9 Culverts

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9 Culverts

9.1 INTRODUCTION

9.1.1 Definition

A culvert is any structure not classified as a bridge, that provides an opening under a roadway, or other types of access or utility.

A culvert is defined as the following:

- A structure that is usually hydraulically designed to take advantage of submergence to increase hydraulic capacity;
- A structure used to convey surface runoff through embankments;
- A structure, as distinguished from bridges, usually covered with embankment and composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert; and
- A structure that is 20 feet or less in centerline span-width between extreme ends of openings for multiple boxes. However, a structure hydraulically designed as a culvert is treated in this chapter, regardless of its span.

The following discusses some of the basic concepts and definitions commonly used in hydraulic design and installation of culverts.

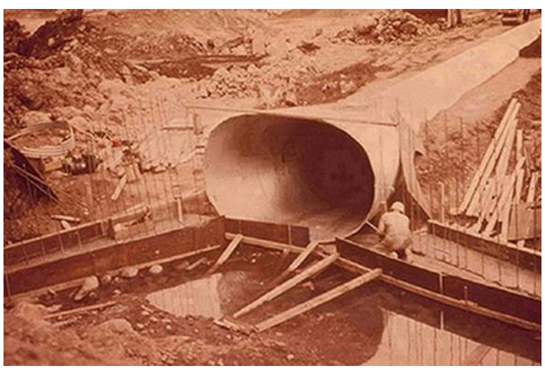


Photo 9.1 Improved inlet during construction on Interstate Highway 70

Photo 9.2 Same improved inlet after construction on Interstate Highway 70

9.1.2 Purpose

A culvert is used primarily to convey water through embankments, or other types of flow obstructions. It can also be used as a passage for pedestrians, stock, wildlife, and fish, as well as for land access and to carry utilities. This chapter focuses on drainage applications of culverts.

Photo 9.3 Culvert outlet

Photo 9.4 Culvert outlet

9.1.3 Concepts and Definitions

The following are discussions of important concepts in culvert design:

Backfill - material used to refill the trench after the pipe and embedment have been placed.

Barrel Roughness - a function of the material used to fabricate the barrel. Typical materials include concrete, plastic, and corrugated metal.

Barrel Area - the cross sectional area perpendicular to the flow.

Barrel Length - the total culvert length, from entrance to exit of the culvert.

Bedding - the material placed at the bottom of a trench on which pipe is laid.

Bottom of Pipe - the point along the pipe vertical axis which is a wall thickness below the invert.

Control Section - the location where there is a unique relationship between flow rate and upstream water-surface elevation.

Cover - the depth of backfill over the top of a pipe. Refer to the CDOT M&S Standards for minimum- and maximum-cover requirements.

Critical Flow - the state of flow where specific energy is minimum for a given discharge. Also, it is the state of flow where the velocity head is equal to one half the hydraulic depth, or where the ratio of inertial forces to gravity forces is equal to unity (Froude number = 1).

Critical Depth - the depth at critical flow. For a given discharge and cross-section geometry, there is only one critical depth. Charts of critical depth for circular pipes and box culverts are included in Appendix A. For other shapes, refer to *Hydraulic Design of Highway Culverts*, FHWA Hydraulic Design Series No. 5.

Critical Slope - a slope that sustains a given discharge at a uniform and critical depth.

Crown - the inside top of the culvert.

Embedment - pipe embedment comprises the soil that is placed under and around the pipe immediately above the bedding, to support the load on the pipe. It includes the haunch fill, the shoulder fill, and the initial cover.

Energy Grade Line - represents the total energy at any point along the culvert barrel. The total energy at any section is the sum of flow depth, velocity head ($V^2/2g$), and all energy losses.

Flexible Pipe - a structure that transmits the load on the pipe to the soil at the sides of the pipe. Examples of flexible pipes are plastic, and thin walled metal pipes.

Flowline - a line running longitudinally with the channel, connecting the lowest points in a series of channel cross sections.

Flow Type - the United States Geological Survey (USGS) has established seven culvert flow types, which assist in determining the flow conditions at a particular culvert site.

Foundation - the in-place or borrow material beneath the bottom of pipe, or layer of bedding material. The foundation material should be removed and replaced if unsuitable.

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

Haunches - the haunches of a pipe are the outside areas between the springline and the bottom of pipe.

Headwater - the depth of the upstream water surface, measured from the flowline at the culvert entrance.

Hydraulic Grade Line - the hydraulic grade line represents the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel.

Improved Inlet - an improved inlet has an entrance geometry which decreases the flow constriction at the inlet, and increases the capacity of culverts flowing under inlet-control conditions.

Invert - the inside bottom of the culvert.

Normal Flow - normal flow occurs in a channel reach when the discharge, velocity, and depth of flow do not change throughout the reach. The water-surface profile and channel-bottom slope will be parallel. This type of flow can exist in a culvert operating on a steep slope, provided the culvert is sufficiently long.

Normal Depth - the depth of water at a steady, uniform, constant velocity and flow at a given channel reach.

Rigid Pipe - a structure that transmits the backfill load on the pipe through the pipe walls to the foundation beneath the pipe. An example of a rigid pipe is a reinforced concrete pipe.

Round Pipe and Corrugation Terminologies - Figure 9.1 shows the terminology commonly used in describing round pipe. Figure 9.2 depicts a profile of a corrugated pipe (3 in by 1 in corrugation), with terminology and sample dimensions.

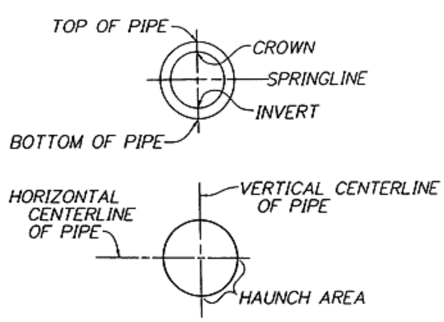


Figure 9.1 Terminology used in describing round pipes

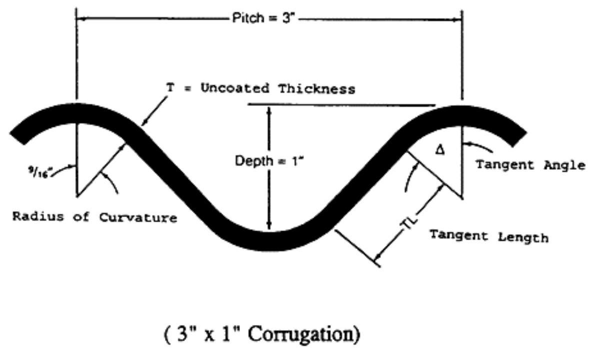


Figure 9.2 Profile of a corrugated pipe

Slope Types - steep slope occurs where the critical depth is greater than the normal depth. Mild slope occurs where critical depth is less than normal depth.

Springline - the horizontal line at the midpoint of the vertical axis of the pipe.

Subcritical Slope - a slope less than the critical slope, which causes a slower flow of the subcritical state for a given discharge.

Submerged Condition - a submerged outlet occurs where the tailwater elevation is higher than the crown of the culvert. A submerged inlet occurs where the headwater is greater than 1.2 times the culvert diameter, or barrel height.

Supercritical Slope - a slope greater than the critical slope, which causes a faster flow of the supercritical state for a given discharge.

Tailwater - the depth of water downstream of the culvert, measured from the outlet flowline. Backwater calculations from a downstream control, a normal-depth approximation, or field observations, are used to define the tailwater elevation.

Top of Pipe - the point along the pipe vertical axis which is a wall thickness above the crown.

Trench - a cut or an excavation made in the ground for the placement of a culvert and required bedding, embedment, backfill, and cover materials.

Trench Terminology - Figure 9.3 shows the different terminologies commonly used in trenches.

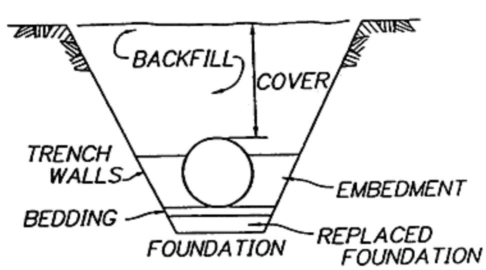


Figure 9.3 Trench terminology

9.1.4 Symbols

To provide consistency within this Chapter and throughout this manual, the symbols given in Table 9.1 will be used. These symbols were selected because of their wide use in culvert publications.

Table 9.1 Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	in or ft
B_f	Width of face section of a tapered inlet	ft
C_d	Coefficient of discharge for flow over an embankment	-
C_r	Coefficient of discharge	-
D	Culvert diameter or barrel height	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_n	Normal depth	ft
g	Acceleration due to gravity	ft/s^2
H	Headloss, sum of $H_e + H_f + H_o$	ft
H_b	Bend headloss	ft
H_e	Entrance headloss	ft
H_f	Friction headloss	ft
H_j	Headloss at junction	ft
H_g	Headloss at grate	ft
H_L	Total energy losses	ft
H_o	Outlet or exit headloss	ft
H_v	Velocity head	ft
h_o	Hydraulic grade line height above outlet invert	ft
HW	Headwater depth (subscript f = face, t = throat)	ft
HW_a	Headwater allowable	ft
HW_i	Headwater depth above inlet invert	ft
HW_o	Headwater depth above the outlet invert	ft
HW_{oi}	Outlet control headwater	ft
HW_{ov}	Height of road above inlet invert	ft
HW_r	Upstream depth, measured above the roadway crest	ft
k_e	Entrance loss coefficient	-
k_t	Submergence coefficient	-
L	Length; Length of roadway crest	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q_0	Overtopping flow rate	ft^3/s
S_o	Slope of streambed	ft/ft
TW	Tailwater depth above outlet invert of culvert	ft
V	Mean velocity of flow with barrel full	ft/s
V_d	Mean velocity in downstream channel	ft/s
V_o	Mean velocity of flow at culvert outlet	ft/s
V_u	Mean velocity in upstream channel	ft/s
γ	Unit weight of water	lb/ft^3

9.1.5 Classification

Culverts can be classified according to their:

- Geometry;
- Construction material; and
- Type of flow control.

In the following sections, each classification and factors used in the classification are discussed.

9.1.6 Geometry

The geometric factors used in culvert classification are barrel shapes and inlet types.

Barrel Shapes

Numerous cross sectional shapes are available. The most commonly used shapes include circular, rectangular, elliptical, pipe arch, and arch. Shape selection is based on the cost of construction, the limitation on upstream water-surface elevation, roadway embankment height, and hydraulic performance.

Inlet Types

A number of different inlet configurations are utilized on culvert barrels. These include both prefabricated and constructed in place installations. Commonly used inlet configurations include: projecting culvert barrels; cast in place concrete headwalls; precast or prefabricated end sections; and, culvert ends mitered to conform to the fill slope. Drawings of these inlet configurations are shown in Figure 9.4. Structural stability, aesthetics, erosion control, and fill retention are considerations in the selection of various inlet configurations.

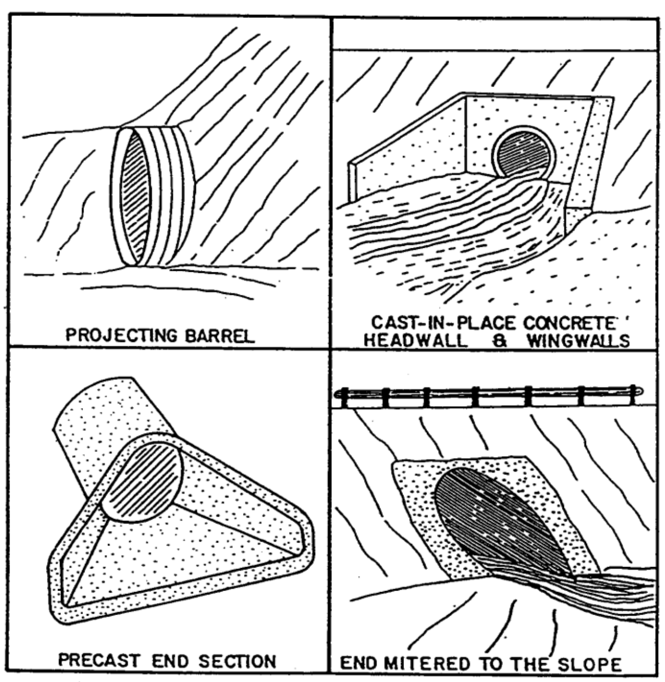


Figure 9.4 Four Standard Inlet Types (Schematic)

The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the natural channel is usually 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, therefore, are more efficient than square edges. Side tapered and slope tapered inlets, commonly referred to as improved inlets, further reduce the flow-contraction losses.

Depressed inlets, such as slope tapered inlets, increase the effective head on the flow-control section, further increasing the efficiency of the culvert. To design tapered inlets, refer to *Hydraulic Design of Highway Culverts*, FHWA Hydraulic Design Series No. 5 (HDS 5).

9.1.7 Culvert Construction Materials

The most common culvert materials are:

- Concrete (reinforced and non-reinforced);
- Aluminum;
- Steel; and
- Thermoplastic (PVC, and high-density polyethylene, HDPE).

Culverts also may be lined with other materials to inhibit corrosion and abrasion, or to reduce hydraulic roughness. Refer to Section 603 - Culverts and Sewers, Section 616 - Siphons, Section 617 - Culvert Pipe, and Section 624 - Corrosion-Resistant Culverts, of the *CDOT Standard Specifications for Road and Bridge Construction* for various construction materials acceptable to CDOT.

Different materials used in constructing culverts result in different structural and performance properties. These properties include:

- Durability;
- Structural strength;
- Hydraulic roughness;
- Embedment conditions;
- Abrasion and corrosion resistance;
- Watertightness requirements.

Durability

Durability (service life) is defined by the number of years a pipe lasts until it becomes structurally or functionally unfit for the intended purpose. The estimated minimum-service life under normal conditions for all types of culvert pipes listed in the *CDOT Standard Specifications for Road and Bridge Construction* must be 50 years. This service life covers pipes used for highway drainage systems, including cross culverts, siphons, and side drains.

Structural Strength

Structural design of the culvert barrel must provide adequate strength to resist the moments, thrusts, and shears determined through structural analysis. The prism load (dead load), dynamic load (traffic load), type of pipe material and pavements (flexible or rigid), and the properties of in situ soil, backfill, embedment, bedding, and foundation materials should be determined before an adequate structural analysis is performed. The designer should refer to the *CDOT M & S Standards* for strength requirements of structures with standard sizes. The Bridge Engineer should be consulted if the proposed structure is not standard, or if, for any reason, it requires special design. The structural requirements specified by the Bridge Engineer, and those found in the *CDOT M & S Standards* should apply during and after construction.

Hydraulic Roughness

Hydraulic roughness represents the hydraulic resistance to flow by culverts. Manning's equation is commonly used to calculate barrel-friction losses in culvert design. The hydraulic resistance coefficients for corrugated-metal conduits are based on the size and shape of the corrugations, spacing of the corrugations, type of joints, bolt or rivet roughness, method of manufacture, size of conduit, flow velocity, and aging. For concrete pipe, the hydraulic resistance varies with the method of manufacture, field installation, quality of joints, and aging. For concrete box culverts, the hydraulic resistance is based on the method of manufacture, quality of the formwork, installation or construction practices, and aging. Typical ranges of recommended Manning's *n* values are given HDS 5.

Table 9.2 Recommended Manning's *n* Roughness Values

Type of Conduit	**Wall Description**	**Manning's *n***
Concrete Pipe	Smooth walls	0.010 - 0.013
Concrete Boxes	Smooth walls	0.012 - 0.015
Corrugated Metal Pipes and Boxes	2.67 by 0.5 in corrugations	0.022 - 0.027
Annular or Helical Pipe (*n* varies with barrel size)	6 by 1 in corrugations 5 by 1 in corrugations 3 by 1 in corrugations 6 by 2 in structural plate 9 by 2.5 in minimum structural plate	0.025 - 0.026 0.025 - 0.026 0.027 - 0.028 0.033 - 0.035 0.033 - 0.037
Spiral Rib Metal	Smooth walls	0.012 - 0.024

Notes:

1. The values indicated in this table are recommended design values for Manning's *n*. Actual field values for older existing pipelines may vary depending on the effects of abrasion, corrosion, deflection and joint conditions. Concrete pipe with poor joints and deteriorated walls may have n values of 0.014 to 0.018. Corrugated metal pipe with joint and wall problems may also have higher n values, and in addition, may experience shape changes which could adversely affect the general hydraulic characteristics of the culvert.
2. For further information concerning Manning's n values for selected conduits, consult FHWA HDS No. 5.

Embedment Conditions

For rigid pipe, the embedment distributes the load over the foundation. For flexible pipe, the embedment resists the deflection of the pipe due to load. The flat surface makes compaction difficult at the very bottom of large structures. Trenches should be wide enough to permit compacting the remainder of the embedment under the haunches of the structure. Refer to *CDOT M&S Standards* for limits for structure excavation backfill, and fill-height requirements (allowable minimum and maximum cover).

Abrasion and Corrosion Resistance

Abrasion is the erosion of culvert material, primarily due to the natural movement of bedload in the stream. Effects of abrasion on the life of culverts in Colorado are poorly documented. In the past, only a few of the Department's culverts required repair as a result of abrasion damage. This changed dramatically in recent years, due to aging of existing installations. When abrasion problems are expected, several options are available to designers:

- Debris-control structures, although requiring periodic maintenance, can be used to reduce or eliminate flow of abrasive material into culverts.
- A liner or bottom reinforcement utilizing additional abrasion-resistant material is another option.
- Concrete or bituminous lining of the invert of corrugated metal pipes is a commonly-employed method to minimize effects from abrasion.
- Concrete culverts may require additional cover over the reinforcement bars, or high-strength concrete mixes to resist abrasion.
- 2 to 6 inches of high-strength concrete, placed over the reinforcing steel, may be used to minimize abrasion of pipes. Using Ultra High Performance Concrete (UHPC) or higher strength concrete for culvert floors might also be a consideration.
- The use of metal or wooden planks attached to the culvert bottom, normal to the flow, will trap and hold bedload materials, thereby providing invert protection.
- Oversized culvert barrels which are partially buried accomplish the same purpose.

Methods of predicting abrasion performance of metal pipe may be available from the manufacturer. If preliminary evaluation indicates that deterioration of pipe by abrasion is likely, the designer may consult with the manufacturer to determine the required pipe thickness.

All available culvert materials are subject to deterioration when placed in certain corrosive environments. The required level of corrosion resistance (C.R.) is determined by the Region and Staff Materials Engineers, using site soil and water tests, and by visual observations of culverts in the area. The C.R. level must be specified in the project plans. The contractor will then be allowed to select alternate culvert materials as specified in Section 624 - Corrosion Resistant Culverts, of the *Standard Specifications for Road and Bridge Construction.*

Watertightness Requirements

Watertightness pertains to the tightness of fit of installed pipes in order to be impermeable to water. Piping caused by seepage along a culvert removes fill material and forms a hollow similar to a pipe. Fine soil particles are freely washed out along this hollow, and erosion inside the fill may ultimately cause failure of the culvert or embankment. Piping may also occur through open joints in the culvert barrel. It is important that culvert joints be as watertight as practical. The designer must ensure that the proposed pipe system is sufficiently watertight to accommodate the hydrostatic pressure resulting from the design headwater. Headwalls, impervious materials at the upstream end of the culvert, and anti-seep or cutoff collars decrease the probability of piping. Anti-seep collars usually consist of bulkhead-type plates or blocks around the entire perimeter of a culvert. They may be of metal or of reinforced concrete and, if practical, their dimensions should be sufficient to key into impervious material.

9.1.8 Types of Flow Control

Culverts are classified into two basic groups, based on the flow-control type: inlet-control culverts, and outlet-control culverts. The basis for this classification is the location of the control section. The hydraulic capacity of a culvert depends on a different combination of factors for each type of control. An accurate theoretical analysis of culvert flow is extremely complex and requires the following:

- Analysis of non-uniform flow, with regions of both gradually-varying and rapidly-varying flow;
- Determination of how flow type changes as flow rate and tailwater elevations change;
- Application of backwater and drawdown calculations, energy and momentum balance;
- Application of the results of hydraulic-model studies; and
- Determination if hydraulic jump occurs, and its location.

Inlet Control

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. 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 region immediately downstream is supercritical. Figure 9.5 shows a typical inlet-control flow condition. 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. Inlet geometry includes the barrel shape, cross sectional area, and the inlet edge. The majority of existing culverts in Colorado highways operate under inlet control.

Three regions of flow for inlet control are shown in Figure 9.6. They are: 1) unsubmerged, 2) transition, and 3) submerged. The equations used to develop the graphical solutions, and to define inlet-control conditions are presented in FHWA publication HDS No. 5.

For an unsubmerged region of flow, headwater elevation is below the inlet crown, and the entrance operates as a weir. A weir is a flow-control section where the upstream water-surface elevation can be predicted for a given flow rate. The relationship between flow and water-surface elevation must be determined by model tests of the weir geometry, or by measuring prototype discharges. These tests are then used to develop equations. Appendix A of FHWA HDS No. 5 contains the equations which were developed from model-test data.

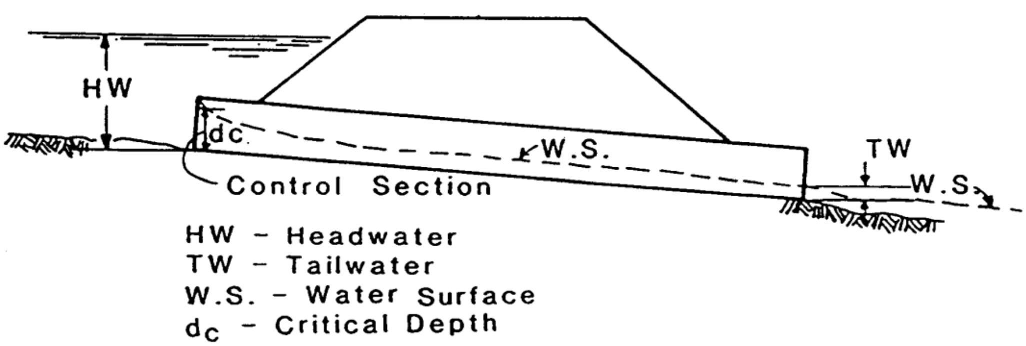


Figure 9.5 Typical inlet control flow condition

For a submerged region of flow, headwater elevation is above the inlet, and the culvert operates as an orifice. An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section. The relationship between flow and headwater can be defined based on results from model tests.

The transition zone is located between the unsubmerged and submerged flow conditions, where the flow is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations, and connecting them with a line tangent to both curves.

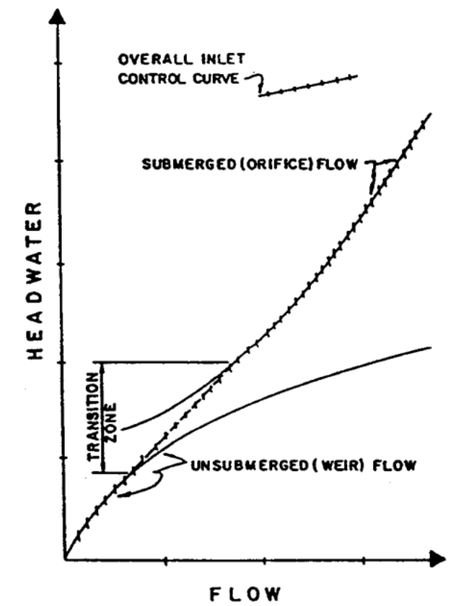


Figure 9.6 Unsubmerged, transition and submerged regions of flow.

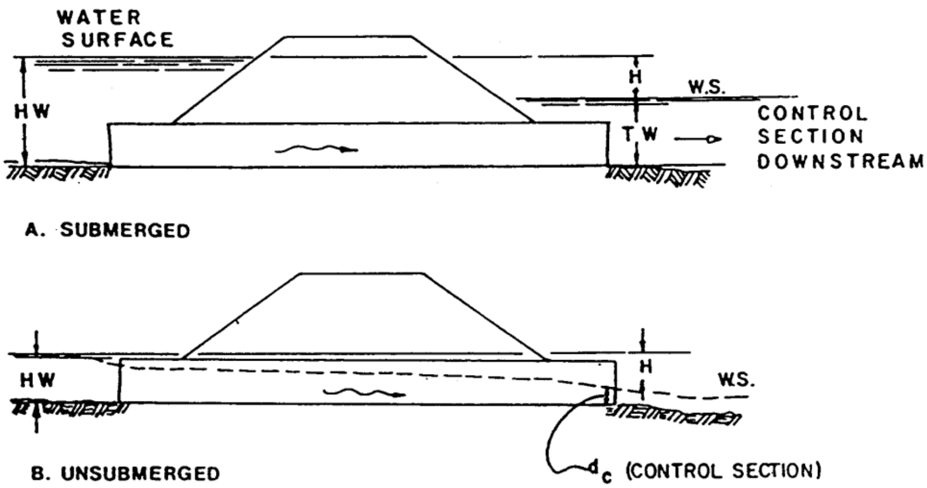
Outlet Control

Outlet-control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet-control flow in a culvert is located at the barrel exit, or further downstream. The culvert may outfall into a pond, lake, gulch, creek, river, or other drainageways. In an outlet-control condition, the water-surface elevations or tailwater on these waterways are high enough to cause backwater upstream of the culvert inlet, and therefore control the flow.

In culvert barrels operating under outlet control, either subcritical or pressure flow exists. Figure 9.7 shows typical outlet-control flow conditions. Under outlet-control flow conditions, all geometric and hydraulic characteristics of the culvert affect its capacity. These characteristics include all factors governing inlet control, water-surface elevation at the outlet (tailwater), slope, length, and hydraulic roughness of the culvert barrel. Graphical representations defining flow in culverts operating in outlet control conditions are presented in FHWA HDS No. 5.

9.1.9 Roadway Overtopping

Roadway overtopping begins when headwater rises to the elevation of the roadway. Overtopping usually occurs at the low point of a sag vertical curve on the roadway.



NOTES:

HW - HEADWATER
TW - TAILWATER
W.S. - WATER SURFACE
H - LOSSES

Figure 9.7 Typical outlet control flow conditions

Various equations

The following equation is used to define the overtopping flow across the roadway. The equation is the same relationship used to define the flow over a broad-crested weir. Flow coefficients for flow overtopping roadway embankments are given in Figure 9.8.

$$Q_r = C_d\, L\, (HW_r)^{1.5} \tag{9.1}$$

Where Q_r = overtopping flow rate, cfs; C_d = overtopping discharge coefficient $(k_t\ C_r)$; k_t = submergence coefficient; C_r = discharge coefficient; L = length of roadway crest, ft; HW_r = upstream depth, measured above the roadway crest, ft.

Crest length is difficult to determine when the crest is defined by a roadway sag vertical curve. It is recommended to subdivide the length into a series of segments. Calculate the flow over each segment for a given headwater. The flows for each segment are then added together to determine the total flow. The length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth of the upstream pool above the roadway.

Total flow is calculated for a given upstream water-surface elevation using the above overtopping equation. The roadway overflow plus culvert flow must equal the total design flow. A trial-and-error process is necessary to determine flow passing through the culvert, and the amount flowing across the roadway. Performance curves for culvert and road overflow may be summed to yield an overall performance (see Section 9.5).

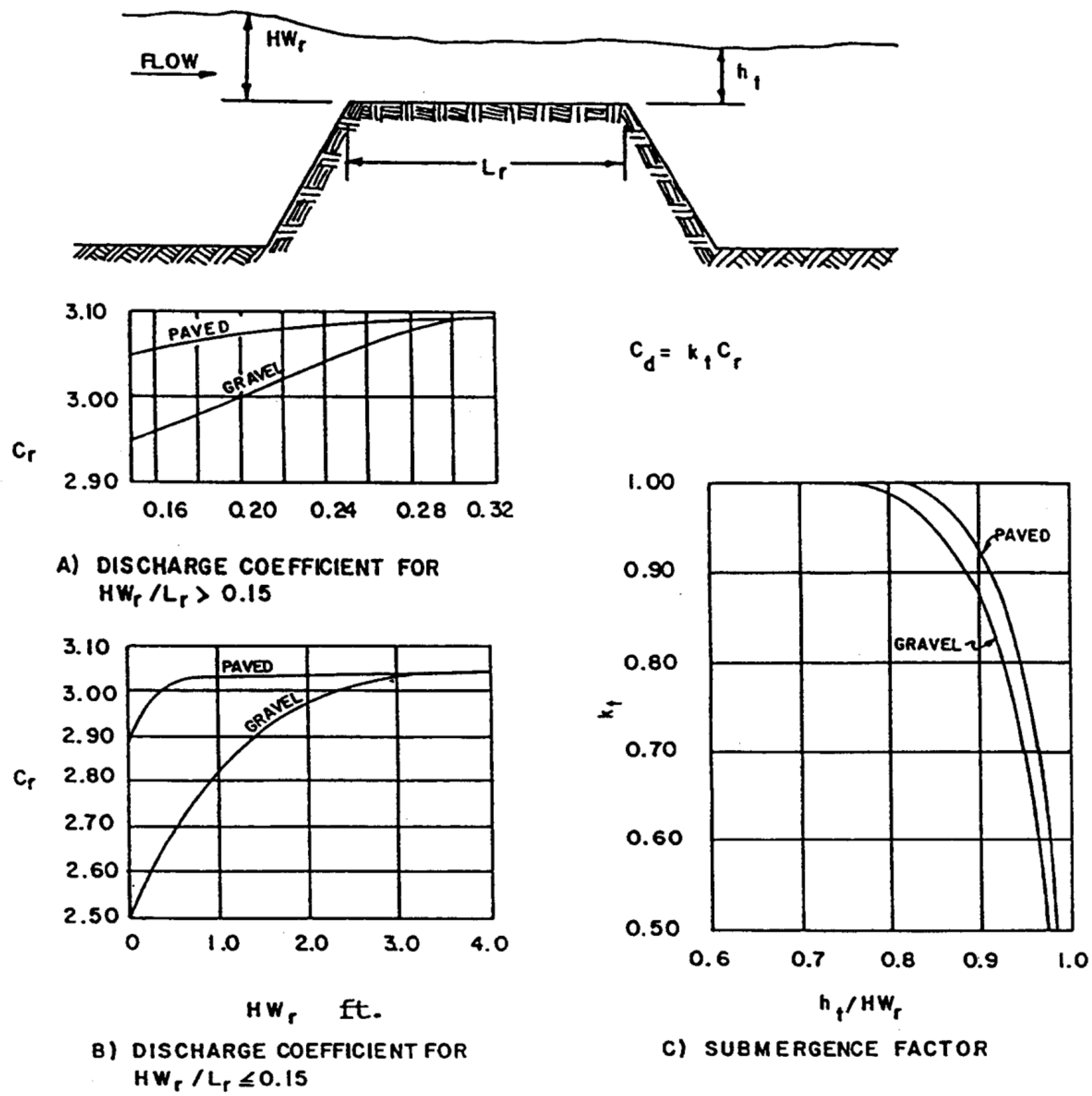


Figure 9.8 Flow coefficients for overtopping roadway

9.2 DESIGN CRITERIA

Design criteria that must be considered for all culvert designs are listed below by categories.

9.2.1 Site and Structure-Selection Criteria

Culvert Location

The Region normally submits field culvert reports, specifying culvert location with supporting survey and topography. Survey transmitted to the Hydraulics Unit must conform with requirements of the CDOT Survey Manual.

Cross culverts must be located as close as possible to the natural drainage waterway. Combining flows from several channels into a common channel, in order to use only one cross culvert, is discouraged. However, if combining is necessary, care must be taken to avoid severe erosion or deposition of silt at the culvert outlet. The same caution applies to concentrating sheet flow from wide or undefined waterways. Flow should not be diverted to another watershed without an evaluation of the legal and physical consequences.

Culvert Alignment

From the standpoint of hydraulic efficiency, durability, and maintenance, abrupt changes in flow direction are undesirable. The maximum angle of bend at any point along a culvert's horizontal or vertical alignment should be 22º 30'. This angle of bend must be referenced to an alignment taken along the side of the culvert nearest to the center of curvature. Any angle of bend greater than this value should be divided into smaller angles, and the miters spaced at a minimum interval of 1.2 times the total diameter or span of the culvert. As an alternative to mitering, and if difficulty of construction dictates, the culvert can be curved using a minimum radius of curvature equal to 3 times the diameter or span. Similar to mitering, curving should be applied along the side of the culvert nearest to the center of curvature.

Any abrupt change of direction at either end of a culvert will retard flow, and may trap debris and cause scouring or silting. The ideal design is to locate a culvert in the existing streambed. Although this is not always feasible, channel changes should be minimized. Water should be intercepted by the cross culvert as close as possible to where the drainage channel first impinges on the highway template. If a channel change cannot be avoided, it should be made at the culvert outlet, rather than at the inlet. In general, when a roadway crosses an irrigation ditch, the skew angle of the crossing cannot be changed in order to reduce the culvert length.

Structure Type Selection

Culverts are used where:

- Bridges are not required hydraulically;
- Debris and ice are tolerable; and
- They are more economical than a bridge.

Bridges are used:

- Where culverts cannot be used;
- Where they are more economical than a culvert;
- To satisfy land-use requirements;
- To mitigate environmental harm caused by a proposed culvert;
- To avoid encroachment on floodways or irrigation canals; and
- To accommodate ice and large debris.

Length, Slope, and Flowline

Culvert length and slope must be chosen to approximate existing topography. To the degree practical, the culvert invert should be aligned with the flowline and skew angle of the stream. The culvert entrance must match the geometry of the roadway embankment.

The length of a culvert is determined from the structure cross section. The end of a culvert is positioned in relation to the fill slope, as shown in the *CDOT M&S Standards*. The culvert length should be increased to compensate for soil sloughing from steep, high fills by projecting the culvert end out an additional 1 foot for each 10 feet of fill height. The length should be increased as required on skewed culverts with ends perpendicular to the flow. These guidelines may be modified when safety, right of way, or physical obstructions dictate.

To reduce sediment deposition within a culvert, slopes should be selected steep enough to maintain or exceed natural-channel velocities. Broken back culvert slopes are discouraged, and abrupt slope changes may trap debris, making culvert cleaning difficult. However, in mountainous regions, a broken-back slope may be necessary to avoid placing the outfall high on the fill. See Section 9.2.3 for a discussion on broken-back culverts.

In cases where the streambed is aggrading, the culvert flowline may be raised accordingly. This reduces sedimentation in the barrel. However, due to the uncertainty of continued aggradation, the culvert height should be increased, leaving the flowline as is. Conditions causing aggradation may be natural, or man- made downstream control. The designer should not select the culvert flowline until upstream and downstream channel-flowline elevations are known. Ground lines of a structure cross section may not represent the channel bottom, causing the designer to erroneously set the culvert flowline on the banks of the channel. The channel flowline may be determined from field survey data or a contour map. The field survey data must include channel cross sections as stated in the *CDOT Survey Manual*.

Ice Buildup

Ice buildup must be mitigated as necessary by:

- Assessing the flood-damage potential resulting from a plugged culvert;
- Increasing the culvert height 1 ft above the total of the maximum observed ice buildup, plus any winter-flow depth; and
- Increasing the culvert width to encompass the observed channel's static-ice width, plus 10% where appropriate, to prevent property damage.

Debris Control

Debris control must be designed using Hydraulic Engineering Circular No. 9, "Debris Control Structures," and must be considered:

- Where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris;
- For culverts located in mountainous or steep regions;
- For culverts under high fills; and
- Where clean-out access is limited. However, access must be available to clean out the debris-control device.

The designer should seek information concerning the type and the amount of debris expected during a major flow. Since it is nearly impossible to calculate the volume by visual observation of the basin, history from previous flows in the proximity of the site is most reliable. The designer may attempt to retain the debris upstream of the entrance, or intentionally pass it through the culvert.

It is not feasible to retain small debris, such as silt, small stones, brush, or trash, upstream of a culvert. Generally, it is also not feasible to retain larger debris, such as large boulders and trees. Debris-control devices are often unsightly and expensive, and they can require considerable maintenance after each flood occurrence. If the storage capacity of a debris trap is too small for a major storm, water may be diverted away from the culvert entrance, causing more damage than under natural conditions.

When debris is passed through a culvert, the size of the culvert must be adequate to prevent excessive ponding at the entrance. If debris is to be retained upstream, a debris-control structure is necessary.

The center web wall in a concrete box culvert can collect excessive debris. If excessive debris is expected, a single-span culvert is favored. Cost comparisons should be made between a single span, and larger double- or triple-cell concrete box culverts. An upstream, sloping web wall is effective in reducing plugging by floating debris. See Chapter 10 Bridges for more information.

Generally, excessive silt is not deposited in a culvert, provided the inlet and outlet are on, or above, the flowline of the channel. Exceptions may occur if the culvert is so wide that the velocity in the culvert is less than in the natural channel, or if the culvert constricts a supercritical channel.

9.2.2 Design Limitations

Allowable Headwater

Allowable headwater is the depth of water that can be ponded or tolerated at the upstream end of a culvert. The allowable headwater does not permit encroachment of water into adjacent roadway, or inundation of upstream property. Both the surrounding features and flow limitations must be considered for each site before the allowable headwater is determined. The potential for future development must also be considered in determining elevation of allowable headwater.

Surrounding features which may control the allowable headwater include:

- Lowest elevation of the roadway subgrade adjacent to the ponding area;
- Flowline of the roadway ditch which passes water along the roadway to another drainage basin; and
- Upstream property, such as buildings or farm crops, which will be damaged if inundated.

Flow-limitation factors that can affect the allowable headwater values include:

When the above factors are insignificant, the ratios of headwater depth to structure depth *(HW/D)* from the flowline, given in Table 9.3, should be used as maximum values in design. These values should be reduced to a ratio of 1.0 or less when design flows are from snowmelt, or in irrigation ditches.

Table 9.3 Maximum Headwater Depth to Structure Depth Ratios, *HW/D*

Range of Diameter or Height or Rise, in	Maximum *HW/D*
< 36 in	2.0
36 in - 60 in	1.7
> 60 in, but < 84 in	1.5
84 in to less than 120 in	1.2
120 in or larger	1.0

The use of *HW/D* ratios greater than those values given in Table 9.3 must be approved by the Region Hydraulic Engineer. For detention ponds, all hydraulic-design work should be performed by the Region Hydraulic Engineer. Refer to Chapter 12 - Storage Facilities, and the U.S. Bureau of Reclamation's, "Design of Small Dams," for additional information on design of detention ponds.

Review Headwater

Review headwater is the flood depth that does not exceed a 1-ft increase over the existing 100-year flood elevation in the National Flood Insurance Program's mapped floodplains, or in the vicinity of insurable buildings. The level of inundation must be tolerable by upstream property and roadway for the review discharge.

Tailwater Relationship (Channel)

To compute a channel's tailwater relationship:

- Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges, which includes the review discharge (see Chapter 8 - Channels).
- Calculate backwater curves at sensitive locations, or use a single cross-section analysis. Backwater curves yield the most-accurate tailwater.
- Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall.
- Use the headwater elevation of any nearby downstream culvert if it is greater than the channel depth.

Tailwater Relationship (Confluence or Large Water Body)

To compute a tailwater relationship for a confluence or large water body:

- Use the high-water elevation with the same frequency as the design flood if events are known to take place concurrently (statistically dependent).
- If statistically independent, evaluate the joint probability of flood magnitudes, and use the likely combination resulting in the greater tailwater depth.

Maximum Velocity

The maximum velocity at the culvert exit must be consistent with the velocity in the natural channel, or must be mitigated with channel stabilization (see Chapter 8 - Channels) and energy dissipation (see Chapter 11 - Energy Dissipator). Velocities greater than 16 fps, along with large discharges, require a concrete stilling basin, or another appropriate type of energy dissipator. Energy dissipators must be designed by the Region Hydraulic Engineer for in house projects.

Minimum Velocity

Minimum velocity in the culvert barrel must result in a tractive force ($\tau = \gamma dS$) greater than critical τ of the transported streambed material at low-flow rates. Use 3 ft/s when streambed material size is not known. If clogging is probable, consider installation of a sediment trap, or a size of culvert that facilitates cleaning.

Storage (Temporary or Permanent)

If storage is being assumed upstream of the culvert, consideration must be given to:

- Limiting the total area of flooding;
- Limiting the average time that bankfull stage is exceeded for the design flood to 48 hours in rural areas, or 6 hours in urban areas; and

- Ensuring that the storage area will remain available for the life of the culvert by purchase of right-of-way or an easement.

Flood Frequency

The flood frequency used to design or review a culvert must be based on:

- Roadway classification;
- Level of risk associated with failure of the crossing, increasing backwater, or redirection of floodwaters;
- An economic assessment or analysis that justifies flood frequencies greater or lesser than the minimum-flood frequencies listed below; and
- Location of FEMA-mapped floodplains.

The flood frequency used to design a culvert must be based on criteria given in Chapter 7 - Hydrology. The minimum design frequencies for various types of roads, terrain, and flood magnitudes are listed in Table 7.2 of Chapter 7 - Hydrology. Frequencies in this table must be used unless an economic analysis indicates otherwise. The minimum design frequency for through-lanes of interstate highways cannot be less than a 50-year flood frequency.

9.2.3 Design Features

Culvert Size and Shape

Selection of culvert size and shape must be based on engineering and economic criteria related to site conditions. The minimum diameters allowed for various types of applications are listed in Table 9.4. Land-use requirements (e.g. need for a cattle pass) can dictate a larger or different barrel geometry than required for hydraulic considerations. Use arch or oval shapes only if required by hydraulic limitations, site characteristics, structural criteria, or environmental criteria.

A circular culvert is the most efficient shape because of its higher ratio of cross-sectional area to the wetted perimeter relative to other shapes with identical cross sectional areas. A narrow but high rectangular culvert is less expensive than a wide, low culvert of the same area. However, the wider culvert has several advantages that are very important. The wider culvert spreads the outlet flow more, and the outlet flow is shallower and has slightly slower velocity, causing less outlet erosion damage. A lower culvert is necessary where clearance is minimal and headwater depths are limited.

Broken-Back Culverts

A broken-back culvert, which combines two different slopes, may be necessary to accommodate a large differential of flow-line elevation, or may result from one or more extensions to an original straight-profile culvert.

Multiple Barrels

Multiple-barrel culverts must fit within the dominant natural channel with minor widening of the channel to avoid conveyance loss through sediment deposition in some of the barrels. They are to be avoided where:

- The approach flow is high velocity, particularly if supercritical. These sites require either a single barrel, or special inlet treatment to avoid adverse hydraulic-jump effects.
- Irrigation canals or ditches are present, unless approved by the canal or ditch owner;

- Fish passage is required, unless special treatment is provided to ensure adequate low flows (commonly one barrel is lowered);
- A high potential exists for debris problems (clogging of culvert inlet); or
- A meander bend is present immediately upstream.

Table 9.4 Minimum Culvert Diameters

Application	**Minimum Diameter, in**
Cross Culvert	
Interstate	36 in
National Highway System-Non Interstate	30 in
State and U.S. Highways and Approaches to Interstate	24 in
Irrigation Crossing and Side Drain	18 in
Storm Drain	
Trunk Line and Single Inlet to Trunk Line	18 in
Median Drain to Cross Culvert	18 in

- **Note:** If site conditions do not allow meeting minimum requirements, the Region Hydraulic Engineer must request approval from Staff Hydraulics for using smaller pipe sizes.

Material Selection

Material selection must consider replacement cost, difficulty of construction, and traffic delay during construction. The material selected must be based on a comparison of the total cost of alternate materials over the design life of the structure, considering the following:

- Durability (service life);
- Structural strength;
- Hydraulic roughness;
- Bedding conditions;
- Resistance to abrasion and corrosion; and
- Water-tightness requirements.

The selection must not be made using first cost as the only criterion. Refer to Appendix C – *CDOT Pipe Material Selection Guide* for an in-depth discussion on material selection.

Culvert Skew

Culvert skew is the acute horizontal angle left or right (looking in the direction of increasing stations) between the centerlines of the roadway and culvert. The culvert skew must not be less than 45 degrees without the approval of the Region Hydraulic Engineer.

End Treatment (Inlet or Outlet)

A culvert's inlet type must be selected from the following list, based on considerations given and inlet coefficients. Refer to FHWA HDS No. 5 for recommended values of entrance-loss coefficients. Consideration must also be given to safety, since some end treatments can be hazardous to errant vehicles. All culverts 48 inches in diameter and larger should have headwalls or slope paving on the inlet end.

Projecting Inlets or Outlets:

- Extend beyond the roadway embankment and are susceptible to damage during roadway maintenance, and from errant vehicles;
- Have low construction cost;
- Have poor hydraulic efficiency for thin materials;
- Are used predominantly with metal pipe; and
- Must include anchoring the inlet to strengthen the weak leading edge for culverts 4 feet in diameter and larger.

Mitered Inlets:

- Are hydraulically more efficient than thin-edge projecting;
- Must be mitered to match the fill slope; and
- Must include anchoring the inlet, to strengthen the weak leading edge for culverts 4 feet in diameter and larger.

Tapered Inlets:

- When practicable, should only be considered for culverts that will operate in inlet control; and
- Are not recommended when fish passage is required.

Refer to Section 9.8 for a discussion on various types of tapered inlets.

Headwalls:

- A full headwall is required for culverts with area ≥ 50 ft^2, diameter ≥ 96 in;
- A concrete Type S headwall is the minimum required for metal culverts with diameters ≥ 42 in, operating under inlet control;
- Headwalls should be placed perpendicular to the culvert centerline for all culverts with span less than 7 feet;
- For wider spans, use the following steps to determine if the headwall is to be placed perpendicular to the culvert, or parallel to the roadway:
    i. Subtract the width of culvert (ft) from the culvert skew (degrees);
    ii. Use headwalls perpendicular to the culvert if the result is greater than 50; or
    iii. Use headwalls parallel to roadway if the result is less than 50.

Headwalls with Bevels:

- Increase the efficiency of metal pipe;
- Provide embankment stability and embankment-erosion protection;
- Provide protection from buoyancy;
- Shorten the required structure length; and
- Reduce maintenance damage.

Improved Inlets:

- Must be considered for culverts with inlet control;
- Can increase the hydraulic performance of the culvert, but may also add to the total culvert cost. They should only be used if practicable.
- With a slope-taper, must not be considered where fish passage is required.

Commercial End Sections:

- All cross culverts and side drains must be installed with end sections, unless other types of end treatments are shown to be more appropriate.
- Are available for both corrugated-metal and concrete pipe;
- Retard embankment erosion and incur less damage from maintenance;
- May improve projecting metal-pipe entrances by increasing hydraulic efficiency, reducing the accident hazard, and improving their appearance; and
- Are hydraulically equal to a headwall, but can be equal to a beveled or side-tapered entrance if a flared, enclosed transition occurs before the barrel.

Wingwalls:

- Are used to retain roadway embankment and avoid a projecting culvert barrel;
- Are used where side slopes of the channel are unstable;
- Are used where the culvert is skewed to the normal channel flow;
- Provide the best hydraulic efficiency if the flare angle is between 30°and 60°; and
- Concrete wingwalls must be used in all box culverts and pipes with full concrete headwalls.

Wingwall geometry is initially determined by a combination of six parameters identified in Figure 9.9. These parameters include:

- The distance in feet from the flowline to the crown (*H*, *Ba*, or Rise generally fixes the value of the height in feet of the upper end of the wingwall);
- The skew angle of the culvert, θ;
- The roadway fill slope, Z;
- The height in feet of the lower end of the wingwall, k, which can be determined from the given equation below. The site topography should dictate the actual value of k that must be used.
- The angle between the wingwall and a line parallel to the roadway, θ_a or θ_b;
- The length of the wingwall, l, in feet.

A schematic drawing of the culvert layout should be shown in the plans, identifying k, θ, and l for each wingwall (m should also be shown if other than standard). The following guidelines should be used in selecting the values for the above six parameters in most situations. However, the designer must set the wings to conform to the culvert site, even though the geometry differs from these guidelines.

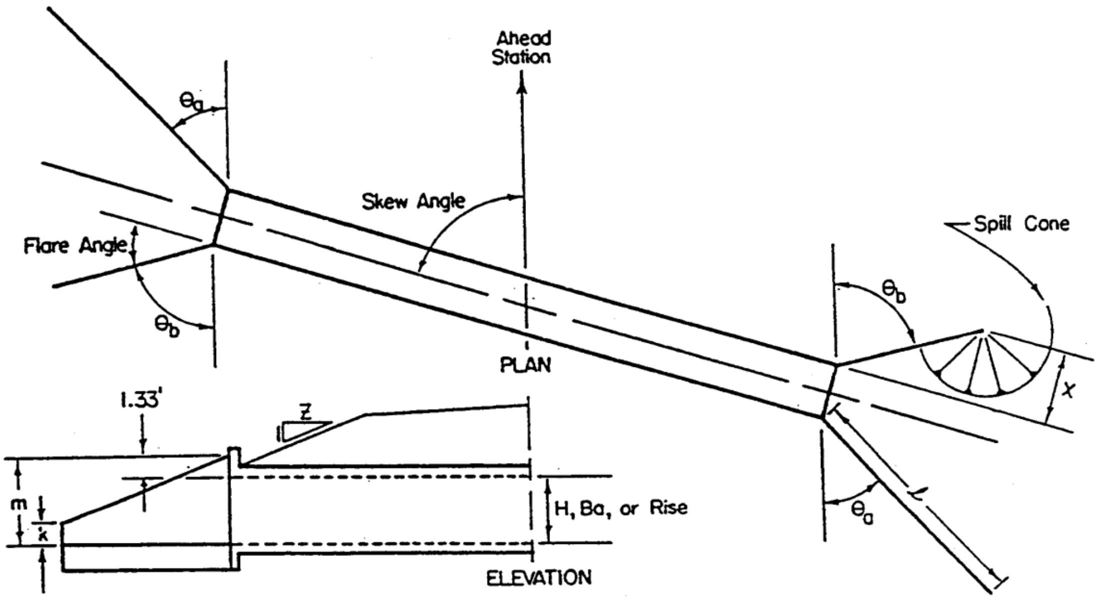


Figure 9.9 Layout of wingwalls

The value of *k* can be determined from the equation:

$$k = 1/2[H, Ba, \text{or Rise}] - 1 \qquad (9.2)$$

Values of *k* and θ must be chosen so that spill cones do not obstruct the culvert inlet and outlet. Higher ends of wings may be used when the spill cone is not subject to erosion. The top of the wingwall should nearly match the final ground elevation. For irrigation structures, the *k* height should be above the design discharge water-surface elevation.

A wingwall flare angle from 20º to 40º normally provides a hydraulically-efficient inlet condition for the culvert. Use of the recommended values for θ_a or θ_b given in Table 9.5, and *k* will generally create a smooth flow transition from channel to culvert, and keep the spill cone out of the projected jet of flow.

The value of *m* can be calculated from the equation:

$$m = [H, Ba, \text{or Rise}] + 1.33 \qquad (9.3)$$

Special headwall designs may dictate some other values for *m*.

The value of wingwall length, *l*, can be obtained from the equation:

$$l = Z\,(m - k) / \sin\theta \qquad (9.4)$$

Wingwall length is constrained by the selection of *Z, m, k,* and θ. Fill slopes flatter than 4:1 should be warped to 4:1 or steeper beyond the culvert headwall to reduce excessive wingwall length. Lengths should be rounded to the nearest foot for l less than 14 feet; to the nearest even foot for l greater than 14 feet but less than 30 feet; and to the nearest four foot increment (30 ft, 34 ft, 38 ft, etc.) for l equal to or greater than 30 feet.

Table 9.5 Recommended wingwall flare angles (θ_a or θ_b)

Skew Angle, θ°	**θ_a°**	**θ_b°**
90	60	60
80	50	70
70	45	80
60	40	90
50	30	90
40	20	100
30	15	105
< 30	Consult the Region Hydraulic Engineer	

A plan showing site contours or spot elevations is often helpful when laying out the final wingwall geometry, especially when the wingwalls must conform to a distinct channel. Allow a spill cone slope of 1.5:1 or flatter at the wingwall ends to assure that the main flow jet, roughly defined by the culvert width, does not impinge on the spill cone.

Culvert headwalls must be perpendicular to the culvert centerline unless excessive cost or aesthetics favor a skewed headwall. A wide culvert with a small skew angle usually justifies the skewed headwall as discussed in the section on headwalls.

Aprons:

- Must be used to reduce scour from high-headwater depths, or from approach velocity in the channel.
- Must extend at least twice the box rise / pipe diamter upstream, but should not be more than 10 ft.
- Must not protrude above the normal streambed elevation.
- Concrete aprons must be used at the outlet of a culvert with wingwalls, and at both inlet and outlet if the culvert also serves as a stockpass.
- Concrete aprons should be considered at the outlet for scour protection.
- If scour damage is anticipated, a concrete apron should be placed between the wingwalls.

Toe walls are required on all wingwalls, except when a concrete apron is used. Toe walls are placed on the end of the apron as shown in the *CDOT M & S Standards*.

Cut-off Walls:

- Are used to prevent piping along the culvert barrel, and undermining at the culvert ends.
- Must be used on all culverts with headwalls or slope paving.
- Must be located as shown in the *CDOT M&S Standards*, or 20 inches below the scour hole.

Safety Considerations

Traffic must be protected from culvert ends as follows:

- Small culverts (diameter ≤ 30 in) must include an end section or slope paving.

- Culverts > 30 inches in diameter must include one of the following:
    i. The culvert must be extended to the appropriate "clear zone" distance per AASHTO's *Roadside Design Guide.*
    ii. The culvert must be safety treated with a grate (90-degree wingwalls) if the consequences of clogging and causing a potential flooding hazard are less than the hazard of vehicles colliding with an unprotected end. If a grate is used, the net area of the grate (excluding the bars) must be 1.5 to 3.0 times the culvert-entrance area. See HDS 5 for information on grate design.
    iii. The culvert must be shielded with a traffic barrier if the culvert is very large, cannot be extended, has a channel that cannot be safely traversed by a vehicle, or has a significant flooding hazard with a grate.
- Periodically inspect each site to determine if safety problems exist for traffic, or for the structural safety of the culvert and embankment.

Median and Entrance Placement

Prepare a cross section of the placement site for each entrance or intersecting road pipe culvert. Pipes should not be installed through permanent maintenance crossovers on an Interstate. Median drainage approaching a crossover should be removed with a pipe across the mainline highway. This will provide for safer crossover inslopes.

Weep Holes

If weep holes are used to relieve uplift pressure in wingwalls and other types of wall structures, they must be designed in a manner similar to underdrain systems. The location of weep holes should be selected carefully to avoid creating an icing hazard.

Performance Curves

Development of performance curves should be considered for evaluating the hydraulic capacity of a culvert where various headwaters, outlet velocities, and scour depths are concerns. These curves will display the consequence of high-flow rates at the site and provide a basis for evaluating flood hazards.

9.2.4 Related Designs

Buoyancy Protection

Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection, must be considered for all flexible culverts. Buoyancy is more serious with steepness of culvert slope (due to higher velocities and hydrodynamic forces), depth of potential headwater (debris blockage may increase), inadequate fill height, large culvert skews, or mitered ends.

Fire Protection

A fully concrete headwall must be considered for any size of thermoplastic culvert pipes if project site is located in areas where risk of fire is relatively high.

Outlet Protection and Erosion Control

Outlet treatment should be designed to restore natural-flow conditions downstream. The outlet should be carefully scrutinized for conditions which can produce scour (see Chapter 11 - Energy

Dissipators). When detrimental scour is expected, protective measures should be utilized. The two common types of channel instability are headcutting, or knickpoint propagation, and scour holes.

Headcutting occurs over a long reach and is influenced insignificantly by the increased culvert velocity. Generally, it is a result of concentrated flow or natural conditions downstream which are independent of the culvert.

Scour holes are caused by high outlet velocities and concentrated flows. Aside from aesthetics, scour holes are not necessarily detrimental, unless structural damage due to undermining occurs. In fact, scour holes at culvert outlets provide efficient energy dissipators. As such, outlet protection for the selected culvert design flood must only be provided where the outlet scour-hole-depth computations indicate:

- The scour hole will undermine the culvert outlet;
- The expected scour hole may cause costly property damage;
- The scour hole causes a nuisance effect (most common in urban areas);
- The scour hole blocks fish passage; or
- The scour hole will restrict land-use requirements.

Headcutting may be controlled by a series of check dams. These will likely be located outside of the right-of-way. Keeping the culvert-outlet elevation low will avoid or reduce the undermining problem. However, lowering should be done only when beneficial, and then with caution so as not to reduce the culvert's capacity. A deep toe wall, a buried rundown, or a concrete apron will also protect the culvert's outlet.

Scour holes can be controlled with riprap, gabions, and plunge basins. Plunge basins are as effective in reducing velocities as natural scour holes.

Relief Opening

Where multiple-use culverts, or culverts serving as relief openings, have their outlet set above the normal stream flow line, special precautions are required to avoid headcuts that would undermine the culvert outlet.

Land-Use Culverts

Consideration must be given to combining drainage culverts with other land-use requirements necessitating passage under a highway:

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

Stock Passes

Economic justification should be determined for all proposed stock passes. Economic analyses should be performed for both cases – with, and without, the proposed stock pass. The value of the parcel of land on either side of the highway should be included in the economic evaluation. The following data are required to determine if stock passes are economically justified, or necessary as safety measures:

- The number of cattle or other livestock that would use the stock pass;
- The frequency of crossings by stock;
- The use of the stock pass for drainage; and
- The required size of drainage structure if the stock pass is not provided.

A proposed stock pass must consist of either a standard box culvert with an opening 6 feet wide and 7 feet high, an 84-inch culvert, or a structural-plate arch culvert 5' - 10" by 7' - 8". Approval of the Region Transportation Director is required to use other sizes of stock-pass structures. Six inches of earth fill material must be placed in the invert of a round or arch culvert after installation. Stock passes that are not needed for drainage can be constructed away from the drainage site to insure the most economical installation with respect to length, cover, and excavation requirement.

If unusual conditions clearly indicate the need for a larger stock pass, full details concerning the proposed size of structures, local conditions, right of way considerations, cost comparisons, and all other pertinent data must be submitted to the Region Transportation Director.

Pedestrian Walkways and Bikeways

A 2-year minimum design-flood frequency is required for any stream crossing of a pedestrian walkway or bikeway. A design frequency of less than 2 years may be used if appropriate, and reasons must be stated in the drainage report. In general, selection of the appropriate design-flood frequency for pedestrian walkways and bike paths should be justified by a cost/benefit ratio analysis of the alternate designs.

Erosion and Sediment Control

Temporary erosion- and sediment-control measures must be included in the construction plans. These measures include use of: silt boxes, straw silt barriers, brush silt barriers, filter cloth, temporary silt fence, and check dams. For more information, see the *CDOT Erosion Control and Stormwater Quality Guide.*

Environmental Considerations And Fishery Protection

Care must be exercised in selecting the location of a culvert site in order to control erosion, sedimentation, debris, and impact on aquatic organism passage. Select a site that will permit the culvert to be constructed, and will limit impact on the stream, wetlands, and aquatic organism passage. For more information, see Chapter 15 - Surface Water Environment.

Irrigation Facilities

Unless legally abandoned, an irrigation structure is required, even if the irrigation canal or ditch is no longer used. The canal or ditch owner must approve the use of multiple-barrel culverts. In general, the inlet and outlet ends of the irrigation structure should extend a maximum of 16.4 feet left and right outside CDOT right of way. In some locations, extensions longer than the maximum may be required. In both cases, construction easements must be obtained from the canal or ditch owner. Provision must be made to accommodate any water escaping the ditch in order to avoid a flood hazard.

Irrigation facilities must be designed to accommodate water rights and intercepted runoff using the following criteria to give the largest culvert size:

- Constrain headwater within the existing canal or ditch banks, unless provision is made for overflow during high flows;
- Provide freeboard to pass expected debris;
- There should be no increase in the velocity beyond what the unprotected ditch material or protection will sustain;
- Avoid a flood hazard created by a canal or ditch failure;
- Provide a width capable of delivering the water and flood right at its existing operating depth; and
- Provide for known winter-ice accumulation problems.

Detour Culvert Pipe Size

Temporary drainage structures, like detour culvert pipes, are normally required during construction to accommodate both traffic and stream flows. From the standpoint of adequate hydraulic design, the higher the design frequency used, the lower the risk of failure will be.

In general, the design flood for a temporary detour structure should be less than that of a permanent drainage structure. The designer should select the design-flood frequency for the detour, and at the same time minimize the cost of culvert pipes. Refer to Table 7.2 of Chapter 7 - Hydrology to determine the design frequency used in sizing the detour culvert pipe. After the design flood is established using reasonable and acceptable hydrology methods, the design steps outlined in sizing permanent culvert structures may be followed to size the detour culvert pipe.

Since detour culvert pipes in general are temporary installations, they are placed without headwalls, and sometimes without adequate cover. For this reason, installation procedures should ensure minimal risk of failure of the detour pipe due to buoyancy forces.

Good construction practices involving detour culvert pipes include locating the detour culvert pipe upstream of the proposed permanent-structure site, and constructing the drainage structure during low-flow seasons.

9.3 MISCELLANEOUS GUIDELINES

9.3.1 Alternatives to Pipe Arches

The use of round pipe with buried inverts is sometimes necessary because of environmental requirements or various other reasons. For example, the pipe invert must be buried under native or borrow materials to improve passage of fish or livestock if the culvert is used as a stockpass. A metal-pipe invert may also be buried under concrete and other lining materials to reduce or eliminate the impact of heavy sediment loads that can cause abrasion and subsequent corrosion.

It is not always necessary to fill the invert up to the natural-channel flowline, as water will transport sediment which will do the backfilling. To use a round pipe with a buried invert, the size of the round pipe should be selected so that its capacity will be equivalent to that of the required unburied pipe arch. Refer to FHWA's publication "Design of Depressed Invert Culverts" for the procedure to determine the required equivalent diameter of buried round pipe. The equivalent diameter is the diameter of a circle which has an area equal to the total area above the bed of a depressed-invert culvert. Figure 9.10 and Tables 9.6(a) and 9.6(b) provide a diagram of a buried round pipe, and a comparison of the areas of unburied metal-arch pipes and partially-buried pipes.

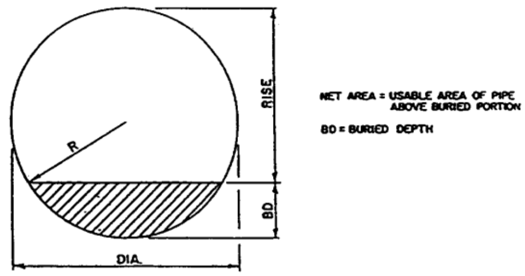


Figure 9.10 Usable area of buried round pipe

Table 9.6(a) Pipe arch area vs. round pipe area

Pipe Arch Data

Equivalent Diameter	Span	Rise	Area
54 in	65 in	40 in	14.3 ft^2
60 in	72 in	44 in	17.6 ft^2
66 in	73 in	55 in	22.0 ft^2
72 in	81 in	59 in	26.0 ft^2
78 in	87 in	63 in	31.0 ft^2
84 in	95 in	67 in	35.0 ft^2
90 in	103 in	71 in	40.0 ft^2
96 in	112 in	75 in	46.0 ft^2
102 in	117 in	79 in	52.0 ft^2
108 in	128 in	83 in	58.0 ft^2
114 in	137 in	87 in	64.0 ft^2
120 in	142 in	91 in	71.0 ft^2

Table 9.6(b) Pipe arch area vs. round-pipe area
Buried Round-Pipe Data

Diameter	Rise	Bd	Area	Net Area
60 in	42 in	18 in	19.64 ft^2	14.7 ft^2
66 in	46 in	20 in	23.76 ft^2	17.7 ft^2
72 in	55 in	17 in	28.27 ft^2	23.2 ft^2
78 in	59 in	19 in	33.18 ft^2	26.9 ft^2
84 in	63 in	21 in	38.49 ft^2	31.0 ft^2
90 in	67 in	23 in	44.18 ft^2	35.3 ft^2
96 in	71 in	25 in	50.27 ft^2	39.9 ft^2
102 in	77 in	25 in	56.75 ft^2	46.0 ft^2
108 in	82 in	26 in	63.62 ft^2	51.8 ft^2
114 in	86 in	28 in	70.88 ft^2	57.4 ft^2
120 in	90 in	30 in	78.54 ft^2	63.2 ft^2
126 in	95 in	31 in	86.59 ft^2	70.0 ft^2

9.3.2 Jacking Welded Steel and Reinforced Concrete Pipe

On roadway, utility, and drainage projects, pipe-jacking operation is commonly used to install a piping system with minimal, or no interruption to the vehicular traffic or any type of utility service. Design and construction guidelines are required to accomplish pipe jacking in a cost effective manner. Guidelines for jacking welded steel and reinforced concrete pipe (RCP) are provided in Appendix B.

9.4 ALTERNATIVE ANALYSIS AND DESIGN METHODS

The design of a culvert system for a highway crossing of a floodplain requires information from the following chapters in this manual: Policy, Documentation, Planning and Location, Hydrology, Channels, Energy Dissipators, Storm Drainage Systems, Surface Water Environment, and Erosion and Sediment Control. Each of these should be consulted as appropriate. The discussion in this section is focused on analysis of alternatives and design methods.

9.4.1 Alternative Analysis

Culvert alternatives must satisfy topography, and design policies and criteria. Alternatives must be analyzed for:

- Environmental impact;
- Hydraulic equivalency; and
- Risk and cost.

Select an alternative that best integrates engineering, economic, and political considerations. The selected culvert must conform with all selected structural and hydraulic criteria, and selection based on:

- Construction and maintenance costs;
- Risk of failure or property damage;
- Traffic safety;
- Environmental or aesthetic considerations;
- Political or nuisance considerations; and
- Land-use requirements.

9.4.2 Design Methods

The designer must choose whether:

- To use a culvert, storm drain, or inverted siphon;
- To assume a constant discharge, or route a hydrograph; or
- To use computer software or charts.

Structure Type

Culvert - a covered structure with both ends open. It is designed using the procedures in HDS 5.

Storm Drain - a covered structure with either end in a manhole. It is usually part of a system of pipes. Storm drains are explained in FHWA Hydraulic Engineering Circular No. 22 (HEC 22), *Urban Drainage Design Manual*, and designed using FHWA Hydraulic Toolbox software (see Chapter 13 – Storm Drains).

Inverted Siphon - a covered structure sometimes termed a sag culvert, with both ends open, and operates at a low head. The invert profile dips below the approach and exit channels. Inverted siphons are designed using procedures in the U.S. Bureau of Reclamation's Design of Small Canal Structures.

Hydrology Methods

Constant Discharge - Constant discharge, usually the peak discharge, is assumed for most culvert designs. This yields a conservatively-sized structure where temporary storage is available, but not normally used.

Hydrograph and Routing - Significant storage will reduce the required culvert size. Storage capacity behind a highway embankment attenuates a flood hydrograph and reduces the peak discharge. It is checked by routing the design hydrographs through the culvert site to determine the outflow hydrograph and stage (backwater) behind the culvert. Procedures are found in Chapter 12 - Storage Facilities, and HDS 5, Section V.

Computational Methods

Computer Software:

See Section 9.11 - Software for Designing Culverts.

Graphical Solution:

- Allow a trial-and-error solution which is easy and provides reliable designs for many applications;

- Require additional computations for tailwater, outlet velocity, and roadway overtopping; and
- May require additional computations for hydrographs and routing.

Charts for circular and box shapes are included at the end of this Chapter. Other shapes and improved inlets are found in HDS 5.

9.5 HYDRAULIC DESIGN

9.5.1 General

An exact theoretical analysis of culvert flow is extremely complex and requires:

- Analyzing non-uniform flow with regions of both gradually-varying and rapidly-varying flow;
- Determining how the flow type changes as the flow rate and tailwater elevations change;
- Applying backwater and drawdown calculations, and energy and momentum balance;
- Applying the results of hydraulic model studies; and
- Determining if hydraulic jumps occur, and if they are inside, or downstream of the culvert barrel.

Most of the above complications are addressed in the FHWA HY-8, Culvert Hydraulic Analysis Program (see Section 9.11 - Software for Designing Culverts).

9.5.2 Standard Practice

HDS 5 describes standard practice for hydraulic design of culverts. The following standard practices apply:

- All culverts should be hydraulically designed;
- The overtopping flood selected should be consistent with the class of highway, and appropriate for the risk at the site;
- Survey information should include topographic features, channel characteristics, aquatic life, high-water information, existing structures, and other related site-specific information. Refer to Chapter 6 - Data Collection;
- Culvert location in both plan and profile should be investigated to minimize the potential for sediment buildup in culvert barrels;
- The cost savings of multiple uses (utilities, stock and wildlife passage, land access, and fish passage) should be weighed against the advantages of separate facilities;
- Culverts should be designed to accommodate debris, or appropriate provisions should be made for debris maintenance;
- Material selection should include consideration of service life that includes abrasion and corrosion;
- Culverts should be located and designed to present a minimum hazard to traffic and people;
- The detail of documentation for each culvert site should be appropriate for the risk and importance of the structure. Design data and calculations should be assembled in an orderly fashion and retained for future reference as provided for in Chapter 4 – Documentation; and

- Where practical, some means should be provided for personnel and equipment access to facilitate maintenance.

Design Discharge - Culverts should be designed for a constant discharge, typically the peak discharge. This will yield a conservatively-sized structure where temporary storage is available but not used.

Control Section - The control section is the location where there is a unique relationship between the flow rate and the upstream water-surface elevation. Inlet control is governed by the inlet geometry. Outlet control is governed by a combination of the culvert-inlet geometry, barrel characteristics, and tailwater or critical depth.

Minimum Performance - Minimum performance is assumed by analyzing both inlet and outlet control, and basing the design on the highest headwater. The culvert may operate more efficiently at times (more flow for a given headwater level), but will not operate at a lower level of performance than calculated.

9.5.3 Inlet Control

Figure 9.11 illustrates the types of inlet control flow. The USGS flow type depends on the submergence of the inlet and outlet ends of the culvert. In all of these examples the control section is at the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high-velocity (supercritical) flow in the culvert barrel. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

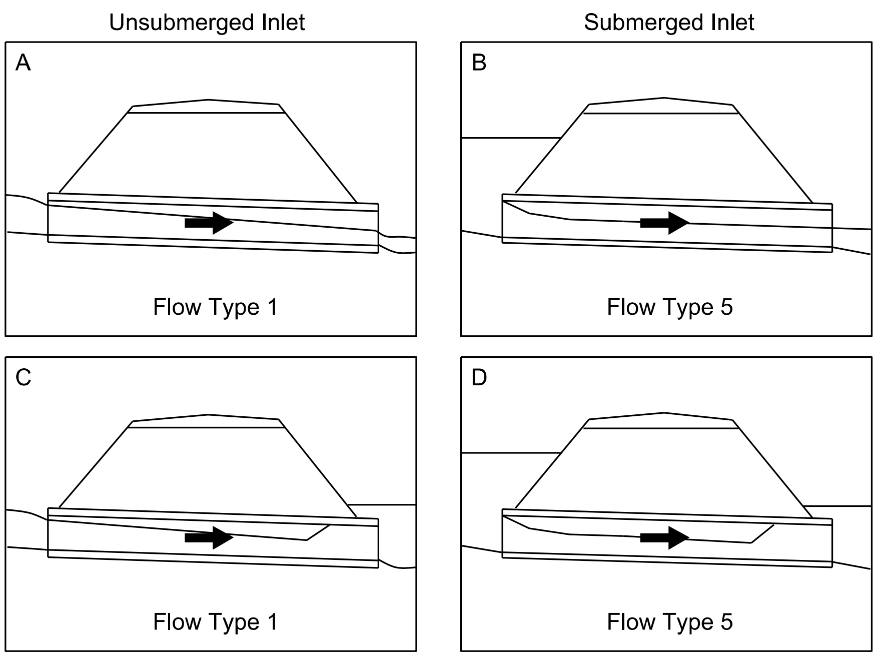


Figure 9.11 Types of inlet control

Since the control is at the upstream end, only the headwater and the inlet factors affect the culvert performance:

Headwater Depth - is measured from the inlet invert of the inlet control section to the surface of the upstream pool.

Inlet Area - is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area. For tapered inlets (Section 9.8) the face area is enlarged, and the control section is at the throat.

Inlet Edge - describes the entrance type. Some typical inlet edge configurations are thin-edge projecting, mitered, square edges in a headwall, and beveled edge.

Inlet Shape - is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical and arch. Check for an additional control section, if different than the barrel.

Barrel Slope - influences inlet control performance, but the effect is small. Inlet control charts assume a slope of 2% for the slope correction term ($0.5S$ for most inlet types). This results in lowering the headwater required by $0.01D$. In the computer program HY-8, the actual slope is used as a variable in the calculation.

Hydraulics

Inlet control performance is defined by the three regions of flow shown in Figure 9.12: unsubmerged, transition and submerged.

Unsubmerged

For headwater between the invert and the culvert height, as shown in Figure 9.13 A and C, the entrance of the culvert operates as a weir. A weir is a flow control section where the upstream water surface elevation can be predicted for a given flow rate. The relationship between flow and water-surface elevation must be determined by model tests of the weir geometry, or by measuring prototype discharges. These tests are then used to develop equations. Appendix A of HDS 5 contains equations developed from model test data.

Submerged

For headwaters submerging the culvert entrance the culvert operates as an orifice, as shown in Figure 9.13 B and D. An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section.

The relationship between flow and headwater can be defined based on results from model tests. Appendix A of HDS 5 contains flow equations developed from model test data.

Transition Zone

The transition zone is located between the unsubmerged (weir control) and the submerged (orifice control) flow conditions, where the flow is poorly defined. This zone is approximated by plotting the unsubmerged- and submerged-flow equations, and connecting them with a line tangent to both curves.

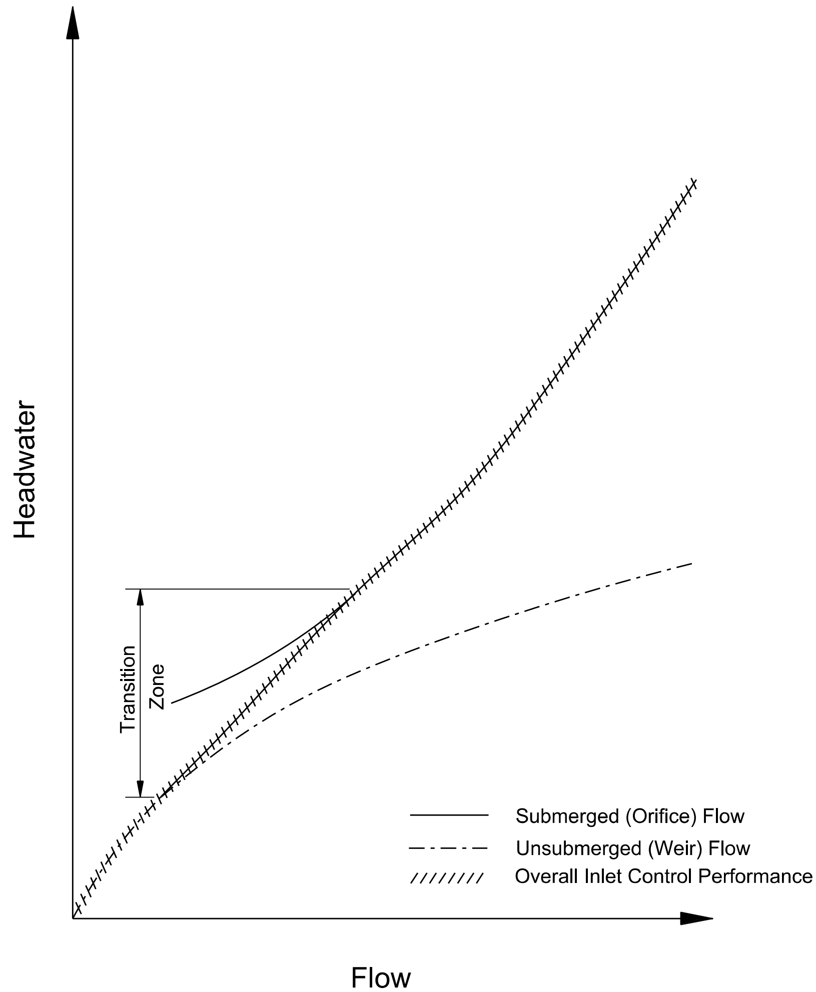


Figure 9.12 Unsubmerged, transition, and submerged

Graphical Solution

The inlet-control flow versus headwater curves which are established using the above procedure are the basis for constructing the inlet-control design charts, and for developing equations used in software. The original equations for computer software were generally 5th order polynomial curve-fitted equations developed to be as accurate as the graphical solution (plus or minus 10%) within the headwater range of 0.5*D* to 3.0*D*. These equations are still being used in HY-8, but have been supplemented with a weir equation from 0.0*D* to 0.5*D*, and an orifice equation above 3.0*D*. Note that in the inlet-control solutions, *HW* is measured to the total upstream energy grade line, including the approach-velocity head.

Inlet Depression

Inlet depression is created by constructing the entrance inlet below the streambed. The amount of inlet depression is defined as the depth from the natural streambed at the face to the inlet invert. The inlet-control equations or charts provide the depth of headwater above the inlet invert required to convey a given discharge through the inlet. This relationship remains constant regardless of the elevation of the inlet invert. If the entrance end of the culvert is constructed below the streambed, more head can be exerted on the inlet for the same headwater elevation.

9.5.4 Outlet Control

Outlet control has depths and velocity that are subcritical. Figure 9.13 illustrates the types of outlet-control flow. The USGS flow type depends on the submergence of the inlet and outlet ends of the culvert. In all cases, the control of the flow is at the downstream end of the culvert (the outlet) or further downstream. The tailwater depth is either assumed to be critical depth near the culvert outlet or the downstream channel depth, whichever is higher. In a given culvert, the type of flow is dependent on all barrel factors. All inlet-control factors also influence culverts in outlet control.

Barrel Roughness - is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. Roughness is represented by a hydraulic resistance coefficient, such as the Manning's *n* value. Typical values for Manning's *n* are presented in Table 9.2. Additional discussion on the sources and derivations of the Manning's *n* values are contained in HDS-5, Appendix B.

Barrel Area - is measured perpendicular to the flow.

Barrel shape - is a function of culvert type and material. Based on the location of the center of gravity for a given area, a box is the most efficient shape, then the arch shape, followed by the circular pipe.

Barrel Length - the total culvert length from the entrance crown to the exit crown of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

Barrel Slope - the actual slope of the culvert barrel, often the same as the slope of the natural stream. However, where the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope. The slope is not a factor in calculating the barrel losses for USGS Flow Types 4, 6, and 7, but is a factor in calculating USGS Flow Types 2 and 3 when a water-surface profile is calculated.

Tailwater Elevation - is based on the downstream water-surface elevation. Backwater calculations from a downstream control, normal-depth approximation, or field observations are used to define the tailwater elevation.

Hydraulics

Full flow in the culvert barrel is assumed for the analysis of outlet-control hydraulics. Outlet-control flow conditions can be calculated based on energy balance from the tailwater pool to the headwater pool. Outlet control can occur as full, partial flow, or a combination of both through the culvert.

A. Losses:

$$H_L = H_E + H_f + H_v + H_b + H_j + H_g \tag{9.5}$$

Where: H_L = total energy loss, ft; H_E = entrance loss, ft; H_f = friction losses, ft; H_v = exit loss (velocity head), *m* (equivalent to H_o; see Equation 9.8d); H_b = bend losses, ft; H_j = losses at junctions, ft; H_g = losses at grates, ft.

These and other losses are discussed in HDS 5.

B. Velocity:

$$V = Q/A \tag{9.6}$$

Where: V = average barrel velocity, ft/s; Q = flow rate, ft^3/s; A = cross sectional area of flow with the barrel full, ft^2.

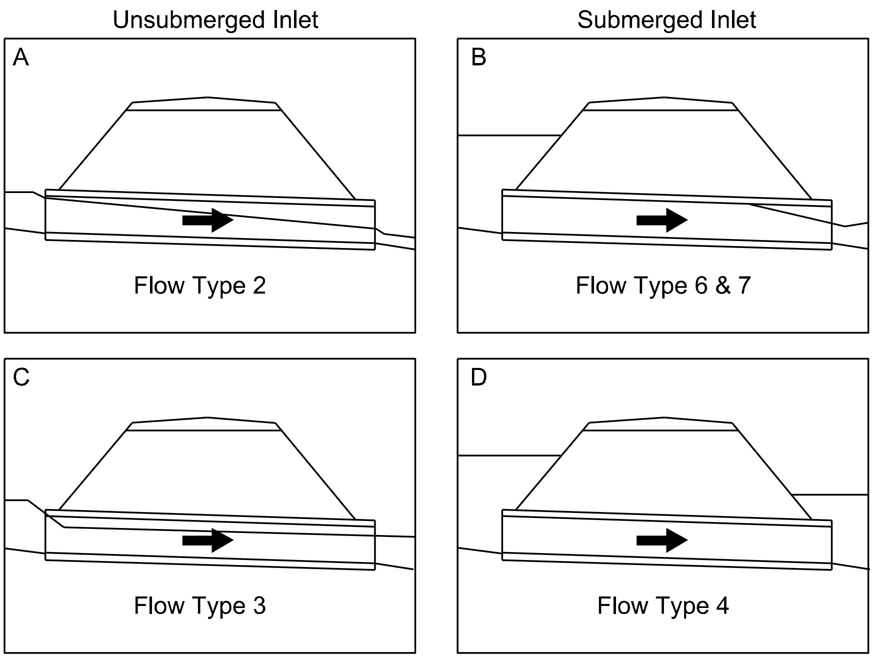


Figure 9.13 Types of outlet control

C: Velocity Head:

$$H_v = V^2/2g \tag{9.7}$$

Where: g = acceleration due to gravity, 32.2 ft/s^2.

D. Entrance Loss:

$$H_E = K_E(V^2/2g) \tag{9.8a}$$

Where: K_E = entrance loss coefficient.

E. Friction Loss:

$$H_f = [(29n^2L)/R^{1.33}]\ [V^2/2g] \tag{9.8b}$$

Where: n = Manning's roughness coefficient; L = length of the culvert barrel, ft; R = hydraulic radius of the full culvert barrel = A/P, ft; P = wetted perimeter of the barrel, ft.

F. Exit Loss:

$$H_o = 1.0\ [V^2/2g) - (V_d^2/2g)] \tag{9.8c}$$

Where: V_d = channel velocity downstream of the culvert, ft/s (usually neglected, in which case the exit loss is equal to the full-flow velocity head in the barrel, as shown in Equation 9.8d).

Equation 9.8c may overestimate exit losses, and a multiplier of less than 1.0 can be used (see FHWA Hydraulic Engineering Circular No. 14, HEC 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* for transition loss).

$$H_o = H_v = V^2/2g \tag{9.8d}$$

G. Barrel Losses:

$$H = H_E + H_o + H_f \quad (9.9)$$

$$H = [1 + K_e + (29n^2L/R^{1.33})]\ [V^2/2g]$$

Energy Grade Line

The energy grade line represents the total energy at any point along the culvert barrel. Equating the total energy at Sections 1 and 2, upstream and downstream of the culvert barrel, the following relationship results:

$$HW_o + (V_u{}^2/2g) = TW + (V_d{}^2/2g) + H_L \quad (9.10)$$

Where: HW_o = headwater depth above the outlet invert, ft; V_u = approach velocity, ft/s; TW = tailwater depth above the outlet invert, ft; V_d = downstream velocity, ft/s; H_L = sum of all losses (Equation 9.1).

This equation is true only if TW is higher than critical depth at the outlet. Additionally, the total available upstream headwater (HW_o) includes the depth of the upstream water above the inlet invert and the approach velocity head. In most instances, the approach velocity is low and the approach velocity head is neglected. However, it can be considered to be a part of the available headwater and used to convey the flow through the culvert. Likewise, the velocity downstream of the culvert (V_d) is usually neglected.

Hydraulic Grade Line

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

HDS 5 Charts (Full Flow)

Graphical solutions have been developed assuming that the culvert barrel is flowing full and:

- $TW \geq D$, Flow Type 4 (see Figure 9.13 D), or $d_c \geq D$, Flow Type 6 (see Figure 9.13 B);
- V_u is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert; and
- V_d is small and its velocity head can be neglected.

Equation (9.10) becomes:

$$HW = TW + H - S_oL \quad (9.11)$$

Where: HW = depth from the inlet invert to the energy grade line, ft; H = the value read from the charts (Equation 9.5), ft; $S_o L$ = drop from inlet to outlet invert, ft.

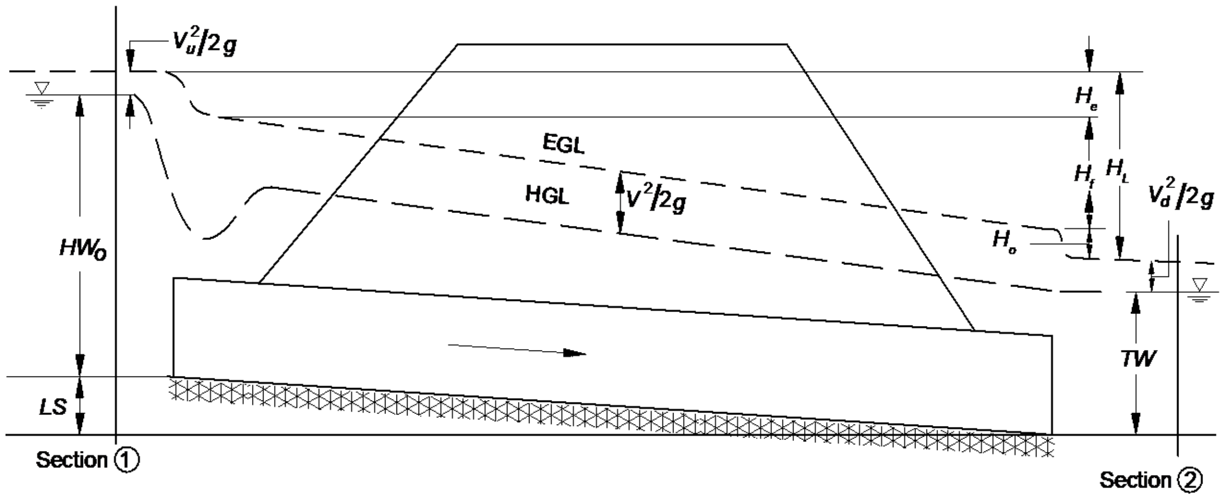


Figure 9.14 Full Flow Energy and Hydraulic Grade Lines

Partial Full Flow Computations

Equations (9.5) through (9.11) were developed for full-barrel flow. The equations also apply to USGS Flow Types 6 and 7, which are effectively full-flow conditions, if $TW < d_c$ (see Figure 9.13 B).

Backwater calculations may be required for partially-full flow conditions (see Figures 9.13 A and C). These calculations begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel, a full flow extends from that point upstream to the culvert entrance.

Partial Full Flow Computations - Approximate Method

Based on numerous backwater calculations performed by FHWA, it was found that the hydraulic grade line pierces the plane of the culvert outlet at a point approximately one half the way between critical depth and the top of the barrel, or $(d_c + D)/2$ above the outlet invert. This approximation should only be used if the barrel flows full for part of its length, or the headwater is at least $0.75D$. If neither of these conditions are met, a water-surface profile should be used to establish the hydraulic grade line. TW should be used if higher than $(d_c + D)/2$. The following equation should be used:

$$HW = h_o + H - S_o L \qquad (9.12)$$

Where: h_o = the larger of TW or $(d_c + D)/2$, ft.

Adequate results are obtained down to $HW = 0.75D$. For lower headwaters, backwater calculations are required. See Figure 9.13 A if $TW < d_c$, and Figure 9.13 C if $TW > d_c$.

9.5.5 Outlet Velocity

Culvert outlet velocities must be calculated to determine the need for erosion protection at the culvert exit. Culverts usually have outlet velocities higher than the natural-stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. If outlet erosion protection is necessary, the flow depths and Froude number may also be needed (see Chapter 11 - Energy Dissipators).

Inlet Control

For inlet control the velocity is calculated from Equation 9.10 after determining the outlet depth. Either of the following methods may be used to determine the outlet depth:

- Calculate the water-surface profile through the culvert. Begin the computation at d_c at the entrance and proceed downstream to the exit. Determine the depth and flow area at the exit.
- Assume normal depth and velocity. This approximation may be used because the water surface profile converges towards normal depth if the culvert is of adequate length. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depths may be obtained by hand computation or by software (e.g. FHWA Hydraulic Toolbox).

Outlet Control

Under outlet control, the cross-sectional area of flow is defined by the geometry of the outlet and either critical depth, tailwater depth or the height of the conduit:

- Critical depth is used where the tailwater is less than critical depth;
- Tailwater depth is used where tailwater is greater than critical depth, but below the top of the barrel; and
- The total barrel area is used where the tailwater exceeds the top of the barrel.

9.5.6 Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. It will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad-crested weir. Flow coefficients for flow overtopping roadway embankments are found in Figure 9.8.

$$Q_0 = C_d L HW_r^{1.5} \tag{9.13}$$

Where: Q_0 = overtopping flow rate, ft^3/s; C_d = overtopping discharge coefficient (weir coefficient) = $k_t C_r$ in which, k_t = submergence coefficient, C_r = discharge coefficient; L = length of roadway crest, ft; HW_r = the upstream depth, measured above the roadway crest, ft.

Roadway Crest Length

Roadway crest length is difficult to determine where the crest is defined by a roadway sag vertical curve. Two methods may be used to overcome this:

- Subdivide the length into a series of segments. The flow over each segment is then calculated for a given headwater. The flows for each segment are added together to determine the total flow.
- The length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth (area/length) of the upstream pool above the roadway.

Total Flow

Total flow is calculated for a given upstream water surface elevation using Equation 9.1.

- Roadway overflow plus culvert flow must equal total design flow;
- A trial-and-error process is necessary to determine the flow passing through the culvert, and the amount flowing across the roadway; and

- Performance curves for the culvert and the road overflow may be summed to yield an overall performance.

9.5.7 Performance Curves

A performance curve is a plot of flow rate versus headwater depth, elevation, velocity, or outlet scour. The culvert performance curve is made up of controlling portions of individual performance curves for inlet, outlet, and roadway-control sections. Performance curves can be developed for all culverts to evaluate hydraulic capacity of a culvert for various headwaters, outlet velocities, and scour depths. These curves display consequences of high flow rates at the site, and provide a basis for evaluating flood hazards.

Inlet Performance Curve - The inlet performance curve is developed using inlet-control charts (see design charts provided at the end of this chapter).

Outlet Performance Curve - The outlet performance curve is developed using Equations 9.5 - 9.13, outlet-control charts (see the appropriate design chart at the end of this chapter), or backwater calculations.

Roadway Performance Curve - The roadway performance curve is developed using the equation for roadway-overtopping flow (Equation 9.13).

Overall Performance Curve - The overall performance curve is the sum of the flow through the culvert and the flow across the roadway, and can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge, and cover the entire flow range of interest. Both inlet- and outlet-control headwaters must be calculated.
2. Combine the inlet- and outlet-control performance curves to define a single performance curve for the culvert.
3. When the culvert-headwater elevations exceed the roadway-crest elevation, overtopping begins. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and the equation for roadway overtopping flow to calculate flow rates across the roadway.
4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert-performance curve as shown in Figure 9.15.

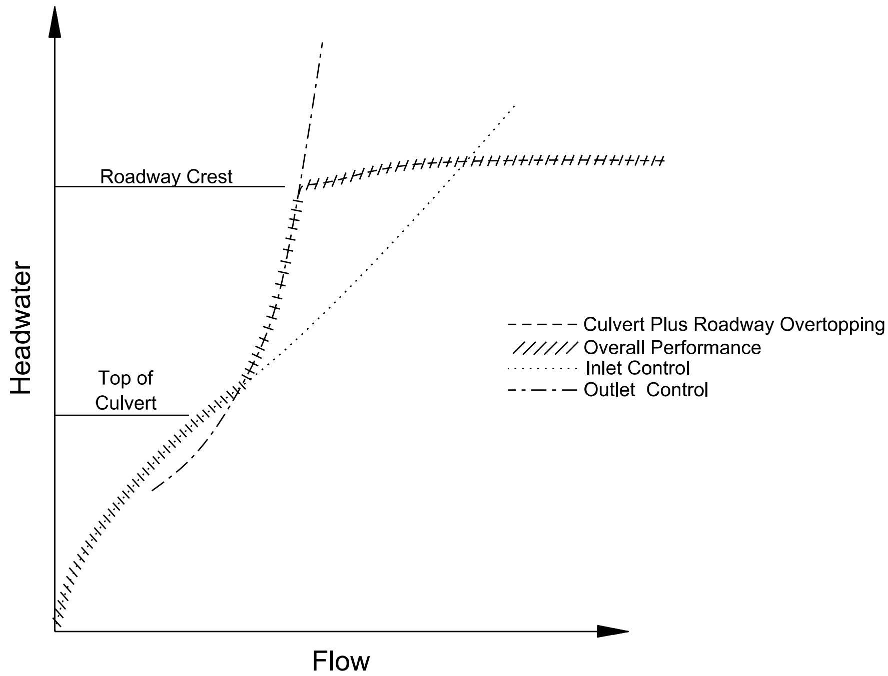


Figure 9.15 Overall performance curve

9.5.8 Culvert Design Form

The Culvert Design Form, shown in Figure 9.16, has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description and the designer's identification. Summaries of hydrologic data are also included. At the top right is a small sketch of a culvert with blanks for inserting important dimensions and elevations.

PROJECT: ________

STATION: ________

SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE: ______ / ______

REVIEWER / DATE: ______ / ______

HYDROLOGICAL DATA

See Add'l Shts.

☐ METHOD: ________

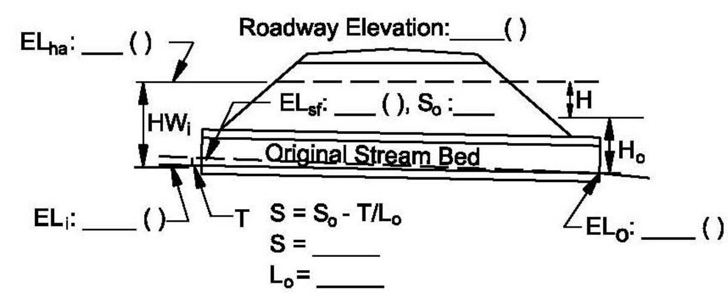
☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____

☐ CHANNEL SHAPE: ________

☐ ROUTING: ____ ☐ OTHER: ____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)



CULVERT DESCRIPTION: MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HWi	T (3)	ELhi (4)	OUTLET CONTROL T W (5)	dc	(dc+D)/2	ho (6)	ke	H (7)	ELho (8)	Control Headwater Elevation	Outlet Velocity	Comments

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i / D = HW / D$ OR HW_i / D FROM DESIGN CHARTS

(3) $T = HW_i - (EL_{hd} - EL_{sf})$
T IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (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) $H = [1 + k_e + (K_u n^2 L) / R^{1.33}] v^2 / 2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
ha. ALLOWABLE HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ________

SHAPE: ________

MATERIAL: ____ n ____

ENTRANCE: ________

Figure 9.16 Culvert design form

9.6 DESIGN PROCEDURE

The following design procedure provides a convenient and organized method for designing culverts for a constant discharge, considering inlet and outlet control. The procedure does not address the effect of storage which is discussed in Chapter 12 - Storage Facilities.

The designer should be familiar with all equations in Section 9.5 before using these procedures. Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structure.

A computation form has been provided in Appendix A to guide the user. It contains blocks for the project description, designer's identification, hydrologic data, culvert dimensions and elevations, trial culvert description, inlet and outlet control *HW*, culvert barrel selected and comments.

Step 1 Assemble Site Data and Project File

a. See Data Chapter - the minimum data are:
   i. USGS, site, and location maps;
   ii. Embankment cross section;
   iii. Roadway profile;
   iv. Photographs;
   v. Field visit (sediment, debris); and
   vi. Design data at nearby structures.

b. Studies by other agencies including:
   i. Small dams - NRCS, USACE, TVA, BLM;
   ii. Canals - NRCS, USACE, TVA, USBR;
   iii. Floodplain - NRCS, USACE, TVA, FEMA, USGS, NOAA; and
   iv. Storm drain - local or private.

c. Environmental constraints including:
   i. Commitments contained in NEPA documents,
   ii. Fish migration, and
   iii. Wildlife passage.

d. Design criteria:
   i. Review Section 9.2 for applicable criteria, and
   ii. Prepare risk assessment or analysis.

Step 2 Determine Hydrology

a. See Chapter 7 - Hydrology.

b. Minimum data are drainage area map and a discharge-frequency plot.

Step 3 Design Downstream Channel

a. See Chapter 8 - Channels.

b. Minimum data are cross section of channel and the rating curve for channel.

Step 4 Summarize Data on Design Form: Collect all data from the preceding steps and record on a single design form (see Figure 9.16).

Step 5 Select Design Alternative: Choose culvert material, shape, size and entrance type.

Step 6 Select Design Discharge, Q_d

a. See Section 9.2.2 - Design Limitations.

b. Determine flood frequency from criteria.

c. Determine Q from discharge-frequency plot (Step 2).

d. Divide Q by the number of barrels.

Step 7 Determine Inlet-Control Headwater Depth, HW_i : Use the inlet control approach in computer modeling or in the charts provided in the appendices.

a. Locate the size or height on the scale.

b. Locate the discharge:

i. For a circular shape, use discharge.

ii. For a box shape, use Q per foot of width.

c. Locate HW/D ratio:

i. Use a straight edge.

ii. Extend a straight line from the culvert size through the flow rate.

iii. Mark the first HW/D scale. Extend a horizontal line to the desired scale and read HW/D and note on Design Form in Appendix A.

d. Calculate headwater depth, HW_i :

i. Multiply HW/D by D to obtain HW to energy grade line.

ii. Neglecting the approach velocity, $HW_i = HW$.

iii. Including the approach velocity, $HW_i = HW$ – approach velocity head.

Step 8 Determine Outlet Control Headwater Depth at Inlet, HW_{oi} :

a. Calculate the tailwater depth TW from the design flow rate, outlet channel geometry, roughness, and slope, using uniform-flow equations such as the Manning's formula. TW can also be obtained using water-surface profile analysis.

b. Calculate the critical depth d_c using software or appropriate charts at the end of this chapter.

c. Calculate the value of $(d_c + D)/2$ and compare with the value of TW. D is the inside diameter or height of the culvert.

d. Determine $h_o = TW$ or $(d_c + D)/2$ whichever is larger.

e. Determine k_e, entrance-loss coefficient from HDS 5 Table C.2.

f. Determine H, which is the sum of the energy losses at the culvert entrance, barrel, and the outlet from the outlet control charts.

g. Calculate HW_{oi} from the equation:

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

where: H and h_o are as defined above, S_o and L are the slope and length of the culvert, respectively. Note, this equation is only applicable where V_u and V_d are neglected.

If HW_{oi} is less than 1.2 times the diameter or rise D and the flow condition is outlet control, the barrel may be flowing partly full and the approximate method of using the greater of tailwater may or may not be applicable. If the headwater depth, $(d_c + D)/2$, falls below 0.75 times D, the approximate method should not be used. Backwater calculations should be used to check the result.

Step 9 Determine Controlling Headwater, HW_c : Compare HW_i and HW_{oi}. If HW_i is greater than HW_{oi} , the culvert is in inlet control. If HW_{oi} is greater than HW_i, the culvert is in outlet control. If HW_i is equal to HW_{oi}, the culvert is both in inlet and outlet control. Use the higher value as the controlling headwater.

Step 10 Compute Discharge Over the Roadway, Q_r:

a. Compute culvert discharge Q, discharge over the roadway, Q_r, and total discharge Q_t, by
trial-and-error method, or graphically by using the performance curve. The culvert discharge Q is the discharge corresponding to the non-overtopping elevation.

b. Determine the overtopping discharge, Q_r, from the equation:

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

$Q_r = 0$ if flow does not overtop the roadway. See section on roadway overtopping for the definition of the variables used in the equation.

c. Calculate depth above the roadway (HW_r):

i. $HW_r = HW_c - HW_{ov}$

ii. HW_{ov} = height of road above inlet invert

d. If $HW_r \leq 0$, $Q_r = 0$; if $HW_r > 0$, determine C_d from Figure 9.8.

e. Determine length of roadway crest, L

f. Calculate Q_r

$$Q_r = C_d\, L\, (HW_r)^{1.5} \tag{9.12}$$

Step 11 Compute Total Discharge, Q_t:

a. Determine the total discharge Q_t from the equation: $Q_t = Q + Q_r$

b. Repeat the calculation process until $Q_t \approx Q_d$. Refer to HDS 5 for procedure to construct performance curves.

Step 12 Calculate Outlet Velocity, V_o and Depth, d_n:

If inlet control is the controlling headwater:

a. Calculate flow depth at culvert exit:

i. Use normal depth, d_n, or

ii. Use water surface profile

b. Calculate flow area, A.

c. Calculate exit velocity, $V_o = Q/A$.

If outlet control is the controlling headwater:

a. Calculate flow depth at culvert exit:

i. Use d_c if $d_c > TW$.

ii. Use TW if $d_c < TW < D$.

iii. Use D if $D < TW$.

b. Calculate flow area, A.

c. Calculate exit velocity, $V_o = Q/A$.

Step 13 Review Results

a. Compare alternative design with constraints and assumptions. If any of the following conditions are not met, repeat steps 5 through 12:

i. The barrel must have adequate cover;

ii. The length must be close to the approximate length;

iii. The headwalls and wingwalls must fit the site;
iv. The allowable headwater must not be exceeded; and
v. The allowable overtopping-flood frequency must not be exceeded.

Step 14 Plot Performance Curve

a. Repeat steps 6 through 12 with a range of discharges.
b. Use the following upper limit for discharge:
   i. Q_{100} if $Q_o \leq Q_{100}$;
   ii. Q_{500} if $Q_o > Q_{100}$;
   iii. Q_{max} = largest flood that can be estimated, if no overtopping is possible.

Step 15 Related Designs:

Consider the following options:

a. Tapered inlets, if the culvert is in inlet control and has limited available headwater;
b. Flow routing, if a large upstream headwater pool exists;
c. Energy dissipators, if V_o is larger than the normal V in the downstream channel (see Chapter 11 – Energy Dissipators);
d. Sediment or debris control storage for sites with sediment concerns (e.g. alluvial fans) or with other debris concerns (see FHWA Hydraulic Engineering Circular No. 9, *Debris Control Structures Evaulation and Countermeasures*, or *CDOT Erosion Control and Stormwater Quality Guide.*);
e. Fish passage or aquatic organism passage; and/or
f. Broken-back culverts (see Section 9.9).

Step 16 Documentation

a. See Chapter 4 – Documentation.
b. Prepare report and file with background information.

All plans for drainage culverts shall have the following information:

D.A.	acres
Q_d = cfs	design
DHW elevation	ft
AHW elevation	ft
Q_{100}	cfs
Q_r	cfs, O.T. when appropriate
HW elevation	ft
OTHW elevation	ft
QWR	cfs (water rights if applicable)

Where: *D.A.* = drainage area of the contributing basin; *Q* = the discharge associated with the frequency indicated by the subscript; *HW* = the headwater elevation associated with the indicated discharge; *DHW* = the design headwater elevation for the design discharge (not to inundate the roadway); *AHW* = allowable headwater elevation. The maximum headwater which can be tolerated due to non-hydraulic features (i.e., private buildings, roadway profile, flowline of ditches which would pass water into an adjacent basin); *O.T.* = overtopping flood. The discharge at the moment water begins to overtop the roadway or flows into an adjacent basin; *OTHW* = headwater elevation for the overtopping flood; QWR = adjudicated water rights.

The above information is not required for culverts 24 in or smaller and with design discharges less than 20 cfs. Only water-right and stage (*DHW*) are required for irrigation structures. This information is to be placed with the structure note on the plan sheet, or in a tabulation. On special-design culverts, this information is to be placed on the layout sheet. The above information is also required for culverts to be extended, unless pipe diameter is 24 in or less. See Chapter 4 - Documentation for additional requirements.

9.7 FLOOD ROUTING FOR CULVERT DESIGN

9.7.1 Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing, it is possible that findings from culvert analyses will be conservative. If the selected allowable headwater is considered acceptable without flood routing, then costly overdesign of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. Special considerations associated with culvert flood routing are discussed in the following subsections:

Culvert Replacement Applications - Normally, a smaller culvert may be used for the same headwater condition.

Environmental - Evaluating environmental concerns may be more realistic.

Flood Hazards - A routing culvert design may require less land for an upstream easement, and assessments of potential flood hazards may be more realistic.

Sediment - Estimation of sediment accumulation is required.

Limitations - Temporary storage must be available.

Culvert Replacement Applications

Improved hydrologic methods or changed watershed conditions are factors that can cause an older, existing culvert to be inadequate. A culvert analysis that relies on findings that ignore any available temporary storage may be misleading. A flood-routing analysis may show that what was thought to be an inadequate existing culvert is, in fact, adequate.

Often existing culverts require replacement due to corrosion or abrasion. This can be very costly, particularly where a high fill is involved. A less-costly alternative is to place a smaller culvert inside the existing culvert. A flood-routing analysis may demonstrate that this is acceptable where there is sufficient storage, as no increase in flood hazard results.

Environmental

With culvert flood routing, a more realistic assessment can be made where environmental concerns are important. The temporary time and extent of upstream ponding can be estimated. This allows environmental specialists to assess whether such ponding is beneficial or harmful to localized environmental features (e.g. fisheries, beaver ponds, wetlands, uplands).

Flood Hazards

Potential flood hazards increase upstream wherever a culvert increases the natural flood stage. Some of these hazards can conservatively be assessed without flood routing. However, some damages associated with culvert backwater are time-dependent, and thus require an estimate of depth versus duration of inundation. Some vegetation and commercial crops can tolerate longer periods of inundation than others, and to greater depths. Such considerations become even more important when litigation is involved.

Sediment

Complex culvert-sediment-deposition (“silting”) solutions require a sediment-routing analysis. This practice requires a time-flood discharge relationship or hydrograph. The flood hydrograph must be coupled to a flood-discharge, sediment-discharge relationship to route the sediment through the culvert site.

Limitations

There are situations where culvert sizes and velocities obtained through flood routing will not differ significantly from those obtained by designing to the selected peak discharge, ignoring any temporary upstream storage. This occurs where:

- There is no significant temporary pond storage available (as in deep incised channels);
- The culvert must pass the design discharge with no increase in the natural channel’s flood stage; and
- Runoff hydrographs last for long periods of time, such as with snowmelt runoff (or irrigation flows).

9.7.2 Routing Equations

In addition to the previous design equations (Section 9.5), the following routing equations must be used. The basic flood routing equation is:

$$I - O = \Delta S / \Delta t, \text{ or} \tag{9.14}$$

$$2S_1 / \Delta t - O_1 + I_1 + I_2 = 2S_2 / \Delta t + O_2 \tag{9.15}$$

For a finite interval of time, Δt, equation 9.14 can be expressed by:

$$\Delta S = Q_i \Delta t - Q_0 \Delta t \tag{9.16}$$

From these equations:

$$(I_1 + I_2)/2 = \Delta S / \Delta t + O_1/2 + O_2/2 \tag{9.17}$$

Where: $\Delta \boldsymbol{S} = S_2 - S_1$; S_1 = storage volume in the temporary pond at the beginning of the incremental time period Δt, ft^3; S_2 = storage volume in the temporary pond at the end of the incremental time period Δt, ft^3; Δt = incremental routing time interval selected to subdivide the hydrograph into finite time elements, *s*; I = average hydrograph inflow to the temporary pond during incremental time period Δt, ft^3/s; I_1 = instantaneous inflow to the temporary pond at the beginning of the incremental time period Δt, ft^3/s; I_2 = instantaneous inflow at the end of the time period Δt, ft^3/s; O = average outflow from the temporary pond during incremental time period Δt, ft^3/s; O_1 = instantaneous outflow at the beginning of the time period Δt, ft^3/s; O_2 = instantaneous outflow at the end of the time period Δt, ft^3/s.

9.7.3 Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained, (Data Collection Chapter) and the analysis of hydrology completed, including estimating a hydrograph (Hydrology Chapter). Once this essential information is available, the culvert can be designed.

Flood routing through a culvert can be time-consuming. It is recommended that appropriate software be used to route floods through a culvert to evaluate an existing culvert (review), or to select a culvert size that satisfies given criteria (design).

However, the designer should be familiar with the culvert flood-routing design process. This familiarization is necessary to:

- Recognize and test suspected software malfunctions;
- Circumvent any software limitations;
- Flood route manually where the software is limited; and
- Understand and credibly discuss culvert flood routing.

The Design Steps are outlined below.

Trial and Error

A multiple, trial-and-error procedure is required for culvert flood routing. In general:

- A trial culvert(s) is selected;
- A trial culvert discharge (outflow) for a particular inflow hydrograph time element is selected;
- Flood-routing computations are made with successive trial discharges until the flood-routing equation is satisfied;
- The hydraulic findings are compared to the selected site criteria; and
- If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

The analysis is simplified if multiple iterations are not necessary, if the culvert performance is in inlet control over the range of appropriate discharges, or if the tailwater is below critical depth for outlet control over the range of discharges.

Design Steps

The design steps are as follows:

Step 1 Hydrograph: Plot the selected design and review hydrograph as computed using the practices from the Hydrology Chapter. Select a time interval, Δt, for use in the flood-routing procedure, and subdivide the hydrograph into these increments.

Step 2 Discharge Curve: Using the practices in the Channel Chapter, compute and plot a stage-discharge curve for the downstream channel.

Step 3 Culvert Performance Curve: Compute and plot a culvert performance curve (headwater versus discharge) for the trial culvert(s). Where overtopping occurs, it is necessary that the upper end of this performance curve be adjusted to reflect this additional discharge. The additional discharge is computed using the weir equation, adjusted to reflect a roadway embankment and any downstream effects where a low roadway fill is involved. The performance curve is the spillway-discharge curve commonly used in reservoir routing.

Step 4 Stage-Storage Curve: Compute and plot a stage-storage curve for the temporary upstream pond.

Step 5 Initial Routing Step: Start with the first inflow hydrograph time increment:

- Determine the average hydrograph inflow and volume discharge corresponding to the first selected time increment.
- Recognizing that, with an increasing headwater, the temporary pond storage will generally reduce this inflow, select a trial outflow (from the culvert) discharge that is less than the inflow discharge.
- With this smaller outflow discharge, estimate the headwater from the trial culvert(s) performance curve.
- Use this headwater to estimate the storage volume corresponding to this headwater from the upstream stage-storage curve.
- Compute the outflow volume corresponding to the selected outflow discharge and the previously-selected time increment.
- Subtract this volume from the foregoing average inflow volume. This is the volume that would have to go into temporary upstream pond storage.
- Compare this volume with the volume corresponding to the previously-estimated headwater. If they are the same (or nearly so), proceed to the next step; if not, repeat this step using a different trial outflow discharge.

Step 6a Increasing *HW*: The procedure for subsequent routing steps where the headwater is increasing is similar to the initial routing step. The difference is in how storage is handled:

- Determine the average hydrograph inflow and volume discharge corresponding to the next selected time increment.
- Recognizing that, with an increasing headwater, the temporary pond storage will generally reduce this inflow, select a trial outflow discharge (from the culvert) that is less than the inflow discharge.
- With this smaller outflow discharge, estimate the headwater from the trial culvert(s) performance curve.
- Use this headwater to estimate the storage volume corresponding to this headwater from the upstream stage-storage curve.
- Compute the outflow volume corresponding to this selected outflow discharge and the previously-selected time increment.
- Subtract this volume from the foregoing average-inflow volume. This is the volume that would have to go into temporary upstream storage.
- Add this volume to the volume already in storage.
- Compare this total volume with the volume corresponding to the previously-estimated headwater. If they are the same (or nearly so), proceed to the next step. If not, repeat this step using a different trial outflow discharge.

Step 6b Decreasing *HW*: This procedure is similar to the routing steps for increasing headwater. The difference is, (1) the selected trial-culvert outflow discharge will be greater than the average inflow hydrograph discharge, and (2) the outflow volume is greater than the inflow volume, so the temporary pond storage volume will be decreasing:

- Determine the average hydrograph inflow and volume discharge corresponding to the next selected time increment.
- Recognizing that, with a decreasing headwater, the temporary pond storage will be decreasing, select a trial outflow (from the culvert) discharge that is larger than the inflow discharge.

- With this larger outflow discharge, estimate the headwater from the trial culvert(s) performance curve.
- Use this headwater to estimate the storage volume corresponding to this headwater from the upstream stage-storage curve.
- Compute the outflow volume corresponding to this selected outflow discharge and the previously-selected time increment.
- Subtract this volume from the foregoing average-inflow volume. This is the volume that would go into temporary upstream pond storage.
- Subtract this volume from the volume already in storage.
- Compare this total volume with the volume corresponding to the previously-estimated headwater. If they are the same (or nearly so), proceed to the next step; if not, repeat this step using a different trial outflow discharge.

Step 7 Criteria Check: Following (or during) the previous routing steps, compare the resulting headwater and outlet velocity, and temporary pond size and duration, with the corresponding criteria selected for the site. If there is a violation of these criteria, return to step 1 and select a larger trial culvert. A smaller trial culvert would be selected if there appeared to be a significant over-design.

9.8 TAPERED INLETS

9.8.1 General

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically-efficient throat section. A tapered inlet may have a depression, or fall, incorporated into the inlet structure or located upstream of the inlet. The depression is used to exert more head on the throat section for a given headwater elevation. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). Tapered inlets are not recommended for use on culverts flowing in outlet control because the simple beveled edge is of equal benefit.

Design criteria and methods have been developed for two basic tapered inlet designs: the side tapered inlet and the slope tapered inlet.

Tapered inlet design charts are available for rectangular-box culverts and circular-pipe culverts.

9.8.2 Side-Tapered

A side tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the side walls (Figure 9.17). The face section is approximately the same height as the barrel height, and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than 10% (1.1D). The intersection of the tapered sidewalls and the barrel is defined as the throat section.

There are two possible control sections, the face and the throat. HW_f, shown in Figure 9.17, is the headwater depth measured from the face-section invert, and HW_t is the headwater depth measured from the throat-section invert. The throat of a side tapered inlet is a very efficient control section. The flow contraction is nearly eliminated at the throat. In addition, the throat is always slightly lower than the face so that more head is exerted on the throat for a given headwater elevation.

The beneficial effect of depressing the throat section below the streambed can be increased by installing a depression upstream of the side tapered inlet. Figure 9.18 depicts a side tapered inlet with the depression contained between wingwalls. For this type of depression, the floor of the barrel should extend upstream from the face a minimum distance of *D*/2 before sloping upward more steeply. The length of the resultant upstream crest, where the slope of the depression meets the streambed, should be checked to assure that the crest will not control the flow at the design flow and headwater. If the crest length is too short, the crest may act as a weir-control section. The barrel is defined as the throat section.

9.8.3 Slope-Tapered Inlets

The slope tapered inlet, like the side-tapered inlet, has an enlarged face section with tapered sidewalls meeting the culvert barrel walls at the throat section (Figure 9.19). In addition, a vertical fall is incorporated into the inlet between the face and throat sections. This fall concentrates more head on the throat section. At the location where the steeper slope of the inlet intersects the flatter slope of the barrel, a third section, designated the bend section, is formed.

A slope-tapered inlet has three possible control sections: the face, the bend, and the throat. Of these, only the dimensions of the face and the throat sections are determined by the design procedures of HDS 5. The size of the bend section is established by locating it a minimum distance upstream from the throat, so that it will not control the flow.

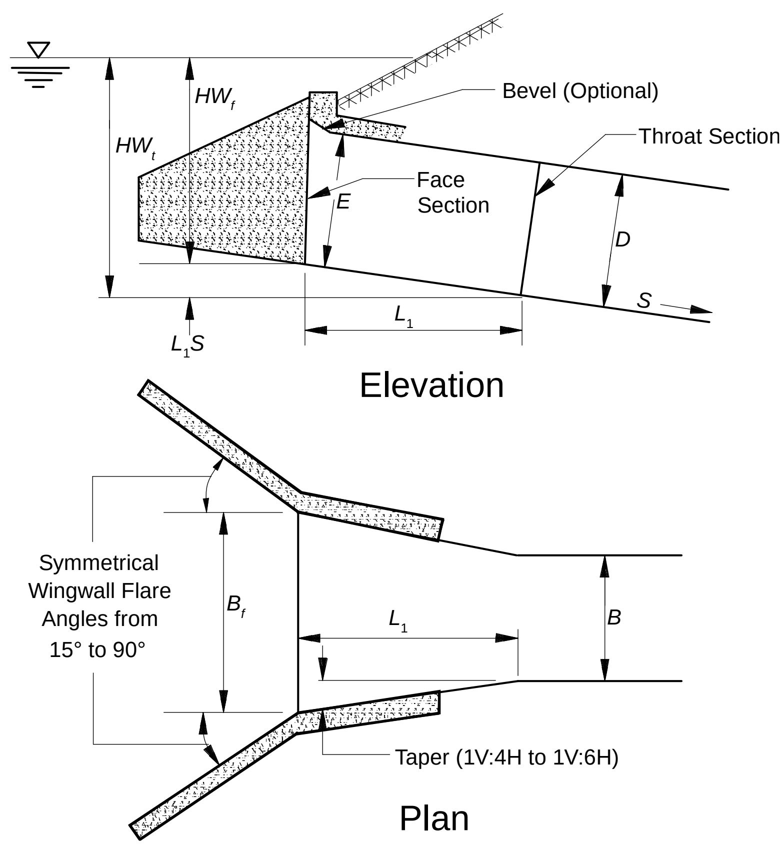


Figure 9.17 Side-tapered inlet

The slope-tapered inlet combines an efficient throat section with additional head on the throat. The face section does not benefit from the fall between the face and throat. The face sections of these inlets are larger than the face sections of equivalent depressed side tapered inlets. The required face size can be reduced by the use of bevels, or other favorable edge configurations. The vertical face slope-tapered inlet design is shown in Figure 9.19.

The slope-tapered inlet is the most complex inlet improvement recommended in this manual. Construction difficulties are inherent, but the benefits in increased performance can be significant. With proper design, a slope-tapered inlet passes more flow at a given headwater elevation than any other configuration. Slope-tapered inlets can be applied to both box culverts and circular-pipe culverts. For the latter application, a square-to-round transition is normally used to connect the rectangular, slope-tapered inlet to the circular pipe.

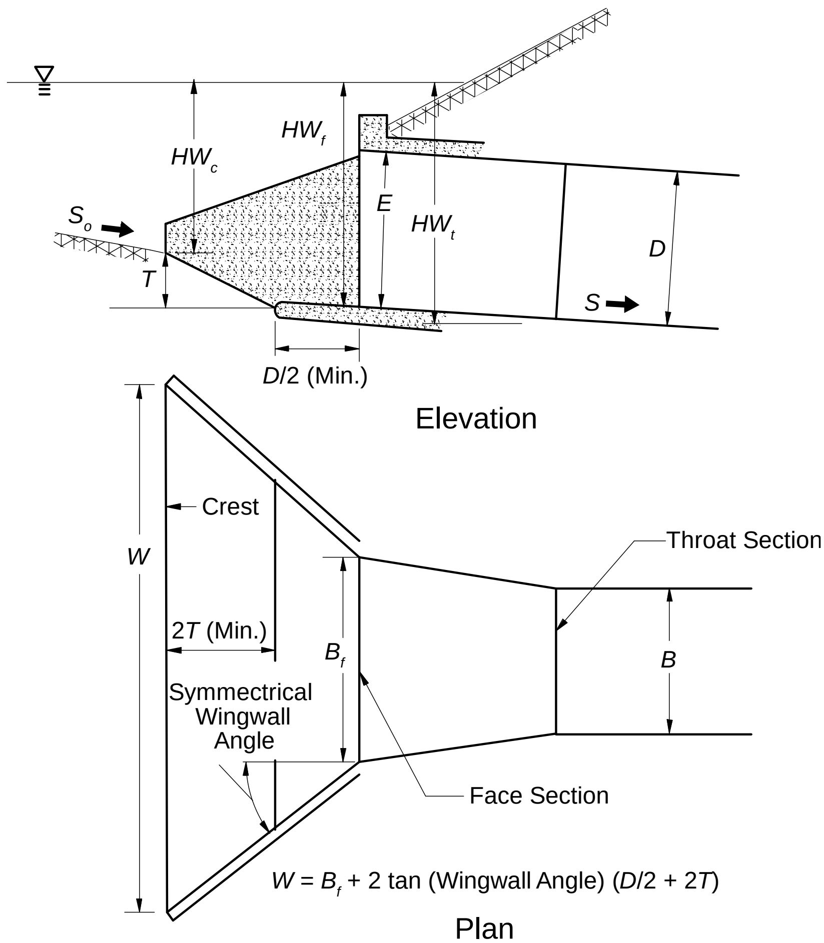


Figure 9.18 Side-tapered inlet with upstream depression contained between wingwalls

9.8.4 Hydraulic Design

Inlet Control

Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets), and the throat. In addition, a depressed side tapered inlet has a possible control section at the crest, upstream of the depression. Each of these inlet-control sections has an individual performance curve. The headwater depth for each control section is referenced to the invert of the section. One method of determining the overall inlet control performance curve is to calculate performance curves for each potential control section, then select the segment of each curve which defines the minimum overall culvert performance (Figure 9.20).

Side-Tapered Inlet: The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters. Because the throat is only slightly lower than the face, it is likely that the face section will function as a weir, or an orifice with downstream submergence within the design range. At lower flow rates and headwaters, the face will usually control the flow.

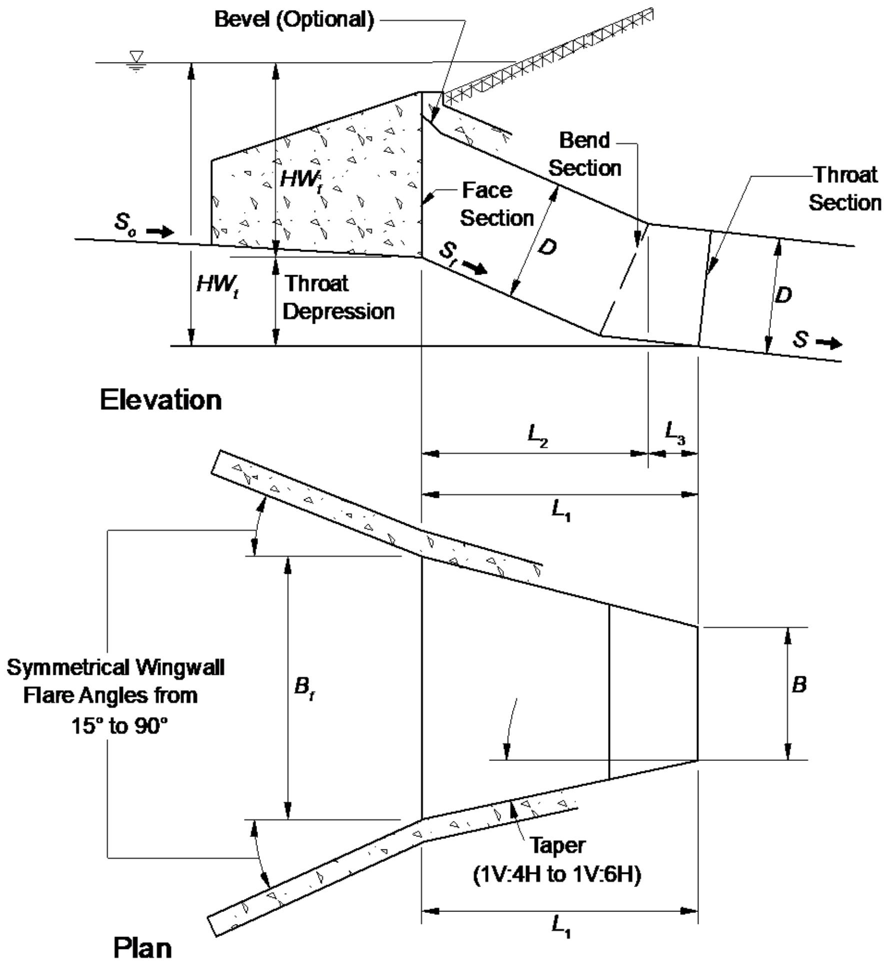


Figure 9.19 Slope-tapered inlet with vertical face

Slope-Tapered Inlet: The slope-tapered inlet throat can be the primary control section, with the face section submerged or unsubmerged. If the face is submerged, the face acts as an orifice with downstream submergence. If the face is unsubmerged, the face acts as a weir, with the flow plunging into the pool formed between the face and the throat. As previously noted, the bend section will not act as the control section if the dimensional criteria of HDS 5 are followed. However, the bend will contribute to the inlet losses that are included in the inlet loss coefficient, k_E.

Outlet Control

When a culvert with a tapered inlet performs in outlet control, the hydraulics are the same as described in Section 9.5 for all culverts. The tapered inlet entrance loss coefficient k_E is 0.2 for both side-tapered and slope-tapered inlets. This loss coefficient includes contraction and expansion losses at the face, increased friction losses between the face and the throat, and the minor expansion and contraction losses at the throat.

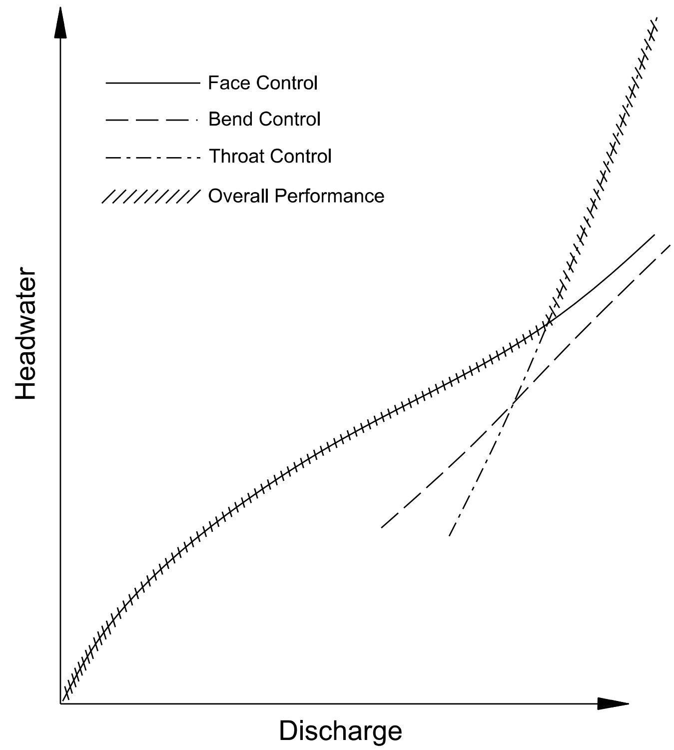


Figure 9.20 Inlet-control performance curves (schematic)

9.8.5 Design Methods

Tapered-inlet design begins with the selection of the culvert barrel size, shape, and material. These calculations are performed using the Culvert Design Form provided in the Appendix. The design procedure is similar to designing a culvert with other control sections (face and throat). The result will be one or more culvert designs, with and without tapered inlets, all of which meet the site's design criteria. The designer must select the best design for the site under consideration.

In the design of tapered inlets, the goal is to maintain control at the efficient throat section in the design range of headwater and discharge. This is because the throat section has the same geometry as the barrel, and the barrel is the most costly part of the culvert. The inlet face is then sized large enough to pass the design flow without acting as a control section in the design discharge range. Slight oversizing of the face is beneficial, because the cost of constructing the tapered inlet is usually minor compared with the cost of the barrel.

Performance Curves

Performance curves are of utmost importance in understanding the operation of a culvert with a tapered inlet. Each potential control section (face, throat and outlet) has a performance curve based on the assumption that the particular section controls the flow. Calculating and plotting the various performance curves results in a graph similar to Figure 9.21, containing the face control, throat control and outlet control curves. The overall culvert performance curve is represented by the hatched line. In the range of lower discharges, face-control governs. In the intermediate range, throat-control governs. And, in the higher discharge range, outlet-control governs. The crest and bend performance curves are not calculated because they do not govern in the design range.

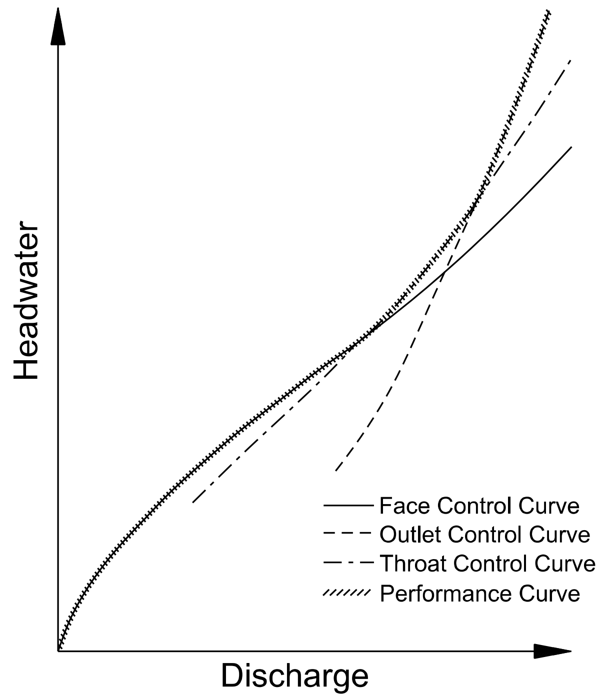


Figure 9.21 Culvert performance curve (schematic)

9.9 BROKEN-BACK CULVERTS

9.9.1 Introduction

An alternative to installing a steeply-sloped culvert is to break the slope into a steeper portion near the inlet followed by a horizontal-runout section. This configuration is referred to as a broken-back culvert. Broken-back culverts can be considered an internal (integrated) energy dissipater if designed so that a hydraulic jump occurs in the runout section to dissipate energy (see HEC 14).

9.9.2 Guidelines

The broken-back configuration is one potential mechanism for creating a hydraulic jump. Two types are depicted in Figures 9.22 and 9.23. When used appropriately, a broken-back culvert configuration can influence and contain a hydraulic jump. However, there must be sufficient tailwater and there should be sufficient friction and length in Unit 3 of the culvert. In ordinary circumstances for broken-back culverts, the designer should employ one or more devices, such as roughness baffles, to create a tailwater that is high enough to force a hydraulic jump.

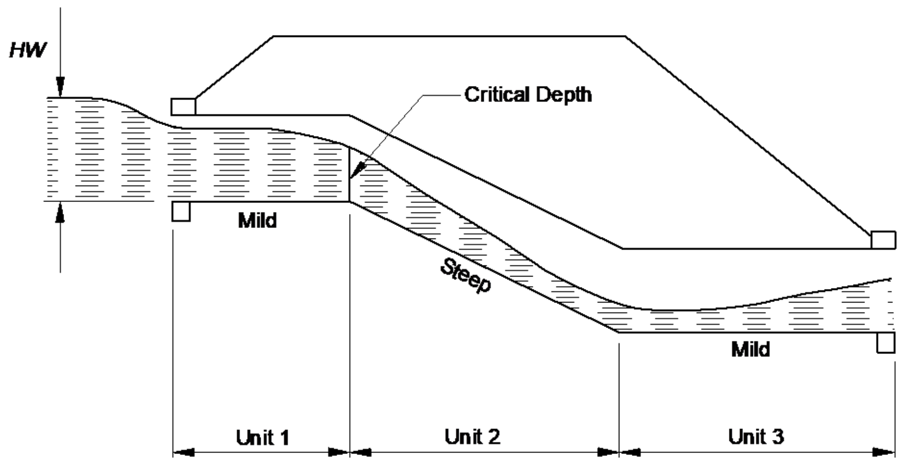


Figure 9.22 Three-unit broken-back culvert

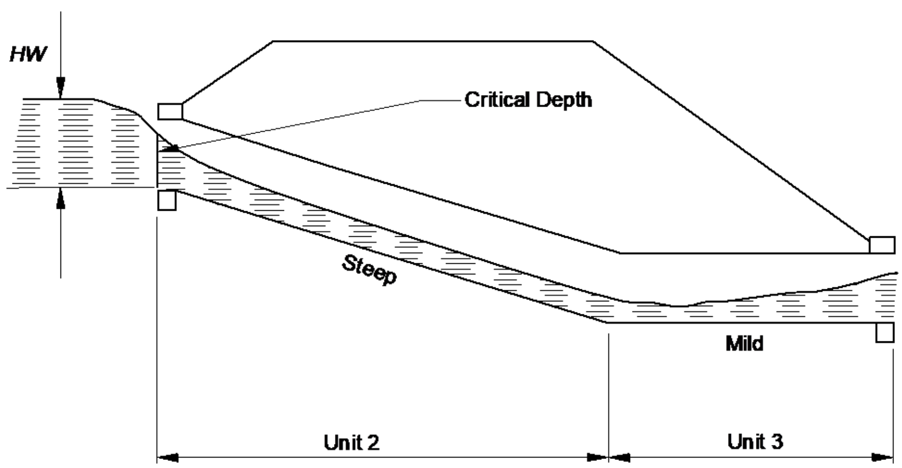


Figure 9.23 Two-unit broken-back culvert

9.9.3 Design Procedure

The design of a broken-back culvert is not difficult, but provisions must be made so that the primary intent of reducing velocity at the outlet is realized. The hydraulics of circular and rectangular culverts can be determined using the FHWA HY-8 software or the Broken-Back Culvert Analysis Program (BCAP) software from the Nebraska Department of Roads. The design of associated energy dissipators is described in HEC 14, Chapter 7.

9.10 CULVERT PROTECTION, REPAIR, REHABILITATION AND REPLACEMENT

This section summarizes the most commonly-available culvert protection techniques and materials, and discusses rehabilitation, repair and replacement methods available to extend culvert service life. This section was based largely on the National Cooperative Highway Research Program (NCHRP) Synthesis 474, *Service Life of Culverts*. NCHRP 474 should be consulted for additional information on these topics.

The processes discussed in this section will aid in:

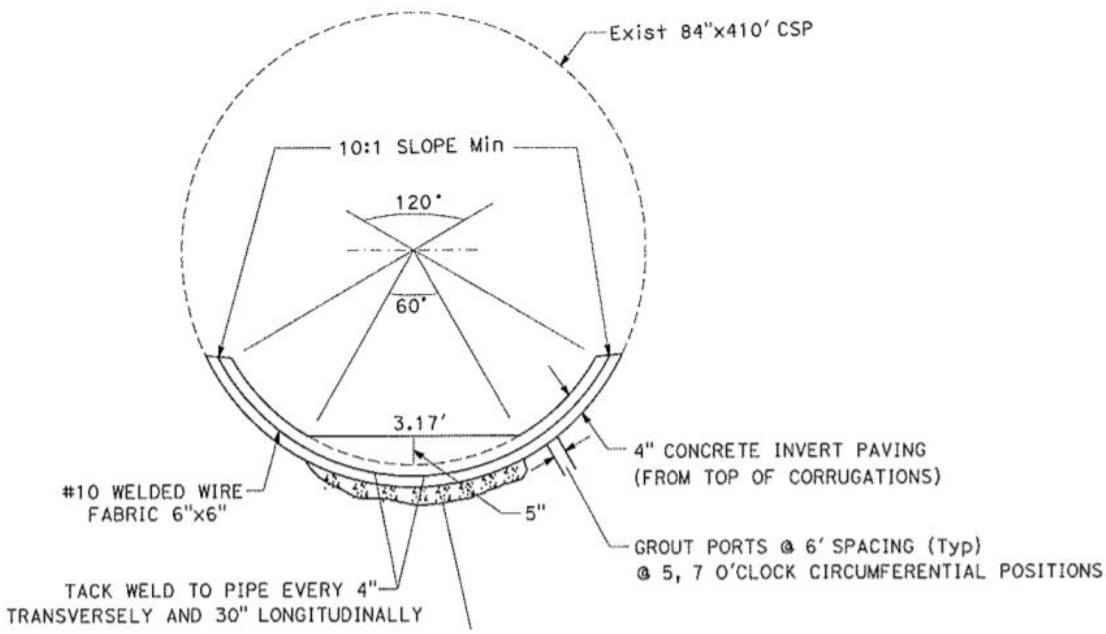
- Ivert paving;
- Sliplining;
- In-situ cured liners; and
- Pipe replacement.

9.10.1 Coatings, Linings, and Paving

Invert Paving (Concrete)

Predominantly used with metal culverts to increase abrasion resistance, concrete can be placed in the invert area of the pipe to a thickness between 3 and 6 in. Thickness and width of coverage varies with typical flow depth and anticipated abrasive potential. Although the concrete may be placed directly on clean pipe material, steel reinforcement, wire fabric, Nelson Studs, or a combination of these are frequently welded to the metal pipe prior to concrete placement.

Although concrete paving is utilized to rehabilitate corroded and severely deteriorated inverts in corrugated metal pipes, it can also be used in concrete culverts if adjustments are made (Figure 9.24). This method consists of placing a concrete lining in the invert, increasing surface roughness (Manning's *n* value), and decreasing flow velocity. The invert paving sections commonly vary from 90° to 180° for the internal angle, depending on the extent of deterioration on both sides of the slope.



Source: Caltrans Supplement to FHWA Culvert Repair Practices Manual 2013

Figure 9.24 Standard detail for concrete invert paving

Concrete invert paving is generally regarded as a temporary repair. However, a survey undertaken by the Minnesota DOT identified a case study where paving had lasted longer than 25 years. Additionally, the Ohio DOT assumes a 20-year add-on service life with the use of concrete paving. The main performance factors include the use of high-strength concrete with durable aggregate, and ensuring that the concrete is sufficiently anchored to the pipe.

In California, the use of steel armor plating 0.25 to 0.50 inches thick, placed over the bottom third of the pipe, is used as an alternative to concrete invert paving. This technique is costly, and mainly suitable for large-diameter (diameter greater than 48 in) corrugated metal pipe with highly abrasive water flows.

Epoxy Coatings for Concrete

A wide variety of epoxy-coated treatments can be applied to protect and extend the life of concrete culverts. These coatings are typically sprayed on, and are suitable for treating minor deterioration of exposed concrete surfaces (i.e. pop-outs, minor scaling, hairline cracks, etc.). However, use of epoxy coatings is not appropriate for severely deteriorated concrete (i.e. where steel reinforcement is exposed).

Epoxy coatings are hard and bond well to property cleaned and prepared concrete. These types of coatings should be regarded as maintenance treatments, but can help retard some forms of concrete degradations, and provide additional service life in appropriate applications.

9.10.2 Rehabilitation and Repair Practices

Pipe rehabilitation and repair technologies are discussed in detail in the literature review of NCHRP Project 14-19 *Culvert Rehabilitation to Maximize Service Life While Minimizing Direct Costs and Traffic Disruption.* NCHRP 14-19 should be consulted for additional information on these topics.

Lining an Existing Pipe

Sliplining: Sliplining is a method of rehabilitation where a new pipe of smaller dimeter is inserted into the deteriorated culvert. The annular space between the host pipe and newly installed pipe is then grouted with cementitious material.

Limitations of sliplining include:

- Need for pit excavation;
- Grouting of annular space (generally required);
- Flow-capacity reduction associated with decrease in cross-sectional area (however, a smooth interior surface of the slipline pipe could restore or increase flow capacity);
- Potential for increased velocity both in-pipe and downstream; and
- Need for sufficient work area.

Correctly sliplined culverts should provide the full service life associated with the type of pipe used in the sliplining. Sliplining is generally equivalent to full pipe replacement in terms of service life.

Spirally-Wound Liner: Spirally-would liners are fabricated in the field from a continuous thermoplastic strip composed of one male and one female edge (Photo 5). During the helical winding process, female and male edges interlock to form a leak-tight joint. Spirally-wound liners typically use nonstructural grout in annular space, or no grout at all.

Source: Caltrans Supplement to FHWA Culvert Repair Practices Manual 2013

Photo 5 Spirally-wound pipe liner being installed

Advantages of spirally-would liners include:

- Elimination of the need for excavation, on-site pipe sorage, and flow diversion;
- Relatively quick installation;
- No chemical processes associated with liners that require grouts and sealants; and
- Spirally-wound liners are applicable with significant radius bends and diameter changes.

Limitations of spirally-wound liners include:

- They are only applicable to circular pipes;
- They are not recommended for use in high-abrasion applications;
- Flow-capacity reduction associated with decrease in cross-sectional area (however, a smooth interior surface of slipline pipe could restore or increase flow cacity); and
- Required watertight sealing at the end of the relined pipe.

Caltrans uses spirally-wound liners for both flexible and rigid pipes to provide a corrosion barrier suitable to meet a 50-year design service life for abrasion levels 1 through 3 (see NCHRP Synthesis 474).

Sprayed-On Liner (Epoxy): Sprayed-on epoxy is typically used for rehabilitation of potable water pipes, but can also be used to line culverts. Sprayed-on epoxy can be applied to protect against corrosion and to make the culvert watertight, and is generally applied manually. Application thickness is normally between 0.06 in and 0.25 in per application layer, with a minimum of two layers recommended.

Advantages of polymer-based coatings include:

- Ability to provide protection against corrosion;
- Excavation is not required for installation; and
- Some polymers provide structural enhancement.

Limitations associated with use of polymer-based coatings include the culvert must be completely free of water for installation, and extensive surface preparations may be necessary for successful application. Epoxy-lining systems are relatively new, no data on service life is currently available.

Cured-in-Place Pipe: Cured-in-place (CIP) relining is a method in which a flexible material (commonly a tube) saturated with thermosetting resin is inserted into the culvert by inversion or wrenching. It is then expanded using air or water pressure, and the resin cured at ambient or increased temperature (using steam or hot water) or with UV light. The resulting product has minimal or no annular space, eliminating the need for grouting. Typically, cured-in-place liners range from 0.2 in to 0.5 in thick.

CIP liners can be conventional or composite. Composite CIP liners are high-strength fiber-reinforced CIP liners. Fiber reinforcement provides increased stiffness and strength, resulting in thinner walls as compared to conventional CIP liners, and can be used to rehabilitate medium to large sewers, drains, and culverts.

Advantages of CIP liners include:

- Elimination of the need for excavation and grouting;
- Installation of continuous jointless products;
- Structural renewal;
- Cost-effective; and
- The process causes minimal traffic distruption.

Limitations of CIP liners include:

- The culvert must be completely free of water (flow diversion may be required);
- A custom tube is required for each installation;
- Trained personnel are required;
- A prolonged cure is required for large diameters;
- CIP liners can cause thermal pollution if hot water is used to accelerate resin cure; and
- CIP liners can damage the environment if styrene-based resins are used.

CIP liners have an expected service life of 50-years.

9.10.3 Pipe Replacement

Numerous trenchless technologies for pipe replacement exist, including jack and bore (see Appendix B), tunneling, and horizontal directional drilling. These methods are not discussed in this section, but the pipe bursting/splitting method is discussed because it reuses the same alignment of the existing culvert.

Pipe Bursting: Pipe bursting is defined as a trenchless replacement method in which an existing pipe is broken either by brittle fracture or by splitting, using an internal mechanically applied force applied by a bursting tool.

HDPE pipe is most commonly used pipe in pipe bursting replacement. Since HDPE pipes are chemically inert, they can readily flex to meet changes in loading along the culvert length while maintaining circular shape. Additionally, fusible PVC pipe, retained jointing PVC pipe, ductile iron pipe, and vitrified clay pipe can be installed using pipe bursting.

Pipe bursting can replace circular pipes up to 54 in in diameter. The length typically is limited to 750 ft. Applicability is not limited by culvert pipe type or condition. Replacement can be performed in active flow conditions. Most bursting projects involve pipes that were originally installed by trenching or open cut because the fill material surrounding them is usually conducive to pipe bursting. The potential for upsizing through pipe bursting depends on soil conditions, overburden cover, and other factors.

Advantages of pipe bursting replacement include:

- Installation of new pipe;
- Ability for pipe upsizing; and
- Reduction of required excavation by 85% or more when compared with open-cut replacement.

Limitations of pipe bursting replacement primarily consist of difficulties that arise when existing pipe composed of brittle material has had point repairs with ductile material. Pipe bursting can cause ground heave or settlement above or at a distance from the culvert. This occurs particularly in dense sand, when the culvert pipe is shallow and ground displacement is primarily directed upwards, cause by significant diameter increases. Additionally, pipe bursting is not applicable when the host pipe has significant sagging or deviation from the original grade.

The service life for pipe replacement by pipe bursting is expected to be equivalent to that for the replacement pipe material.

9.11 SOFTWARE FOR DESIGNING CULVERTS

Computer software is available for computing the hydraulics of culverts and energy dissipators in open channels that follow the procedures outlined in this chapter and Chapter 11 - Energy Dissipators. The primary software used is HY-8. This software assumes a headwater pool at the entrance (no approach velocity), and that all the velocity head is loss at the exit (standard method), or conserved (Utah State University method). If the culvert being designed is for a constructed channel where energy conservation is important, the HEC-RAS software should be used. If the site conditions warrant the use of a broken-back culvert, the HY-8 software should be used.

The software versions shown in Table 9.7 are the most recent when this manual was prepared. For current versions of software and documentation, the hydraulic engineer should consult the software source.

Available user and reference manuals are listed in the references. However, many software developers provide extensive help within the software in place of a user’s manual.

Table 9.7 Software for Designing Culverts

Software Name	Features	Source
HY-8	The FHWA Culvert Hydraulic Analysis Program (HY-8) enables users to analyze culvert performance for a highway crossing that has multiple culvert barrels that share the same tailwater, and roadway overtopping. HY-8 uses HDS-5 procedures and includes: • Culvert configurations with constants in HDS-5 Appendix A: i. Table A.1 - circles, boxes, and tapered ii. Table A.2 - pipe-arches, ellipses, metal boxes, and arches iii. Table A.3 SDDOT - RCB iv. Table A.4 CON/SPAN v. Table A.5 embedded circular vi. Table A.6 embedded elliptical vii. User-defined shapes use Chart 52 • Embedded or depressed invert design; • Utah State University outlet transition loss; • Energy dissipator analysis (HEC-14); • Internal hydraulic jumps; • Broken-back barrel analysis; and • Project documentation.	FHWA website
HEC-RAS	For culverts, HEC-RAS Hydraulic Reference Manual Chapter 6 references HDS-5 inlet control constants. HEC-RAS includes: • Culvert configurations with constants in HDS-5 Appendix A (Tables A.1, A.2, and A.4); • Embedded invert analysis; and • Inlet and outlet transition loss (e.g. coefficient times velocity head difference).	HEC website

REFERENCES

1. AASHTO, "Highway Drainage Guidelines," Chapter 4, *Hydraulic Design of Highway Culverts,* Task Force on Hydrology and Hydraulics, 2007.
2. AASHTO, *Roadside Design Guide*, Task Force on Roadside Safety, 2011.
3. Bodhaine, G. L., 1968, *Measurement of Peak Discharge at Culverts by Indirect Methods, U.S. Geological Survey Techniques of Water-Resources Investigations,* Book 3, Chapter A3.
4. California Department of Transportation, *Caltrans Supplement to FHWA Culvert Repair Practices Manual*, Design Information Bulletin No. 83-03, Sacramento, 2013.
5. Federal Highway Administration, "Debris Control Structures," Hydraulic Engineering Circular No. 9, FHWA-IF-04-016, Washington DC, 2005.
6. Federal Highway Administration, *HY-7, WSPRO, Bridge Waterways Analysis Model*, (Version P60188), 1999.
7. Federal Highway Administration, *HY-8*, *Culvert Hydraulic Analysis Program,* (Version 7.50), 2016.
8. Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels,* Hydraulic Engineering Circular No. 14, FHWA-NHI-06-086, 2006.
9. Federal Highway Administration, "Hydraulic Design of Highway Culverts," Hydraulic Design Series No. 5, FHWA-HIF-12-026, Washington DC, 2012.
10. Federal Highway Administration, Hydraulic Design Series No. 1, *Hydraulics of Bridge Waterways*, FHWA-EPD-86-101, 1978.
11. Federal Highway Administration, Hydraulic Design Series No. 3, *Design Charts for Open-Channel Flow*, FHWA-EPD-86-102, 1961.
12. Federal Highway Administration, "Urban Drainage Design Manual," Hydraulic Engineering Circular No. 22, FHWA-NHI-10-009, 2009.
13. King, H.W. and Brater, E.F., *Handbook of Hydraulics,* Sixth Edition, McGraw-Hill Book Co., 1976.
14. Transportation Research Board of the National Academies, NCHRP Project 14-19, *Culvert Rehabilitation to Maximize Service Life while Minimizing Direct Costs and Traffic Disruption*, Literature Review, Final draft, Washington D.C., 2010.
15. U.S. Bureau of Reclamation, *Design of Small Canal Structures,* Denver CO, 1978.
16. Wyoming Highway Department, "Culvert Design," Section 3-04, *Road Design Manual*, Cheyenne WY 82001, 2011.

APPENDIX A – DESIGN CHARTS

CDOT Culvert Design Data Form

COLORADO DEPARTMENT OF TRANSPORTATION

CULVERT DESIGN DATA

Existing structure

Station	State Highway	Designer	Project
			Station
Skew	Mile post		Skew
Size	Type	Date	Q (Design)= cfs

Discharge	Proposed structure			Structure sizing									End treatment and comments
	Size	Type	L	Flow line elevation		So	Sc	Control section*	HW/D	HW	DHW elevation	Vo	
				in	out								
								inlet					
								outlet					
								inlet					
								outlet					
								inlet					
								outlet					
								inlet					
								outlet					
Q_{100}								inlet					
								outlet					

Allowable headwater elevation (AHW)	AHW control location

Comments: (Discuss inlet and outlet conditions, channel changes, debris, possible corrosion, etc.)

Hydraulic information required to be placed on plans:

D.A.=

Q___= cfs

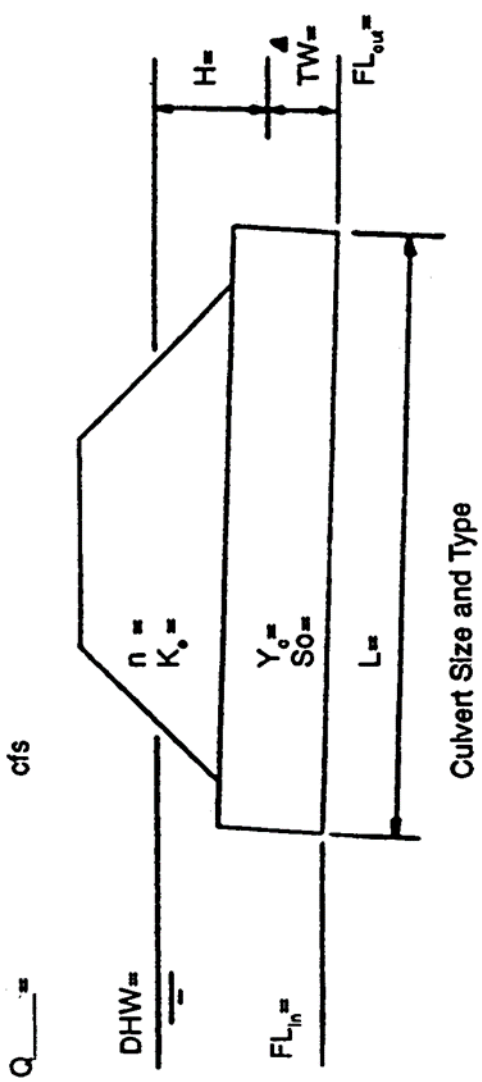
DHW=

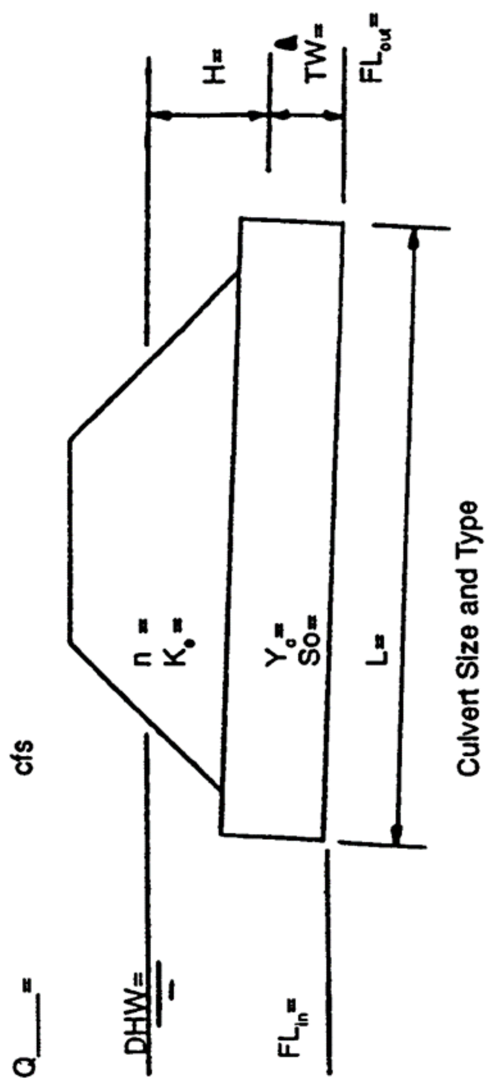
AHW=

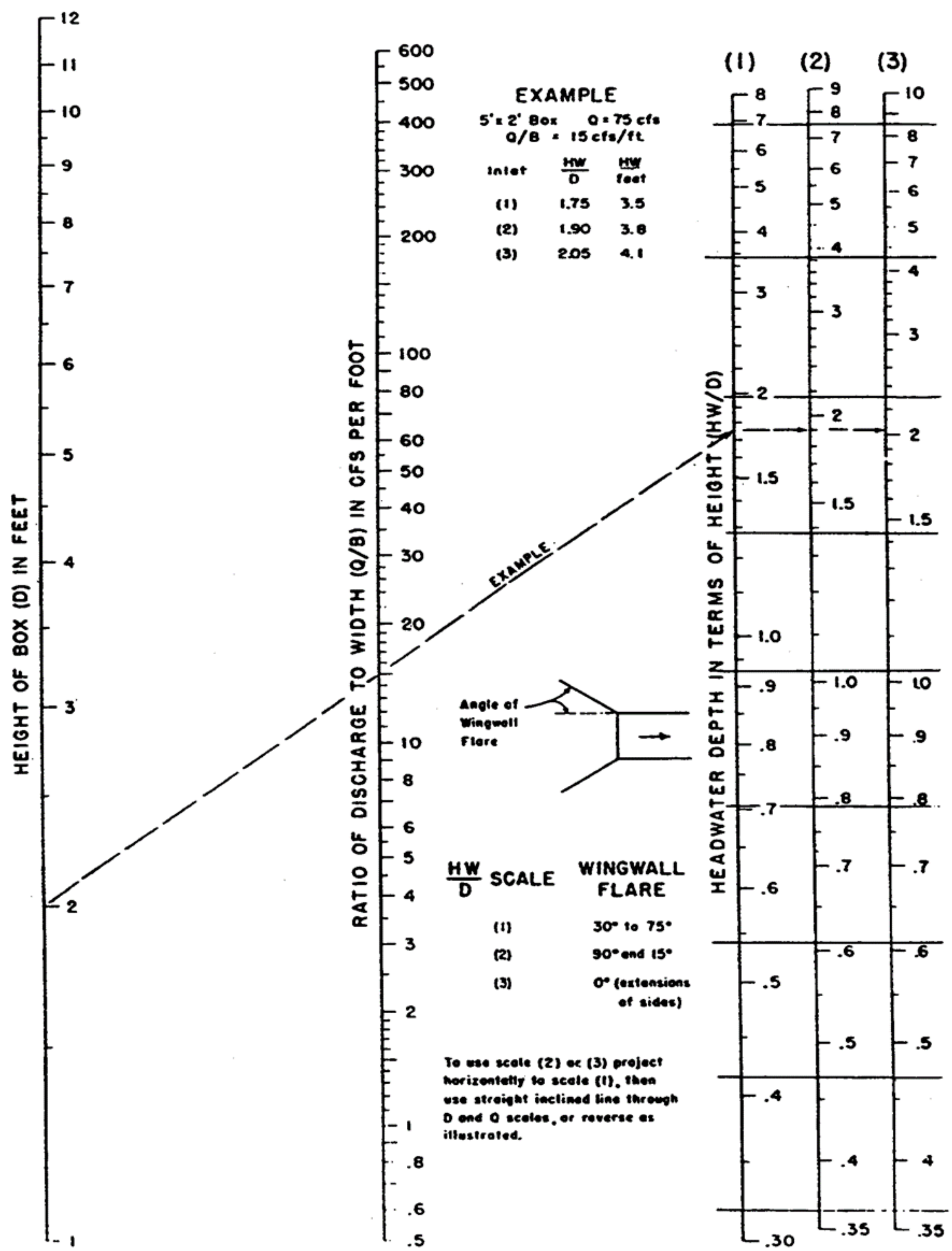
Q_{100}= cfs

HW=

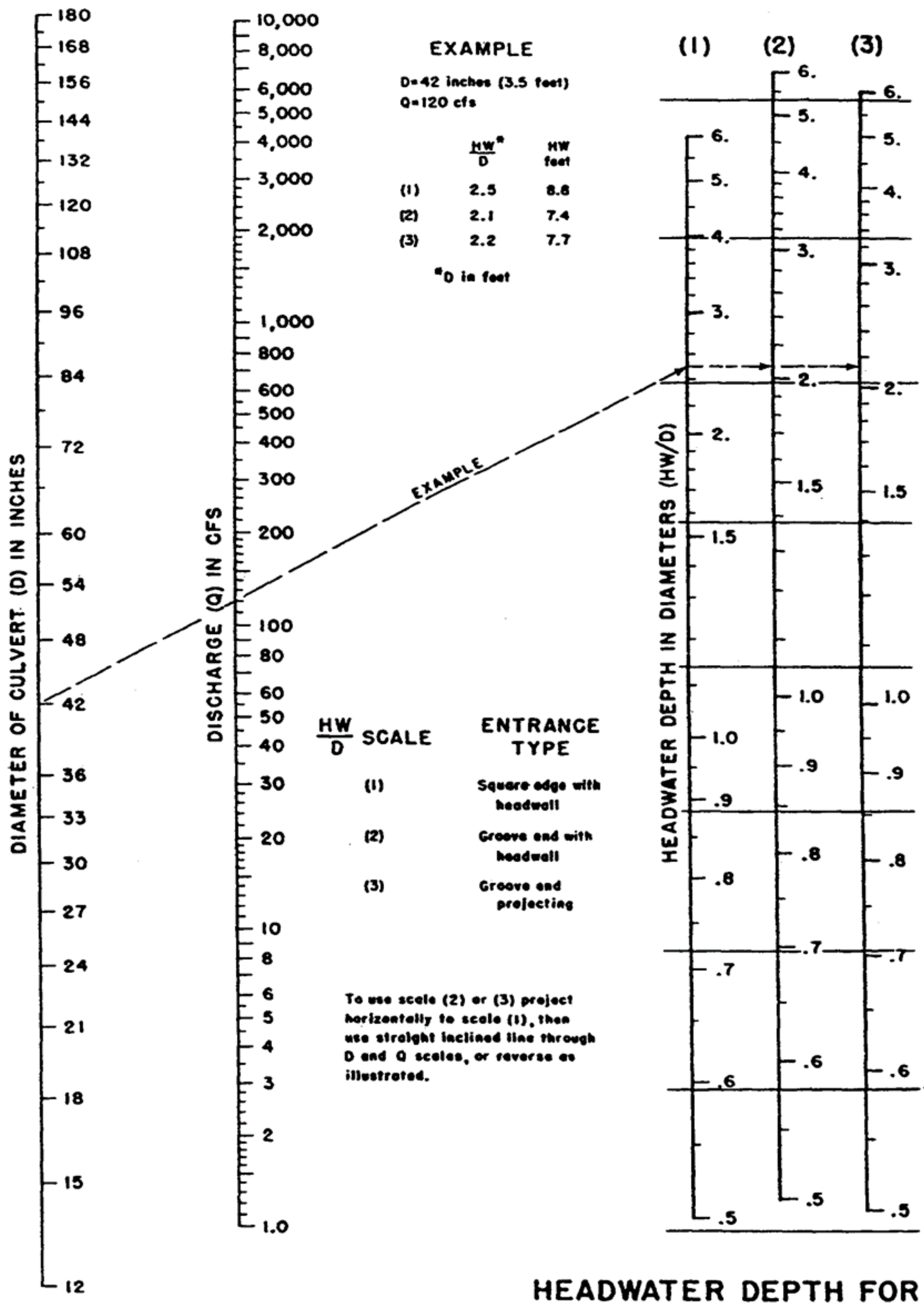
CDOT Culvert Design Data Form (Outlet Control Headwater Computations)



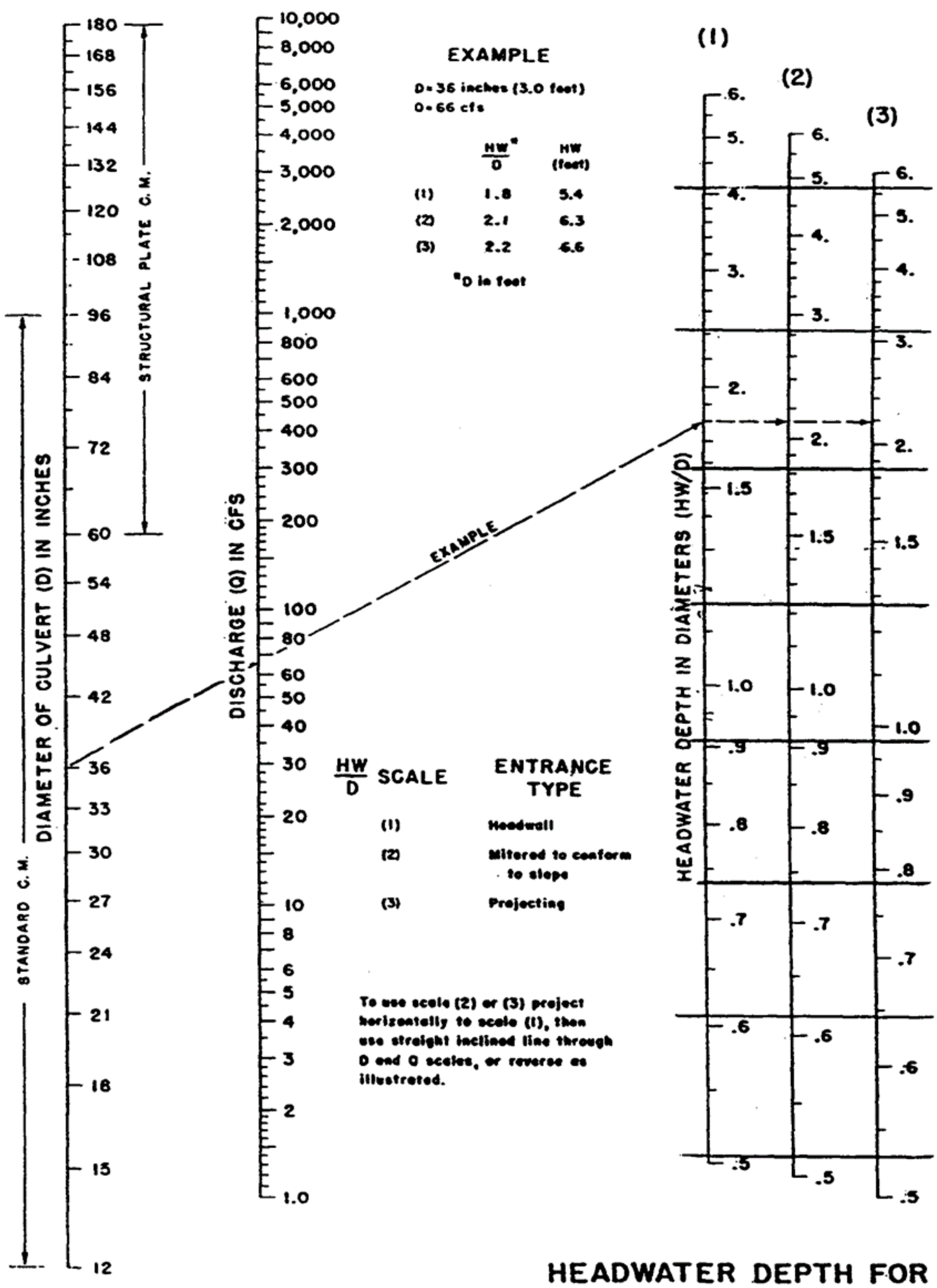




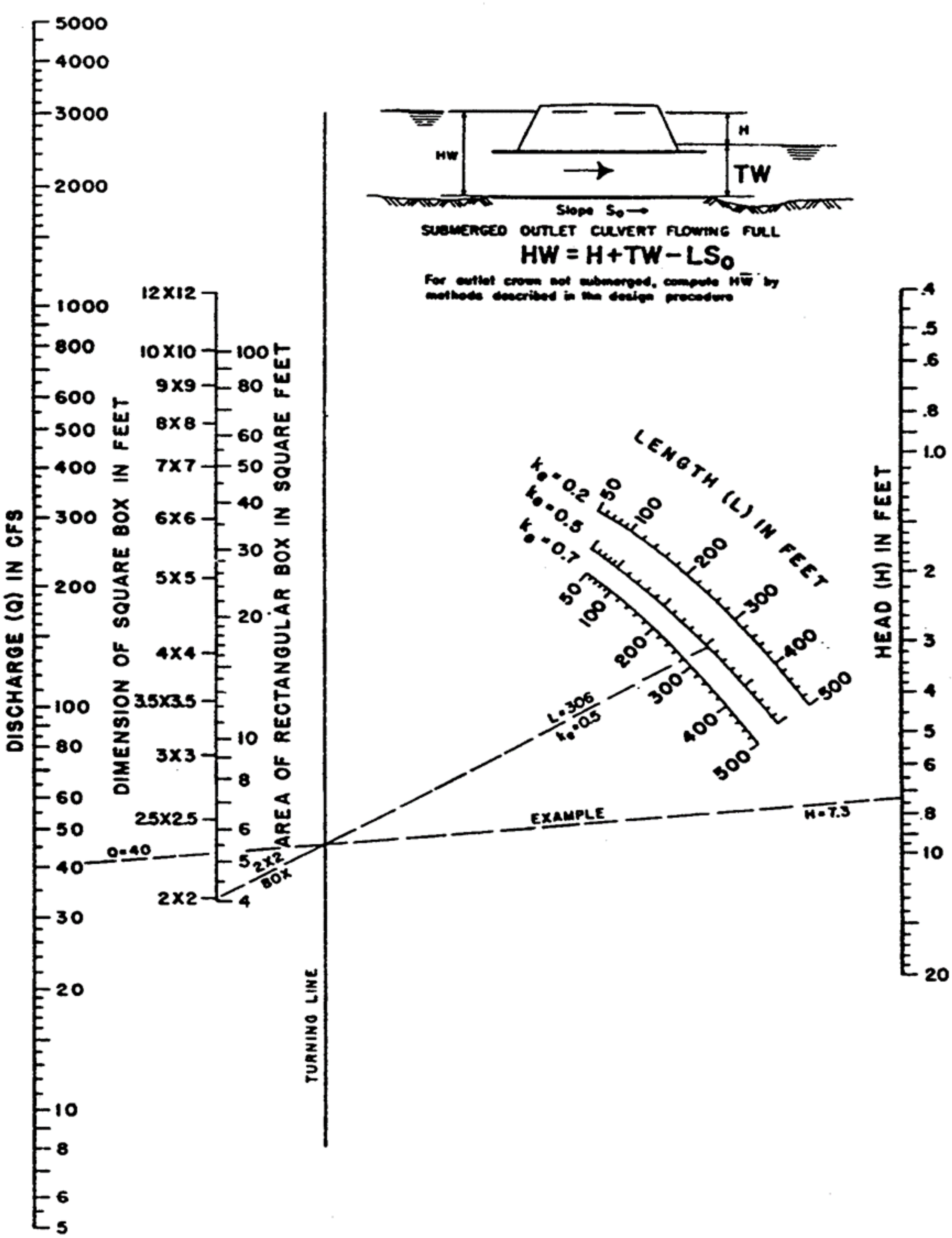
HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL



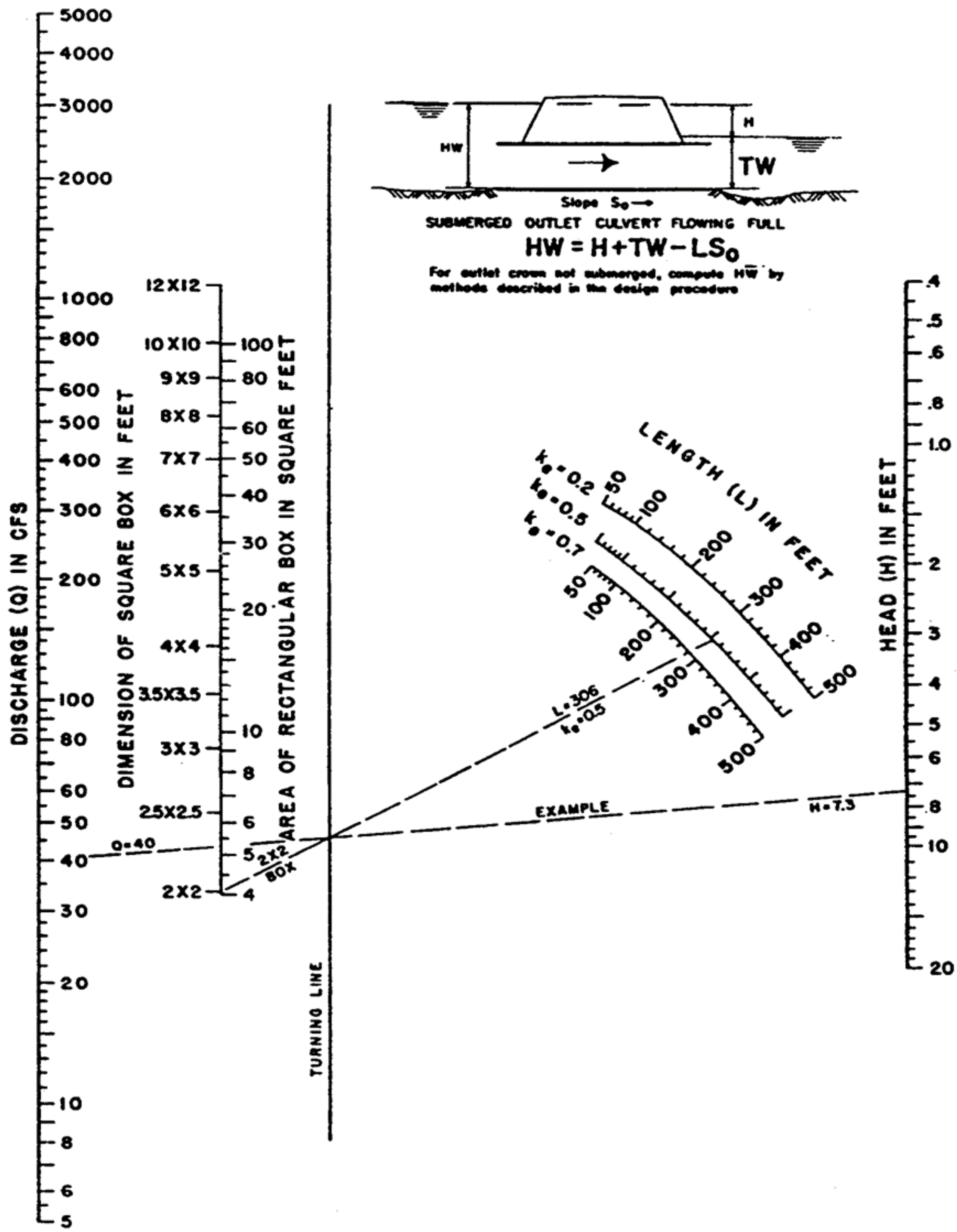
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL



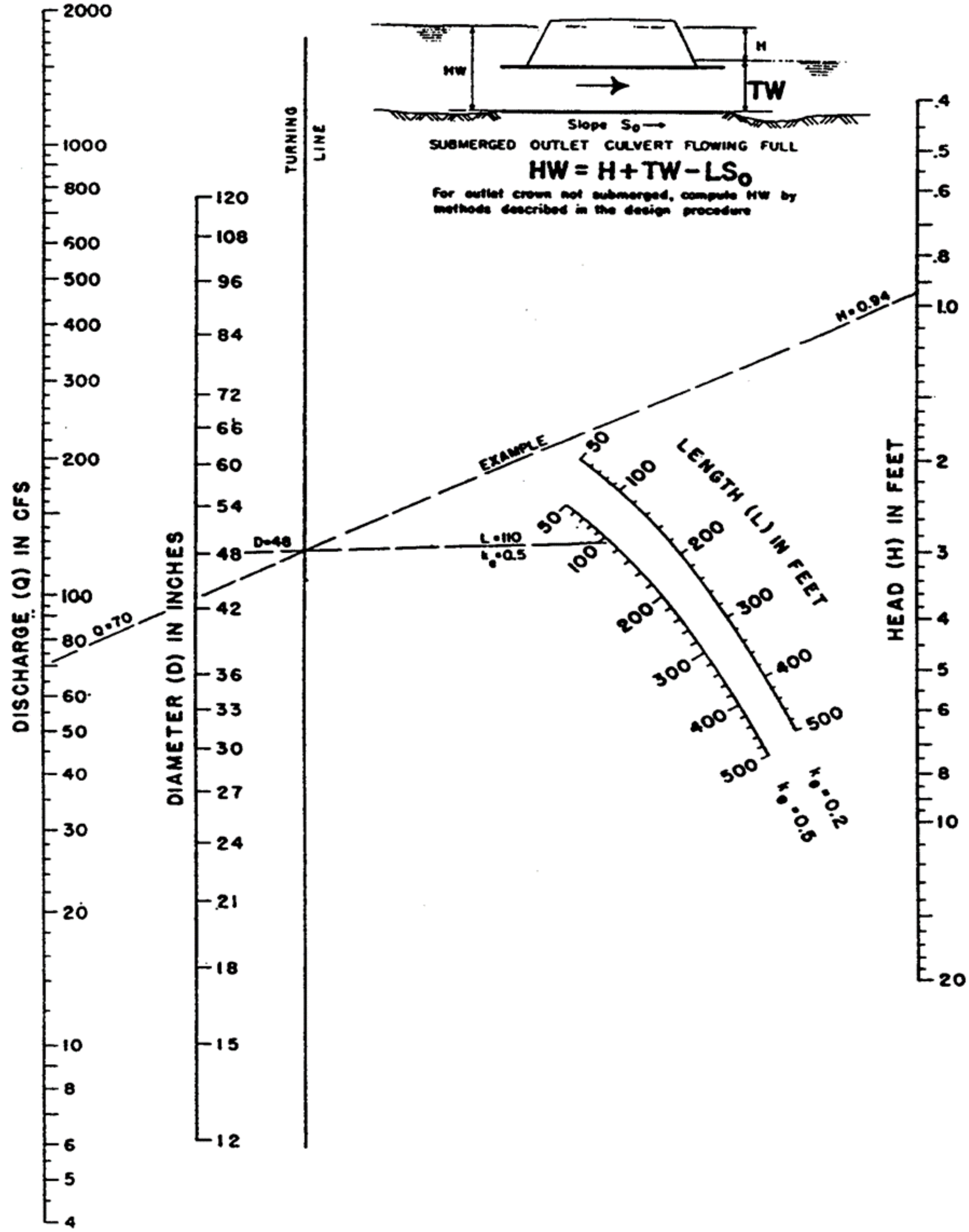
HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL



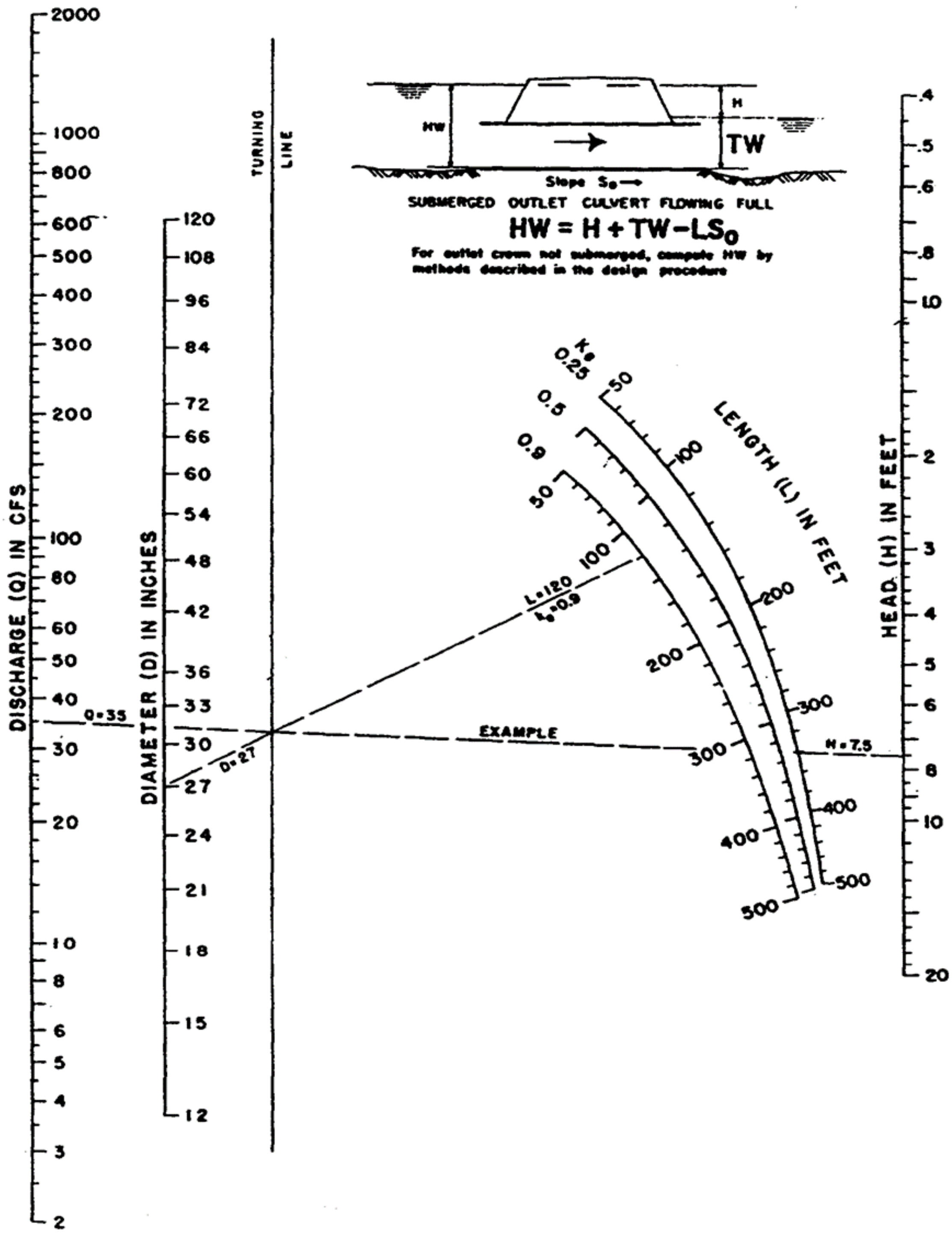
HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012



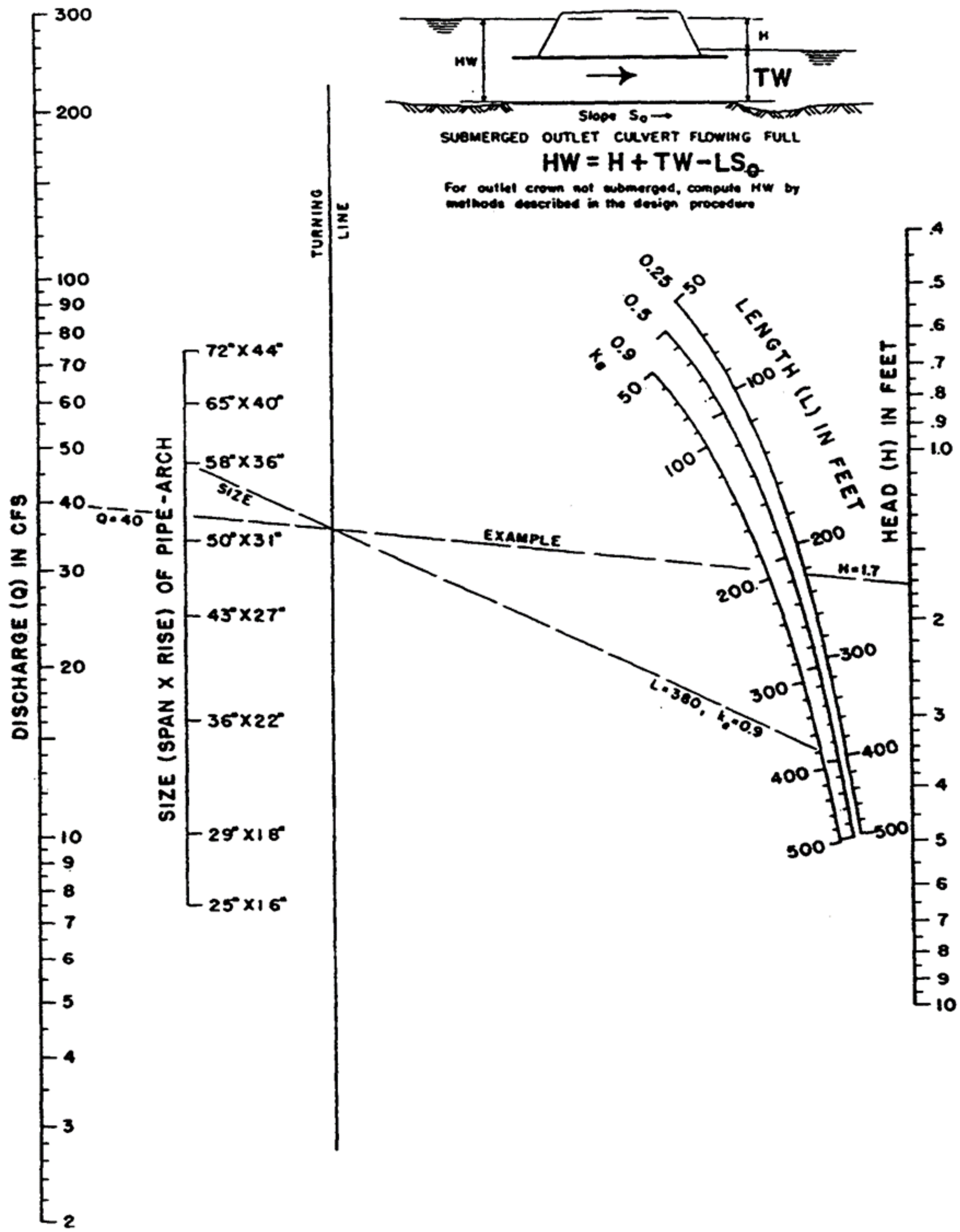
HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012



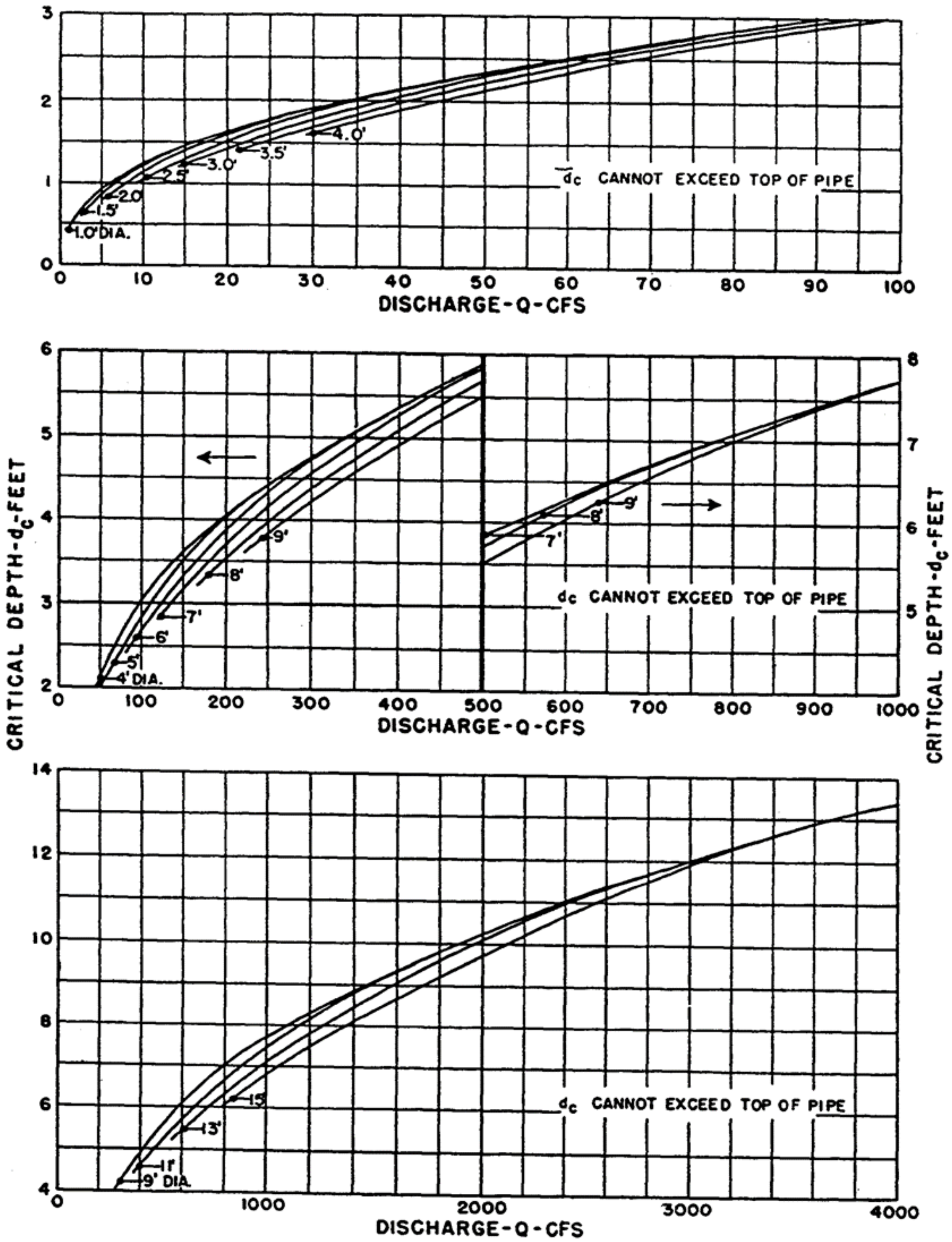
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
$n = 0.012$



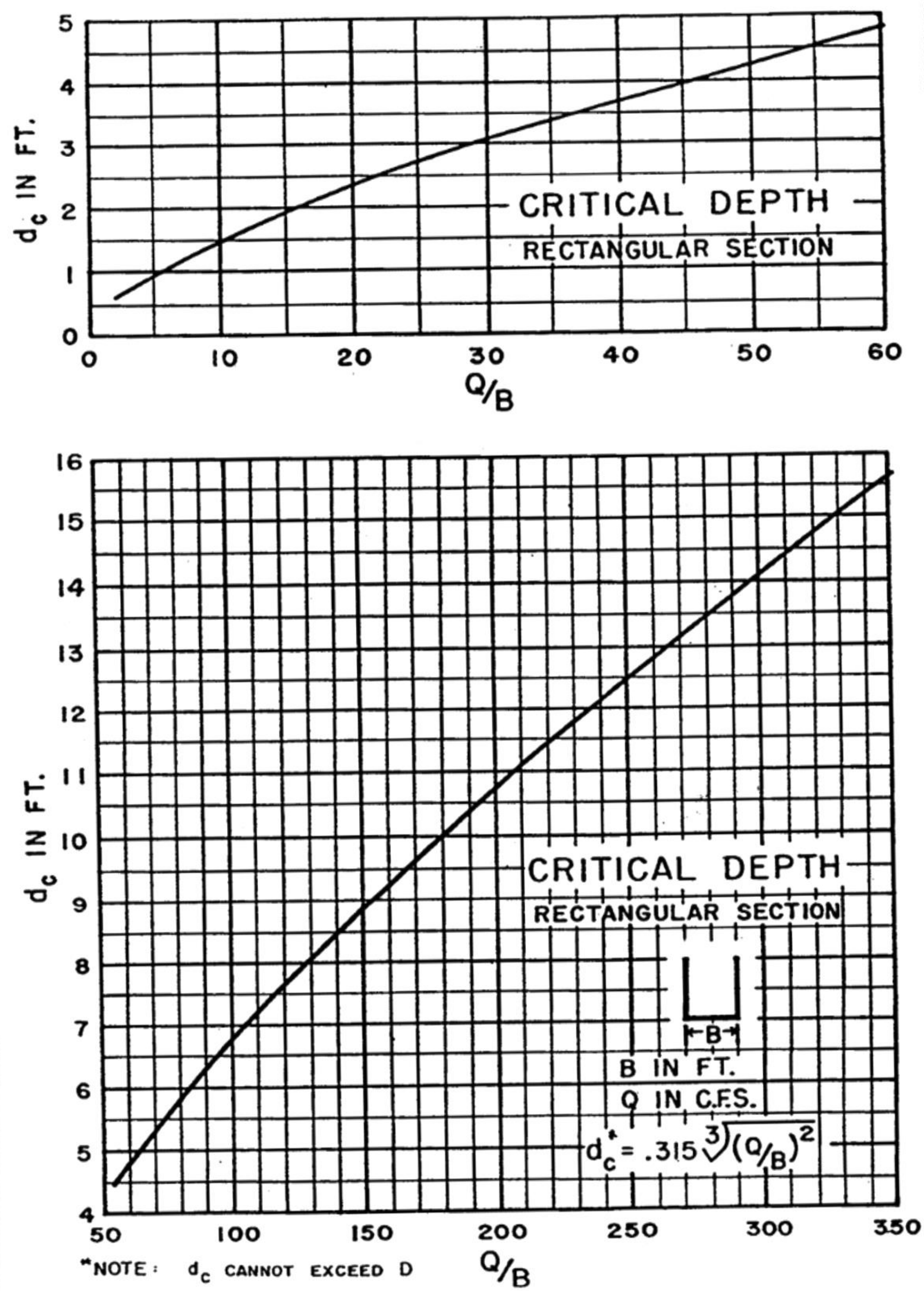
HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
$n = 0.024$

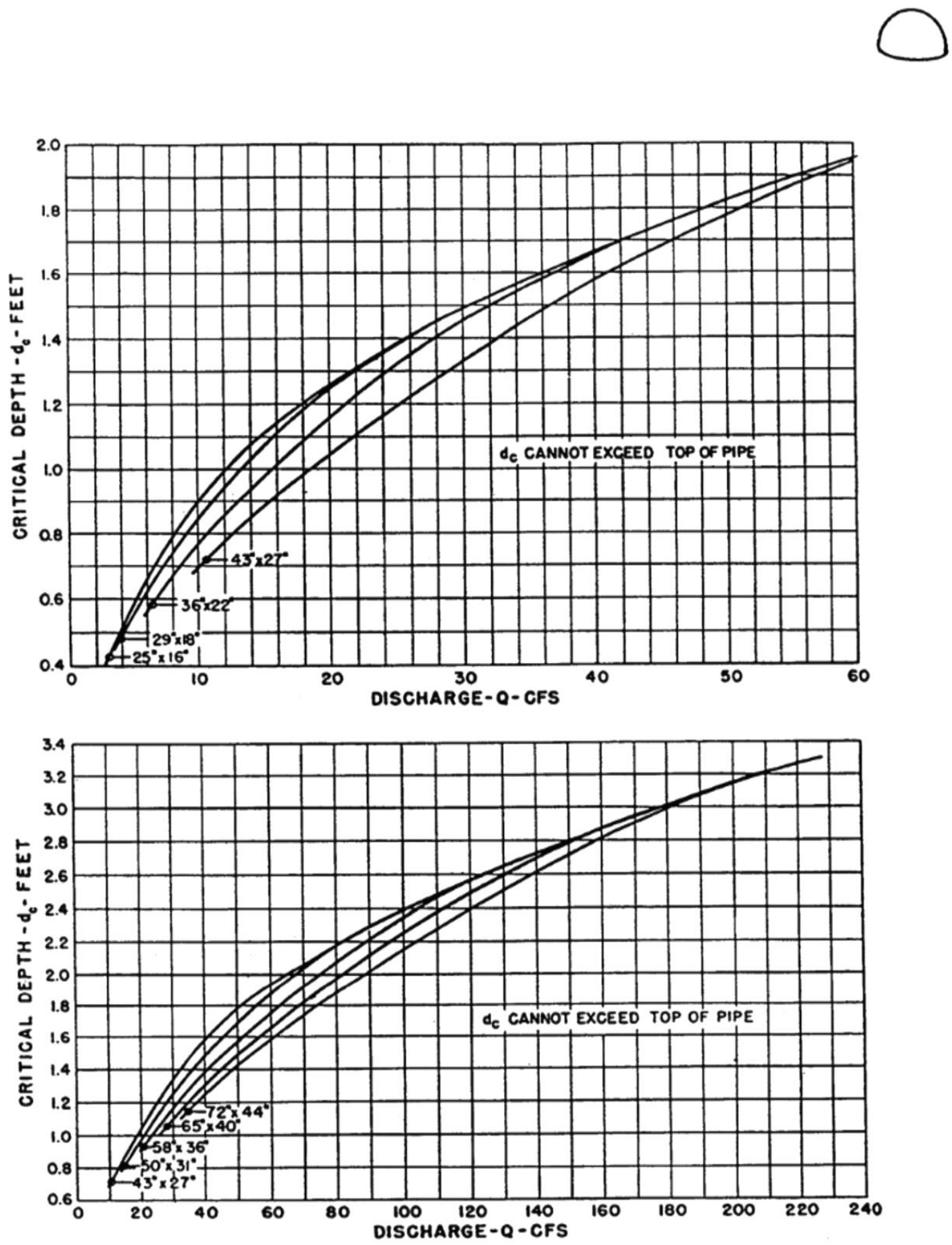


HEAD FOR STANDARD C. M. PIPE-ARCH CULVERTS FLOWING FULL n=0.024

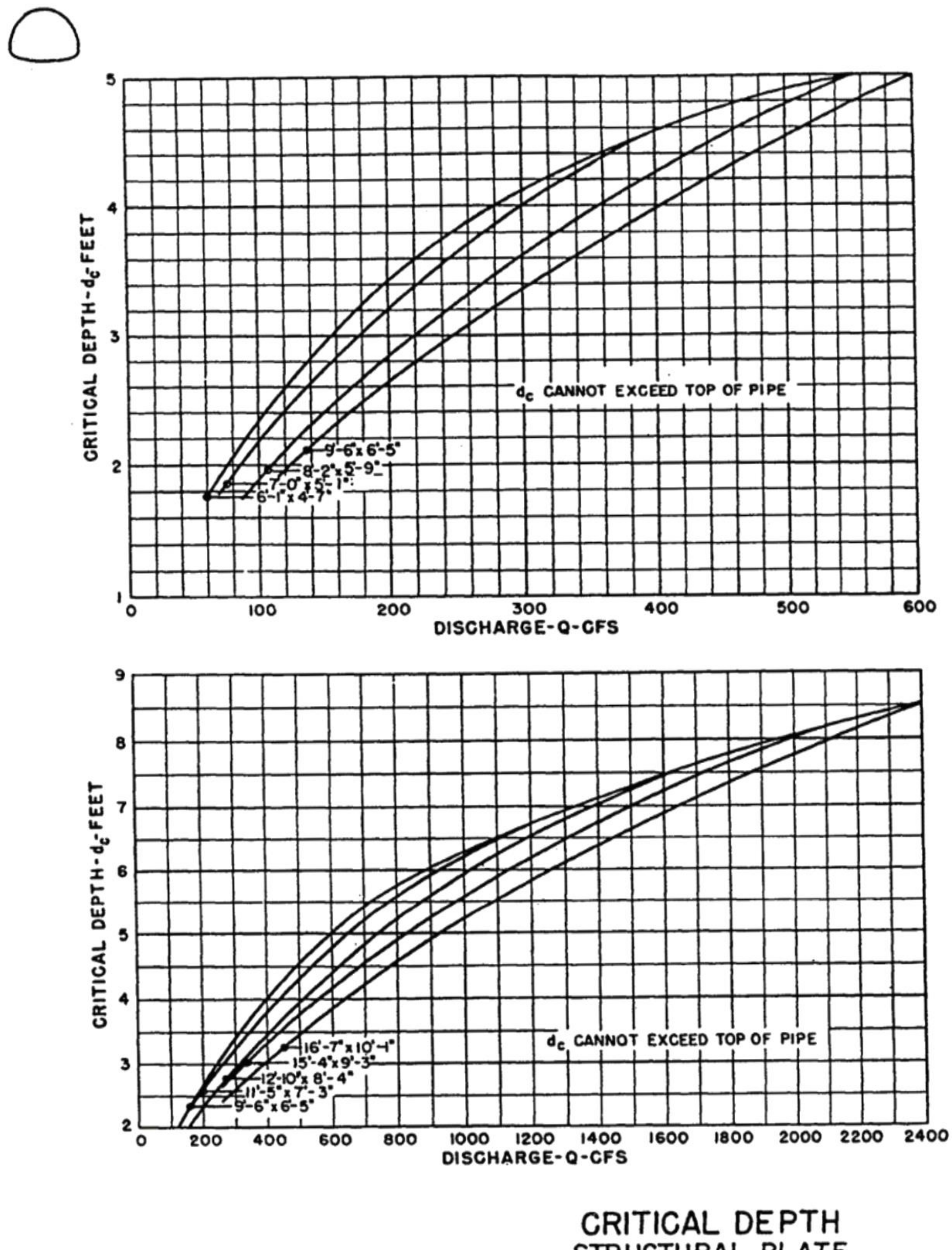


CRITICAL DEPTH
CIRCULAR PIPE


CRITICAL DEPTH
RECTANGULAR SECTION
d_c IN FT.
Q/B
CRITICAL DEPTH
RECTANGULAR SECTION
B
B IN FT.
Q IN C.F.S.
$d_c^* = .315\sqrt[3]{(Q/B)^2}$
d_c IN FT.
Q/B
*NOTE: d_c CANNOT EXCEED D



CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH



CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE–ARCH
18 INCH CORNER RADIUS

APPENDIX B - JACKING WELDED STEEL AND REINFORCED CONCRETE PIPE

Pipe Jacking and/or boring is done to install pipelines under roadways, railroads, runways, canals, etc., without interrupting surface traffic. The choice of jacking pipe either by boring out the inside of the pipe with rotary equipment, or by mining out the material within the pipe, is the option of the contractor performing the installation. Jetting with water is not allowed.

CDOT’s bid system does not differentiate between mining and boring. All pipes placed without trenching are identified in the bid items as jacked.

A jacked pipe may be used as a carrier pipe, or as a casing to protect a carrier pipe. Utilities such as electrical wiring, water, natural gas, petroleum products, and sewage are transmitted through the carrier pipe. The carrier pipe may be within a jacked casing pipe, allowing the utility pipe to be removed, repaired, or replaced from within the casing without disruption to traffic. Storm drainage generally is run directly through the carrier pipe.

Jacked pipe generally is reinforced concrete pipe, precast concrete boxes, or welded steel pipe. Thirty six inch diameter pipe is the smallest practical diameter for mining jacked concrete or steel pipe. The controlling criteria for the diameter of jacked pipe is the ability for workmen to maneuver within the pipe. Concrete pipe as large as 132-inch diameter can be jacked. Precast concrete boxes may be jacked in sizes up to approximately 10 feet by 10 feet. Welded steel pipe may be bored in diameters up to 48 inches. Small diameter pipe is generally smooth welded steel.

Pipe jacking requires sufficient surface area to provide an access pit. This pit is used to set up jacking equipment, and to remove the earth excavated from the jacked pipe. All utilities must be located prior to commencing the jacking operation.

A safe working pit and a safe working area near the pit are essential to jacked pipe installation. The beginning and end of the casing pipe should extend to a point at least 1 ft away from the edge of the roadway for every 1 ft of depth. Surface working areas must be adequate to enable the contractor to use a crane, trucks, and backhoe. A storage area for the excavated earth from the pit and for the casing pipe should be available at the site.

The distance that a pipe can be jacked varies with the pipe size, pipe material, soil types, and alignment.

Welded Steel Pipe

Bare welded steel pipe is normally used for jacked casing pipe. A wall thickness of 0.25 inch for pipe diameters of 12 in to 20 in provides a collapse pressure of 80 psi or more, as well as providing handling rigidity. As sizes increase in diameter, the 80-psi collapse pressure can be used as a minimum “wall thickness / diameter guide.” Evaluation of the 80-psi minimum-collapse pressure must be made for special load conditions. Casing pipe for railroad crossings usually requires heavier wall thickness for steel pipe, and the wall thickness is specified by the railroad owner.

Wall Thickness - Welded Steel Pipe

Table B.1, containing values of wall thicknesses, is based on the 80-psi collapse pressure. It can be used as a guide for minimum wall thickness.

Table B.1 Minimum Wall Thickness for Welded Steel Pipe

Pipe Outside Diameter, in	Minimum Wall Thickness, in (For Highway Use)
12 in	0.250 in
16 in	0.312 in
18 in	0.312 in
20 in	0.312 in
24 in	0.344 in
30 in	0.375 in
36 in	0.438, 0.500 in
42 in	0.500, 0.538, 0.625 in
48 in	0.625 in, or special design

Note: Welded steel pipe is measured by outside diameter in sizes above 12 in. Special conditions, such as rocky ground or highly corrosive soils, may require a heavier wall thickness.

Casing pipe is joined by field welding. CDOT Standard Specifications for Road and Bridge Construction require a certified welder unless waived by a Project Special Provision. When the casing pipe is bored, grouting the outside of the casing is not required, unless voids are created or present. When a casing pipe is jacked by mining out the material, voids are usually created, and must be grouted to avoid settlement of the ground above. This can be done from the casing pipe interior, through 2-inch grout plugs spaced at given intervals throughout the casing. Grouting pressures to pump the soil cement, or sand, cement, and water-slurry mix normally need not exceed 20 psi to adequately fill all earth voids. The grouting or sand filling of the void between the outside of the jacking pipe and the over-excavation creates an almost-perfect bedding condition and may be superior to normal trench-bedding conditions.

Carrier pipe is installed after completion of the casing installation by threading through the casing. The carrier pipe is supported on wooden skids banded to the carrier pipe, or with factory-installed skid bands that have runners around the band to support the carrier. Steel casing pipe placed by boring or mining should be bare steel. The integrity of pipe coatings inside or outside of the pipe cannot be maintained with jacking operations, and they are impossible to replace or repair. Extra pipe thickness should be used to protect against corrosive environments.

Reinforced Concrete Pipe

Reinforced concrete pipe cannot be placed by boring due to the thick walls of the pipe. The material within the area to be occupied by the jacked pipe, including the walls, must be removed by hand mining.

Loading Criteria - Jacked Concrete Pipe

The class of reinforced concrete jacking pipe is determined from the table for the height of fill in CDOT Standard M 603-2.

Reinforced concrete pipe is subject to the axial forces of the jacking operations, dead load, and live load. The jacking forces must be evenly distributed around the periphery of the ends of the pipe by use of a heavy rope wrap or plywood. The dead load on the jacked pipe is usually less than a trench installation (with a trench width equal to the bore), due to the cohesive force of the soil reducing overburden load.

Reinforced concrete jacking pipe is manufactured in all joint types, with a smooth outside surface of the pipe barrel. Although tongue and groove (TG) or rubber and concrete (RC) joints can be used as a jacking pipe, rubber and steel (RS) joints are often used to increase the integrity of the joint under the severe installation conditions of the jacking operation.

Jacking Installation Procedures

The leading edge of reinforced concrete pipe is usually protected with a steel cutting guide or shoe, to protect the pipe joint. Material is excavated in front of the lead pipe, while an axial force is applied along the pipe centerline. A lubricant such as bentonite is applied to the outside of the pipe at the jacking pit, or is pumped through the grout nipples as the jacking operation progresses. The jacking operation is often a continuous operation from beginning to completion. Alignment rails in the jacking pit can be set in concrete to assist in the control of the alignment and grade. Careful excavation at the lead pipe to prevent over-excavation, allow alignment of the guiderails in the jacking pit, and equal distribution of axial jacking load, all contribute to a good final pipe alignment and grade.

As the pipe is advanced, the jack ram is retracted and another section of pipe is set on the guide rails, joined to the preceding pipe, and the jacking pressure reapplied. The size and number of jacks required, and the size of the jacking abutment, are a function of soil types encountered, size of pipe, and length of jacked installation.

Excavated material is usually removed through the jacking pipe installation in carts, or by a conveyor system. Once the installation is complete, soil cement, grout, or sand is pumped through grout nipples into the void between the excavated bore and the outside of the pipe. Nipples are plugged after grouting.

Casing Pipe Size

The size of a jacked casing pipe is a function of the carrier-pipe size. The size of the casing pipe is determined by the largest dimension of the carrier pipe. This is the outside diameter of the bell, flange or other joint detail. To this must be added the dimensions of any skids used to install the carrier pipe within the casing. The inside diameter of the casing pipe should be adequate to easily accommodate the largest diameter of the flange or bell, allowing for 2 in by 4 in, or 4 in by 4 in redwood skids, which are banded to the carrier pipe to facilitate sliding it through the casing pipe.

The designer must allow ample room between the carrier and the casing pipe. A minimum of 1-inch of clearance between the casing and the skids must be allowed. A 6-inch difference between bell outside diameter (O.D.) and carrier pipe inside diameter (I.D.) is suggested. The dimensions of the carrier pipe joint details must be known before the casing-pipe diameter can be determined. The designer should contact the utility or owner of the carrier pipe to determine the type of pipe, and the dimensions of the joint details. It may be necessary to obtain manufacturers' catalogs of the exact pipe to obtain these details.

It is not uncommon for jacking contractors to oversize the casing pipe to compensate for unanticipated alignment problems.

APPENDIX C – CDOT PIPE MATERIAL SELECTION GUIDE

Implementation

The *CDOT Pipe Material Selection Policy* was initially developed by the Project Development Branch for approval by the Chief Engineer. However, this document is no longer required to be a separate policy document that requires the Chief Engineer's approval. Therefore, it will now be referred to as the *CDOT Pipe Material Selection Guide* and incorporated as a design procedure in the *CDOT Drainage Design Manual.*

These Procedures for Pipe Material Selection (as updated April 30, 2015) supersede and replace all previous procedures, guidelines, and policies regarding the selection of pipe materials used by CDOT.

These procedures also replace the CDOT Chief Engineer's memo dated February 8, 1984, *Pipe to be Used in Storm Drains.*

Introduction

This guide will enable Project Managers (PMs) to select the allowable pipe material options for each installation on a specific project. The contractor will choose the final pipe material from the list of options provided in the Contract and as specified in applicable sections of the *CDOT Standard Specifications for Road and Bridge Construction*. Any pipe that meets the corrosion and abrasion criteria in this guide and which is installed per the plans and specifications is assumed to have a 50-year service life.

Background

This policy/guide was originally developed to comply with the provisions of the Final Rule published in 23 CFR 635.411(b) published in the Federal Register on November 15, 2006. On July 6, 2012, the Moving Ahead for Progress in the 21st Century Act (MAP-21) was signed into law. With passage of MAP-21 the federal requirement for this policy/guide was nullified. However, CDOT has determined the additional performance criteria outlined in the original policy (now guide) is beneficial to the State. Therefore, this revised guide retains much of the original policy and is to be incorporated into all CDOT design projects. CDOT will follow its standard practices for the hydraulic and structural design of pipes. This guide replaces all previous policies regarding the selection of pipe material for storm drains, cross drains, and side drains.

Selection Considerations

CDOT will evaluate the risk associated with the performance of pipe materials. Risk will be considered to the extent that it is influenced by the pipe, other materials, or installation techniques as they are used in construction.

This guide identifies the specific engineering and performance criteria used to evaluate the acceptability of alternate pipe materials. CDOT allows alternate pipe materials where appropriate. A record of the determination of abrasion and corrosion levels must be documented and maintained in the project design files.

The following exemptions are not intended to be covered by this guide:

- Subsurface drains and Embankment Protector Type 3 (M-Standard 615);

- Extensions of existing pipes or systems must be completed using similar material and sizes. Exceptions to this may be made when conditions and engineering justifications merit.
- Local agencies and other organizations that will own and maintain the new pipe should be consulted for guidance on selection of type of pipe material. Only pipe material types that have been evaluated and approved for use by CDOT may be used. In the event a local agency or organization will own and maintain the new pipe and the guidance provided differs from this guide, the guidance from the local agency or organization shall govern.

Definitions

Cross Drain – pipes or culverts that convey flows from one side of a road to the other, typically open on each end. This is also known as a cross culvert.

Side Drain – a pipe or culvert typically parallel to the roadway and under a driveway or road approach to the mainline roadway.

Storm Drain – a network of pipes that connects inlets, manholes, and other drainage features to an outfall.

Subsurface Drain – a network of pipes used to collect groundwater, or relieve water pressure from a wall or structure, and transport it to a location where it will not harm the roadway features, or where it can be conveyed by another system, often a storm sewer. A common example is a French drain.

Type III Embankment Protector – see M-Standard 615-1.

Durability – the ability of a pipe or culvert to resist wear or decay. Although structural condition is an important element in the performance of pipes, durability problems are a common cause for replacement. Pipes are more likely to wear away than fail structurally. Durability is affected by two mechanisms – corrosion and abrasion. Each is discussed in the following sections.

Corrosion – deterioration of material due to chemical or electrochemical reaction with the environment. Corrosion of pipe materials may occur in many different types of soil and water which may contain acids, alkalis, dissolved salts, organics, industrial wastes or chemicals, mine drainage, sanitary effluents, and dissolved or free gases. Pipe corrosion is generally related to water and the chemicals that have reacted to, dissolved in, or have been transported by the water.

Abrasion – the process of wearing down or grinding away surface material of pipes. It is caused by sand, gravel, or stones within water flowing through a pipe. The abrasive force increases with rising velocity within the pipe.

Alternate Materials – various pipe materials that will meet a project's requirements. Alternate materials are identified in the Contract, and the contractor may select any of them for use on the project.

Selection Process Responsibility

All decisions regarding pipe material type must be based on best engineering practices and engineering judgment. The PM is responsible for all aspects of the design and schedule of the project. The PM must schedule work associated with the pipe-material selection process to ensure compliance with the overall project schedule. Factors which must be considered include durability, environmental considerations, soil conditions, fill heights, the need for watertight joints, pipe minimum and maximum slope (i.e. pipe velocity), hydraulic characteristics of inside surfaces, and other factors relevant to the project or specific pipe application.

The PM must specify on the plans or in the special provisions when watertight joints are required. Siphons, irrigation systems, and storm drain systems require watertight joints.

In some cases the results of the material-type selection process may include alternate material types and different pipe diameters. In such cases the PM may specify in the plans the appropriate diameter for each material type, or specify only the largest pipe diameter determined by the selection process regardless of material type.

When a specific Manning's *n* value is critical to a pipe's performance, the maximum and minimum values must be shown on the plans. If the larger diameter will not meet minimum-cover requirements, or the material will not meet the Manning's *n*-value range, that material type must be prohibited at the relevant locations. Any material type prohibited at a specific location during design must be clearly designated on the plans.

Step 1 – Determine Application

The PM must use the latest version of the *CDOT Drainage Design Manual* and *Project Development Manual*. The pipe-selection process begins when the PM determines the location of new pipe. The PM must then determine and document the specific use of the pipe: cross drain, side drain, or storm drain.

Step 2 – Determine Abrasion Level

An estimate of the potential for abrasion is required to determine acceptable pipe types and whether there is a need for invert protection. With concurrence of the project hydraulics engineer, the PM must estimate and document the abrasive forces that will affect the pipe material, and document the following items:

- Measure or estimate the velocity of the water based upon 2-year flow and less;
- Estimate bed loading as:
  - i. No bed load
  - ii. Minor bed load – silt and sand
  - iii. Moderate bed load – silt, sand, and gravel
  - iv. Heavy bed load – silt, sand, gravel, and rock
- Determine the abrasion level:
  - i. *Abrasion Level 1* – conditions are nonabrasive. Nonabrasive conditions exist in areas of no bed load and very low velocities. This is the level assumed for the soil surface of drainage pipes. This is also the level assumed for inverts of cross drains and side drains installed in drainages that are typically dry.
  - ii. *Abrasion Level 2* – low abrasive conditions existing in areas of minor bed loads of sand where velocities are 5 fps or less.
  - iii. *Abrasion Level 3* – moderately abrasive conditions existing in areas of moderate bed loads of sand and gravel, with velocities between 5 fps and 15 fps.
  - iv. *Abrasion Level 4* – severely abrasive conditions existing in areas of heavy bed loads of sand, gravel, and rock, with velocities exceeding 15 fps.

Abrasion levels are intended to help the PM consider the impacts of bed-load wear on the inverts of pipe materials. The PM must determine the expected level of abrasion through visual examination and documentation of the size of materials in the stream bed, and the average slope of the channel. In some cases, sampling of the streambed material may be required to enable the PM to determine the level of abrasion.

Where existing pipes are in place in the same drainage, the conditions of their inverts should be documented and used as guidance. The expected stream velocity should be based upon 2-year flows and less.

Step 3 – Determine Corrosion Level

The station of each proposed pipe must be determined by the PM. The PM must schedule soil and water testing to ensure meeting the project schedule. Resistivity, pH, and moisture levels must be determined in the field by Region personnel. These tests are most efficiently and effectively conducted at the time of sampling. CDOT Materials and Geotechnical staff are available to perform sulfate and chloride testing. The PM must schedule this work appropriately to avoid project delays. Region personnel should develop their ability to perform these simple tests to expedite the design process. Sample testing information is used in flowcharts (Figures 1 and 2) to select appropriate material.

The PM must document the following properties of the soil and water using the designated test procedure:

- Sulfate levels – CPL 2103
- Chloride levels – CPL 2104
- Resistivity – ASTM G57
- pH – ASTM G51
- Moisture levels

This information must be obtained at all pipe locations supplied by the PM and documented in the project records. If the alluvium of the area is sufficiently homogeneous, a reduced sampling schedule is acceptable. This determination should only be made with input from the Region Materials Engineer (or Staff Materials) and the Region Hydraulics Engineer.

Table C.1 Guidelines for selection of corrosion resistance levels

CR Level	**Soil**			**Water**		
	Sulfate (SO_4) % max	Chloride (Cl) % max	pH	Sulfate (SO_4) ppm (max)	Chloride (Cl) ppm (max)	pH
*CR 0	0.05	0.05	6.0-8.5	50	50	6.0-8.5
CR 1	0.10	0.10	6.0-8.5	150	150	6.0-8.5
CR 2	0.20	0.20	6.0-8.5	1,500	1,500	6.0-8.5
CR 3	0.50	0.50	6.0-8.5	5,000	5,000	6.0-8.5
CR 4	1.00	1.00	5.0-9.0	7,500	7,500	5.0-9.0
CR 5	2.00	2.00	5.0-9.0	10,000	10,000	5.0-9.0
CR 6	> 2.00	> 2.00	< 5** or > 9	> 10,000	> 10,000	< 5** or > 9

* No special corrosion protection recommended when values are within these limits.

** Concrete pipe used when the pH of either the soil or water is less than 5 must be coated in accordance with subsection 706.07. When needed, specify the coating in a special provision or plan note.

Table C.2 Minimum pipe thickness for metal pipes based on the resistivity and pH of the adjacent soil

Soil Side		Minimum Required Gauge Thickness for Metal Pipe Material
Resistivity, R (ohm – cm)	**pH**	
≥ 1,500	5.0-9.0	0.052 in (18 gauge) Aluminized Type 2
≥ 250	3.0-12.0	0.052 in (18 gauge) Polymer Coated

For storm drains, use Standard Specification 603 and write a project special provision stating the required corrosion classification as determined by this guide (i.e. sulfate class). Use appropriate pay items in these cases.

Step 4 – Selection of Pipe Material Type

Use the flowcharts in this document to identify acceptable pipe material types. Use Figure 1 to determine if metal pipe is an allowable material type. Then use Table C.2 to determine whether there are additional requirements for metal pipes.

Step 5 – Verify Fill Height

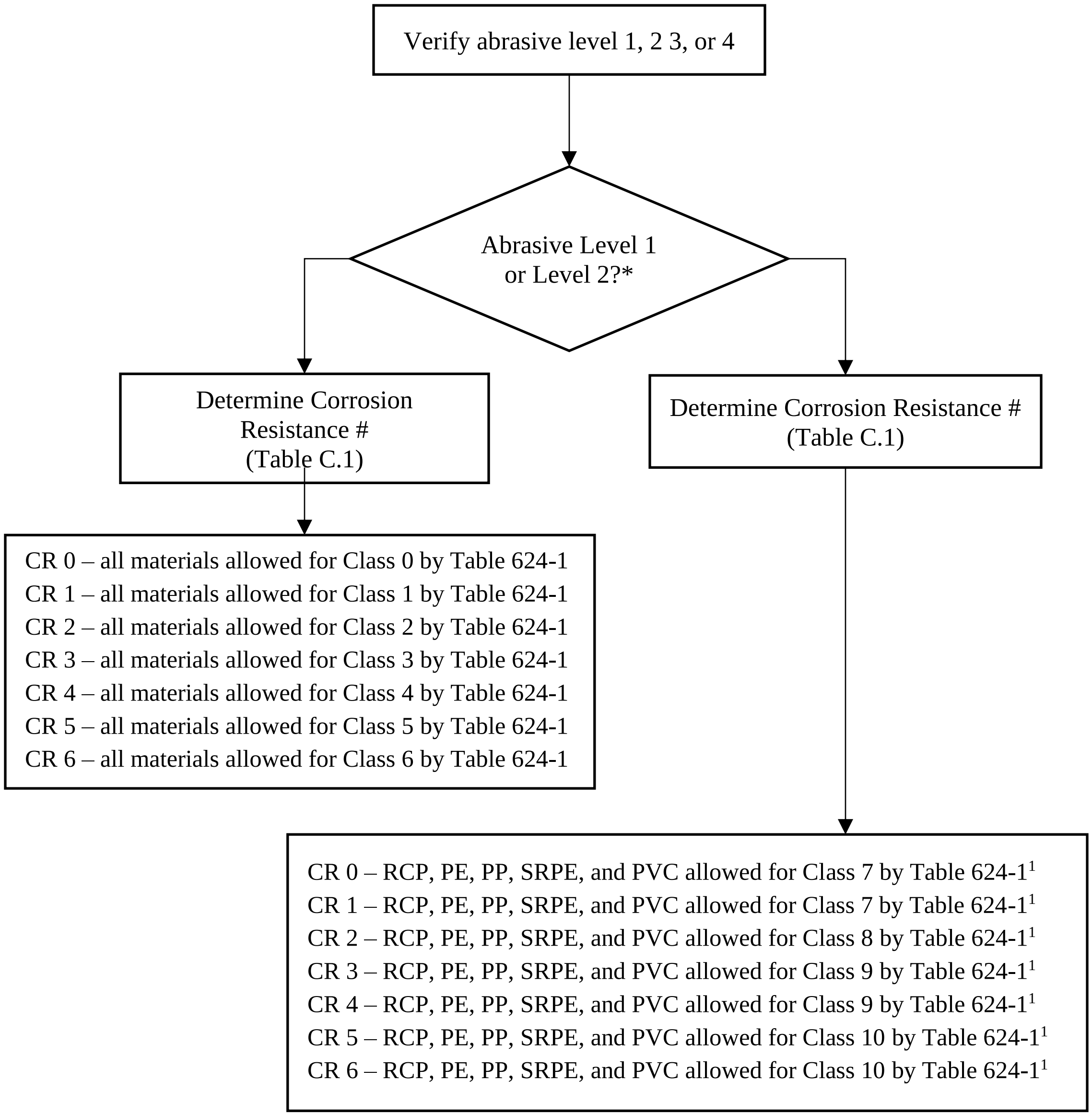
Check fill height tables in the Standard Plans. Determine if project special provisions are required and/or if any other Standard Special Provisions are applicable. Use the latest versions of these specifications, found at http://www.coloradodot.info/business/designsupport/construction-specifications/2011-specs.

Step 6 – Address Exceptions to *CDOT Pipe Materials Selection Guide*

When sound engineering judgment justifies an exception to this guide, the PM must document this in a justification letter. All justification letters must be approved by the Region Program Engineer (PE III) or their designee prior to final design.

Step 7 – Documentation

All design decisions regarding pipe material type selection must be documented and a letter placed in the project file. Prior to making final design decisions, send copies of all selection letters to the Region Program Engineer or their designee for guidance and to verify consistency.



* Aluminum alloy pipe not allowed in environments with an Abrasion Level greater than 1.

[1] **When concrete pipe is selected the sulfate content dictates the CR level**. Cementitious requirements for Sulfate Protection Classes are listed in 601.04. A higher level of protection may be used. Concrete shall have a minimum compressive strength of 4,500 psi and maximum water-to-cementitious ratio (w/cm) listed in 601.04. Concrete may be used when the pH and chlorides exceed levels listed in Table C.1.

Figure C.1 Cross Drains and Side Drains

For Metal pipes, see "Minimum Pipe Thickness for Metal Pipes Based on the Resistivity and pH of the Adjacent Soil" (Table C.2) in this guide.

When extending an existing pipe, the same size and type of material must be specified. If conditions are Abrasive Level 1 or 2 **and** CR 0, specify material type from Section 603 pay items.

For storm drains CDOT allows the use of only reinforced concrete pipe (RCP), polypropylene (PP), steel-reinforced polyethylene (SRPE), or polyvinyl chloride pipe (PVC) in accordance with Standard Plans M-603-2 and M-603-5 for storm drains.

Determine Corrosion Resistance # (Table 1)

↓

CR 0 – RCP, PP, SRPE, or PVC allowed for Class 7 by Table 624-1[1]
CR 1 – RCP, PP, SRPE, or PVC allowed for Class 7 by Table 624-1[1]
CR 2 – RCP, PP, SRPE, or PVC allowed for Class 8 by Table 624-1[1]
CR 3 – RCP, PP, SRPE, or PVC allowed for Class 9 by Table 624-1[1]
CR 4 – RCP, PP, SRPE, or PVC allowed for Class 9 by Table 624-1[1]
CR 5 – RCP, PP, SRPE, or PVC allowed for Class 10 by Table 624-1[1]
CR 6 – RCP, PP, SRPE, or PVC allowed for Class 10 by Table 624-1[1]

[1] If Abrasion Level is 3 or 4, concrete must have a minimum compressive strength of 4,500 psi. Cementitious requirements for Sulfate Protection Classes are listed in 601.04. A higher level of protection may be used.

Figure 2 Storm Drains

When extending an existing pipe, the same size and type of material must be specified. If conditions are Abrasive Level 1 or 2 **and** CR 0, specify material type from Section 603 pay items.

Trial Installations and Evaluation Process for New Pipe Material

At any time, manufacturers may request in writing to have materials not approved herein evaluated for a specific application. Requests for trial installations must follow the requirements of Procedural Directive 1401.1. Contact information for that procedure is given below:

Product Evaluation Coordinator
Colorado Department of Transportation
Materials and Geotechnical Branch
4670 Holly Street, Unit A
Denver CO 80216
303 398-6500

- Manufacturers must provide all of the materials, equipment, and labor required for the pipe material to be evaluated at no cost to CDOT.
- The pipe material to be evaluated must meet applicable AASHTO and ASTM design and material standards.
- Manufacturers are responsible for all coordination with the contractor, and any additional cost incurred by the contractor as a result of the trial installation.
- CDOT will determine a suitable location for the trial installation.
- During installation, the manufacturer must have a representative at the installation site. The manufacturer must provide documentation to CDOT that the pipe material was designed and installed per all current and applicable AASHTO and CDOT design and installation standards.

- Trial installations must perform satisfactorily for at least one year before conclusions regarding product performance are made.
- During the one-year evaluation period, at a time chosen by CDOT, the manufacturer must provide laser video inspection services on the trial installation utilizing an inspection contractor approved by CDOT.
- The results of the laser video inspection will be used to evaluate trial installations. The results must demonstrate compliance with CDOT and AASHTO deflection, joint separation, buckling, tearing, sagging, and cracking standards.
- Monitoring may include research of the trial material in use in other states.
- If further evaluation is required beyond one year, the supplier will be notified of the justification for this evaluation extension.
- An independent evaluation performed by a local agency or other organization may be substituted for this trial installation and evaluation process if all of the following are true:
    i. The local agency or other organization owns and maintains the material being evaluated.
    ii. A representative with the local agency or organization can be contacted to verify the information supplied.
    iii. The installation specifications are available for CDOT to review.
    iv. A trial installation was performed in Colorado on site applications similar to CDOT projects.
    v. A laser video inspection was performed (or can be performed) a minimum of 1 year after installation that produced satisfactory results.
- Upon successful completion of the monitoring period, CDOT's Drainage Advisory Committee will review the performance and determine the acceptability of the material for future inclusion into this guide.
- If changes to this guide, including the introduction of new materials or drainage products are requested, they will be evaluated by the following process:
    i. The Drainage Advisory Committee will evaluate documentation concerning changes to this guide.
    ii. Documentation supporting the proposed change must be submitted by the supplier to the Product Evaluation Coordinator at the above address.
    iii. The Product Evaluation Coordinator will compile all submitted documentation and submit it to the chairperson and secretary of the Drainage Advisory Committee.
    iv. The Drainage Advisory Committee will determine the future acceptability of the material for inclusion in this guide. The Drainage Advisory Committee will forward recommendations to the Chief Engineer for signature.