasing the barrel roughness with baffles. Energy dissipaters and outlet protection devices sometimes are required to avoid excessive scour at the culvert outlet. When a culvert is operating under inlet control and the culvert barrel is not operating at capacity, it often is beneficial to reduce the barrel slope or add a roughened section to reduce outlet velocities.

Similar to the allowable headwater, the allowable outlet velocity is a design criterion which is unique to each culvert site. The types and characteristics of soil can vary considerably from site to site. Energy dissipaters or rock stilling basins may be required in erodible soils to decrease the velocity of flow to prevent damage to the channel.

Velocities at which soils erode may vary widely. The designer should attempt to estimate the threshold of erosive velocity for each culvert location. This may be done by observing storm flows on various soil types and estimating those velocities at which erosion is occurring (see HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 1988), HEC-11, *Design of Riprap Revetment* (FHWA, 1989a), HDS-6, *River Engineering for Highway Encroachments* (FHWA, 2001b), HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001c), and HEC-20, *Stream Stability at Highway Structures, Third Edition* (FHWA, 2001d)).

The designer should exercise extreme caution when considering culvert designs with outlet velocities of greater than 4.5 m/s (14.8 ft/s). The designer should provide riprap or control devices in situations where outlet velocity poses potential erosion problems. Section 9.5 describes different velocity protection and control devices.

If the culvert has been sized properly according to allowable headwater criteria, it almost always is more economical to protect against excessive outlet velocity with riprap and/or velocity protection or control devices than to try to adjust the culvert size to reduce the excessive outlet velocity.

Velocities of less than about 0.6 m/s (2 ft/s) usually foster deposition of sediments at flow depth of 20% of culvert diameter (HEC-23 (FHWA, 2001c)). Therefore, 0.6 m/s (2 ft/s) is suggested as a minimum for culvert design and operation where sediment deposit would present a problem.

I. End Treatments. End treatments serve several different purposes, but typically they act as a retaining wall to keep the roadway embankment material out of the culvert opening. Some secondary characteristics of end treatments include improvements to:

- Performance.
- Public safety.
- Debris control.
- Flood protection.
- Piping prevention (flow through the embankment outside of the culvert).

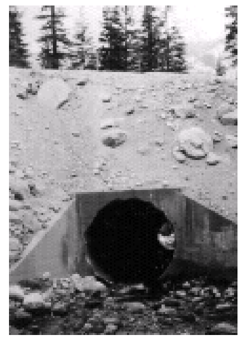
Figure 9.5 shows sketches of various end treatment types.

Figure 9.5 Typical Culvert End Treatments

Projected end

Projected Bell Inlet

Head wall with beveled edge inlet



Head wall with wing walls

J. Safety Considerations. Cross-drainage and longitudinal drainage facilities usually are necessary in any highway project. These facilities control highway drainage and natural runoff from areas adjacent to the highway. However, due to their inherent mass and fixed nature, they can affect the safety of the motoring public.

Three practical methods are available for addressing safety issues associated with highway culverts:

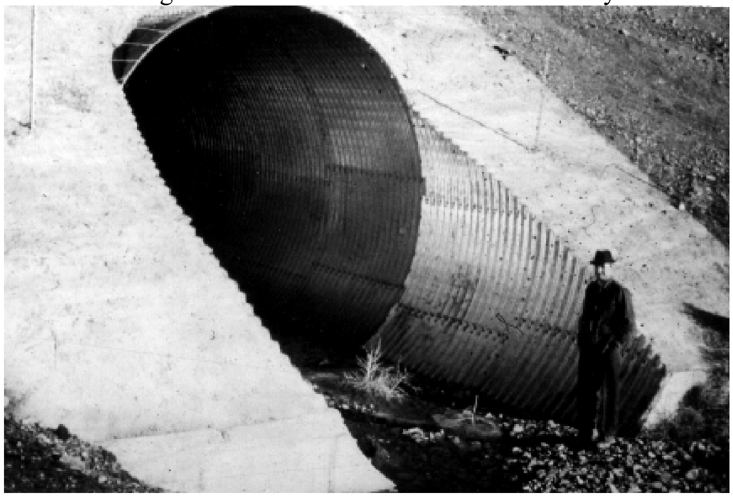
- Safety treatment at culvert ends.
- Satisfaction of clear-zone requirements.
- Shielding by guide fences and/or other types of barriers.

Safety treatment of culvert ends is effective in avoiding the hazards created by protruding culvert ends; however, it also represents a significant interference with the original purpose of the drainage structure. A safety end treatment has a tendency to accumulate trash and flood debris, thus blocking flow into and out of the culvert.

Mitered end sections should be used carefully for several reasons. First, the use of mitered end sections may increase hydraulic head losses; the hydraulic performance of this type of inlet is approximately the same as a thin-edged projecting inlet. Additionally, a non-reinforced mitered end may affect the structural integrity of the culvert. With the use of mitered end sections, where practicable, the designer should incorporate safety end treatment standards, such as metal guides.

The simple step of removing the headwall and applying a mitered end section alone (see Figure 9.6) offers relatively little obstacle for passage of drift or debris.

Figure 9.6 Mitered End Treatment for Safety



K. Culvert Type Selection. Culvert type selection includes the choice of material to meet design life, shape and cross section, and number of culvert barrels. Total culvert cost can vary considerably depending upon the culvert type selection. The following are some of the factors which influence culvert type selection:

- Fill height.
- Terrain.
- Foundation condition.
- Shape of the existing channel.
- Roadway profile.
- Allowable headwater.
- Stream stage discharge.
- Frequency-discharge relationships.
- Cost.
- Service life.
- Fish passage.

Generally, the primary factors affecting culvert type are economics, hydraulic properties, durability, and strength. A summary of acceptable criteria for specifying alternate types of culvert pipes based on the type of use is presented in Table 9.1.

Table 9.1. Alternate Pipes Selection Criteria Based on Type of Installation

Types of Drainage Installations	**Types of Pipe**	**Minimum Number of Alternates Required**
Cross Drains Under All Highways Including Pipes Under Pavements	CP or RCP or CCGS or FBCGS or ECP or SLCPP	2
Parallel Storm Sewer Outside of Pavement	All pipes listed in Table Legend	3
Side Drains (Driveways, etc.)	All pipes listed in Table Legend	3
Slope Pipes	AA or CSPMC	2
Combination Storm Sewer and Underdrain and Other Special Drainage Systems	CP open joint or RCP open joint or perforated CSPMC or perforated AA or perforated CPP	3

Table Abbreviations:
PSLCGS -- Plastic (PVC) Smooth-Lined Corrugated Galvanized Steel
SSP -- Stainless Steel Pipe
CP -- Non-reinforced Concrete Pipe (36" max.)
RCP -- Reinforced Concrete Pipe
SLCGS -- Smooth-lined Corrugated Galvanized Steel
AA -- Aluminum Alloy
Ext. Str. VC -- Extra Strength Vitrified Clay (36" max.)
FBCGS -- Fiber-Bonded Corrugated Galvanized Steel Pipe
DIP -- Ductile Iron Pipe
CSPMC -- Corrugated Steel Pipe Metallic Coated
CCGS -- Coated (Polymer) Corrugated Galvanized Steel
ECP -- Epoxy-Lined Concrete Pipe
SLCPP -- Smooth-Lined Corrugated Polyethylene Pipe
CPP -- Corrugated Polyethylene Pipe (36" max.)

Table Notes:
1. Pipe alternates may be eliminated for the following engineering reasons: (1) unstable support, (2) high impact and concentrated loading, (3) high embankments, (4) limited clearance, (5) steep gradients, (6) high acidity or alkalinity of soils and water or other corrosive elements, (7) high erosive forces, or (8) for other pertinent reasons.
2. When conditions are such that the pipe requires coating, galvanized pipes shall be polymeric coated in accordance with Publication 408, *Highway Specifications*. Polymeric coated galvanized steel pipe is available only in 10 gage (3.5 mm) or lighter; if a coating is required for 8 gage (4.3 mm) or heavier, this alternate should be eliminated.

The selection of pipe alternates is dependent upon environmental factors as presented in Table 9.2. Consideration to the future land use should be given. For example, pipe placed in an area not being mined presently, but which ultimately may be mined, should be designed to handle the mine acid drainage.

Table 9.2. Pipe Selection Criteria for Corrosion Protection Based on pH and Resistivity Values

TYPE OF PIPE	COATING	WATER AND/OR SOIL pH	SOIL RESISTIVITY (OHM-M)	ABRASION COATING REQUIRED
Aluminum Alloy	Uncoated	4.0 to 8.5	> 15	Paved Invert
Concrete	Uncoated	4.0 or Greater	All	Epoxy Lined
Concrete	Vitrified Clay	< 4.0	All	None Required
Thermo-Plastic		All	All	None Required
Steel	Metallic Coated	5.5 to 8.5	> 60	Paved Invert
Steel	10 mil Polymer-Type C	5.5 to 8.5	>60	None Required

Table Notes:
1. Selection of pipe alternates and requisitions for pipe shall be supported with the pH of the effluent and pH and resistivity of the soil. For design purposes, the pH of the water at the construction site shall be determined in the field by ASTM D-1293. If the pH is below 5.5, a one (1) quart sample shall be furnished to the Materials and Testing Division (MTD) for exact identification. Testing should be done seasonally, if possible, and the worst set of conditions used in making determination of the proper type of pipe. Additionally, a six (6) to eight (8) pound (2.7 to 3.6 kg) sample of the site soil shall be sent to the MTD for determination of the soil pH and resistivity by AASHTO T-288, for further consideration of the proper pipe type.
2. All pipe designs shall vary based on location as stated in Design Manual Part 2, Table 10.5.5 and will range from a 25- to 100-year design life. If an analysis is not feasible, the designer shall assume worst-case conditions. For steel pipe, this would require a minimum of 10 gage (3.5 mm) for corrosion protection in the pH range 5.5 to 8.5 or a polymeric coating at the gage required in the appropriate fill-height table, with the exception that side drains may be supplied in 14 gage (2.0 mm). If the fill-height criteria indicates the need for heavier than 10 gage (3.5 mm), the steel pipe alternate shall be eliminated unless appropriate site analysis is provided which indicates a minimum 50-year service life.

L. Economics. The designer should select a material that satisfies hydraulic, structural, and other design criteria with the lowest overall cost. One must keep in mind that both material availability and ease of construction influence the total cost of the structure, as well as the timing of project delivery. Choosing culvert components which are readily available to construction contractors or maintenance forces may result in lower bid prices and faster completion of the project. Some common combinations are:

- Pipe (concrete, steel, aluminum, plastic):
  - Circular.
  - Pipe-arch (CMP only).
  - Elliptical.
  - Precast concrete arches.
- Structural-plate (steel or aluminum):
  - Circular.
  - Pipe-arch.
  - Elliptical.
  - Arch.
- Box culverts (single or multiple barrel):
  - Concrete box culvert.
  - Steel or aluminum box culvert.
- Long span (structural-plate [steel or aluminum]):
  - Low profile arch.
  - High profile arch.
  - Elliptical.

In those cases where Departmental design criteria specifies that alternate pipes shall be included in the plans and proposal, design computations shall be submitted for each alternate. If the design computations determine that, for one or more of the alternates, different sizes are adequate, the construction drawings and quantities should be developed for the larger size.

Alternate sizes should be indicated on the tabulation and/or summary sheet when a choice in size exists.

M. Hydraulic Properties. Each shape has distinct hydraulic properties, and each material has an associated wall roughness. Both factors influence the hydraulic operation of a structure, in addition to inlet geometry, as discussed earlier.

N. Operation. The designer should consider the feasibility of operating the roadway and culvert over the period of anticipated service life. Each of the factors listed below depends on the type of material and the shape of the culvert and therefore requires consideration.

- Ease of access for maintenance purposes.
- Repair and rehabilitation costs.
- Durability.

O. Fill Heights. Fill heights conform to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.4 and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Appendix B.

9.2 HYDRAULIC ANALYSIS OF CULVERTS

A. General - Hydraulic Analysis of Culverts. Flow through a culvert is classified by one of two types of operation: outlet control or inlet control. The two basic culvert design criteria are allowable headwater and allowable outlet velocity. Allowable headwater is usually the most important in influencing the overall configuration of the culvert. By consideration of various parameters, the designer can arrive at the appropriate calculation procedure. In this chapter, an attempt has been made to illustrate the major conditions affecting culvert analysis. Some minor variations of the identified conditions presented here may be found, but these variations should not alter the result appreciably.

Actual culvert flow conforms to the laws of open-channel flow and closed-culvert flow. The procedures set forth in this section assume steady flow but still can involve extensive calculations which lend themselves to computer application. Computer models should be used for all final design applications; however, for a simple cross pipe to be replaced by maintenance forces or for initial planning, simplified hand methods and nomographs may be appropriate.

Culvert analysis involves computing headwater elevations for both inlet and outlet control for a given discharge. These elevations are compared, and the larger of the two is used as the controlling headwater elevation. Tailwater effects are taken into consideration when calculating these elevations. If the controlling headwater elevation overtops the roadway embankment, an overtopping analysis is done in which flow is balanced between the culvert discharge and the flow over the roadway.

For multiple conveyance systems, a headwater balancing technique is used to compute the discharge through each opening.

B. Computations for Culvert Hydraulics. The flow through a culvert is classified as either inlet or outlet control. Under inlet control, the headwater elevation is a function of only the inlet configuration, which includes the size, shape, and material as well as entrance conditions such as headwalls. Under outlet control the energy equation is solved in terms of an orifice with an empirical coefficient of discharge to account for the inlet losses. For the outlet control the energy equation is balanced between the inlet and outlet of the pipe accounting for the minor losses at the entrance and exit and the friction losses within the culvert barrel.

In order to understand the equations the following sections dealing with subcritical and supercritical flow and uniform flow are presented. For a more detailed discussion see Chapter 8, *Open Channels*.

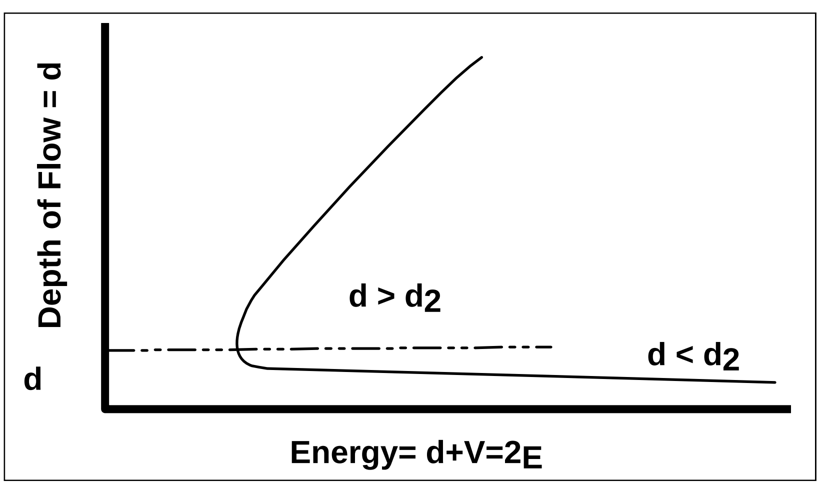
C. Supercritical versus Subcritical Flow. The specific energy, E, of a fluid is defined as the energy per unit weight of the fluid measured relative to the bottom of the channel and comprises the sum of depth of flow and velocity head.

(Equation 9.1)

$$E = d + \frac{v^2}{2g}$$

where: E = specific energy, m (ft)
d = depth of flow, m (ft)
v = average velocity of flow, m/s (ft/s)
g = gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

Figure 9.7 Plot of Depth of Flow versus Specific Energy for a Constant Flow Rate per Unit Width (q)



When specific energy is plotted against the depth of flow, a parabolic curve, as seen in Figure 9.7, is formed. Specific energy is at a minimum where the depth of flow is at critical depth (d_c). This critical state represents the threshold between "subcritical" flow and "supercritical" flow.

With respect to Figure 9.7, supercritical flow occurs when depth of flow is lower than critical depth and subcritical flow occurs when the depth of flow is greater than critical depth of flow. Refer to Chapter 8, *Open Channels*, for additional information regarding types of flow in channels.

It is apparent from the specific energy diagram that when the flow conditions are close to critical, a relatively large change of depth occurs with small variations in specific energy. Flow under these conditions is unstable, and excessive wave action or undulations of the water surface may occur.

When the flow is subcritical, computations of water surface elevations proceed from downstream to upstream; for supercritical flow, the computations are reversed beginning with upstream and proceeding downstream. For culvert hydraulics, inlet control often indicates that supercritical flow occurs in the culvert while outlet control indicates that flow computations are controlled by friction in the system and the flow regime is subcritical.

A determination of the appropriate flow regime is accomplished by evaluating a dimensionless value called the Froude number, F_r.

(Equation 9.2)

$$F_r = \frac{v}{\sqrt{g d_m}}$$

where: d_m = hydraulic depth, m (ft)

The hydraulic depth is calculated by dividing the cross sectional flow area (A) by the width of the free water surface (T). When $F_r > 1.0$, the flow is supercritical and is characterized as rapid. When $F_r < 1.0$, the flow is subcritical and is characterized as smooth and tranquil. If $F_r = 1.0$, the flow is said to be critical.

D. Critical Depth in Culverts. Critical depth can be illustrated best as the depth at which water flows over a weir; this depth is attained automatically where no other backwater forces are involved. This is because it is the depth at which the energy content of flow is at a minimum. Critical depth is a direct function of discharge and geometry of the culvert. The general equation for critical flow in any shape culvert or channel is:

(Equation 9.3)

$$\frac{Q_c^2}{g} = \frac{A_c^3}{T}$$

where: Q_c = critical discharge, m^3/s (cfs)
A_c = cross sectional area of flow at critical discharge, m^2 (ft^2)
T = water surface width of critical discharge, m (ft)

The formula applicable for calculating critical depth in rectangular channels is:

(Equation 9.4)

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

where: q = discharge per unit width of rectangular culvert, $m^3/s/m$ (cfs/ft)

Critical depth of circular and pipe-arch or irregular shapes can be calculated by iterative use of Equation 9.5. Equations 9.6 and 9.7 allow determination of the area, A, and top width, T, of flow in a circular pipe, respectively. For other shapes, the designer should acquire or derive relationships between depth of flow, area and top width.

(Equation 9.5)

$$\theta = \cos^{-1}\left(1 - \frac{2d}{D}\right)$$

Note: Equation 9.5 is for use in Degrees only.

(Equation 9.6)

$$A = \frac{D^2}{8}\left[\frac{\pi}{90}\theta - \sin(2\theta)\right]$$

(Equation 9.7)

$$T = D\sin(\theta)$$

where:
A = section area of flow, m^2 (ft^2)
T = width of water surface, m (ft)
d = depth of flow, m (ft)
D = pipe diameter, m (ft)

E. Uniform Depth in Culverts. Uniform depth is the depth of water when the geometry, roughness characteristics, slope, and discharge are all constant for a sufficiently long reach of open channel conveyance. Uniform depth sometimes is referred to as normal depth. The characteristics of culvert flow closely approximate the culvert characteristics involved in uniform flow. Since culverts are usually hydraulically short there may not be a sufficient length of reach for attainment of normal depth.

Manning's Equation (represented by Equation 9.8) describes uniform flow and is used for the determination of uniform depth.

(Equation 9.8)

$$Q = \frac{k}{n}AR^{2/3}S^{1/2}$$

where:
Q = culvert barrel discharge, m^3/s (cfs)
k = 1.0 (metric), 1.486 (U.S. Customary)
n = Manning's roughness coefficient (see Table 7.4 for suggested values)
A = cross section flow area, m^2 (ft^2)
R = hydraulic radius of the culvert, R= A/WP, m (ft)
WP = wetted perimeter, m (ft)
S = slope of water surface - taken to be the culvert slope, m/m (ft/ft)

For most shapes, a direct solution of Equation 9.10 for normal depth is not possible and an iterative solution is required. For rectangular shapes area and wetted perimeter are simple functions of flow depth. For circular pipe, area is computed using Equation 9.5 and wetted perimeter is computed using Equation 9.9. For other shapes, the designer should acquire or derive the relationship between depth of flow, area and wetted perimeter.

(Equation 9.9)

$$WP = \frac{\pi}{180}D\theta$$

Notes:
1. θ is from Equation 9.5
2. Equation 9.9 is for use in Degrees only

F. Friction Slope. The friction slope is a theoretical value which describes the slope of the energy grade line and is based upon Manning's Equation, rearranged as follows:

(Equation 9.10)

$$S_f = \left(\frac{Q_n}{xR^{2/3}A}\right)^2$$

where: S_f = friction slope, m/m (ft/ft)

G. Steep Slope versus Mild Slope. When critical depth (d_c) is higher than normal depth (d_u or d_n), the slope is considered steep. The culvert may flow completely full (pressure flow) or partly full (free surface flow). Whether the free surface flow is supercritical or subcritical depends on tailwater conditions.

In considering each factor more critical, judgment is necessary if it is kept in mind that any condition that causes turbulence and retards flow results in a greater value of "n."

Outlet velocity for bituminous paved inverts shall be determined based on a 25% reduction in Manning's roughness coefficient, "n."

When critical depth is lower than uniform depth, the slope is termed mild. Pressure flow or free surface flow may occur. Free surface flow is most likely to be subcritical within the culvert.

The shape of the free water surface is dependent on whether the culvert slope is steep or mild. The methods described in this chapter accommodate the differences in water surface shape.

H. Headwater Under Inlet Control. The design equations used to develop the inlet control nomographs presented in the HDS-5, *Hydraulic Design of Highway Culverts*, (FHWA, 2005b) are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration).

The two basic conditions of inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between the tangent of the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Equations 9.11 through 9.13 present the unsubmerged and submerged inlet control design equations developed by FHWA. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply. Either form of unsubmerged inlet control equation will produce adequate results.

The constants for the equations are given in Table 9.3.

Inlet Control Design Equations by FHWA presented in HDS-5 (FHWA, 2005b) are as follows:

<u>Unsubmerged</u>

(Equation 9.11)

Form (1)

$$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M - 0.5S^2$$

(Equation 9.12)

Form (2)

$$\frac{HW_i}{D} = K\left[\frac{K_u Q}{AD^{0.5}}\right]^M$$

Table 9.3. Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomo-graph Scale	Inlet Edge Description	Eqn. Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y
1	Cir. Concrete	1	Square edge w/hw	1	.0098	2.0	.0398	.67
		2	Groove end w/hw		.0018	2.0	.0292	.74
		3	Groove end projecting		.0045	2.0	.0317	.69
2	Cir. CMP	1	Headwall	1	.0078	2.0	.0379	.69
		2	Mitered to slope		.0210	1.33	.0463	.75
		3	Projecting		.0340	1.50	.0553	.54
3	Cir.	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74
		B	Beveled ring, 33.7° bevels*		.0018	2.50	.0243	.83
8	Rect. Box	1	30° to 75° ww	1	.026	1.0	.0347	.81
		2	90° and 15° ww flares		.061	.75	.0400	.80
		3	0° ww flares		.061	.75	.0423	.82
9	Rect. Box	1	45° ww flare d=.043D	2	.510	.667	.0309	.80
		2	18° to 33.70 ww flare d=.083D		.486	.667	.0249	.83
10	Rect. Box	1	90° hw w/3/4" chamfers	2	.515	.667	.0375	.79
		2	90° hw w/45° bevels		.495	.667	.0314	.82
		3	90° hw w/33.7° bevels		.486	.667	.0252	.865
11	Rect. Box	1	¾" chamfers; 45° skewed hw	2	.545	.667	.04505	.73
		2	¾" chamfers; 30° skewed hw		.533	.667	.0425	.705
		3	¾" chamfers; 15° skewed hw		.522	.667	.0402	.68
		4	45° bevels; 10°-45° skewed hw		.498	.667	.0327	.75
12	Rect. Box ¾" chamfers	1	45° non-offset ww flares	2	.497	.667	.0339	.803
		2	18.4° non-offset ww flares		.493	.667	.0361	.806
		3	18.4° non-offset ww flares - 30° skewed barrel		.495	.667	.0386	.71
13	Rect. Box Top Bevels	1	45° ww flares – offset	2	.497	.667	.0302	.835
		2	33.7° ww flares – offset		.495	.667	.0252	.881
		3	18.4° ww flares – offset		.493	.667	.0227	.887
16-19	C M Boxes	2	90° hw	1	.0083	2.0	.0379	.69
		3	Thick wall projecting		.0145	1.75	.0419	.64
		5	Thin wall projecting		.0340	1.5	.0496	.57

Table Abbreviations:

- CMP – Corrugated Metal Pipe
- Cir. – Circular
- hw – headwall
- Eqn. – Equation
- CM – Corrugated Metal
- Rect. – Rectangular
- ww – wingwall
- Ref. – References

Submerged

(Equation 9.13)

$$\frac{HW_i}{D} = H_c\left[\frac{K_u Q}{AD^{0.5}}\right]^2 + Y - 0.5S^2$$

where

HW_i	=	Headwater depth above the inlet control section invert, m (ft)
D	=	Interior height of culvert barrel, m (ft)
H_c	=	Specific head at critical depth, m (ft)
Q	=	Discharge, m^3/s (cfs)
A	=	Full cross sectional area of culvert barrel, m^2 (ft^2)
S	=	Culvert barrel slope, m/m (ft/ft)
K, M, c, Y	=	Constants (see Table 9.3)
K_u	=	1.811 (metric), 1.0 (U.S. Customary)

A culvert operates with inlet control when the flow capacity is controlled at the entrance by the depth of headwater and the entrance geometry, including the barrel shape, cross sectional area and the inlet edge. Sketches to illustrate inlet control flow for unsubmerged and submerged projecting entrances are shown in Figure 9.8.

For a culvert operating with inlet control, the roughness and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic capacity. The entrance edge and the overall entrance geometry have much to do with culvert capacity in this type of flow; therefore, special entrance designs can improve hydraulic capacity and result in a more efficient and economical culvert.

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. Inlet control is most likely to occur when the culvert configuration is hydraulically steep ($d_c > d_u$). The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Depending on conditions downstream of the culvert inlet, a hydraulic jump may occur in the culvert.

Under inlet control, hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. The upstream water surface elevation and the inlet geometry represent the major flow controls. The inlet geometry includes the barrel shape, the cross sectional area, and the inlet edge.

It is important to note that, although inlet control conditions are more prevalent when the culvert slope is greater than the critical slope, inlet control can occur under mild slope conditions. Such a condition is likely when the tailwater is lower than critical depth in the culvert exit of a short culvert.

A fifth-degree polynomial equation was developed by FHWA, based on regression analysis, to model the inlet control headwater for a given flow. The regression equation was developed for the range of inlet heads from one-half to three times the culvert rise. Analytical equations, based on minimum energy principles, are matched to the regression equations to model flows that create inlet control heads outside of the regression data range.

For $0.5 \le \frac{HW_{ic}}{D} \le 3.0$, Equation 9.14 applies:

(Equation 9.14)

$$HW_{ic} = [a + bF + cF^2 + dF^3 + eF^4 + fF^5]D - 0.5DS_0$$

where:

HW_{ic}	=	inlet control headwater, m (ft)
D	=	rise of culvert barrel, m (ft)
a to f	=	regression coefficients for each type of culvert (Table 9.4)
S_0	=	culvert slope, m/m (ft/ft)
F	=	function of the average outflow discharge being routed through a culvert; culvert barrel rise; and, for box and pipe arch culverts, the width of the barrel, W, as shown in Equation 9.15

(Equation 9.15)

$$F = 1.8113\frac{Q}{WD^{3/2}}$$

where: W = width or span of culvert, m (ft)

Figure 9.8 Inlet Control Conditions
(Condition A – Inlet unsubmerged; Condition B – Inlet submerged; Condition C – Outlet submerged)

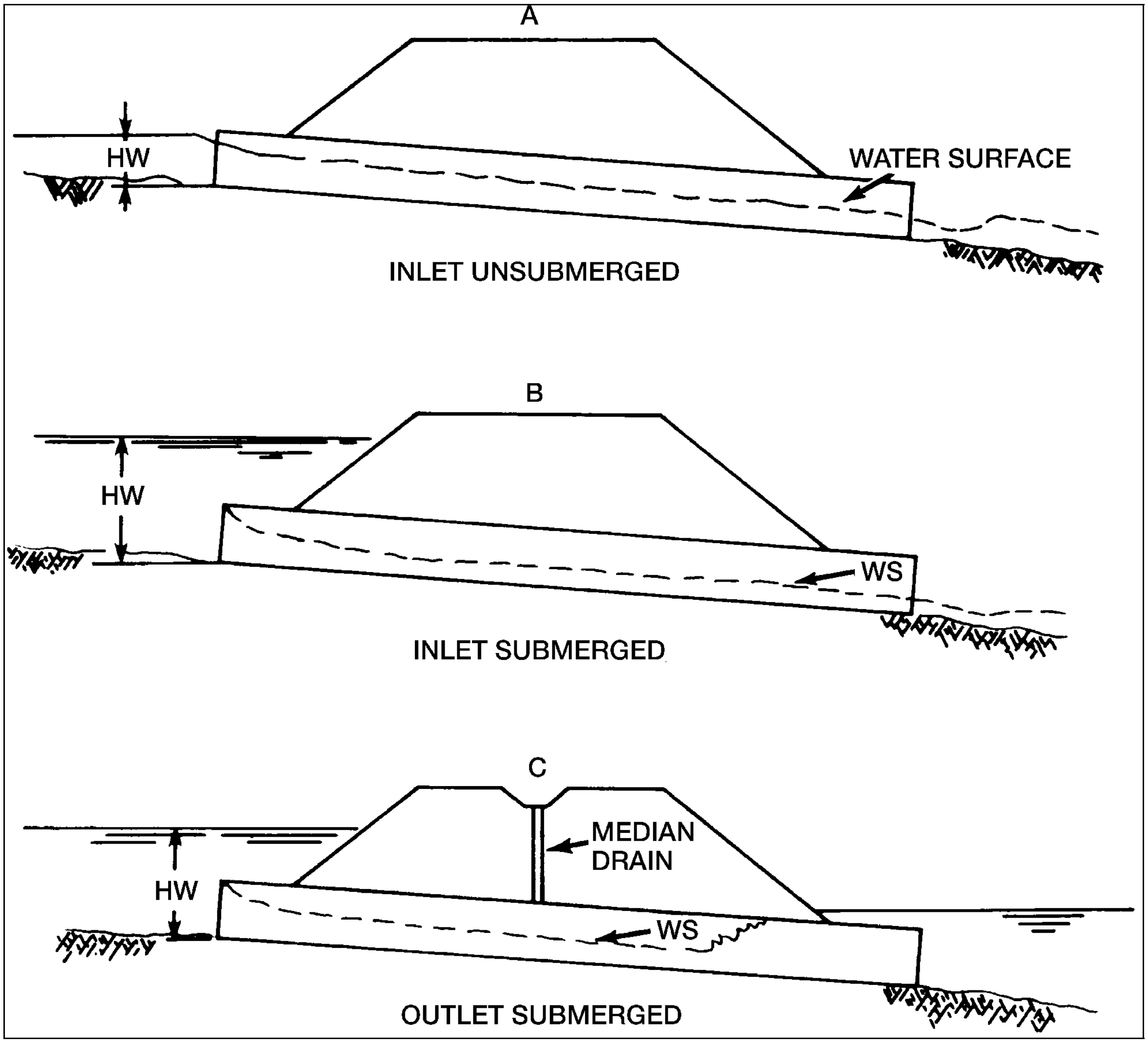


Table 9.4. Regression Coefficients for Inlet Control Equations

Shape & Material	Entrance type	a	b	c	d	e	F
RCP	Square edge w/hw	0.087483	0.706578	-0.2533	0.0667	-0.00662	0.000251
	Groove end w/hw	0.114099	0.4653562	-0.2336	0.059772	-0.00616	0.000243
	Groove end proj.	0.108786	0.662381	-0.2338	0.057959	-0.00558	0.000205
	Beveled Ring	0.63343	0.766512	-0.316097	0.08767	-0.00984	0.000417
	Improved (flared) inlet	0.2115	0.3927	-0.0414	0.0042	-0.0003	-0.00003
CMP	Headwall	0.167433	0.53859	-0.14937	0.039154	-0.00344	0.000116
	Mitred	0.107137	0.757789	-0.3615	0.123393	-0.01606	0.000767
	Proj.	0.187321	0.567719	-0.15654	0.044505	-0.00344	0.00009
	Improved (flared) inlet	0.2252	0.3471	-0.0252	0.0011	-0.0005	-0.00003
Box	30-70° Flared ww	0.072493	0.507087	-0.11747	0.02217	-0.00149	0.000038
	Parallel to 15° ww	0.122117	0.505435	-0.10856	0.020781	-0.00137	0.0000346
	Straight ww	0.144138	0.461363	-0.09215	0.020003	-0.00136	0.000036
	45° ww w/top bevel	0.156609	0.398935	-0.06404	0.011201	-0.00064	0.000015
	Parallel hw w/bevel	0.156609	0.398935	-0.06404	0.011201	-0.00064	0.000015
	30° skew w/chamfer edges	0.122117	0.505435	-0.10856	0.020781	-0.00137	0.000034
	10-45° skew w/bevel edges	0.089963	0.441247	-0.07435	0.012732	-0.00076	0.000018
Oval B > D	Square edge w/hw	0.13432	0.55951	-0.1578	0.03967	-0.0034	0.00011
	Groove end w/hw	0.15067	0.50311	-0.12068	0.02566	-0.00189	0.00005
	Groove end proj.	-0.03817	0.84684	-0.32139	0.0755	-0.00729	0.00027
Oval D > B	Square edge w/hw	0.13432	0.55951	-0.1578	0.03967	-0.0034	0.00011
	Groove end w/hw	0.15067	0.50311	-0.12068	0.02566	-0.00189	0.00005
	Groove end proj.	-0.03817	0.84684	-0.32139	0.0755	-0.00729	0.00027
CM pipe-arch	Headwall	0.111281	0.610579	-0.194937	0.051289	-0.00481	0.000169
	Mitred	0.083301	0.795145	-0.43408	0.163774	-0.02491	0.001411
	Proj.	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293
Struct pl pipe arch	Proj. - 450mm corner pl	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293
	Proj. - 780mm corner pl	0.12263	0.4825	-0.00002	-0.04287	0.01454	-0.00117
CM Arch (flat bottom)	Parallel hw	0.111281	0.610579	-0.1949	0.051289	-0.00481	0.000169
	Mitred	0.083301	0.795145	-0.43408	0.163774	-0.02491	0.001411
	Thin wall projecting	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293

Table Abbreviations:

- CMP – Corrugated Metal Pipe
- RCP – Reinforced Concrete Pipe
- hw – headwall
- pl – plate
- CM – Corrugated Metal
- Proj. – Projecting
- ww – wingwall

For $HW_i/D > 3.0$, an orifice equation, Equation 9.16, is used to estimate headwater. The potential head is determined from the centroid of the culvert opening, which is approximated as the sum of the invert elevation and one-half the rise of the culvert.

(Equation 9.16)

$$HW_i = \left[\frac{Q}{k}\right]^2 + \frac{D}{2}$$

where: HW_i = inlet control headwater depth, m (ft)
Q = design discharge, m^3/s (cfs)
k = orifice equation constant ($\sqrt{2g}AC$)
D = rise of culvert, m (ft)

The effective area, A, and orifice coefficient, C, are implicit.

The coefficient, k, is determined by rearranging Equation 9.16 using the discharge that creates a HW/D ratio of 3 in the regression equation (i.e., the upper limit of the Equation 9.11):

(Equation 9.17)

$$k = 0.6325\frac{Q_{3.0}}{D^{1/2}}$$

where: $Q_{3.0}$ = discharge in m^3/s (cfs) at which $HW_i/D = 3$

Generally, for Department designs, it is not considered efficient to design culverts for $HW_i/D < 0.5$. However, if such a condition is likely ($HW_i/D < 0.5$), an open channel flow minimum energy equation (weir equation) is used with the addition of a velocity head loss coefficient. The minimum energy equation, with the velocity head loss adjusted by an entrance loss coefficient, generally describes the low flow portion of the inlet control headwater curve. However, numerical errors in the calculation of flow for very small depths tend to increase the velocity head as the flow approaches zero. This presents little or no problem in most single system cases since the flows that cause this are relatively small.

In many of the required calculations for the solution of multiple culverts, the inlet control curve must decrease continuously to zero for the iterative calculations to converge. Therefore, in computer models, modifications to this equation have been made to force the velocity head to continually decrease to zero as the flow approaches zero.

Refer to the "Charts" in HDS-5 (FHWA, 2005b) for graphical solution of headwater under inlet control. The fifth degree polynomials are an approximation of the original research performed by FHWA. Rather than have the engineer solve using Equation 9.14, FHWA developed the charts for hand computations. Those charts have since been modified to be equivalent to the Equations 9.11 through 9.13, which were developed from the same data as the fifth degree polynomials. Within the accuracy of reading the charts the solutions are equivalent to those equations.

I. Headwater Under Outlet Control. Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. Outlet control is likely only when the hydraulic grade line inside the culvert at the entrance exceeds critical depth. Therefore, outlet control is most likely when the flow in the culvert is on a mild slope ($d_n > d_c$). It is also possible to experience outlet control with a culvert on a steep slope ($d_n < d_c$) and a high tailwater such that subcritical flow or full flow exists in the culvert.

The headwater resulting from flow through a culvert in outlet control is a function of the following:

- Discharge.
- Culvert section geometry.
- Culvert roughness characteristics.
- Length of the culvert.
- Profile of the culvert.
- Entrance geometry (to a minor extent).
- Tailwater level (possibly).

For practical purposes, when a culvert is under outlet control, the headwater can be adjusted by modifying either the culvert geometry or roughness.

Outlet control headwater HW_{oc} depth (from the flowline of the entrance) is computed by balancing energy between the culvert exit and the culvert entrance as indicated by Equation 9.18.

(Equation 9.18)

$$HW_{oc} + h_{va} = h_e + h_{vi} + \sum h_f - S_o$$

where:
- HW_{oc} = headwater depth due to outlet control, m (ft)
- h_{va} = velocity head of flow approaching the culvert entrance, m (ft)
- h_{vi} = velocity head at entrance as calculated using Equation 9.19, m (ft)
- h_e = entrance head loss as calculated using Equation 9.20, m (ft)
- h_f = friction head losses as calculated using Equation 9.21, m (ft)
- S_o = culvert slope, m/m (ft/ft)

When applying Equation 9.18, the velocity at the entrance often is assumed to be negligible (i.e. $h_{vi} = 0$) so that the headwater and energy grade line are coincident at the upstream face of the culvert. This is conservative for most Department needs. Some conditions may warrant consideration of the approach velocity, in which case the approach velocity head could be subtracted. Possible scenarios under which it may be necessary to consider the approach velocity include:

- Estimating the impact of a culvert on FEMA designated flood plains.
- Designing or analyzing a culvert used as a flood attenuation device where the storage volumes are very sensitive to small changes in headwater.
- A culvert that has an effective flow area similar to the approach channel section so that approach velocities and through culvert velocities are similar.

The velocity head at any location in the culvert is computed using Equation 9.19. The velocity at the entrance (v_i) is used to compute the velocity head at the entrance.

(Equation 9.19)

$$h_v = \left[\frac{v^2}{2g}\right]$$

where:
- v = flow velocity in culvert, m/s (ft/s)
- g = the gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

The entrance loss, h_e, is dependent on the velocity of flow at the inlet, v_i, and the entrance configuration, which is accommodated using an entrance coefficient, C_e.

(Equation 9.20)

$$h_e = C_e\left[\frac{v_i^2}{2g}\right]$$

where:
- C_e = entrance loss coefficient
- v_i = flow velocity inside culvert inlet, m/s (ft/s)

Values of C_e are selected by the designer from the values in Table 9.5 (entrance loss coefficients) based on culvert shape and entrance condition. The velocity, v_i, is equal to the outlet velocity (v_o) for submerged conditions. For free surface flow, the designer should determine the velocity at the entrance using backwater calculations through the culvert.

The outlet depth, H_o, is established based on the following conditions:

- For steep slope: if the Manning's Equation capacity of the culvert is lower than the discharge (i.e., $d_u > D$), the culvert is expected to flow full and the outlet depth, H_o, is taken as the higher of the barrel depth, D, and the tailwater depth, TW. Otherwise (i.e., $d_u < D$), the outlet depth is set to the tailwater depth when the tailwater exceeds critical depth (d_c). If the tailwater is below critical depth in a steep-slope culvert, outlet control headwater is unlikely.
- For a culvert on a mild slope, if critical depth exceeds the barrel depth ($d_c > D$), the outlet depth (H_o) is taken as the higher of the barrel depth and the tailwater depth. Otherwise (i.e., $d_c < D$), the outlet depth is taken as the higher of critical depth and the tailwater depth.

J. Outlet Control Headwater Due to Full Flow in Culverts. The friction slope is constant over the length of the barrel that is flowing full. The frictional headloss, h_f, is computed using Equation 9.21.

(Equation 9.21)

$$h_f = S_f L$$

where:
- h_f = head loss due to friction in the culvert barrel, m (ft)
- S_f = friction slope, m/m (ft/ft) (See Equation 9.10)
- L = length of culvert containing full flow, m (ft)

Table 9.5. Entrance Loss Coefficients (C_e)

Concrete Pipe	**C_e**
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Corrugated Metal Pipe or Pipe-Arch	**C_e**
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Reinforced Concrete Box	**C_e**
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

** "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design, have a superior hydraulic performance.*

Full flow at the outlet occurs when the outlet depth (H_o) is equal to, or higher than, the barrel depth, D. Refer to the discussion above regarding the outlet depth.

For a completely submerged culvert, as shown in Figure 9.9, Equation 9.18 is used directly to determine the outlet control headwater. Figure 9.9 includes all the energy conditions associated with a straight, prismatic culvert. The magnitude of the energy loss at the exit, h_o, does not affect the outlet control headwater under the assumption that the hydraulic grade line inside the culvert is equal to the tailwater depth or critical depth (whichever controls). In some publications, the velocity head at the entrance is termed exit loss.

If the friction slope is less than the culvert slope, it is possible that full flow may not occur along the entire length of the culvert. Free surface flow begins at the point of intersection of the hydraulic grade line and the soffit of the culvert barrel as shown in Figure 9.9(C). If this condition occurs, the outlet control headwater is determined using the free surface flow approach described below with the starting depth (d_1) equal to the barrel rise (D) and starting at the location along the barrel at which free surface flow begins. The total friction losses ($\sum h_f$) include the head loss due to full flow friction (h_{ff}) and the friction losses associated with partially full or free-surface flow (h_{fp}). The energy loss at the exit, h_o, does not affect the outlet control headwater under the assumption that the hydraulic grade line at the culvert exit is equal to the tailwater elevation.

The distance from the outlet at which free surface flow begins (L_f) is determined using the geometric relationship shown in Equation 9.22.

(Equation 9.22)

$$x = \frac{h_o + H_o - D}{S_p - S_f}$$

If x is greater than the culvert length, or S_f is greater than S_o, the entire length of the culvert is full.

When the outlet is not submerged, if the culvert is long enough and the flow high enough, full flow will begin within the culvert. The point at which full flow begins is determined using the procedures described in Section 9.2.K. Figure 9.9(C) illustrates this condition. Since the hydraulic grade line has already been established as being at a depth of D, the entrance control headwater is determined using Equation 9.23.

(Equation 9.23)

$$HW_{oc} = D + S_f L_f - S_o L_f + h_e + h_{vi} - h_{va}$$

where: L_f = length from beginning of full flow in culvert to culvert entrance in m (ft)

This constitutes an implicit use of Equation 9.15 since the backwater calculation accommodates the friction losses (h_{fp}), and change in elevation for the free surface flow length ($L - L_f$). As discussed earlier, the velocity head associated with the flow approaching the culvert entrance often is ignored.

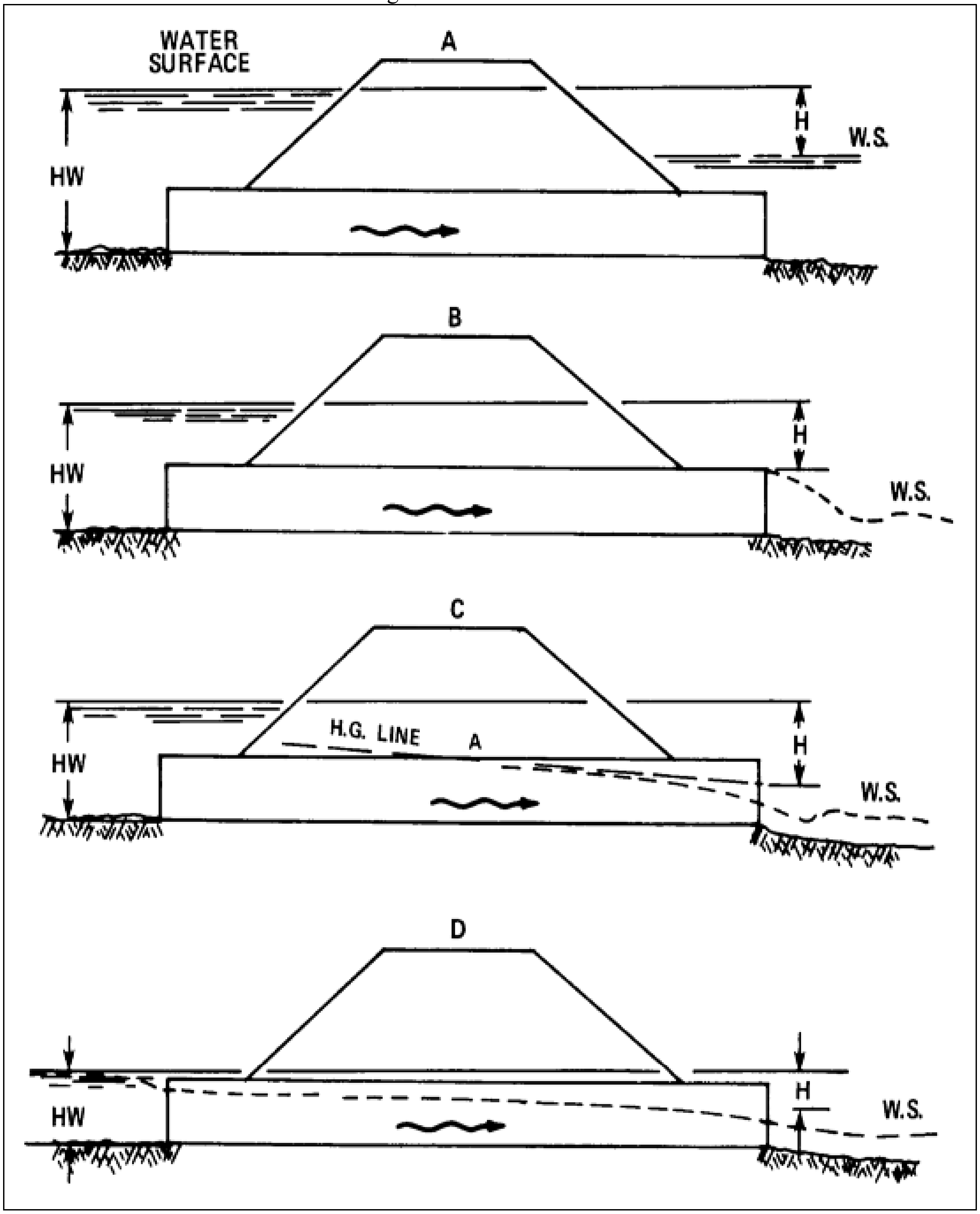
K. Outlet Control Headwater Due to Free-Surface Flow in Culverts. If free surface flow is occurring in the culvert, the hydraulic parameters are changing with flow depth along the length of the culvert. One of two combinations of free surface flow is likely to occur in the culvert:

1. Entire culvert maintains free surface flow, as shown in Figure 9.9(D)

2. Free surface at exit but backwater reaches barrel rise within culvert, establishing full flow for the remaining portion, as shown in Figure 9.9(C)

For each of the above conditions, it is necessary to calculate the backwater profile based on a starting depth, d_i. The following discusses the selection of appropriate starting depths.

Figure 9.9 Outlet Control



By definition, a free-surface backwater from the outlet end of a culvert may only affect the headwater when subcritical flow conditions exist in the culvert. Subcritical, free-surface flow at the outlet will exist if either the culvert is on a mild slope with a hydraulic grade line (H_o) lower than the outlet soffit or the culvert is on a steep slope with a hydraulic grade line higher than critical depth at culvert outlet and lower than the outlet soffit. Therefore, when free surface flow exists at the outlet, the starting depth should be taken as the higher of critical depth (d_c) and the tailwater depth (TW).

As indicated in Section 9.2.J., if full flow exists at the outlet and free-surface flow begins upstream in the barrel, the backwater calculations should begin at the barrel rise (d_1 = D) and continue to the inlet to get the depth at the inlet, d_i.

The Direct Step Method, which is discussed in Section 9.2.N., is appropriate for determining water surface profiles in culverts. The calculations begin at the outlet and proceed in an upstream direction until either the end of the culvert is reached (at which $d_i = d_2$), or the calculated depth (d_2) reaches the barrel rise (D).

When subcritical free surface flow exists at the inlet, the outlet control headwater is calculated using Equation 9.24.

(Equation 9.24)

$$HW_{oc} = d_i + h_e + h_{vi} - h_{va}$$

This is applying Equation 9.15 implicitly because the backwater calculation process accommodates the friction losses ($\sum_{hf}$) and the change in elevation (S_oL). The approach velocity head (h_{va}) often is ignored.

If the backwater reaches the barrel depth before reaching the end of the culvert, the remaining length, L_f, will flow full and the outlet control headwater is computed using Equation 9.23.

L. Slug Flow. When the flow becomes unstable, a phenomenon termed slug flow may occur. In this condition the flow varies from inlet control to outlet control and back again in a cyclic pattern under the following circumstances:

- Flow is indicated as supercritical but the tailwater level is relatively high.
- Uniform depth and critical depth are relatively high with respect to the culvert barrel depth.
- Uniform depth and critical depth are within about 5% of each other.

The methods discussed in this chapter accommodate the potential for slug flow by assuming the higher of inlet and outlet control headwater.

M. Determination of Outlet Velocity. The outlet velocity, v_o, is dependent on the culvert discharge (Q) and the cross sectional area of flow at the outlet (A_o).

(Equation 9.25)

$$v_o = \frac{Q}{A_o}$$

The variable d_o is assigned as the depth with which to determine the cross sectional area of flow at the outlet.

For outlet control, the depth, d_o, is set equal to the higher of critical depth (d_c) and tailwater depth (TW) as long as the value is not higher than the barrel rise (D) as shown in Figure 9.9. If the culvert will flow full at the outlet, usually due to a high tailwater or a culvert capacity lower than the discharge, d_o is set to the barrel rise (D) so that the full cross sectional area of the culvert is used.

For inlet control, under steep slope conditions, the designer may estimate the depth at the outlet using one of the following approaches:

1. Employ a step backwater method (such as the Direct Step Method discussed in Section 9.2.N.) starting from critical depth (d_c) at the inlet and proceeding downstream to the outlet. If the tailwater is lower than critical depth at the outlet, calculate the velocity resulting from the computed depth at the outlet. If the tailwater is higher than critical depth, a hydraulic jump within the culvert is possible. Section 9.2.O. discusses a means of estimating whether the hydraulic jump occurs within the culvert. If the hydraulic jump does occur within the culvert, determine the outlet velocity based on the outlet depth, $d_o = H_o$.

2. Assume uniform depth at the outlet. If the culvert is long enough and tailwater is lower than uniform depth, uniform depth will be reached at the outlet of a steep slope culvert. For a short, steep culvert with tailwater lower than uniform depth, the actual depth will be higher than uniform depth but lower than critical depth, so this assumption will be conservative in that the estimate of velocity will be somewhat higher than the actual velocity. If the tailwater is higher than critical depth, a hydraulic jump is possible and the outlet velocity could be significantly lower than the velocity at uniform depth. Outlet velocities should be checked if a hydraulic jump is present since outlet protection may no longer be necessary.

N. Direct Step Backwater Method Applied to Culverts. The Direct Step Backwater Method uses the same basic equations as the Standard Step Backwater Method (see Section 8.5), but is simpler to compute because no iteration is necessary. In the Direct Step Method, the designer chooses an increment (or decrement) of water depth (d) and computes the distance over which the depth change occurs. The accuracy is dependent on the size of d. The method is appropriate for prismatic channel sections such as occur in most culverts. It is useful for estimating supercritical profiles and subcritical profiles. The following provides some direction on its application for free surface flow in culverts.

1. Choose a starting point with a known starting water depth (d_1). This starting depth is dependent on whether the profile is supercritical or subcritical.

- For a mild slope ($d_c < d_u$) and free surface flow at the outlet, begin at the outlet end. Select the higher of critical depth (d_c) or tailwater depth (TW). This is not to imply that supercritical flow will not occur in a culvert on a mild slope; however, most often, the flow will be subcritical when mild slopes exist. A check of this assumption may be necessary.
- For a steep slope ($d_c > d_u$), where the tailwater exceeds critical depth but does not submerge the culvert outlet, begin at the outlet with the tailwater as the starting depth.
- For a steep slope in which tailwater depth is lower than critical depth, begin the water surface profile computations at the culvert entrance starting at critical depth and proceeding downstream to the culvert exit. This implies inlet control in which case the computation may be necessary to determine outlet velocity but not headwater.
- For a submerged outlet in which free surface flow begins within the barrel, use the barrel depth, D, as the starting depth. Begin the backwater computations at the location where the hydraulic grade line is coincident with the soffit of the culvert.

The following steps assume subcritical flow on a mild slope culvert for a given discharge, Q, through a given culvert of length L, at a slope S_o.

2. Calculate the following at the outlet end of the culvert based on the selected starting depth (d_1):

- Cross section area of flow, A.
- Wetted perimeter, P.
- Velocity, v (= Q/A).
- Velocity head, h_v, using Equation 9.19.
- Specific energy, E, using Equation 9.1.
- Friction slope, S_f, using Equation 9.10.
- Assign the subscript 1 to the above variables (A_1, P_1, etc.).

3. Choose an increment or decrement of flow depth, Δd; if $d_1 > d_u$, use a decrement (negative Δd); otherwise use an increment. The increment, Δd, should be such that the change in adjacent velocities is not more than 10%.

4. Calculate the parameters, A, P, v, E, and S_f at the new depth, $d_2 = d_1 + \Delta d$, and assign the subscript 2 to these (e.g., A_2, P_2, etc.).

5. Determine the change in energy, ΔE, using Equation 9.26.

(Equation 9.26)

$$\Delta E = E_2 - E_1$$

6. Calculate the arithmetic mean friction slope using Equation 9.27.

(Equation 9.27)

$$S_f = \frac{(S_{f2} + S_{f1})}{2}$$

7. Using Equation 9.28, determine the distance, ΔL, over which the change in depth occurs.

(Equation 9.28)

$$\Delta L = \frac{\Delta E}{S_o - S_f}$$

8. Consider the new depth and location to be the new starting positions (assign the subscript 1 to those values currently identified with the subscript 2) and repeat steps 3 to 7, summing the incremental lengths, Δ L, until the total length, $\sum$L, equals or just exceeds the length of the culvert. The same increment may be used throughout or the designer may modify the increment to achieve the desired resolution. Such modifications are necessary when the last total length computed far exceeds the culvert length and when high friction slopes are encountered. If the computed depth reaches the barrel rise (D) before reaching the culvert inlet, skip step 9 and determine the outlet control headwater using Equation 9.23.

9. The last depth (d_2) established is the depth at the inlet (d_i) and the associated velocity is the inlet, v_i. Calculate the headwater using Equation 9.24.

The procedure for subcritical flow ($d > d_c$), but steep slope ($d_c > d_u$) is similar, with the following exceptions:

- Choose a decrement in depth, d.
- If the depth, d, reaches critical depth before the inlet of the culvert is reached, the headwater is under inlet control (Section 9.2.H.) and a hydraulic jump may occur in the culvert barrel. Refer to Section 9.2.O. for discussion of the hydraulic jump.
- If the depth at the inlet is higher than critical depth, determine the outlet control head water using Equation 9.24 as discussed in Section 9.2.K. A hydraulic jump may occur within the culvert. Refer to Section 9.2.O. for discussion of the hydraulic jump.

The procedure for supercritical flow ($d < d_c$) and steep slope is similar, with the following exceptions:

- Begin computations at critical depth at the culvert entrance and proceed downstream.
- Choose a decrement of depth, d.
- If the tailwater is higher than critical depth, a hydraulic jump may occur within the culvert. Refer to Section 9.2.O. for discussion of the hydraulic jump.

O. Hydraulic Jump in Culverts. For a given discharge in any channel, when water flows at a depth which is less than critical depth (supercritical flow), there is a "sequent" (or "conjugate") depth in subcritical flow such that the sum of the forces due to momentum and hydrostatic pressure at respective cross sections will be the same. With a proper configuration, the water flowing at the lower depth in supercritical flow can "jump" abruptly to its sequent depth in subcritical flow. This is called a hydraulic jump. With the abrupt increase in flow depth, there is a corresponding increase in cross sectional area of flow and a decrease in average velocity.

The balance of forces is represented using a momentum function, as appears in Equation 9.29.

(Equation 9.29)

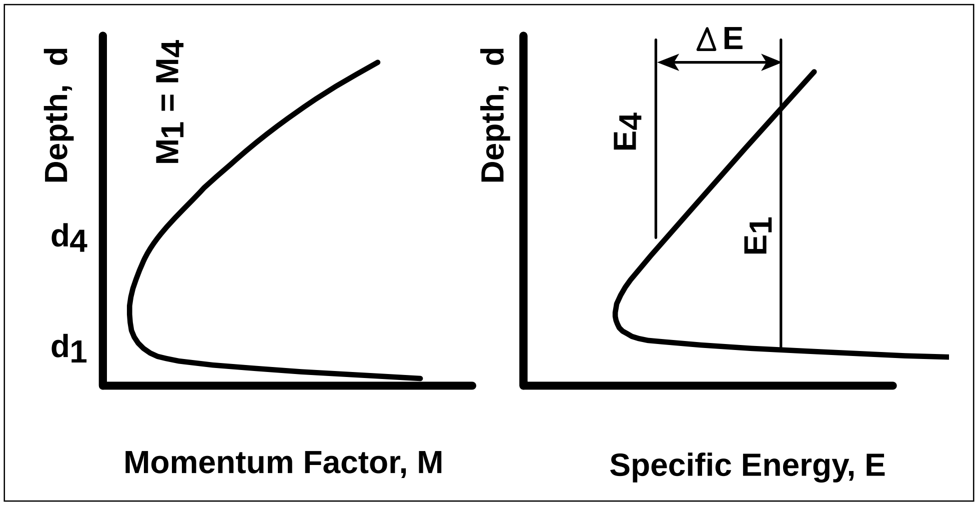
$$M = \frac{Q^2}{gA} + A\bar{d}$$

where:	M	=	momentum function
	Q	=	discharge, m^3/s (cfs)
	A	=	section area of flow, m^2 (ft^2)
	$\bar{d}$	=	height from water surface to centroid of flow area, m (ft)

The term $A\bar{d}$ represents the moment of the area about the water surface. Assuming no drag forces or frictional forces at the jump, conservation of momentum maintains that the momentum function at the approach depth, M_1, is equal to the momentum function at the sequent depth, M_s.

Figure 9.10 provides a sample plot of depth and momentum function and an associated specific energy plot. By comparing the two curves at a supercritical depth and its sequent depth it can be seen that the hydraulic jump involves a loss of energy. Also, the momentum function defines critical depth as the point at which minimum momentum is established.

Figure 9.10 Momentum Function and Specific Energy



The potential occurrence of the hydraulic jump within the culvert is determined by comparing the outfall conditions with the sequent depth of the supercritical flow depth in the culvert. The conditions under which the hydraulic jump is likely to occur depend on the slope of the culvert.

Under mild slope conditions ($d_c < d_u$), two typical conditions could result in a hydraulic jump:

1. The backwater profile in the culvert caused by the tailwater is higher than the sequent depth computed at one location in the culvert.

2. The supercritical profile reaches critical depth upstream of the culvert outlet.

Under steep slope conditions, the hydraulic jump is likely only when the tailwater is higher than the sequent depth.

P. Sequent Depth for Rectangular Culvert. A direct solution for conjugate depth, d_s, is possible for free surface flow in rectangular culverts using Equation 9.30.

(Equation 9.30)

$$d_s = 0.5d_1\left(\sqrt{1+\frac{8v_1^2}{gd_1}}-1\right)$$

where:
d_s = sequent depth, m (ft)
d_1 = depth of flow, m (ft)
v_1 = velocity of flow at depth d_1, m/s (ft/s)

Q. Sequent Depth for Circular Culvert. A direct solution for conjugate depth in a circular culvert is not feasible. However, an iterative solution is possible by selecting a trial sequent depth, d_s, and applying Equation 9.31 until the calculated discharge is equal to the design discharge.

(Equation 9.31)

$$Q^2 = \frac{g\left(A_s\overline{d}_s - A_1\overline{d}_1\right)}{\frac{1}{A_1} - \frac{1}{A_s}}$$

where:
- Q = discharge, m^3/s (cfs)
- A_s = area of flow at sequent depth, m^2 (ft^2)
- $A_s\,\overline{d}_s$ = moment of area about surface at sequent depth, m^3 (ft^3)
- $A_1\,\overline{d}_1$ = moment of area about surface at supercritical flow depth, m^3 (ft^3)

R. Sequent Depth for Other Shapes. Equation 9.31 is applicable to other culvert shapes. The moment of area about the surface, $A\overline{d}$, is dependent on the shape of the culvert and depth of flow. The designer should acquire or derive a relationship between flow depth and second moment of area.

S. Roadway Overtopping. Where water flows both over the roadway and through a culvert, a definition of hydraulic characteristics requires a flow distribution analysis. This is a common problem where a discharge of low probability of occurrence (e.g., 500-year return period) is applied to a facility which may have been designed for a higher probability (e.g., 10-year return period). For example, a complete design involves the application and analysis of a 100-year discharge to a hydraulic facility which may have been designed for a much smaller flood.

In such a case, the headwater may exceed the low elevation of the roadway, causing part of the water to flow over the roadway embankment while the remainder flows through the structure. The headwater components of flow form a common headwater level. An iterative process is necessary to establish this common headwater.

The following is one iterative approach that is reasonable for hand computations and computer programs.

Step 1 Initially assume that all the runoff (analysis discharge) passes through the culvert and determine the headwater using the procedures outlined in Section 9.3.D. If the headwater is lower than the low roadway elevation, no roadway overtopping occurs and the analysis is complete; otherwise, proceed to Step 2.

Step 2 Record the analysis discharge as the initial upper flow limit and zero as the initial lower flow limit. Assign 60 or 40% of the analysis discharge to the culvert and the remaining portion to the roadway as the initial apportionment of flow.

Step 3 Using the procedures outlined in Section 9.3.D., determine the headwater and the apportioned flow for the culvert.

Step 4 Compute the roadway overflow (discharge) associated with the headwater level determined in step 3 using the Weir Equation (Equation 9.32).

(Equation 9.32)

$$Q = k_t C L H_h^{1.5}$$

where:
- Q = discharge, m^3/s (cfs)
- k_t = over-embankment flow adjustment factor for friction
- C = discharge coefficient (for roadway overtopping use 1.66 (metric), 2.6 (U.S. Customary))
- L = horizontal length of overflow, m (ft). This length should be perpendicular to the overflow direction. For example, if the roadway curves, the length should be measured along the curve.
- H_h = average depth between headwater and low roadway elevation, m (ft)

The value H_h assumes that the effective approach velocity is negligible (less than 1.5 m/s (4.92 ft/s)), similar to the culvert headwater procedure. For estimation of maximum headwater, this is a conservative assumption; however, under some conditions, such as the need to provide adequate detention storage, the designer may need to consider the approach velocity head ($v^2/2g$). That is, replace H_h with $H_h + v^2/2g$ in Equation 9.32.

Tailwater will affect the over-embankment flow if its excess (H_t) over the highway is lower than critical depth of flow over the road, which is approximately 0.67 H_h. For practical purposes, H_t/H_h may approach 0.8 without any correction coefficient. For H_t/H_h values above 0.8, use Figures 9.11(a) and 9.11(b) to determine k.

For most cases of flow over highway embankments, the section over which the discharge must flow is parabolic or otherwise irregular. In such cases, it becomes necessary to divide the section into manageable increments and to calculate individual weir flows for the incremental units, summing them for total flow.

If the tailwater is sufficiently high, the flow over the roadway can be affected. In fact, at high depth, the flow over the road may become open channel flow and weir calculations are no longer valid. At extremely high depth of roadway overtopping, it may be reasonable to ignore the culvert opening and compute the water surface elevation based on open channel flow over the road.

Figure 9.11(a) Metric Discharge Coefficients for Roadway Overtopping
(Source: *The Model Drainage Manual*, Chapter 9 (AASHTO, 2005))

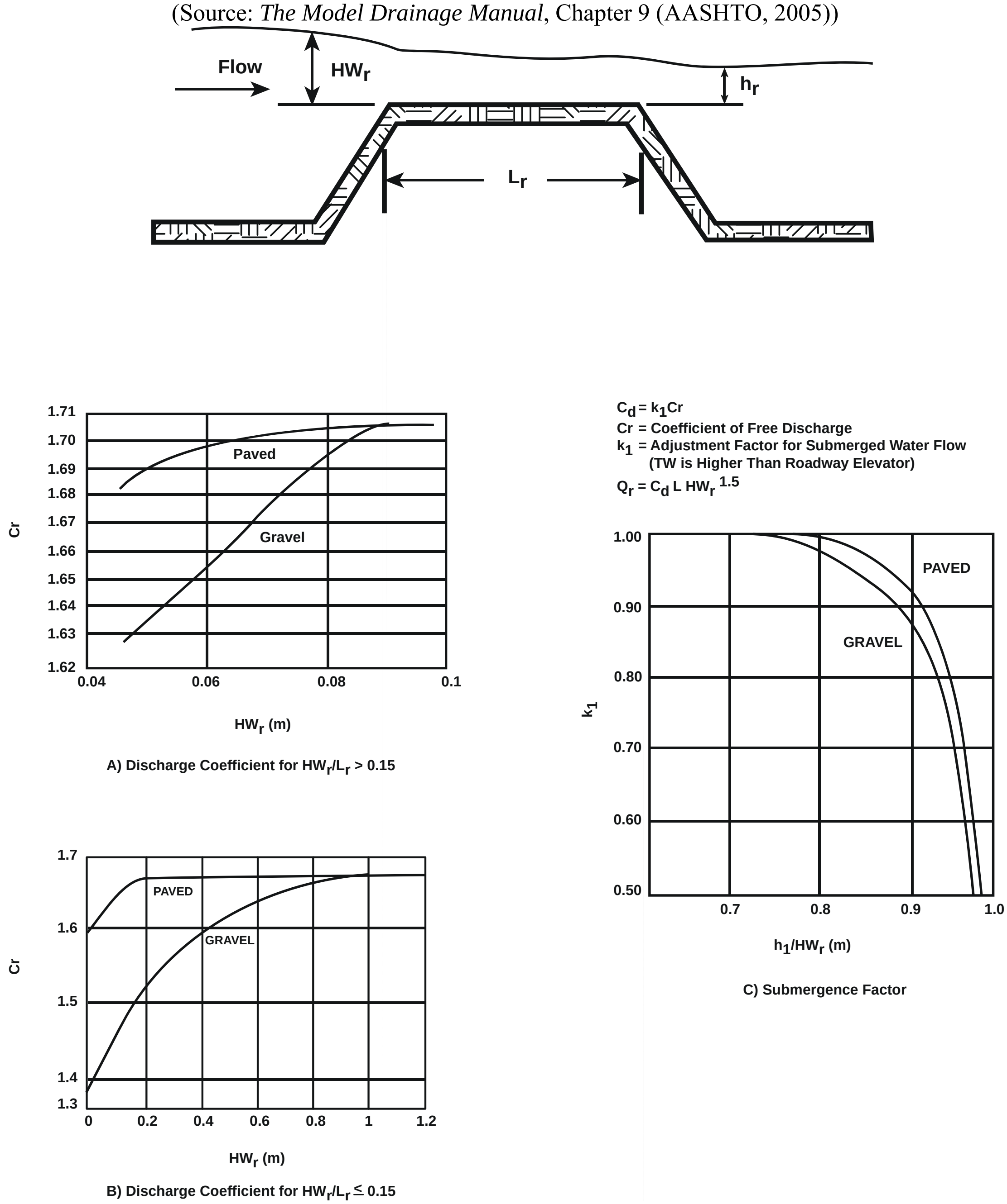
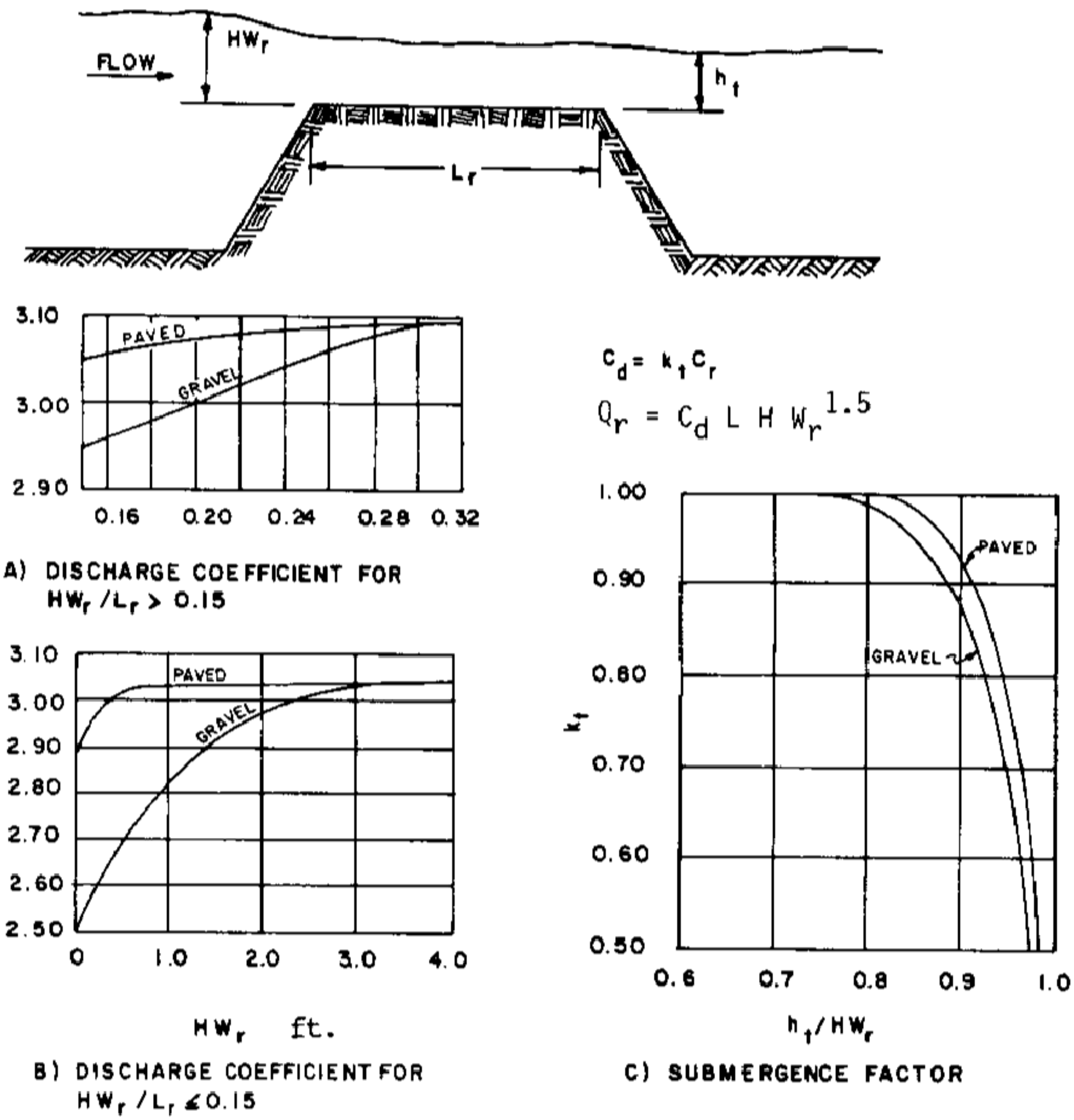


Figure 9.11(b) U.S. Customary Drainage Coefficients for Roadway Overtopping
(Source: *The Model Drainage Manual*, Chapter 9 (AASHTO, 2005))



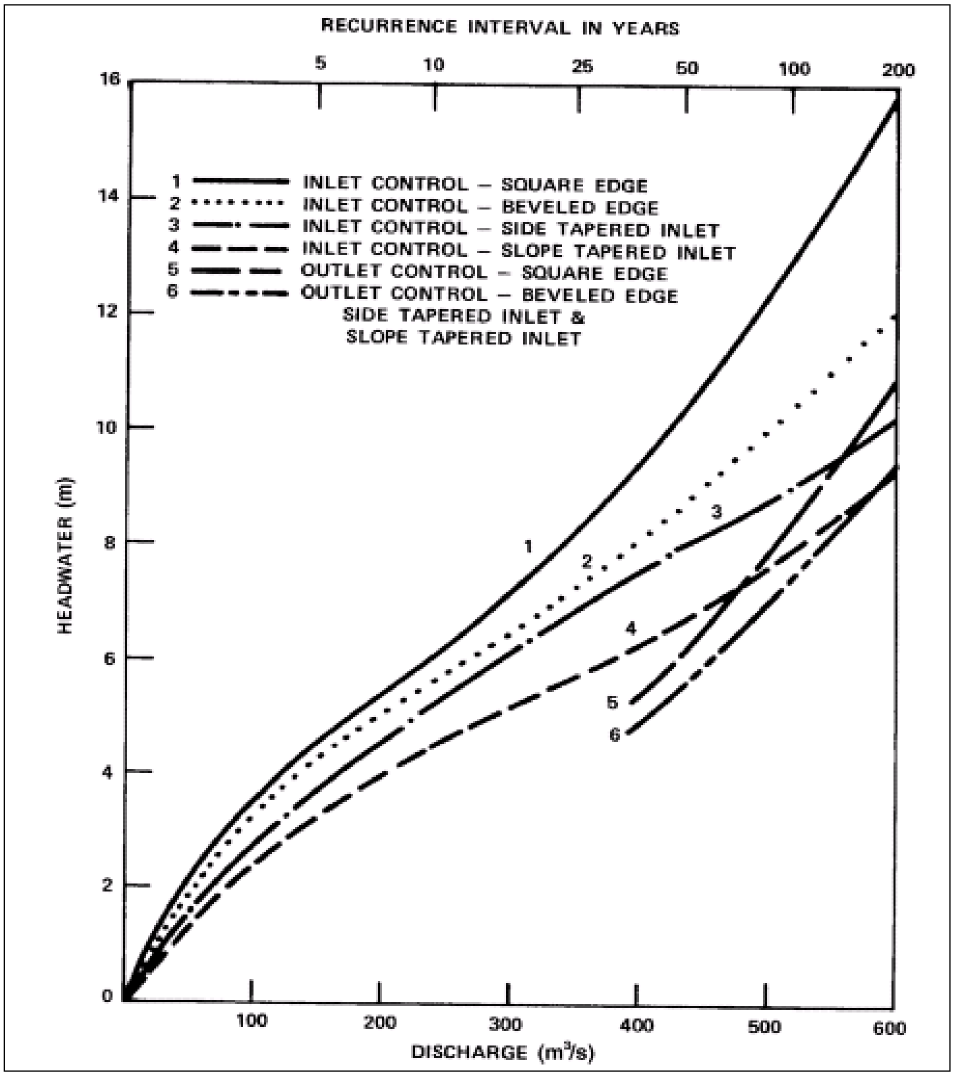
Step 5 Add the calculated roadway overflow to the culvert flow. If this is greater than the analysis discharge, record the current culvert flow apportionment as the current upper flow limit and set the new culvert flow apportionment at a value halfway between the current upper and lower flow limits. If the calculated total is less than the analysis discharge, record the current culvert flow apportionment as the lower flow limit and set the new culvert flow apportionment at a value halfway between the current upper and lower flow limits.

Step 6 Repeat Steps 3 to 5, using the culvert flow apportionment established in Step 5, until the difference between the current headwater and the previous headwater is less than a reasonable tolerance. For computer programs, consider a tolerance of about 3 mm (0.1 in). Consider the current headwater and current assigned culvert flow and calculated roadway overflow as the final values.

T. Performance Curves. For any given culvert, the control (outlet or inlet) might vary with the discharge. Figure 9.12 shows sample plots of headwater versus discharge for inlet and outlet control. The envelope (shown as the bold line) represents the highest value of inlet and outlet headwater for any discharge in the range. This envelope is termed a performance curve. In this example, inlet control prevails at lower discharges and flow transitions to outlet control as the discharge increases. The flatter portion represents the effect of roadway overflow.

The performance curve is generated by performing culvert headwater computations for increasing values of discharge. Such information is particularly useful for performing risk assessments and for hydrograph routing through detention ponds and reservoirs.

Figure 9.12 Typical Performance Curve



9.3 CULVERT DESIGN/ANALYSIS PROCEDURE

A. General - Culvert Design/Analysis Procedure. The following basic steps are necessary. The procedure contained in HDS-5, *Hydraulic Design of Highway Culverts* (FHWA, 2005b), as noted below, shall be used for the design of pipe culverts:

- Define the location, orientation, shape, and material for the culvert to be designed. In many instances, more than one shape and/or material may be considered.
- With consideration of the site data, establish a maximum limit for barrel depth.
- Based upon the discharges of interest, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (as part of the iterative design process of trial and error).
- Determine the flow type (supercritical or subcritical) to determine headwater and outlet velocity.
- Reconsider and refine the culvert configuration if necessary.
- Treat any excessive outlet velocity separately from headwater. Energy dissipators may be warranted to reduce the outlet velocity to acceptable limits.

B. Multiple Barrels. As a general rule, if there will be more than one barrel in the culvert, use shapes of uniform geometry and roughness characteristics. The flow distribution for uniform multiple barrels is then a simple equal distribution of flow through each barrel. However, if the inverts are set at different elevations, then the flow distributions are not equal and computer modeling using HY-8 (FHWA, 1997) or HEC-RAS should be used for the analysis.

C. Overview of Culvert Hydraulic Design. The procedure for culvert hydraulic design is usually an iterative process. The following must be done until all design criteria are satisfied economically:

- Analyze a trial configuration.
- Compare the results with the design criteria.
- Adjust the configuration.
- Perform another analysis.

The flow charts shown in Figures 9.13 and 9.14 provide guidance throughout the culvert design process. The flow chart may not address every nuance which the designer may encounter as the culvert is designed. However, it does lend guidance to the computational process for the vast majority of culvert design situations.

Figure 9.13 Flow Chart A – Culvert Design Procedure

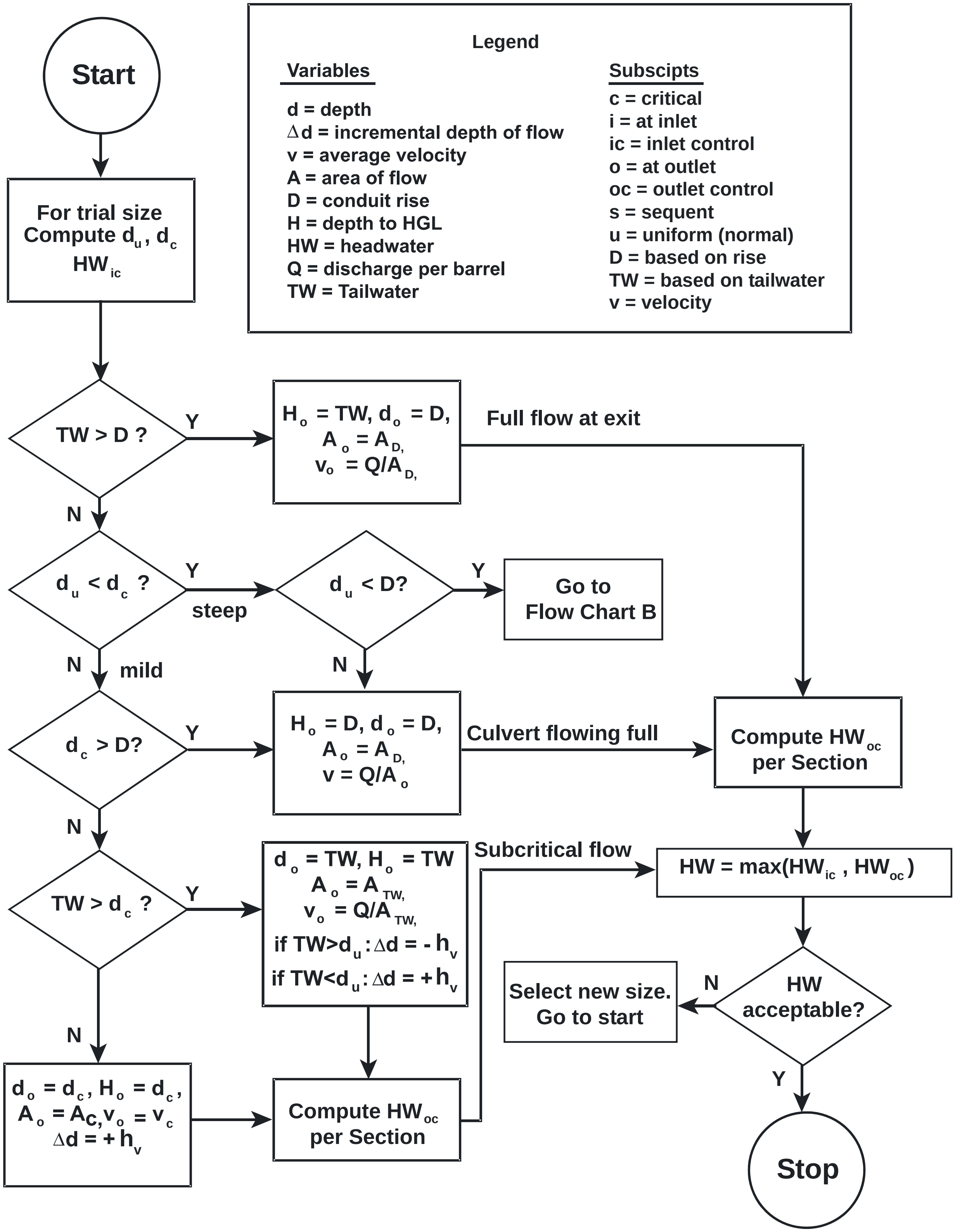
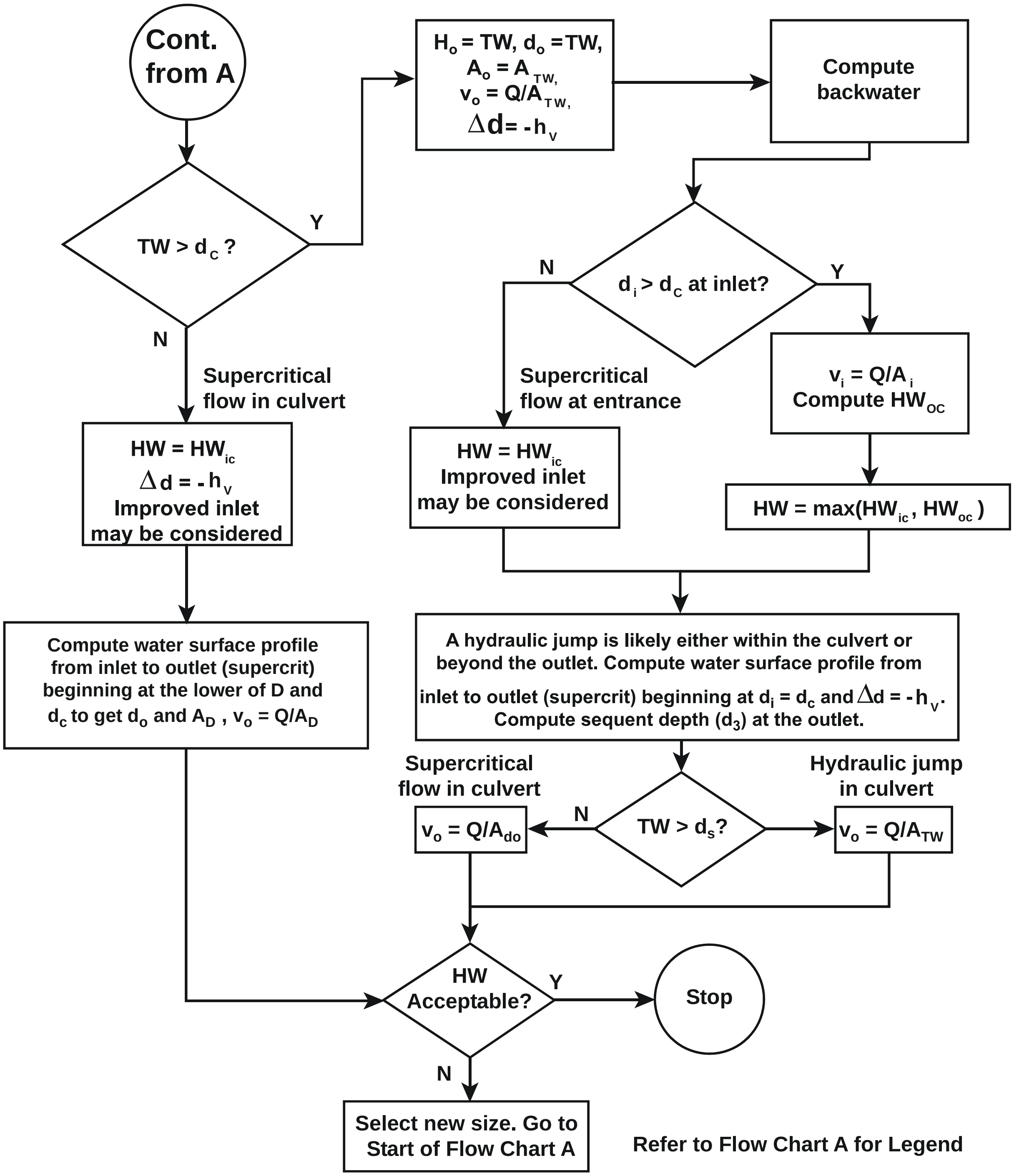


Figure 9.14 Flow Chart B – Culvert Design Procedure (continued)



While the important criterion of allowable outlet velocity is considered, it has little or no influence on the culvert barrel configuration in the design process. Any problem with excessive outlet velocity should be treated separately in most cases (e.g., the use of energy dissipators). The reader is referred to Chapter 12, *Erosion and Sediment Pollution Control* and Publication 72M, *Roadway Construction Standards* for guidance on designing outlet protection.

As mentioned previously, the designer must choose design storms for which the culvert will be analyzed. Storms with larger return periods should be checked to see if the performance of the proposed structure is within acceptable tolerance, and to check for compliance with the regulatory criteria.

D. Design Procedure for Culverts. The following is a step-by-step culvert design procedure for a standard culvert configuration. That is, straight in profile and, if multiple barrels are used, the barrels are parallel and of equal size. Variations from normal culvert design are covered elsewhere in this chapter. Any of the configurations considered in the iterative process of design will influence a unique flow type. Each new iteration requires a determination of whether there is inlet or outlet control.

1. Establish an initial trial size assuming inlet control.

Determine the maximum practical rise of culvert (D_{max}) and the maximum allowable headwater depth (HW_{max}). Determine a trial head using Equation 9.33.

(Equation 9.33)

$$h = HW_{max} - \frac{D_{max}}{2}$$

where:
h = allowable effective head, m (ft)
HW_{max} = allowable headwater depth, m (ft)
D_{max} = maximum culvert rise, m (ft)

Use Equation 9.34 (a form of the orifice equation) to determine the required area, A, for the design discharge, Q. This assumes an orifice coefficient of 0.5 which is reasonable for initial estimates only.

(Equation 9.34)

$$A = k\frac{Q}{h^{0.5}}$$

where:
A = approximate sectional area required, m^2 (ft^2)
Q = design discharge, m^3/s (cfs)
k = 0.45 (Metric), 0.25 (U.S. Customary)

Decide on the culvert shape.

For a box culvert:

a. Determine the required width, W, as A/D_{max}.

b. Round W to the nearest value which yields a whole multiple of standard box widths.

c. Divide W by the largest standard span S for which W is a multiple. This yields the number of barrels, N.

d. At this point, the determination has been made that the initial trial configuration will be N - S D_{max} L.

For a circular pipe culvert:

a. Determine the ratio of area required to maximum barrel area as:

(Equation 9.35)

$$\frac{4A}{\pi D_{max}^2}$$

b. Round this value to the nearest whole number to get the required number of barrels, N.

c. At this point, the determination has been made that the initial trial size culvert will be N - DL circular pipe.

For other shapes, provide an appropriate size such that the cross section area is approximately equal to A.

2. Determine the design discharge per barrel as Q/N. This assumes that all barrels are of equal size and parallel profiles. The computations precede using one barrel with the appropriate apportionment of flow.

3. Perform a hydraulic analysis of the trial configuration. Generally, the designer should employ a computer program or spreadsheet.

For the trial configuration, determine the inlet control headwater (HW_{ic}), the outlet control headwater (HW_{oc}), and outlet velocity (v_o) using Flow Chart A shown in Figure 9.13. Flow Chart A references Flow Chart B, which is shown in Figure 9.14. Table 9.6 provides references for some of the variables required.

Table 9.6. Quick References for Culvert Variables

Variable	**Reference Section (s)**
Critical depth (d_c)	9.2.D.
Uniform depth (d_u)	9.2.E.
Outlet depth (H_o)	9.2.J.
Outlet velocity (v_o)	9.2.M.
Full flow headwater (HW_{oc})	9.2.J.
Free surface flow headwater (HW_{oc})	9.2.K.
Inlet control headwater (HW_{ic})	9.2.H.
Friction Slope (S_f)	9.2.F.
Sequent depth (d_s)	9.2.P., 9.2.Q. & 9.2.R.

4. Evaluate trial design. At this step in the design process, a headwater and outlet velocity have been calculated for the design discharge through a trial culvert configuration.

a. If the calculated headwater is equal to or is not appreciably lower than the allowable headwater (an indication of culvert efficiency), the design is complete. A good measure of efficiency is to compare the calculated headwater with the culvert depth D. If the headwater is less than the depth, the configuration may not be efficient.

b. If the calculated headwater is considerably lower than the allowable headwater or lower than the culvert depth D, a more economical configuration may be possible. The trial culvert configuration should be changed by reducing the number of barrels, span widths, diameter, or other geometric or material changes. The calculations must then be repeated; go back to step 2.

- If the calculated headwater is equal to or is not appreciably lower than the allowable headwater, and the culvert is operating in inlet control, an improved inlet may be in order.
- If the operation is not inlet control, then the culvert geometry design is complete.
- If the calculated headwater is greater than the allowable headwater, the trial culvert configuration should be changed to increase capacity by adding barrels, widening spans, and increasing diameter. Regardless of the changes made here, the calculations must be repeated. Go back to Step 2.

If the culvert is operating with inlet control, the possibility exists for improving the entrance conditions with the aim of reducing the overall cost of the structure. This may be done by investigating the design of a flared (or tapered) inlet and associated structure. The design procedures for improved inlets are discussed in Section 9.4, Improved Inlets.

Due to the cost of the improved inlet, a careful economic comparison should be made between the design with a normal entrance and the design with an improved inlet.

The culvert for which the calculated headwater is satisfactory may have an excessive outlet velocity. The definition of an "excessive" outlet velocity is normally an engineering judgment based on local conditions (HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, FHWA, 2006).

In comparison to adjusting the culvert barrel configuration, it is usually more economical to provide riprap, sills, or a stilling basin at the outlet end to control any excessive velocity.

Any outlet control or protective device which may be required is considered part of the hydraulic design of the culvert. It is normal for a properly designed culvert to have an outlet velocity which is greater than the natural stream velocity (see Section 9.5).

5. Develop a performance curve and check the 100-year flood conditions using the procedures outlined in Section 9.2.

An overall hydraulic performance analysis for the designed culvert will indicate headwater and outlet velocity characteristics for a wide range of discharges. Through analysis, consider applying potential headwater levels and outlet velocities with respect to their probabilities of occurrence to the designed culvert. Such application can lead to a determination of potential risks and costs associated with the long-term operation of the culvert.

For any culvert design, the minimum additional analysis required will be the application of the 100-year discharge to the culvert. The design is considered complete if the results of the headwater and outlet velocity represent an acceptable risk and conform with regulatory requirements.

A performance curve can be used for further evaluation of culvert risks. Culvert performance analysis consists of a definition of headwater and outlet velocity for each of a variety of discharges.

If a performance curve is desired, apply a range of discharges both lower and greater than the design discharge to make this determination. For example, if the culvert was designed for the 50-year discharge, apply the 5-, 10-, 25- and 100-year discharges with the corresponding tailwater for each discharge. With each of these discharges and tailwaters, determine the resulting headwater level and outlet velocity. A tabulation or plot of the resulting performance curves then can be made.

Life cycle costs generally are not needed to determine cost-effective designs. Culverts are expected to last without maintenance for the design life specified. However, the following long-term cost components (reduced to an annual cost on the basis of the anticipated service life) may be considered if a life cycle cost analysis is requested under an unusual circumstance.

- Initial cost of the culvert.
- Expected cost of damage to the roadway.
- Expected cost of damage to the culvert and associated appurtenances.
- Expected cost of damage to the stream (approach and exit).
- Expected cost of damage to upstream and downstream private or public property.
- Expected cost of traffic detours.
- Debris removal.

The cost of traffic detours may be the most important since the extension of detoured vehicles, detour distance, and cost of operation per vehicle mile can amount to extremely heavy expense. This is particularly true if there is a large average daily traffic rate.

9.4 IMPROVED INLETS

A. Conditions for Improved Inlets. If the culvert is operating under inlet control, it may be economical to apply an improved inlet. An improved inlet serves to "funnel" the flow into the culvert so that the point of control is removed from the face of the inlet to a throat located downstream from the face. The normal contraction of flow is included in the transition from the face to the throat of the inlet. If the culvert is operating under outlet control, side tapered and slope tapered improved inlets are not effective and should generally not be considered. However, beveled inlets can be useful in the circumstance of outlet control.

Several factors must be weighed in the use of improved inlets:

- Improved inlets offer the potential advantage of increasing the capacity of existing inadequate culverts. This is important where an inlet control culvert serves a watershed which has changed in character from rural to urban, or otherwise has experienced an increase in the design discharge rate.
- For inlet control, the improved inlet design process typically will allow a reduction in the culvert size. It is important to check that this reduction in culvert size does not cause the culvert to revert to outlet control.
- Improved inlets usually are costly; therefore, it is important to develop a complete economic analysis of such designs. The analysis should weigh the additional costs associated with the installation and maintenance of the improved inlet against the savings affected by reduced culvert size.
- Available design procedures require improved inlets to be placed with the face normal to the direction of flow. That is, the procedures do not accommodate improved inlets when the face of the inlet is skewed relative to the stream channel.
- Where heavy debris loads are anticipated, improved inlets can become serious maintenance problems. This is because of a tendency for some debris to pass the face of the inlet and become lodged in the throat.

The typical types of improved inlets are:

- Beveled edges.
- Depth transition.
- Side-tapered inlet.
- Slope-tapered inlet.

For design procedures for improved inlets, refer to HDS-5, *Hydraulic Design of Highway Culverts*, (FHWA, 2005b).

B. Beveled Edges. Beveled edges effectively reduce the contraction downstream of the culvert face, resulting in a more efficient conveyance of water by the available barrel area (see Figure 9.15). Generally, little or no enlargement of the culvert inlet is required to gain the hydraulic advantage of beveled edges; thus, structural problems are minor. Beveled edges may be implemented at little additional expense and are effective for culverts operating under either inlet or outlet control. The effect of beveled edges can be a significant improvement in culvert capacity and/or reductions in the subtended headwater. They may be easily adapted to either pipe or box culverts. Table 9.4 provides polynomial coefficients for some beveled entrance conditions for use in Equation 9.13 (inlet control headwater).

Figure 9.15 Beveled Entrance

C. Top-Tapered Transition. A simple transition of depth in a rectangular box culvert may improve the hydraulic efficiency. If the box culvert is operating under inlet control, it follows that the barrel of the culvert is more hydraulically efficient than the entrance geometry. The barrel depth may be reduced in the transition from the original depth of the inlet to a minimum of 0.3 m (1 ft) greater than the uniform depth of flow. The transition length should be a minimum of 6 m (20 ft) (see Figure 9.16).

Figure 9.16 Top-tapered Box Culvert

This method is arbitrary and should be used carefully only when the culvert is definitely operating in inlet control. In terms of design and construction, the method is effective, economical, and simple to perform.

This method may be preferred when designing a multiple barrel box culvert. Other inlet improvement methods are not feasible for multiple barrel box culverts because of the need to taper or flare the side walls of the barrels.

D. Side-Tapered Inlet. Side-tapered inlets involve a widening of the face area of the culvert by tapering the sidewalls. Such inlets have two possible control sections: the face and the throat. Control should be maintained at the throat for design discharge in order to realize significant cost savings in the culvert barrel. This type of improvement is similar in operation to the flared inlet for pipes discussed in the segments that follow.

E. Slope-Tapered Inlet. The slope-tapered inlet incorporates the efficient flow characteristics of side-tapered inlets with a concentration of more of the total available culvert fall at the throat control section. Slope-tapered improvements are not practical for pipe culverts because of their complexity and lack of availability.

Some of the drawbacks of slope-tapered inlets are described as follows:

- Slope-tapered inlets have a tendency to allow sediment deposition. This can result in maintenance problems.
- Slope-tapered inlets imply a reduced slope of the culvert. A reduced slope often leads to a change of hydraulic flow type in the culvert from supercritical to subcritical. In such cases, the application of the improved inlet may be ineffective.
- Because of the lowering of the upstream end of the culvert, the use of slope-tapered inlets can result in increased costs of structural excavation.

F. Flared Entrance Design for Circular Pipe. In certain instances, if a circular pipe culvert of sufficient barrel length is operating under inlet control, a flared entrance as an inlet improvement may serve to increase the hydraulic capacity with a corresponding savings in the initial cost of the culvert barrel. A sufficient barrel length would be such that the reduced cost of the smaller diameter barrel more than offsets the additional cost of the flared inlet or pipe liner.

A flared entrance for a pipe culvert is practical only when steep slope, inlet control conditions exist.

For any circular pipe culvert operating under inlet control, there is a possible use for a flared inlet to reduce the size of the barrel. The design procedure outlined in Section 9.3.D. is applicable. For the inlet control headwater, Table 9.5 provides coefficients for concrete and corrugated metal circular pipe with flared inlets for use in Equation 9.20. The following conditions should be noted:

- If the culvert is on a mild slope ($d_c < d_u$), a flared inlet is not likely to be effective.
- If the inlet analysis procedure indicates outlet control, a flared inlet is not an efficient application.
- Trial size is verified when all three of the following criteria are met:
  - $HW_{ic} < AHW$
  - $d_u < d_c$
  - $HW_{oc} < HW_{ic}$
- If the trial size is verified, compare costs with a culvert designed without a flared inlet. Calculate the culvert outlet velocity in accordance with the procedure outlined for an inlet control culvert.
- The flared inlet unit should not be cut to a skew even if the culvert is skewed with respect to the roadway.
- If the trial size is not verified, then simply design the culvert without a flared inlet in accordance with the usual procedure given in Section 9.3.D.

9.5 VELOCITY PROTECTION AND CONTROL DEVICES

A. General - Velocity Protection and Control Devices. While allowable headwater influences the overall configuration of the culvert, the allowable outlet velocity is the governing criterion in the selection and application of various downstream fixtures and appurtenances.

If the designer considers the outlet velocity to be excessive, several possible solutions which may minimize the negative effects of velocity are available. The excessive velocity may be accommodated, reduced, or controlled.

- Minor configuration changes in the culvert barrel may reduce an excessive velocity to a more acceptable exit velocity. For situations involving excessive outlet velocities in culverts operating under inlet control, it is possible to roughen the culvert or even change geometry of the culvert and yet not affect the headwater characteristics.

- Historically, the most widely used control has been the use of riprap which covers the channel area immediately downstream from the culvert outlet.
- An efficient, but usually expensive, countermeasure is an energy dissipator. Some energy dissipators have an analytical basis for design, while others are intended to cause turbulence in unpredictable ways. With turbulence in flow, energy is dissipated and velocity can be reduced.
- Certain special culvert types may be used successfully to minimize excessive outlet velocities. One such special culvert type is the broken-back culvert.

Velocity control appurtenances for culverts may be classified broadly as either protection devices or control devices.

B. Velocity Protection Devices. A velocity protection device does not necessarily reduce excessive velocity but does protect threatened features from damage. Such devices usually are economical and effective in that they serve to provide protection from the flow for specific sensitive features.

1. Riprap. Riprap, when used as an outlet velocity protection measure, should be applied to the channel area immediately downstream of the culvert outlet for some distance, possibly to the edge of right-of-way. This limit may be tempered by engineering judgment based on the severity of the velocity and the potential for erosion or scour. Rip-rap is rock of various sizes as specified in Section 850 of Publication 408, *Highway Specifications*.

2. Pre-formed Outlet. A very effective protection device consists of a pre-formed scour hole in the area threatened by excessive outlet velocities. These holes should be lined with some type of riprap.

3. Channel Recovery Reach. Similar to a pre-formed outlet, a channel recovery reach provides a means for the flow to return to an equilibrium state within the natural, unconstricted stream channel. The recovery reach should be well protected against the threat of scour or other damage.

C. Velocity Control Devices. A velocity control device serves to effectively reduce an excessive culvert velocity to an acceptable level. The design of some control devices is based on theoretical analysis while, for others, the specific control may be unpredictable. In increasing order of their probable expense, some velocity control devices are:

- Natural hydraulic jumps (most control devices are intended to force a hydraulic jump).
- Sills.
- Roughness baffles.
- Impact basins.
- Stilling basins.
- Specialized energy dissipators.

Most velocity control devices rely on the establishment of a hydraulic jump. Since an excessive outlet velocity from a culvert usually is the result of the culvert being on a relatively steep slope, the depth downstream of the culvert exit usually is not great enough to induce a hydraulic jump. However, some mechanisms may be available to provide a simulation of a greater depth necessary to create a natural hydraulic jump.

D. Other Control Devices. Other controls are described in the FHWA publication, HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006).

9.6 SPECIAL HYDRAULIC CONSIDERATIONS

In addition to the hydraulic considerations discussed in the preceding sections, other factors may be considered in order to assure the integrity of culvert installations and the highway.

A. Anchorage. The forces acting on a culvert inlet during high flows are variable and highly indeterminate. Vortices and eddy currents cause scour which can undermine the culvert inlet, erode the embankment slope and make the inlet vulnerable to failure. Flow is usually constricted at the inlet, and inlet damage (Figure 9.17) or lodged drift can accentuate this constriction. The large unequal pressures resulting from this constriction are, in effect, buoyant forces which can cause entrance failures, particularly on a corrugated metal pipe with mitered, skewed or projecting ends.

Scour for a particular drainage feature or constriction may be assessed using the guidance provided in BD-632M, BD-633M and RC-30M.

Figure 9.17 Damage to Culvert Inlets Due to Hydraulic Forces and Drift

Anchorage at the culvert entrance helps to protect against these failures by increasing the dead load on the end of the culvert, thus protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. End anchorage can be in the form of slope paving, concrete headwalls, or grouted stone, but the culvert end must be anchored to the end treatment to be effective. In some locations, prefabricated metal end sections also should be anchored to increase their resistance to failure.

Culvert ends need anchorage at many locations. Sectional rigid pipe is susceptible to separation at the joints when scour undermines the ends. Commercially available tiebars can be used to prevent separation of concrete pipe joints. Metal culvert ends projected into ponds, tidal waters or through levees are susceptible to failure from buoyant forces if tide gates are used or if the ends are damaged by debris. Figure 9.18 shows a culvert which failed from buoyant forces at the inlet end.

Figure 9.18 Culvert and Roadway Fill Failure from Buoyant Forces: Culvert Carried Downstream

B. Piping. Piping is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a void similar to a pipe, hence the term "piping" (Figure 9.19). Fine soil particles are washed out freely along the void, and the erosion inside the fill ultimately may cause failure of the culvert or the embankment. Piping also may occur through open joints into the culvert barrel.

The possibility of piping can be reduced by decreasing the velocity of the seepage or by decreasing the quantity of seepage flow. Methods of achieving these objectives are discussed in the following sections.

Figure 9.19 Void From Piping Along Culvert Barrel

C. Joints. In order to decrease the velocity of the seepage flow, it is necessary to increase the length of the flow path and thus decrease the hydraulic gradient. The most direct flow path for seepage and thus the highest hydraulic gradient is through open pipe joints. Therefore, it is important that culvert joints be as watertight as practical. If piping through joints could become a problem, flexible, long-lasting joints should be specified as opposed to mortar joints.

D. Anti-seep Collars. Piping should be anticipated along the entire length of the culvert when ponding above the culvert is planned. Anti-seep or cutoff collars increase the length of the flow path, while decreasing the hydraulic gradient and the velocity of flow, and thus the probability of pipe formation. Anti-seep collars usually consist of bulkhead type plates or blocks around the entire perimeter of the culvert. They may be of metal or of reinforced concrete, and, if practical, dimensions should be sufficient to key into impervious material. Figure 9.20 shows anti-seep collars installed on a culvert under construction.

Figure 9.20 Anti-seep Collars

E. Weep Holes. Weep holes sometimes are used to relieve uplift pressure. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and to prevent the formation of piping channels. The filter materials should be designed as underdrain filter so they will not become clogged and so piping cannot occur through the pervious material and the weep hole. Geotextile filter material should be placed over the weep hole in order to keep the pervious material from being carried into the culvert.

Weep holes may not be required in culverts, and their use is becoming less prevalent. For guidance on when to use weep holes, see Design Manual Part 4 and BD-632M. If additional drainage of the fill behind the culvert wall is believed necessary, a separate underdrain system may be installed.

F. Junctions and Bifurcations. It sometimes is necessary to combine the flow of two culverts into a single barrel. The junction should be designed so that a minimum amount of turbulence and adverse effect on each branch will result. This is accomplished by considering the flow momentum in each branch and other variables such as the timing of peak flows, e.g., low flow in one branch and high flow in the other. Supercritical flow velocities add to the complexity of the problem. Chow (1970) and Behlke and Pritchett (1966) address the subject of junctions for supercritical flow. In critical locations, laboratory verification of junction design is advisable.

If a bifurcation in flow is necessary or desirable, it is suggested that the flow division be accomplished outside the culvert barrel. Problems with clogging by debris and the desired proportioning of flow between branches can be handled much more easily outside of the culvert.

G. Training Walls. Where supercritical flow conditions prevail in a curved approach to a culvert, training walls may be needed to align flow with the culvert inlet and to equalize flow rates in the barrels of multiple barrel culverts. In locations where overtopping of the channel or culvert, or inefficient operation, could result in catastrophic failure, laboratory verification of the training wall design is advisable.

Training walls also may be required at culvert outlets to align flow with the downstream channel if this alignment cannot be accomplished in the culvert barrel. Design of the training wall shown in Figure 9.21 was verified by laboratory testing, and the wall has been proven by operation during floods.

Figure 9.21 Training Walls

H. Sag Culverts. A sag culvert, often called an inverted siphon, is not a siphon because the pressure in the barrel is not below atmospheric. Sag culverts of pipe or box section are used extensively to carry irrigation water under highways. They are used infrequently for highway drainage and should be avoided on intermittent or alluvial streams because of problems with siltation and stagnation.

Hydraulically, a sag culvert operates with outlet control, and losses through the culvert can be computed by the procedures used for conventional culverts.

I. Irregular Alignment. At some locations, it may be desirable to incorporate bends, either in plan or profile, in the culvert alignment. When irregular alignment is advisable or desirable, bends should be as gradual and as uniform as is practical to fit site conditions. Changes in alignment may be accomplished either by curves or angular bends. When large changes are necessary, mild bends, e.g., 15° at intervals of 15 m (50 ft), should be used. Passage of debris should be considered in selecting the angle, interval and number of bends used to accomplish the change in alignment.

If the culvert operates with inlet control, bend losses do not enter into the headwater computation. If it operates with outlet control, typically, bend losses will be small. In critical locations, they should be calculated and added to the usual losses. Bend loss coefficients can be calculated using Equation 9.36. More information on bend losses can be found in Bureau of Reclamation (1987) and AASHTO (2005) publications.

(Equation 9.36)

$$h_b = 0.0033(\Delta)\left(\frac{V_o^2}{2g}\right)$$

where: h_b = bend loss coefficient
Δ = angle of curvature, Degrees
V_o = initial velocity

J. Cavitation. The phenomenon known as cavitation occurs as a result of local velocity changes at surface irregularities which reduce the pressure to the vapor limit of the liquid. Tiny vapor bubbles form at the point of lowest pressure and are carried downstream into a zone of higher pressure where they collapse. As the countless bubbles collapse, extremely high local pressure is transmitted radially outward at the speed of sound, followed by a negative pressure wave which may lead to a repetition of the cycle. Boundary materials in the vicinity are subjected to rapidly repeated stress reversals and may fail through fatigue (Rouse, 1949). Surface pitting is the first sign of such a failure.

Cavitation is seldom a problem in highway culverts because of relatively low velocities and because flow rates are not sustained for a long period. Abrasion damage sometimes is mistaken for cavitation damage.

K. Tidal Effects and Flood Protection. Where areas draining through culverts are affected by tide or flood stages, flap gates may be desirable to prevent backflow. Sand, silt, debris or ice will cause these gates to require considerable maintenance to keep them operative. Head losses due to the operation of flap gates may be computed using loss coefficients furnished by the manufacturer.

9.7 MULTIPLE USE CULVERTS

Culverts often serve purposes in addition to drainage. There are cost advantages of multiple use culverts, but one purpose or the other is often served inadequately. The cost advantages of multiple use culverts should be weighed against the possible advantages of separate facilities for each use.

A. Utilities. It is sometimes convenient to locate utilities in culverts, particularly if jacking, boring or an open cut through an existing highway can be avoided by such a location. The space occupied in the culvert is usually relatively small, and the obvious effects on culvert hydraulic performance can be insignificant. Consideration of this multiple use, however, should include recognition of the flood flow and debris hazard to the utility and the probability of reduced culvert capacity from debris caught on the utility line. Also, increased stream scour often occurs at pipelines at the upstream and downstream ends of culverts. This multiple use generally is not suggested if separate facilities are practicable.

B. Stock and Wildlife Passage. Culverts can serve both for drainage and for stock and wildlife passes. Culvert size may be determined either by hydraulic requirements or by criteria established for the accommodation of the stock or game which will use the structure.

C. Land Access. Culverts often serve both as a means of land access and drainage, particularly on highways with controlled access. This use is common in areas where land use on both sides of the highway is under common control. The culvert size generally will be determined by the physical dimensions of the equipment or vehicles which will make use of the facility. Scour protection not considered necessary for hydraulic reasons may be required at the outlet to facilitate access to the culvert. Where a low flow culvert is placed at a lower elevation than the multiple use culvert, precautions against headcutting from the stream to the outlet of the multiple use culvert may be necessary. Good drainage at the culvert ends is necessary for culverts used for land access.

D. Fish Passage. Many resource agencies have established design criteria for fish passage through culverts. These include maximum allowable velocity, minimum water depth, maximum culvert length and gradient, type of structure, and construction scheduling.

Several types of culvert installations have been used satisfactorily for fish passage. These include:

- Open Bottom Culverts. Culverts supported on spread footings to permit retention of the natural stream bed. The culvert size must be adequate to maintain natural stream velocities at moderate flows, and the foundation must be in rock or scour resistant material (Figure 9.22).

Figure 9.22 Culvert on Spread Footings to Retain Streambed for Fish Passage

- Oversized or Countersunk Culverts. Oversized culverts with the bottom of the culvert placed below the stream bed so that gravel will deposit and develop a nearly natural stream bed within the culvert (Figure 9.23). Sometimes, baffles are necessary to hold gravel and rock in place, especially at stream grades > 4%.

Figure 9.23 Culvert Invert Placed Below Streambed
Baffles used to hold gravel in place and provide natural stream bed for fish passage

- Special Treatment. In wide, shallow streams, one barrel of a multiple-barrel culvert can be depressed to carry low flow, or weirs can be installed at the upstream end of some barrels to provide for fish passage through other barrels at low flow.

- Timing. When fish passage is required, consideration must be given to the time of the year that the culvert will be installed. Fishery agencies usually will provide dates when spawning will occur in order to limit stream disturbance during this period.

For additional information regarding the design of fish passages, refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.11.

9.8 DEBRIS CONTROL

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. The consequences may be damages from inundation of the road and upstream property.

The designer has three options for coping with the debris problem: retain the debris upstream of the culvert, attempt to pass debris through the culvert, or use a bridge (Reihsen & Harrison, 1971).

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. If debris is to be passed through the structure or retained at the inlet, a relief opening should be considered, either in the form of a vertical riser or a relief culvert placed higher in the embankment (Figure 9.24).

Figure 9.24 Vertical Riser for Relief

It often is more economical to construct debris control structures after problems develop since debris problems do not occur at all suspected locations.

A. Debris Control Structure Design. The design of a debris control structure must be preceded by a thorough study of the debris problem. Among the factors to be considered are:

- Type of debris.
- Quantity of debris.
- Expected changes in type and quantity of debris due to future land use.
- Streamflow velocity in the vicinity of the culvert entrance.
- Access requirements for maintenance.
- Availability of flow storage area and volume.
- Annual maintenance plan including debris removal.
- Assessment of damage due to debris clogging, if protection is not provided.

For more information on debris control structures, refer to HEC-9, *Debris Control Structures Evaluation and Countermeasures* (FHWA, 2005a).

B. Maintenance. Provisions for maintenance access are necessary for debris control structures. For high embankments, this may be difficult. If access to the debris control structure is not practical, a parking area for mechanical equipment, such as a crane, may be necessary in order to remove debris without disrupting traffic.

Many debris barriers require cleaning after every storm. The frequency of maintenance should be considered in selecting the debris control structure. If a low standard of maintenance is anticipated, the designer should choose to pass the debris through the structure.

9.9 SERVICE LIFE

Commonly used culvert materials are durable at most locations, but some soil and water environments are hostile. Service life must be a consideration in material selection and culvert design. Processes which affect the service life of culvert materials are corrosion, abrasion, and freeze-thaw cycles. Measures to increase service life are sometimes costly. The total expected annual cost should be considered when designs are prepared. Periodic culvert replacement may be a practical alternative. Driveway culverts, for instance, are generally easy to replace, and traffic service would not be a problem when replacement becomes necessary. Culverts under highways with high traffic volumes or under high fills are more difficult and costly to repair or replace. In such cases, more precaution against failure from a hostile environment is warranted.

Many of the conditions which affect service life can be evaluated and service life estimated prior to the selection of culvert material. The type and degree of protection needed can then be determined using Beaton & Stratfull (1962); Berg (1965); Braley (1951); FHWA (1991); Haviland et al., (1968); Lowe & Koeph (1964); Nordin & Stratfull, (1965); and, Peterson (1973). One of the most reliable methods available to the designer is to examine existing culverts in the same stream channel or in similar streams in the same area.

Additional information pertaining to service life of materials are provided in PennDOT's Design Manual publications.

A. Abrasion. Abrasion is the erosion of culvert materials by the bed load carried by streams (Figure 9.25). The principal factors to be considered are the frequency and duration of runoff events which transport significant amounts of abrasive materials, the character and volume of the bed load, and the resistance of the culvert material to abrasion. In some locations, culverts can be protected from abrasion by use of debris control structures to remove the abrasive sediment load from the flow.

Provision for abrasive wear can be made by the use of invert paving. The paving is to be reinforced and must be at least 5 cm (3 in) thick if it is to be counted on for increased durability.

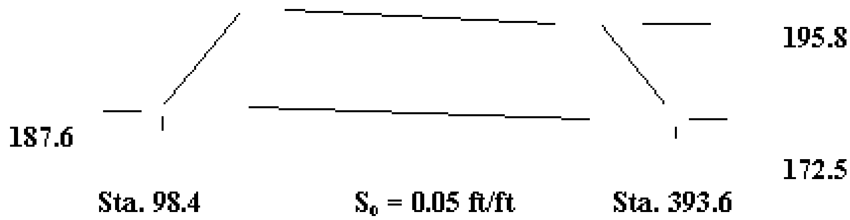
Figure 9.25 Loss of Culvert Material from Abrasion

9.10 EXAMPLE

The following example problem follows the Design Procedure Steps:

Step 1 Assemble Site Data and Project File

1. Site survey Project file contains:
- USGS, site and location maps.
- Roadway profile.
- Embankment cross section.



Site visit notes indicate:
- No sediment or debris problems.
- No nearby structures.

2. Studies by other agencies — none

3. Environmental, risk assessment shows:
- No buildings near floodplain.
- No sensitive floodplain values.
- No FEMA involvement.
- Convenient detours exist.

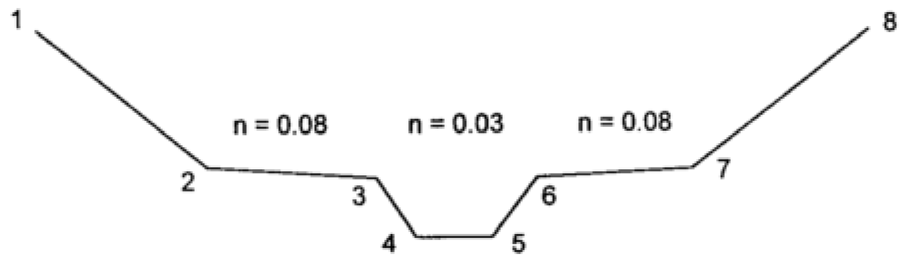
4. Design criteria:
- 50-year frequency for design.
- 100-year frequency for check.

Step 2 Determine Hydrology

1. USGS Regression equations yield:
- Q_{50} = 11.33 m^3/s (400 cfs)
- Q_{100} = 14.16 m^3/s (500 cfs)

Step 3 Design Downstream Channel

Cross section of channel (Slope = 0.05 ft/ft)



Point	Station, m (ft)	Elevation, m (ft)
1	3.7 (12.1)	54.86 (180.0)
2	6.7 (21.9)	53.34 (175.0)
3	9.8 (32.1)	53.19 (174.5)
4	10.4 (34.1)	52.58 (171.5)
5	11.9 (39.0)	52.58 (171.5)
6	12.5 (41.0)	53.19 (174.5)
7	15.5 (50.8)	53.34 (175.0)
8	18.6 (61.0)	54.86 (180.0)

The rating curve for the channel calculated by normal depth yields:

Q, m^3/s (cfs)	TW, m (ft)	V, m/s (ft/s)
2.83 (100)	0.43 (1.41)	3.39 (11.1)
5.66 (200)	0.63 (2.06)	4.18 (13.7)
8.50 (300)	0.76 (2.50)	4.87 (15.9)
11.33 (400)	0.85 (2.79)	5.34 (17.5)
14.16 (500)	0.93 (3.05)	5.73 (18.8)

Step 4 Summarize Data On Design Form

(See Figure 9.26)

Step 5 Select Design Alternative

Shape: box — Size: 2135 mm x 1830 mm (7ft x 6 ft)
Material: concrete — Entrance: beveled

Step 6 Select Design Discharge

$Q_d = Q_{50} = 11.33\ m^3/s$ (400 cfs)

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use inlet control nomograph - Chart 10

1. D = 1.83 m (6.0 ft)
2. Q/B = 11.33/2.13 = 5.32 m^2/s (400 / 7 = 57.1 ft^2/s)
3. HW/D = 1.33 for 19 mm (0.75 in) chamfer
HW/D = 1.27 for 45 bevel
4. HW_i = (HW/D)D = (1.27) 1.83 = 2.32 m ((1.67) 6.0 = 7.62 ft) (Neglect the approach velocity.)

Step 8 Determine Outlet Control Headwater Depth at Inlet (HW_{oi})

1. TW = 0.85 m (2.79 ft) for Q_{50} = 11.33 m^3/s (400 cfs)
2. d_c = 1.43 m (4.70 ft) from Chart 14
3. (d_c + D)/2 = (1.43m + 1.83m)/2 = 1.63 m, ((4.70 + 6.0)/2 =5.35 ft)
4. h_o = the larger of TW or (d_c + D/2)
   h_o = (d_c + D)/2 = 1.63 m (5.35 ft)
5. K_E = 0.2 from Table 2
6. Determine (H) - use Chart 15
   - K_E scale = 0.2
   - culvert length (L) = 90 m (295 ft)
     n = 0.012 same as on chart
   - area = 3.90 m^2 (41.95 ft^2)
   - H = 0.85 m (2.79 ft)
7. HW_{oi} = H + h_o - S_oL = 0.85 + 1.63 - (0.05) 90 = -2.02 m (2.79 + 5.35 - (0.05) 295 = -6.61 ft)
   HW_{oi} is less than 1.2D, but control is inlet control. Outlet control computations are for comparison only.

Step 9 Determine Controlling Headwater (HW_c)

- HW_c = HW_i = 2.32 m (7.60 ft) > HW_{oi} = -2.02 m (-6.61 ft)
- The culvert is in inlet control.

Step 10 Compute Discharge over the Roadway (Q_r)

1. Calculate depth above the roadway:
   HW_r = HW_c - HW_{ov} = 7.61 - 8.50 = -0.27 m (-0.89 ft)
2. If HW_r = 0, Q_r = 0

Step 11 Compute Total Discharge (Q_t)

$Q_t = Q_d + Q_r$ = 11.33 m^3/s (400 cfs) + 0 = 11.33 m^3/s (400 cfs)

Step 12 Calculate Outlet Velocity (V_o) and Depth (d_n)

INLET CONTROL

1. Calculate normal depth (d_n):

$$Q = (1.00/n)\, A\, R^{2/3}\, S^{1/2} = 11.33\ m^3/s\ (400\ cfs) \quad (1)$$
$$= (1.00/0.012)(7.0\times d_n)[7.0\times d_n/(7.0+2d_n)]^{2/3}(0.05)^{0.5} = 11.33 m^3/s\ (400\ cfs) \quad (2)$$

After rearranging Equation (2) above,

$(7.0\times d_n)[7.0\times d_n/(7.0+2d_n)]^{2/3}$ = .608 m (14.40 ft)
Try d_n = .60 m (1.96 ft), .675 (15.98) > .608 m (14.40 ft)
Use d_n = .55 m (1.80 ft), .596 (14.2) ≈ .608 m (14.40 ft)

2. A = (2.13m)(0.55m), (7ft)(1.80ft) = 1.17 m^2 (12.6 ft^2)

3. V_o= Q/A = 11.33m^3/s /1.17m^2 = 9.68 m/s, (400/12.6 = 31.75 ft/s)

Step 13 Review Results

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

- Barrel has 2.59-1.83 = 0.76 m, (8.5-6.0 = 2.5 ft) of cover.

- L = 90 m (295 ft) is OK, since inlet control.
- Headwalls and wingwalls fit site.
- Allowable headwater (2.59 m), (8.50 ft) > 2.32 m (7.60 ft) is OK.
- Overtopping flood frequency > 50-year.

Step 14 Plot Performance Curve

Use Q_{100} for the upper limit. Steps 6 through 12 should be repeated for each discharge used to plot the performance curve. These computations are provided on the computation form, (see Figure 9.17).

- No flow routing, a small upstream headwater pool exists.
- Consider energy dissipators since V_o = 9.68 m/s (31.75 ft/s) > 5.34 m/s (17.5 ft/s) in the downstream channel.
- No sediment problem.
- No fishery.

Step 15 Documentation

Report prepared and background filed.

Figure 9.26 Chart 17 and Performance Curve for Design Example (Metric)

PROJECT: EXAMPLE PROBLEM
AASHTO Chapter 9

STATION: TEST 0 + 00
SHEET 1 1

CULVERT DESIGN FORM
DESIGNER / DATE: MJM **OF** 7/24/06
REVIEWER / DATE: ______ **OF** ______

HYDROLOGICAL DATA

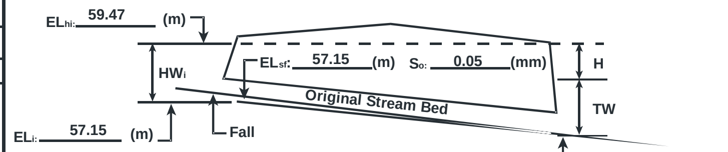
SEE ADDITIONAL SHEETS

- ■ METHOD: USGS
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE ______
- ☐ CHANNEL SHAPE: ______
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOW & TAILWATER

R.I.(YEARS)	FLOW (m^3/s)	TW (m)
50	11.33	0.85
100	14.16	0.93

ROADWAY ELEVATION: 59.74 (m)



$S = S_o$ - Fall/L_o
S = 0.05 (m/m)
L_o = 90 (m)
EL_o: 52.58 (m)

CULVERT DESCRIPTION MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (m^3/s)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HW	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
	11.33	5.32	1.27	2.32	0	59.47	0.85	1.43	1.63	1.63	0.2	0.85	55.06	59.47	9.68	59.49 ≤ 59.47 ok
	14.16	6.65	1.56	2.87	0	60.02	0.93	1.65	1.74	1.74	0.2	1.31	55.63	60.02	10.36	60.02 > 59.74 calc=Q_r
Performance Curve	2.83	1.33	0.46	0.85	0	58.00										
	5.66	2.66	0.78	1.43	0	58.58										
	8.50	3.99	1.01	1.85	0	59.00										

TECHNICAL FOOTNOTES

(1) USE Q/NB FOR BOX CULVERTS
(2) HW_i/D = HW/D OR HW_s/D FROM DESIGN CHARTS
(3) FALL = HW_i - (EL_{hd} - EL_{sf}): FALL IS ZERO FOR CULVERTS ON GRADE
(4) EL = HW_i + EL_i (INVERT OF INLET CONTROL SECTION)
(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL
(6) h_o = TW or (d_c + D)/2 (WHICHEVER IS GREATER)
(7) H = [1+k_e + (19.63 n^2L)/$R^{1.33}$] $V^2/2g$
(8) EL_{ho} = EL_o + H + h_o

SUBSCRIPT DEFINITIONS
- a. Approximate
- f. Culvert Face
- hd. Design Headwater
- hi. Headwater in Inlet Control
- ho. Headwater In Outlet Control
- i. Inlet Control Secton
- o. Outlet
- sf. Streamed at Culvert Face
- tw. Tailwater

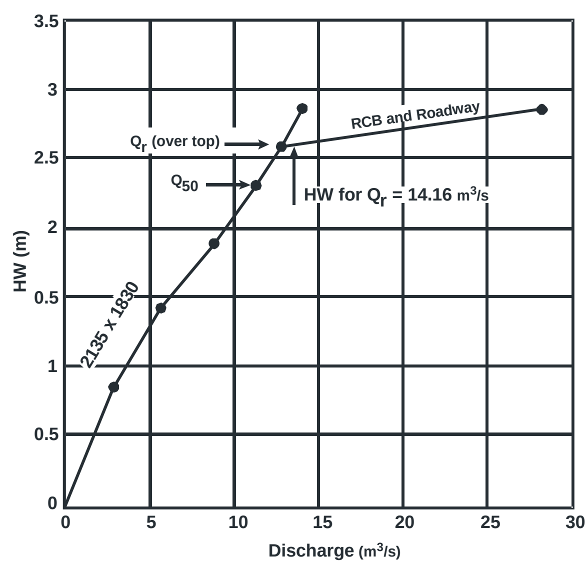
COMMENTS / DISCUSSION:

Use 14.16 (m^3/s) in culvert

$Q_p = C_d L HW_r^{1.5} = 1.67(60)(0.27)^{1.5} = 14.06\ m^3/s$

$Q_T = 14.16 + 14.06 = 28.22\ m^3/s$

CULVERT BARREL SELECTED
SIZE 2135 x 1830 (mm x mm)
SHAPE RCB
MATERIAL Concrete n 0.012
ENTRANCE Beveled



9.11 CHAPTER 9 NOMENCLATURE

Symbol	Definition	Units
A	Area of cross section of flow	m^2 (ft^2)
A_c	Cross sectional area of flow at critical discharge	m^2 (ft^2)
AHW	Allowable HW	m (ft)
B	Barrel width	m (ft)
D	Culvert diameter or barrel height	m (ft)
d	Depth of flow	m (ft)
d_m	Hydraulic depth	m (ft)
d_c	Critical depth of flow	m (ft)
d_n	Normal Depth	m (ft)
d_u	Uniform Depth	m (ft)
g	Acceleration due to gravity	m/s^2 (ft/s^2)
H	Sum of $H_E + H_f + H_o$	m (ft)
H_b	Bend headloss	m (ft)
H_E	Entrance headloss	m (ft)
H_f	Friction headloss	m (ft)
H_L	Total energy losses	m (ft)
H_o	Outlet or exit headloss	m (ft)
h_v	Velocity head	m (ft)
h_o	Hydraulic grade line height above outlet invert	m (ft)
HW	Headwater depth (subscript indicates section)	m (ft)
K_E	Entrance loss coefficient	dimensionless
L	Length of culvert	m (ft)
n	Manning's roughness coefficient	dimensionless
P	Wetted perimeter	m (ft)
Q	Rate of discharge	m^3/s (cfs)
Q_c	Critical discharge	m^3/s (cfs)
q	Discharge per unit width	$m^3/s/m$ (cfs/ft)
R	Hydraulic radius (A/P)	m (ft)
S	Slope of culvert	m/m (ft/ft)
S_f	Friction Slope of culvert	m/m (ft/ft)
S_o	Slope of culvert	m/m (ft/ft)
TW	Tailwater depth above invert of culvert	m (ft)
V	Mean velocity of flow with barrel full	m/s (ft/s)
V_d	Mean velocity in downstream channel	m/s (ft/s)
V_o	Mean velocity of flow at culvert outlet	m/s (ft/s)
V_u	Mean velocity in upstream channel	m/s (ft/s)
γ	Unit weight of water	N/m^3 (lb/ft^3)
τ	Tractive force	N/m^2 (lb/ft^2)

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