# Section 1000 Culverts and Bridges
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# Section 1000 Culverts and Bridges

## 1001 INTRODUCTION

Culverts and bridges convey surface water through or beneath an embankment such as a highway, railroad, or canal. The size, alignment, and support structures of a culvert or bridge directly affect the capacity of the drainage system. An undersized culvert or bridge will force water out of the channel and cause flooding and damage. Culverts and bridges may significantly influence upstream and downstream flood risks, floodplain management, and public safety.

A Boulder County floodplain development permit is required for all work within a floodplain regulated by the county. FEMA regulations will apply to all work in a FEMA-regulated floodplain. Limited guidance on current FEMA and county floodplain regulations is included in Section 1400. These regulations are both nuanced and subject to change over time. MANUAL users are encouraged to consult the FEMA website and the county for guidance on regulations at the start of each project.

Because trails often follow drainageways such as creeks and rivers, a new bridge, bridge replacement, or large box culvert sometimes presents the opportunity to remove an at-grade trail crossing and replace it with a designated pathway under a bridge or through an additional box culvert section designed to house the trail. If this option is considered, design guidance can be found in the USDCM (UDFCD, 2016). Additionally, some flood control improvements may be eligible for UDFCD maintenance assistance. The UDFCD should be contacted to determine if a project qualifies.

The criteria presented in this section shall be used to evaluate and design culverts and bridges for public roadways and private driveways, whether they are located within the public right-of-way or not. The review of all submittals outlined in Section 200, Submittals will be based on the criteria herein. Stormwater crossings of CDOT roadways may have additional requirements.

## 1002 CULVERT DESIGN STANDARDS

Culverts shall be designed and constructed to the following standards. If any criteria for culverts are provided in the Boulder County Multimodal Transportation Standards, the criteria in this MANUAL will take precedence. All proposed culverts are subject to review and approval by the Boulder County Transportation Department. This includes culverts to be placed in both the public right-of-way and on private lands. Review must be conducted and approved by the Transportation Department regardless of whether a Building Permit, Floodplain Development Permit, or Access Permit is required.

### 1002.1 Construction Materials

All roadway cross culverts within Boulder County shall be reinforced concrete unless otherwise approved by the County Engineer and Road Supervisor. Culverts under private driveways may be reinforced concrete or corrugated steel. Aluminum and plastic culverts are not permitted. Construction materials used for irrigation and raw water lines within the county right-of-way must be approved by the county before installation. The types of pipe, materials, and related items shall meet the most recent versions of the standards listed in Table 1000-1.

**Table 1000-1. Types of Pipe, Materials, and Related Items Standards**

| Item | Standard |
|---|---|
| Reinforced Concrete Pipe—Round | ASTM C76 or AASHTO M 170 |
| Reinforced Concrete Pipe—Elliptical | ASTM C507 or AASHTO M 207 |
| Reinforced Concrete Pipe—Joints | ASTM C443 or AASHTO M 198 |
| Reinforced Concrete Box Culvert—Joints | ASTM C1677 |
| Reinforced Concrete Pipe—Arch | ASTM C506 or AASHTO M 206 |
| Precast Concrete Box Culverts | ASTM C1433/C1577 or AASHTO M 259/M 273 |
| Concrete for Cast-in-place culverts | CDOT 601 |
| Corrugated Steel Pipe—Galvanized | AASHTO M 36 |
| Structural Plate | ASTM A761 or ASHTO M 167 |

### 1002.2 Minimum Culvert Size

The minimum culvert size shall be an 18-inch-diameter round pipe or equivalent. Equivalents are a 22-inch by 12-inch arch or a 23-inch by 14-inch elliptical section.

### 1002.3 Roadway Overtopping

A major factor to be considered when designing a culvert under a roadway is the roadway's functional classification and the associated allowable roadway encroachment or overtopping. Table 900-1 in Section 900, Roadways presents allowable roadway encroachment and overtopping criteria for culvert design for both the minor and major storm event for each of the county's roadway classifications. Design standards and criteria for bridges is separate and is discussed in Section 1004.

Where roadway overtopping is anticipated to occur, the depth of overtopping can be assumed as the difference between the headwater elevation and the roadway crown elevation. This is a conservative estimate, as water will tend to spread out once it leaves the channel banks. Where overtopping is not permitted but some amount of encroachment is permitted, the culvert headwater elevation can be set at the elevation corresponding to the limits of encroachment.

For allowable overtopping, the roadway crown is assumed to act as a broad-crested weir. A weir coefficient of 2.8 shall be assumed along with a weir length not to exceed 100 feet, regardless of roadway geometry. The designer should first calculate weir flow using the allowable overtopping depth for the major storm as given in Table 900-1. The designer should then calculate flow through the culvert per Section 1005, with culvert headwater set at the allowable overtopping elevation. If the calculated weir flow plus the flow through the culvert exceeds the culvert's 100-year design flow, the allowable overtopping condition has been met. Weir flow is discussed in the design example in Section 1006.2.

### 1002.4 Allowable Headwater

The maximum headwater for the 100-year design flow shall be 2.0 times the culvert diameter or culvert rise dimension for shapes other than round ($HW/D \leq 2.0$) for culverts with a rise dimension less than or equal to 48 inches. For culverts with larger rise dimensions, the headwater to depth ratio for the 100-year design flow shall be less than 1.5. There is no maximum headwater value for the minor storm.

### 1002.5 Velocity and Outlet Protection

A minimum flow velocity within the culvert of 3 feet per second is required so that sediment will not accumulate in the culvert. The minimum flow velocity should be calculated using the Manning's equation and assuming open channel flow and a normal depth equal to 0.25 times the vertical dimension of the culvert. Manning's *n* values are presented in Table 1000-2.

**Table 1000-2. Manning's *n* Values for Culverts**

| Pipe Material | Manning's *n* | | | |
|---|---|---|---|---|
| RCP | 0.013 | | | |
| Annular CMP (2⅔- × ½-inch corrugation)[a] | 0.024 | | | |
| Annular CMP (3 × 1-inch corrugations)[a] | 0.027 | | | |
| Helical CMP[a] | 18 Inch | 24 Inch | 36 Inch | 48 Inch |
| 2⅔- × ½-inch corrugation | 0.014 | 0.016 | 0.019 | 0.020 |
| 3- × 1-inch corrugations | N/A | N/A | 0.021 | 0.023 |

(a) The *n* value for CMP shall be 0.027 unless pattern and corrugations are specified on the drawings.

The design must also include revetment to protect the outlet from erosion caused by the maximum velocity exiting a culvert. Table 1000-3 presents the required culvert outlet protection based on maximum culvert exit velocity. Maximum exit velocity shall be calculated using the major storm design flow and the methods described in Section 1005.

The most common type of culvert outlet protection is riprap, either as a riprap apron or as a low tailwater basin. Procedures for designing a riprap apron or low tailwater basin downstream of a culvert outlet, including for multiple conduit installations, can be found in the UDFCD's USDCM. Both of these procedures are applicable for Froude numbers up to 2.5.

**Table 1000-3. Required Culvert Outlet Protection**

| Culvert Exit Velocity (V) | Protection Required |
|---|---|
| V < 5.5 fps | None |
| 5.5 fps < V < 16 fps | Riprap apron or low tailwater basin |
| > 16 fps | Energy dissipator structure |

An economical culvert design that meets allowable headwater requirements should not normally result in a Froude number larger than 2.5 when design velocities are kept below 16 feet per second. The designer should generally strive to keep culvert slopes as flat as practicable to limit the amount of revetment that is required at the outlet. A riprap apron is typically used when the culvert is discharging to a well-defined channel that can be expected to have a tailwater elevation equal to at least one-third of the height of the discharging conduit. A low tailwater basin is typically used when the receiving channel may have little or no tailwater or where the receiving channel is not well defined.

### 1002.6 Headwalls, Wingwalls, and End Sections

All culverts in the public right-of-way shall be designed with headwalls and wingwalls or flared end sections at the inlet and outlet, with the exception of private driveway culverts that are less than 36 inches in diameter, which may have projecting ends. Headwalls, wingwalls, and end sections shall be in accordance with the most recent edition of the CDOT M&S Standard Plans. Headwalls or end sections shall be located to provide a grade no steeper than 3H:1V between the back of the structure and the edge of the shoulder or back of walk. Outlet protection shall be provided at both the inlet and outlet of every culvert as required by Section 1002.5 until velocities fall below 5.5 feet per second. Figure 1000-1 is an example of a concrete box culvert in Boulder County.

**Figure 1000-1.** Example of a Concrete Box Culvert Adjacent to a Trail (Boulder County, 2016).

### 1002.7 Structural Design and Minimum Cover

Culvert installations shall be designed for the vehicular bridge loadings listed in the Boulder County Multimodal Standards, in accordance with the design procedures in the latest edition of the AASHTO Standard Specifications for Highway Bridges, or appropriate ASTM standard, and with the pipe manufacturer's recommendations. The minimum cover shall be 12 inches in all cases. Trench installations shall be in accordance with the most recent edition of the CDOT M&S Standard Plans.

### 1002.8 Fish Passage

Depending on the site location, a culvert may need to accommodate migrating fish. The U.S. Fish and Wildlife Services and the Colorado Parks and Wildlife should be consulted early in the planning process to determine requirements related to fish passage. Some locations may require a bridge to span the natural channel, but culvert modifications, including oversizing the culvert or placing it below grade and filling the lower portion with native streambed material, can often be used to meet regulatory design criteria. All culvert projects should examine the potential to incorporate ecological components into the design.

## 1003 LOW WATER CROSSING DESIGN STANDARDS

A low water crossing is a privately constructed and maintained embankment structure that provides property ingress and egress through a floodplain. Low water crossings should be used only as a last resort and are only allowed for Local, Local Secondary, and Townsite Roads or private driveways at elevations less than 6,000 feet. Low water crossings are generally not feasible above 6,000 feet for several reasons. Typical stream cross sections in mountainous areas would require driveway slopes to be significantly above the maximum allowed by the Boulder County Multimodal Transportation Standards. In addition, if a low water crossing in a canyon were to block debris, the resulting backup in floodwater has less space available to spread out before causing damage to roadways and adjacent properties. The low water crossing is designed to allow access across the drainageway during a minor storm event. Access generally will not be possible during larger storm events.

If the low water crossing is located in a FEMA-regulated floodplain, it is subject to FEMA regulations, and design flow rates may be determined from the FEMA Flood Insurance Study (FIS), available by entering the crossing address into the FEMA Flood Map Service Center's interactive website at (*https://msc.fema.gov/portal*). Flow rates may also be available from studies published by Boulder County or from USGS gaging stations. For low water crossings on unstudied or ungauged streams, the design flow rates may be determined using the USGS program Streamstats that can be found at (*http://water.usgs.gov/osw/streamstats/colorado.html*).

Culverts in a low water crossing must meet the culvert design standards of Section 1002 for construction materials, minimum culvert size, velocity, and outlet protection. Low water crossings have unique design standards for allowable roadway overtopping, allowable headwater, and end treatments, as well as additional criteria for roadway surfacing, embankment materials, and backwater. The standards for design of low water crossings are below. Some are also included as part of Figure 1000-2, Low Water Crossing Schematic, located at the end of this section.

### 1003.1 Embankment and Roadway Surfacing

Embankment slopes shall not be steeper than 3H:1V, regardless of the surface treatment used. Embankment slopes within the public right-of-way shall be slope paved with concrete a minimum of 4 inches thick. Low water crossings outside the public right-of-way may be slope paved with 4 inches of concrete or protected with 18 inches of grouted, Type M riprap installed in accordance with UDFCD specifications. The roadbed for the approach to the crossing and over the crossing must be designed to withstand the forces of the 100-year overtopping event.

### 1003.2 Slope Paving and Headwall

The slope paving and headwall shall be as shown on Figure 1000-2, Low Water Crossing Schematic, located at the end of this section. Slope paving shall extend to the edge of the proposed roadbed. A vertical headwall and wingwalls will not be allowed for use in a low water crossing.

### 1003.3 Allowable Backwater

Backwater from the low water crossing shall not increase the 100-year water surface elevation by more than 1.0 feet on streams that are not regulated by FEMA.

### 1003.4 Maximum Crossing Height

The maximum crossing height for a low water crossing is the distance between the highest point on the road surface and the channel invert. The maximum crossing height shall be determined by comparing the channel capacity at the crossing location with and without the proposed crossing. The 100-year capacity above the proposed crossing, including the 1.0-foot allowable increase in the 100-year water surface elevation, shall be greater than the 100-year capacity without the crossing. The low water crossing design example in Section 1006.2 details the calculations required for this analysis. The existing water surface elevation for the 100-year event may be calculated using HEC-RAS or an iterative process using the Manning's equation. A site survey must be conducted to ensure accurate calculations.

### 1003.5 Structural Design and Minimum Cover

Minimum cover over low water crossing culverts shall be 12 inches for round corrugated metal pipe, 6 inches for round concrete pipe, and 18 inches for arch pipes or 12 inches for arch pipes if an HS 10-44 loading is applied. If the manufacturer recommends a larger minimum cover, that minimum shall be required.

### 1003.6 Culvert Sizing

The culvert sizing for a low water crossing shall be completed once the maximum crossing height is determined. The culvert size shall be determined using the minor storm event. Weir flow over the crossing is allowed up to a depth of 6 inches during the minor storm event. The culverts shall be sized for a capacity equal to the minor storm event minus the allowable weir overflow. The procedure used for low water crossing culvert sizing is presented in Section 1006.2.

## 1004 BRIDGE DESIGN STANDARDS

All bridges shall be designed in accordance with the latest edition of the Boulder County Multimodal Standards, although the criteria in this MANUAL will take precedence if there is a discrepancy. The majority of the criteria in this section apply to bridges on public roads. Section 1004.5 is dedicated specifically to private driveway bridges, and Section 1004.6 is dedicated specifically to pedestrian bridges. All bridges adjacent to roadways, regardless of category, must also adhere to the encroachment and overtopping requirements of Table 900-1. Design flow rates may be determined in the same manner as in Section 1003 for low water crossings. Figure 1000-3 shows an example of a bridge in Boulder County with an aesthetic component.

It is possible that a bridge designed to meet the criteria of this MANUAL may be on a roadway that becomes flooded during the storm event the bridge is designed to pass. New bridges shall be designed to the standards of this MANUAL regardless of adjacent roadway flooding because roadways that experience frequent flooding may be reconstructed at a higher elevation in the future to achieve an overall greater level of protection.

### 1004.1 Public Bridge Sizing Criteria

In addition to the criteria set forth in Section 1002, the low chord of any public bridge shall provide a minimum freeboard. If any criteria for freeboard are provided in the Boulder County Multimodal Transportation Standards, the criteria in this MANUAL will take precedence.

All bridges on Collector, Residential Collector, Local, and Local Secondary roadways, or with a 100-year flow that is less than 1,000 cfs, shall have a low chord elevation set at or above the energy grade line (EGL). All bridges on Minor Arterial and Principal Arterial roadways, or where the 100-year flow is higher than 1,000 cfs, shall have a low chord elevation set at least 1 foot above the EGL.

**Figure 1000-3.** Example of a Bridge With an Aesthetic Component (Boulder County, 2016).

### 1004.2 Hydraulic Analysis

The hydraulic analysis of a bridge opening is a complicated undertaking. Design calculations for all bridges must be prepared and certified by a licensed Colorado Professional Engineer. The procedures for design as outlined in the Federal Highway Administration (FHWA) publication *Hydraulic Design of Safe Bridges* shall be used for the hydraulic analysis of the proposed design. HEC-RAS may be used to complete the hydraulic analysis of bridge openings provided the guidance in the publication is followed. All bridges are assumed to remain in place during all storm events and shall not be assumed to break away or otherwise be removed from any modeling scenario.

### 1004.3 Inlet and Outlet Configuration

Where bridge abutments and foundations are located below the 100-year water surface elevation, concrete wingwalls shall be tied to the existing side slopes to prevent erosion behind the abutments and to provide slope stabilization from the top of the embankment to the toe of slope. Riprap protection on the inlet and outlet transition slopes shall be provided to prevent erosion caused by eddy currents.

### 1004.4 Scour Analysis and Countermeasures

Velocity limitations through the bridge opening are intended to limit potential scour. Regardless of the results of the scour analysis, a maximum 100-year average channel velocity of 16 feet per second shall be allowed through a bridge opening.

If any criteria for scour analysis are provided in the Boulder County Multimodal Transportation Standards, the criteria in this MANUAL will take precedence. Whenever a new or replacement bridge is designed, it is critical that scour depths at piers and abutments be estimated. The scour estimate must consider subsurface data and a hydraulic analysis of the proposed design.

The FHWA has published a set of Hydraulic Engineering Circulars to provide guidance for bridge scour and stream stability analysis. The set includes HEC-18, Evaluating Scour at Bridges, HEC-20, Stream Stability at Highway Structures, and HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance. Latest editions of each shall be used in concert with each other to evaluate stream stability, potential scour, and appropriate scour countermeasures. HEC-RAS may be used to provide the raw data required for the HEC-18 equations. HEC-RAS may also be used to evaluate scour, but the user must be experienced in the nuances HEC-RAS presents in evaluating scour and the potential errors that can occur. Using HEC-RAS default values will cause inaccurate results.

The potential for local scour (pier and abutment) and general scour (contraction, stream degradation, and pressure) should be evaluated using HEC-18 to determine the extent of the various types of scour as applicable to each site. HEC-20 should be consulted to determine the general stability of the stream and whether lateral channel movement should be anticipated. If there is potential for scour during the scour design storm shown in Table 1000-4, countermeasures shall be designed in accordance with HEC-23. In all cases, the length of bridge piles shall be such that the design structural load may be safely supported entirely below the probable scour depth.

**Table 1000-4. Bridge Scour Design Standards**

| Roadway Classification | Design Storm for Abutment, Pier Cap, and Retaining Wall Design | Design Storm for Foundation Design |
|---|---|---|
| Principal Arterial (PA) | 500-year | 500-year |
| Minor Arterial (MA) | 500-year | 500-year |
| Collector (C) | 500-year | 500-year |
| Residential Collector (RC) | 100-year | 500-year |
| Local (L) | 100-year | 500-year |
| Local Secondary (LS) | 50-year | 500-year |
| Townsite Road | 50-year | 500-year |

### 1004.5 Design Standards for Private Driveway Bridges

According to the FHWA, scour at bridge foundations is the most common cause of bridge failure. Private driveway bridges are usually constrained by cost and other factors from meeting typical bridge scour design standards and are therefore, at a larger risk of failure from scour. To mitigate the scour threat, a risk- and resiliency-based approach has been adopted for private driveway bridges. This approach factors in the importance of the bridge foundation stability in ensuring a safe and reliable waterway crossing and is referenced in HEC-18. The objective of these guidelines is to protect bridges from failure during relatively common flood events, such as the 5-, 10-, and 25-year events, while remaining within the constraints inherent in private driveway bridge design.

This MANUAL contains different design criteria for private bridges located in mountainous areas and those located in the plains areas of the county. When floodwaters overtop a private bridge or when a private bridge is knocked off its foundations during a storm event, the effects can be experienced by the entire community. Overtopping causes an increase in backwater upstream of the bridge, an increase in velocity through the bridge opening that could cause erosion downstream of the bridge, and the potential for debris to accumulate at the upstream face of the bridge, further increasing upstream backwater and downstream velocity. When a bridge is completely dislodged, it can cause an enormous channel blockage. These conditions are more critical in mountainous areas because of the reduced available floodplain width that exists in canyon roadways. A small blockage can result in a large increase in water surface elevation. By contrast, a reduction in capacity cause by blockages in the plains areas results in a smaller increase in water surface because the water has more room to spread out. Tables 1000-5 and 1000-6 provide criteria for the design private driveway bridges.

**Table 1000-5. Design Recurrence Intervals for Private Driveway Bridges**

| Type of Crossing or Street Classification | Minimum Hydraulic Capacity Design Event(a) | Foundation and Scour Calculations Design Event(b) |
|---|---|---|
| Private Driveways below 6,000 feet of elevation | 5-year flow | 25-year |
| Private Driveways above 6,000 feet of elevation | 10-year flow | 50-year |

(a) Minimum hydraulic capacity shall be calculated using the freeboard requirements in Table 1000-6.
(b) Should the required bridge foundation depth be unreasonable based on the predicted scour, appropriate scour countermeasures should be implemented to mitigate the predicted scour depth.

**Table 1000-6. Freeboard Requirements for Private Driveway Bridges**

| Average Channel Flow Depth(a) | Minimum Required Freeboard (FB) |
|---|---|
| ≥ 1.5 feet | Lesser of 1.5 feet and $(0.1Q^{0.3}+0.008V^2)$ |
| < 1.5 feet | 0.5 feet |

(a) Average channel flow depth shall be for the design event specified in Table 1000-5 and shall be the average distance from the channel thalweg to the water surface taken at 50 feet upstream of the bridge, at the bridge, and at 50 feet downstream of the bridge.

The type of bridge foundation and foundation elevations should be determined by the bridge structural design engineer. During the design of the bridge foundations, the design engineer shall consider the design loading, the findings of the geotechnical investigation, scour depth as calculated using the procedures in HEC-18, anticipated frost depth, pressure flow during the 100-year event, and any other factors the engineer considers appropriate in his or her professional judgement. If scour is anticipated, the engineer can either design scour countermeasures using the procedures in HEC-23 for the design storm listed in Table 1000-5 or locate the bridge foundations below the anticipated depth of scour by a distance that provides a sufficient factor of safety in his or her professional judgement. Scour countermeasures will be required if anticipated scour depth is in excess of 5 feet.

The superstructure and abutments of all private bridges shall be designed to withstand the buoyant and lateral forces generated by the 50-year event so that the superstructure will not become dislodged from the abutments during this event. The types of lateral forces to be considered include drag and impact forces from floating debris and ice and any other forces the engineer considers appropriate. The bridge will be designed to withstand twice the calculated buoyant and lateral forces of clear water to provide for the accumulation of debris.

Structural, scour, and foundation design calculations must be accompanied by a certification statement that is signed and sealed by a professional engineer licensed in the State of Colorado and submitted to the county for review. The certification statement shall read as follows.

> I hereby affirm that the design calculations and plans for the private access bridge at [insert address] were prepared by me, or under my direct supervision, for the owners thereof, in accordance with the requirements of the International Building Code, the Boulder County Land Use Code, the Boulder County Multimodal Transportation Standards, the Boulder County Storm Drainage Criteria Manual, any approved variances and exceptions thereto, and my professional engineering judgment. I understand that Boulder County does not and will not assume liability for facilities, structures, or improvements designed by others.
>
> Registered Professional Engineer [Affix Seal]
>
> State of Colorado No. ________________________________

All assumptions made by the bridge design engineer shall be provided in the calculations. Furthermore, the design of all private bridges may be subject to review by a third party at the county's discretion. When located within a FEMA floodplain, all private driveway crossings are subject to requirements of the National Flood Insurance Program (NFIP) and local floodplain management regulations.

The county recognizes that in certain limited instances, it may be exceptionally difficult to conform to these standards. In these instances, the applicant will document in writing good and sufficient cause for a requested Design Exception on the most recent Boulder County Private Driveway Design Exception Request Form, which must then be signed and sealed by a professional engineer licensed in the State of Colorado.

Upon receipt of a written request for a Design Exception from a particular provision of this MANUAL, the County Engineer will issue a determination on whether the Design Exception should be granted or denied given the specific circumstances for which it was requested. The County Engineer will provide a copy of the determination to the applicant. Determinations made by the County Engineer in interpreting and enforcing the standards in this MANUAL involve the considered application of professional engineering and transportation planning judgment and skill in the context of each particular situation and are not appealable

### 1004.6 Design Standards for Pedestrian Bridges

Several types of pedestrian bridges can be constructed in the county. Table 1000-7 below describes each type and the applicable criteria. The design of bridges crossing raw water ditches shall be in consultation with and approved by the ditch owner(s).

**Table 1000-7. Pedestrian Bridge Criteria**

| Pedestrian Bridge Type | Description | Criteria |
|---|---|---|
| A – Small Trail Bridge | A private or public bridge crossing a stream, raw water ditch, or major drainage, not intended to carry motorized vehicles, and generally in low density and/or open areas | There are no capacity or minimum span length requirements. They may be tethered or designed to break away during a storm event less than the minor storm. |
| B – Private Access Pedestrian Bridge | A bridge crossing not intended to carry motorized vehicles, providing primary and/or sole access for users to private property. | These must comply with the private driveway bridge standards set forth in Section 1004.5. Breakaway or tethered designs are not permitted. When located within a FEMA floodplain, all private crossings are subject to the requirements of the National Flood Insurance Program (NFIP) and local floodplain management regulations. |
| C – Large Multiuse Bridges | A large bridge on a multiuse path or regional trail system, or used for maintenance access to or along a raw water ditch or major drainageway. These bridges are rated to carry small service vehicles including light duty pick-up trucks. | These must also comply with design standards for private bridges as set forth in Section 1004.5. Break away or tethered designs are not permitted. When located within a FEMA floodplain, all private crossings are subject to the requirements of the National Flood Insurance Program (NFIP) and local floodplain management regulations. |

## 1005 CULVERT HYDRAULICS

Presented in this section are the general procedures that shall be used for hydraulic design and analysis of culverts. The user is assumed to possess a basic working knowledge of culvert hydraulics and is encouraged to review the technical literature on the subject that is included in FHWA HDS-5, Hydraulic Design of Highway Culverts. The two primary types of culvert flow are inlet control and outlet control. Under inlet control, the cross-sectional area of the barrel, inlet geometry, and headwater are the factors that affect capacity. Outlet control involves the additional consideration of tailwater and the slope, roughness, and length of the culvert barrel. The Culvert Design Form, shown in Figure 1000-4, is a template for culvert hydraulic analysis that can be used with the information and equations below. It can be found at the end of this section.

### 1005.1 Inlet Control Condition

Under inlet control conditions, the slope of the culvert is steep enough that the culvert does not flow full. The control section of a culvert operating under inlet control is located just inside the entrance. Inlets may be either unsubmerged or submerged. In an unsubmerged condition, the headwater is not sufficient to submerge the top of the culvert and the culvert slope is supercritical, as shown in Figure 1000-5. In this situation, the culvert inlet acts like a weir.

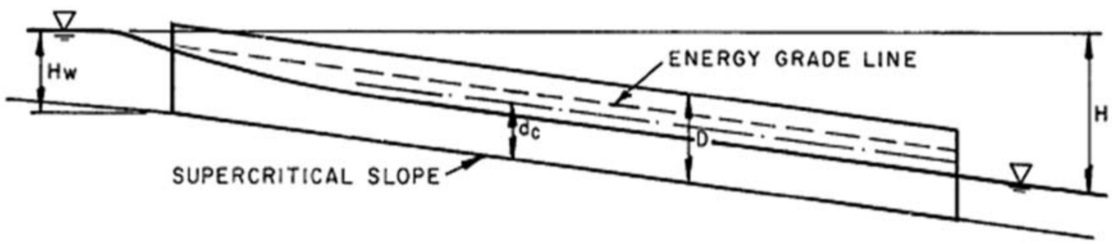



**Figure 1000-5.** Inlet Control – Unsubmerged Inlet (UDFCD, 2016).

In a submerged condition, the headwater submerges the top of the culvert but the pipe does not flow full, as shown in Figure 1000-6. In this situation, the culvert inlet acts like an orifice.

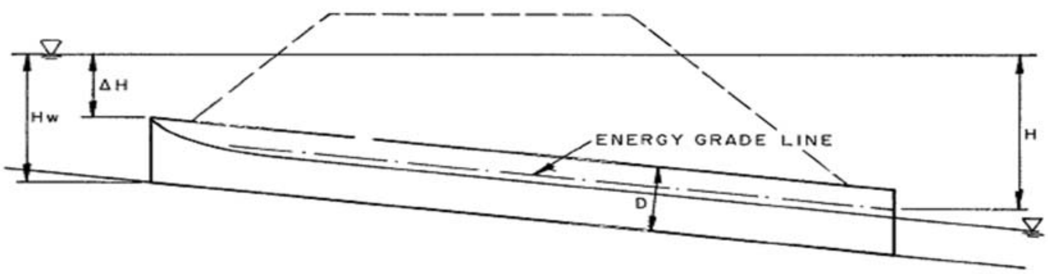



**Figure 1000-6.** Inlet Control – Submerged Inlet (UDFCD, 2016).

In the submerged inlet condition, the equation governing the culvert capacity is the orifice flow equation. However, because of the uncertainty in estimating the orifice coefficient for a submerged culvert inlet, it is recommended that the inlet control nomographs published in HDS-5 be used to determine headwater for submerged inlets operating under inlet control. Some of the more commonly used nomographs are included at the end of this section, cumulatively designated as Figure 1000-7, Common Nomographs. The remainder can be found in the second edition of HDS-5, publication number FHWA-NHI-01-020. Table 1000-8 provides the appropriate chart to use for various types of culverts and end treatments.

**Table 1000-8. Inlet Control Nomograph Selection**

| Material | Cross Section | End Treatment | Chart |
|---|---|---|---|
| Concrete | Circular | None (Projecting), Headwall | 1B |
| Concrete | Circular | Flared end section[(a)] | 1B |
| Concrete | Horizontal Elliptical (Oval) | Headwall or Projecting (use scale 1 for end section) | 29B |
| Corrugated Metal | Circular | None (Projecting), Headwall, Mitered | 2B |
| Corrugated Metal | Circular | Flared end section [(a)] | 2B |
| Concrete | Rectangular | Wingwalls, angle and headwall bevel varies | 8B-13B |

(a) End sections conforming to fill slope are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control (HDS-5, 2012).

### 1005.2 Outlet Control Condition

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. Either subcritical or pressure flow exists in the culvert barrel under these conditions. Outlet control will govern if the headwater is deep enough, the culvert slope is sufficiently flat, or the culvert is sufficiently long.

Outlet control generally exists under two conditions. The first, and less common, occurs when headwater is not high enough to submerge the top of the culvert and the culvert slope is subcritical, as shown in Figure 1000-8.

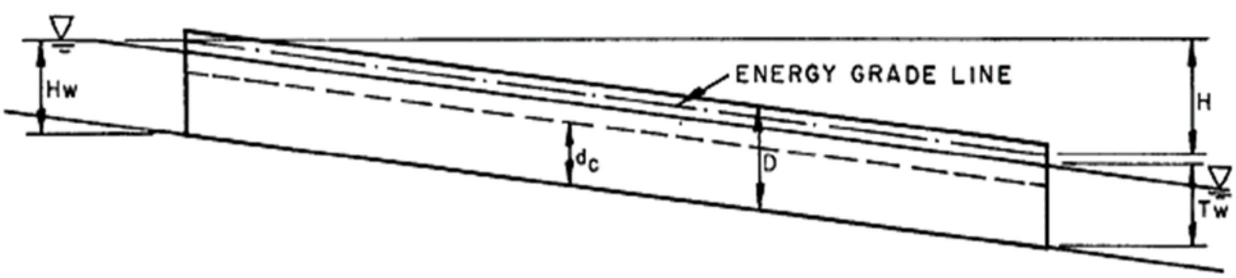



**Figure 1000-8.** Partially Full Conduit (UDFCD, 2016).

The more common outlet control condition exists when the culvert is flowing full, as illustrated in Figure 1000-9. A culvert with a submerged inlet and an unsubmerged outlet may also operate under outlet control, especially if it has a long barrel length or a flat enough slope. Culverts under outlet control may flow full or partly full, depending on various combinations of hydraulic factors.

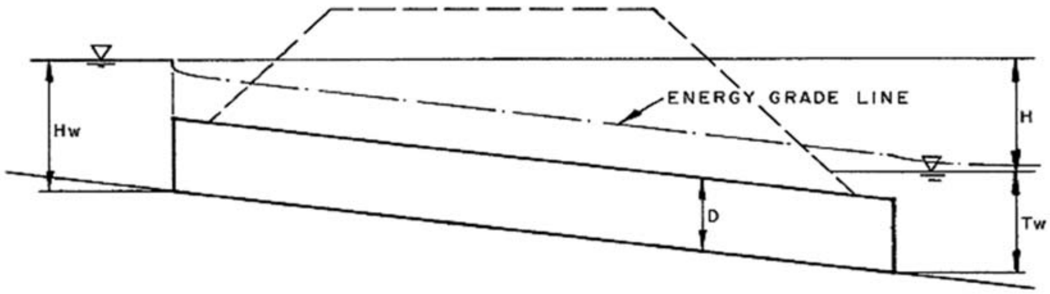



**Figure 1000-9.** Full Conduit (UDFCD, 2016).

Culvert capacity under outlet control is calculated using Bernoulli's equation for the conservation of energy. An energy balance is determined between the headwater at the culvert inlet and at the culvert outlet and includes inlet losses, friction losses, and velocity head. The general equation is expressed as:

$$H = h_e + h_f + h_v \tag{1000.1}$$

where

$H =$ total energy head $(\text{ft})$

$h_e =$ entrance head loss $(\text{ft})$, $K_e V^2 / 2g$

$h_f =$ friction losses $(\text{ft})$

$h_v =$ velocity head $(\text{ft})$, $V^2 / 2g$

$K_e =$ entrance loss coefficient per Table 1000-8.

Friction loss is the energy required to overcome the culvert barrel roughness and is calculated by the following equation.

$$h_f = \left(29n^2L/R^{1.33}\right)\left(V^2/2g\right) \tag{1000.2}$$

where

$n =$ Manning's coefficient per Table 1000-3

$L =$ Length of culvert $(\text{ft})$

$R =$ Hydraulic radius $(\text{ft})$

$V =$ Velocity of flow $(\text{fps})$

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

**Table 1000-9. Culvert Entrance Loss Coefficients, $K_e$, for Outlet Control Calculations (HDS-5)**

| Structure and Entrance Type | $K_e$ | Structure and Entrance Type | $K_e$ |
|---|---|---|---|
| RCP | | RCB | |
| Headwall, socket end of pipe | 0.2 | Wingwalls at 30° to 75° to barrel | |
| Headwall, square edge | 0.5 | Square edge at crown | 0.4 |
| Projecting from fill, socket end | 0.2 | Rounded or beveled top edge | 0.2 |
| Projecting from fill, square cut end | 0.5 | Wingwalls at 10° to 25° to barrel | |
| Mitered to conform to fill slope | 0.7 | Square edge at crown | 0.5 |
| Side- or slope-tapered inlet | 0.2 | Wingwalls parallel (side extensions) | |
| Beveled edges, 33.7° or 45° bevels | 0.2 | Square edge at crown | 0.7 |
| Rounded (radius = D/12) | 0.2 | Side- or slope-tapered inlet | 0.2 |
| End section that conforms to fill slop[(a)] | 0.5 | No wingwalls | |
| CMP[(b)] | | Square edge on 3 sides | 0.5 |
| Projecting from fill | 0.9 | Rounded or beveled on 3 sides | 0.2 |

(a) End sections that conform to fill slope are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, which incorporate a closed taper in their design, have a superior hydraulic performance. These latter sections can be designed by using the information given for the beveled inlet.

(b) Conditions not listed specifically for CMP have the same coefficient as RCP.

Combining the equations yields the following equation, which can be used to calculate culvert capacity directly only when the tailwater is at or above the crown of the culvert outlet.

$$H = \left(K_e + 1 + 29n^2L / R^{1.33}\right)\left(V^2/2g\right) \qquad (1000.3)$$

When the tailwater is below the culvert outlet crown, the tailwater depth used for calculations shall be the larger of the tailwater anticipated in the downstream channel at the culvert outlet and the average of the critical depth in the culvert and the culvert diameter, $(D+d_c)/2$. The FHWA has determined the average of the critical depth and the culvert diameter to be an adequate approximation for tailwater depth for culverts that flow partially full. Critical depth calculation is a direct process for a box culvert but an iterative one for a circular pipe that is easily accomplished with a spreadsheet. Critical depth occurs when the Froude number is equal to 1.0. The flow area and top width will be those that occur at critical depth in the pipe. Many online tutorials are available.

$$Fr = \frac{v}{\sqrt{gD_h}} \qquad (1000.4)$$

where

$$Fr = \text{Froude number } (\text{dimensionless})$$
$$v = \text{velocity } (\text{ft/s})$$
$$g = \text{gravitational acceleration } (32.2 \text{ ft/s}^2)$$
$$D_h = \text{hydraulic depth, } D_h = A / T \ (\text{ft})$$
$$A = \text{flow area } (\text{ft}^2)$$
$$T = \text{top width of flow area } (\text{ft}).$$

In addition to equation 1000.3, outlet control nomographs published by the FHWA in HDS-5 can also be used to calculate the required headwater under outlet control conditions where the outlet is submerged. Some of the more commonly used nomographs are included at the end of this section, included within Figure 1000-7. The remainder can be found in the second edition of HDS-5, publication number FHWA-NHI-01-020. Table 1000-10 provides the appropriate chart to use for various types of culverts. End treatments do not affect outlet control.

**Table 1000-10. Outlet Control Nomograph Selection**

| Material | Cross Section | Chart |
|---|---|---|
| Concrete | Circular | 5B |
| Corrugated Metal | Circular | 6B |
| Concrete | Rectangular | 15B |
| Concrete | Horizontal Elliptical (Oval) | 33B |

When using the outlet nomographs for corrugated steel pipe, the data must be adjusted to account for the variation in the *n* value between the nomographs and the culvert being evaluated. The adjustment is made by calculating an equivalent length according to the following equation:

$$L' = L(n'/n)^2 \tag{1000.5}$$

where

$$L' = \text{Equivalent length}$$
$$L = \text{Actual length}$$
$$n = \text{Manning's } n \text{ value, shown on the HDS-5 nomographs}$$
$$n' = \text{Actual } n \text{ value of a culvert from Table 1000-2.}$$

Culvert capacity shall be computed using the Culvert Design Form in Figure 1000-4 at the end of this section. Three example calculations for culvert sizing are provided in Sections 1006.1 through 1006.3. The first is for a roadway crossing culvert, the second is for a low water crossing, and the third is for private driveway culvert in a roadside ditch. The FHWA HDS-5, Hydraulic Design of Highway Culverts, offers extensive guidance on the design of culverts that are under roadways and that may be used in conjunction with the requirements of this MANUAL.

### 1005.3 Computer Applications

Although the procedures and nomographs for analyzing culvert hydraulics are still used, engineers increasingly design culverts by using computer applications. Among the applications approved for use by Boulder County is the FHWA's HY-8 Culvert Analysis Program and the UDFCD's UD-Culvert spreadsheet, both of which may be used to calculate roadway overtopping, inlet and exit velocity, and hydraulic grade line.

## 1006 DESIGN EXAMPLES

Three design examples are included in this section. The first example details the analysis of an existing roadway crossing culvert by using the Culvert Design Form. The second is for design of a private driveway culvert in a roadside ditch. The third is for the design of a low water crossing.

### 1006.1 Crossing Culvert Analysis Example

The procedure to evaluate culverts is based on the procedures presented in HDS-5. The methodology consists of evaluating the culvert headwater requirements for both inlet and outlet control. The type of flow control that results in a larger required headwater is the governing flow condition.

An example calculation for rating an existing culvert is presented in Figure 1000-10, Culvert Design Form Example, located at the end of this section. The culvert is a 48-inch CMP with $2^{2}/_{3}$ × ½-inch annular corrugations. The length is 150 feet. The upstream invert elevation is 5540.0, and the downstream invert elevation is 5535.5. The slope is 0.030. The low point of the embankment over the culvert has an elevation of 5551.9. The *n* value is 0.024 (from Table 1000-2). The culvert has flared end sections. All depths in Figure 1000-10 are in feet unless noted otherwise.

The tailwater rating values are provided for this example and shown in Column 5 of Figure 1000-10. If the tailwater condition is unknown, it must be computed using the normal depth (subcritical or critical only) of a trapezoidal channel approximating the existing drainageway. A HEC-RAS model of the site could also be used to determine the tailwater rating curve.

The entrance loss coefficient, $K_e$, can be determined from Table 1000-9 as 0.5 for an end section that conforms to fill slope, which is the category used to represent a common flared end section. The full flow and the velocity are calculated from these values for comparison. The rating then proceeds in the following sequence:

- **Step 1:** The culvert design process begins with selecting a discharge range or a headwater depths range and then using an inlet control nomograph to determine the corresponding values. This example begins with a range of headwater depths that are entered in Column 3. Headwater to pipe diameter ratios ($H_w/D$) are calculated and entered in Column 2. If the culvert is not circular, the culvert height is used for the calculation.

- **Step 2:** For each $H_w/D$ ratio, inlet capacity is read from the appropriate inlet control nomograph (Chart 2B for this example) and entered into Column 1. Flared end sections are hydraulically equivalent to headwalls according to HDS-5. Scale (1) should be used on Chart 2B to determine discharges, which then completes the inlet control rating.
- **Step 3:** For outlet control, the Q values that have been entered in Column 1 are used to determine the head values (H) in Column 4 from the appropriate outlet control nomograph (Chart 6B for the example).
- **Step 4:** The tailwater depths ($T_w$) are then entered into Column 5 for each Q value in Column 1. The depths have been provided in this example, but must be calculated if they are not available. If the tailwater depth is less than the diameter of the culvert, Columns 6 and 7 must be calculated per Step 5, and the larger of the tailwater depth and the value of Column 7 shall be used as $h_o$. If the tailwater depth is greater than the diameter of the culvert, the tailwater values in Column 5 are entered into Column 8 as the values for $h_o$, and Step 6 should begin (Step 5 being skipped).
- **Step 5:** Approximate tailwater depths are calculated if tailwater depths from Step 4 are less than the diameter of the culvert. The critical depth, $d_c$, for each Q value in Column 1 is calculated and entered into Column 6. The average of the critical depth and the culvert diameter is calculated and entered into Column 7 as the approximate $h_o$ value.
- **Step 6:** The headwater values (HW) are calculated according to Equation 1000.6:

  $$H_w = H + h_o - LS_o \qquad (1000.6)$$

  where H is from Column 4 and $h_o$ is either the value from Column 8 where $T_w \geq D$ or the larger value of Column 5 and Column 7 where $T_w < D$. L is the length of the culvert barrel and $S_o$ is its slope.
- **Step 7:** The final step is to compare the inlet and outlet control headwater requirements (Columns 3 and 9) and record the higher of the two values in Column 10. The type of flow control is recorded in Column 11. The headwater elevation is calculated by adding the controlling headwater (Column 10) to the upstream invert elevation. A culvert's rating curve can then be plotted from the values in Columns 12 and 1.

Outlet velocity for designing downstream protection can be computed using V = Q/A. For full flow conditions, the culvert area is the full cross sectional area of the culvert. For partially full conditions, the culvert area is the area calculated at a depth of $h_o$. Channel protection shall be as described in Section 1002.5. Velocity values are not shown in Figure 1000-10 but should be calculated for the 100-year event. To size a culvert crossing, the same form can be used, with some variation in the basic data. First, a design Q is selected and the maximum allowable headwater is determined, subject to the conditions of Section 1002.4. An inlet type is selected and the invert elevations and culvert slope are estimated based on site constraints. A culvert type and size is then selected and rated for both inlet and outlet control. If the controlling headwater exceeds the maximum allowable headwater, design data should be modified and the procedure repeated until the desired results are achieved.

### 1006.2 Low Water Crossing Design Example

A low water crossing is planned in an ungauged stream with steep vegetated banks and an invert composed mostly of gravel and cobble. StreamStats indicates the major storm discharge is 1430 cfs and the minor storm discharge is 200 cfs. The existing channel has an average slope of 1.6 percent and a typical cross section is shown in Figure 1000-11, Low Water Crossing Design Example, at the end of this section. A Manning's *n* value of 0.040 is assigned to the channel in accordance with Section 700. Design of a low water crossing is an iterative approach described in the steps below. AutoCAD, spreadsheet software, and HEC-RAS can all be used to assist with the analysis.

- **Step 1:** Calculate the existing 100-year water surface elevation. Determine the flow area, wetted perimeter, and hydraulic radius of the typical section at several different water surface elevations. Use Manning's equation to determine discharge at each of those water surface elevations. Adjust the water surface elevation until the calculated discharge is equal to the 100-year flow rate. In this example, a water surface elevation of 5406.0 resulted in a discharge of 1,431 cfs, which is acceptably close to the 100-year flow rate of 1,430 cfs. This water surface is shown in Figure 1000-11 at the end of this section.

- **Step 2:** Calculate the proposed 100-year water surface elevation. A low water crossing may cause a maximum increase of 1.0 feet in the 100-year water surface elevation. The proposed 100-year water surface elevation is 5407.0 feet, which is also shown in Figure 1000-11 at the end of this section.

- **Step 3:** Determine the proposed road grade elevation. Using Manning's equation, calculate the discharge above several potential road grade elevations using the existing conditions *n* value and the proposed 100-year water surface elevation of 5407.0. Do not account for flow in any potential culverts. The entire 100-year flow should pass over the low water crossing. Select the highest road grade elevation that will convey a flow equal to or slightly greater than the 100-year discharge at a water surface of 5407. In this example, a discharge of 1,462 cfs was calculated at a corresponding road grade elevation of 5404.1, shown in Figure 1000-11 at the end of this section. This flow rate is considered acceptably close to the 100-year flow rate of 1,430 cfs.

- **Step 4:** Calculate the discharge over the road grade during the minor storm. The maximum allowable overtopping depth (H) is 0.5 feet during the 5-year event. Use the weir equation, equation 1000.7, to determine the 5-year discharge over the low water crossing, $Q_o$, using a weir coefficient (C) of 2.8. The length of the weir (L) can be determined from the typical channel cross section and the selected road grade elevation, up to a maximum of 100 feet. In this example, the weir length is 38 feet.

$$Q_o = CLH^{3/2} \quad (1000.7)$$

$$Q_o = (2.8)(38)(0.5)^{3/2} = 37.6cfs \quad (1000.8)$$

- **Step 5:** Determine the minimum required culvert capacity, $Q_p$. This is the difference between the channel 5-year design flow and the allowable discharge over the low water crossing.

$$Q_p = Q_5 - Q_o = 200 - 37.6 = 162 \text{ cfs} \quad (1000.9)$$

- **Step 6:** Select the culvert size. The proposed road grade elevation must accommodate the selected culvert size with the minimum allowable cover over the top of the pipe. Assume a 53-inch by 34-inch HERCP, which has a wall thickness of 5 inches. The proposed road grade is 49 inches above the channel invert (5404.1 – 5400.0 = 4.1 feet = 49 inches). The distance from the pipe invert to the top of the pipe is 39 inches (34 + 5 = 39 inches). This results in 10 inches of cover between the top of the pipe and the road grade. This distance is greater than the allowable minimum of 6 inches, and the 53-inch by 34-inch Horizontal Elliptical Reinforced Concrete Pipe (HERCP) is an acceptable alternative.
- **Step 7:** Calculate the number of culverts required. Assume inlet control and use the nomograph in Chart 29B to calculate capacity of the 53-inch by 34-inch HERCP. Allowable headwater is equal to the proposed crossing height plus 6 inches of allowable overtopping, or 55 inches (49 + 6 = 55 inches). The ratio of headwater to interior pipe height, HW/D, is 1.62 (55 inches / 34 inches = 1.62). Because there is no scale for an entrance that is mitered to conform to the slope, use Scale 1 to the right of the nomograph. The nomograph indicates the capacity of a single culvert to be 84 cfs. The number of culverts needed is calculated by dividing the total required culvert capacity by the individual culvert capacity, or 162 cfs / 84 cfs = 1.9. Rounding up, two culverts are required.

The low water crossing will use two 53-inch by 34-inch HERCP culverts with a road grade elevation of 5404.1. Figure 1000-11 shows the final design data.

### 1006.3 Private Driveway Culvert Design Example

A driveway is planned to provide access to a new residence from a collector roadway with an existing roadside ditch. The collector has a transverse slope of 2 percent. The roadside ditch is trapezoidal with 1:1 side slopes, a 2-foot bottom width, and a 3.5-foot depth as shown in Figure 1000-12.

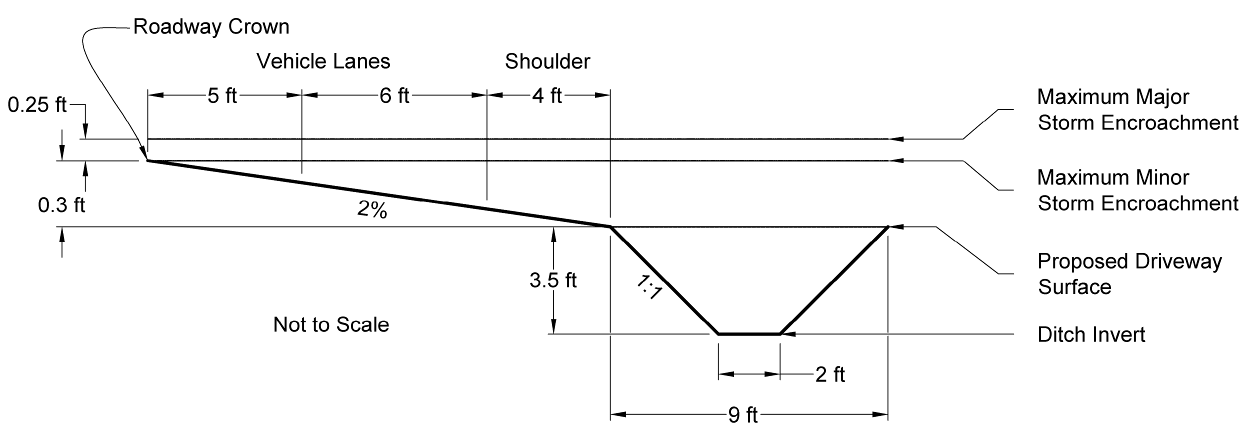



**Figure 1000-12.** Private Driveway Culvert Example Ditch Geometry.

The driveway is assumed to have no slope over the ditch for calculation purposes. The calculated discharge for the major design storm is 28 cfs. The minor design storm discharge is 12 cfs. Floodwater encroachment onto the road must not exceed the limitations set forth in this MANUAL. Inlet control is assumed for this example; however, actual projects should use Figure 1000-4, Culvert Design Form, at the end of this section to determine if culverts are under inlet or outlet control.

- **Step 1:** During the major storm, water on a collector may be 3 inches deep at the crown of the road as shown in Table 900-1. The depth of the water from the crown of the road to the ditch invert is 4.05 feet. This value is assumed as the headwater depth.

- **Step 2:** Calculate the discharge through an 18-inch CMP (D = 1.5 feet) with flared end sections and a headwater depth of 4.05 feet using Chart 2B.

$$Hw/D = 4.05\,ft / 1.5\,ft = 2.7 \quad (1000.10)$$

  HDS-5 states that flared end sections are hydraulically equivalent to headwalls. A HW/D ratio of 2.7 on Scale 1 of Chart 2B gives a discharge of 15.5 cfs for an 18-inch CMP.

- **Step 3:** Calculate weir flow over the road and driveway during the major storm. Flow over the road and ditch are calculated independently. Because the road grade is sloped across the cross section, the average depth of flow over the road is used. Flow outside the top of the ditch side slope is assumed to be negligible for this example, but it may be considered if the designer feels it is appropriate. Assume a weir coefficient of 2.8.

$$\begin{aligned}
Q_{weir} &= Q_{road} + Q_{driveway} \\
Q_{weir} &= CLH^{3/2} + CLH^{3/2} \\
Q_{weir} &= (2.8)(15)\left((0.55+0.25)/2\right)^{3/2} + (2.8)(9)(0.55)^{3/2} \\
Q_{weir} &= 16.9\ cfs.
\end{aligned} \quad (1000.11)$$

  Total flow over the road and driveway is 16.9 cfs.

- **Step 4:** The combined flow through the 18-inch CMP and over the road/driveway is 32.4 cfs, which is more than the major design storm flow. Encroachment onto the collector will not exceed allowable and the chosen culver is acceptable. If the combined flow would have been less than the major storm flow, a larger culvert would be required, and Steps 2–3 would be repeated using a 24-inch CMP.

- **Step 5:** Verify that the minor storm meets criteria. During the minor storm, flow may spread to the crown of a collector. Assuming encroachment extends to the roadway crown yields a headwater depth of 3.8 feet.

$$Hw/D = 3.8\,ft / 1.5\,ft = 2.53 \quad (1000.12)$$

  Chart 2B indicates a capacity of 15 cfs, which is greater than the minor storm flow. Encroachment will meet criteria.

- **Step 6:** Verify that the culvert has a minimum 12 inches of cover. The driveway surface is 3.5 feet or 42 inches above the ditch invert.

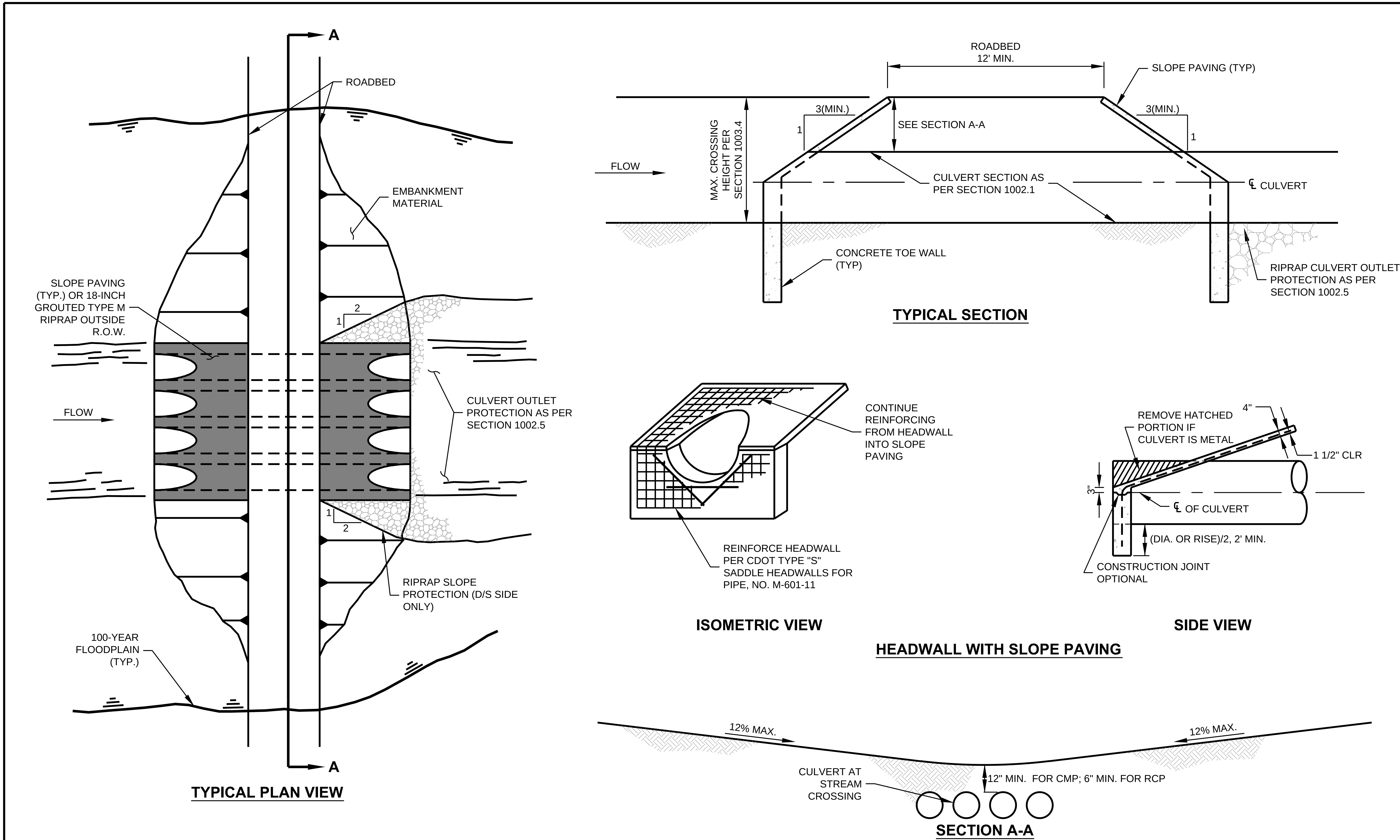
$$42\text{in} - 18\,in = 24\,in \quad (1000.13)$$

The 18-inch CMP has 24 inches of cover, which is greater than the 12-inch minimum. An 18-inch CMP meets all criteria for this location.

## 1007 REFERENCES

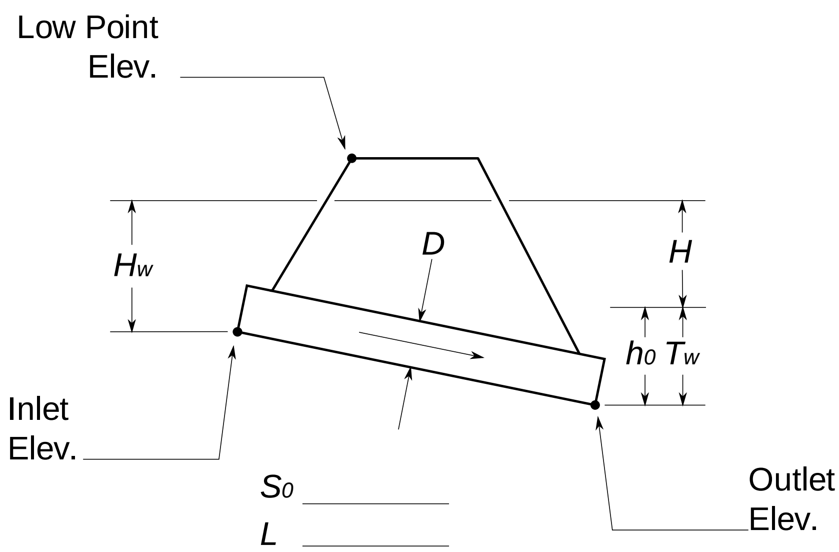
**Urban Drainage and Flood Control District, 2016.** *Urban Storm Drainage Criteria Manual: Volume 2 Structures, Storage, and Recreation*, prepared by the Urban Drainage and Flood Control District, Denver, CO.

**Schall, J. D., P. L. Thompson, S. M. Zergers, R. T. Kilgore, and J. L. Morris, 2012.** Hydraulic Design Series Number 5 (HDS-5) *Hydraulic Design of Highway Culverts*, Third Edition, HIF-12-026, prepared by Ayres Associates, Fort Collins, CO, for the U.S. Federal Highway of Administration, Washington, DC.




| | | | |
|---|---|---|---|
| | | **BOULDER COUNTY STORM DRAINAGE CRITERIA MANUAL** | **FIGURE 1000-2 LOW WATER CROSSING SCHEMATIC** |

| **Figure 1000-4**<br>**Culvert Design Form** | Project: | Consultant: | Date: |
|---|---|---|---|
| | Route/Highway: | Engineer: | |
| | Mile Marker: | Other: | |




### Culvert Data

| | |
|---|---|
| Size: | $n$: |
| Material: | $Q_{Full}$: |
| Inlet Style: | $V_{Full}$: |
| $k_e$: | |

### Outlet Control Equations

(1) $H_w = H + h_0 - LS_0$

(2) For $T_w < D, h_0 = \frac{d_c+D}{2}$ or $T_w$ (whichever is greater)

$T_w > D, h_0 = T_w$

(3) For Box Culvert: $d_c = 0.315\left(\frac{Q}{B}\right)^{2/3}$ (B = width of the box culvert)

| $Q$ | Inlet Control | | Outlet Control | | | | | | Design Curve | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | $T_w < D$ | | $T_w > D$ | | | | |
| | $\frac{H_w}{D}$ | $H_w$ | $H$ | $T_w$ | $d_c$ | $h_0 = \frac{d_c + D}{2}$ | $h_0$ | $H_w$ | $H_w$ | Control Method | WSEL |
| Col 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
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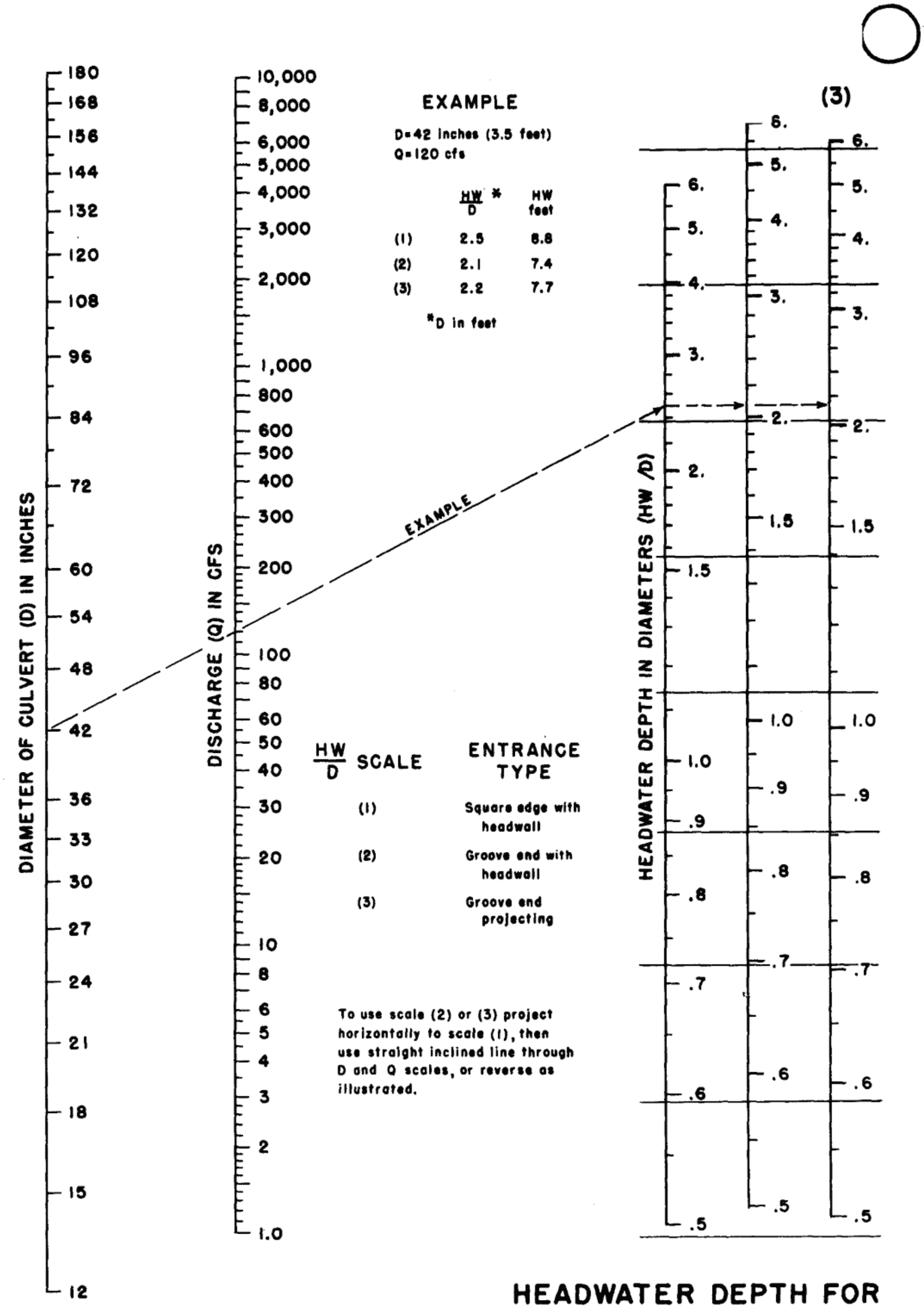
# CHART 1B




HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

## HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

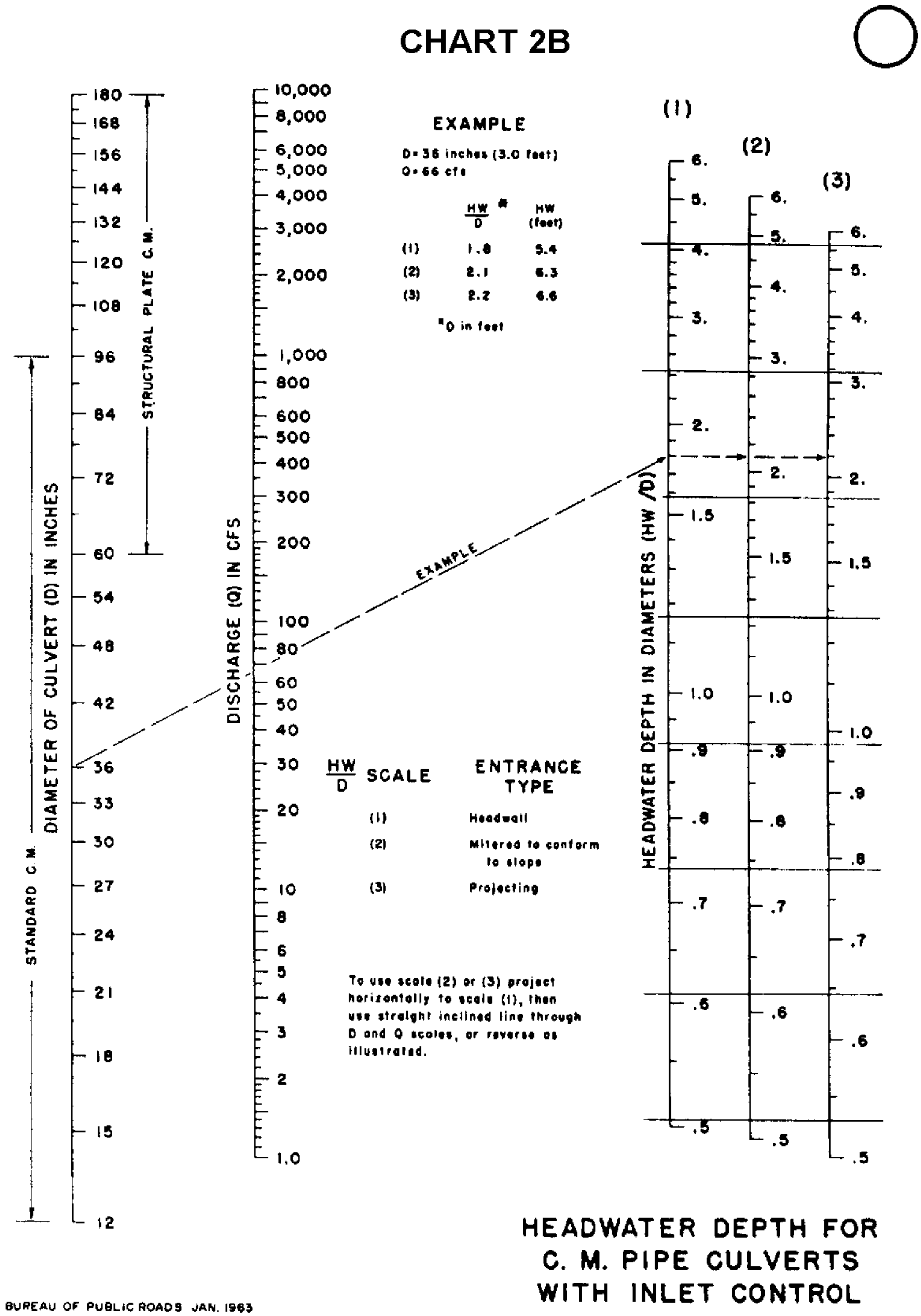
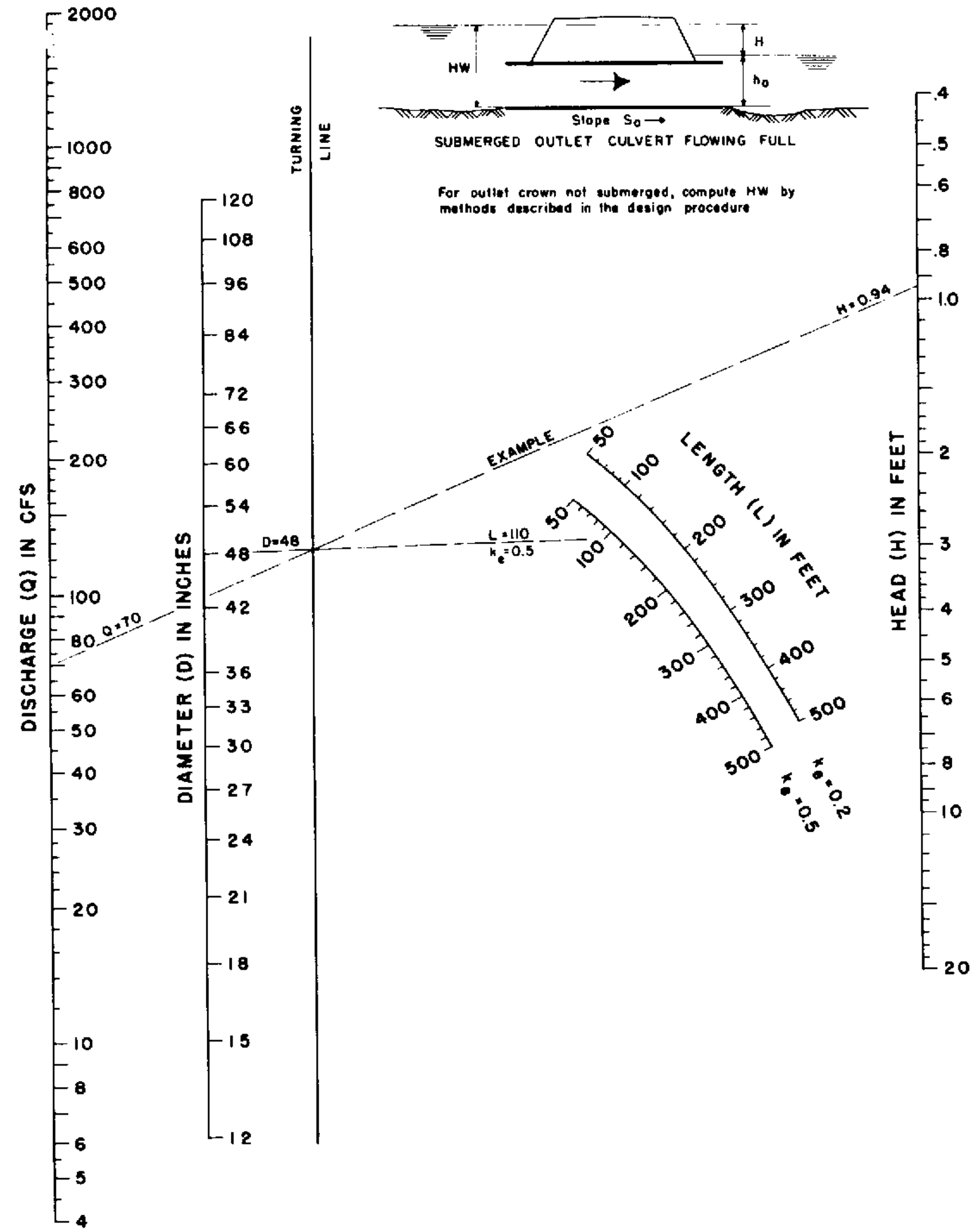


CHART 2B
EXAMPLE
D=36 inches (3.0 feet)
Q=66 cfs
HW/D* HW (feet)
(1) 1.8 5.4
(2) 2.1 6.3
(3) 2.2 6.6
*D in feet
DIAMETER OF CULVERT (D) IN INCHES
STANDARD C.M.
STRUCTURAL PLATE C.M.
180 168 156 144 132 120 108 96 84 72 60 54 48 42 36 33 30 27 24 21 18 15 12
DISCHARGE (Q) IN CFS
10,000 8,000 6,000 5,000 4,000 3,000 2,000 1,000 800 600 500 400 300 200 100 80 60 50 40 30 20 10 8 6 5 4 3 2 1.0
EXAMPLE
HEADWATER DEPTH IN DIAMETERS (HW/D)
(1) (2) (3)
6. 5. 4. 3. 2. 1.5 1.0 .9 .8 .7 .6 .5
HW/D SCALE ENTRANCE TYPE
(1) Headwall
(2) Mitered to conform to slope
(3) Projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL
BUREAU OF PUBLIC ROADS JAN. 1963

# CHART 5B




HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

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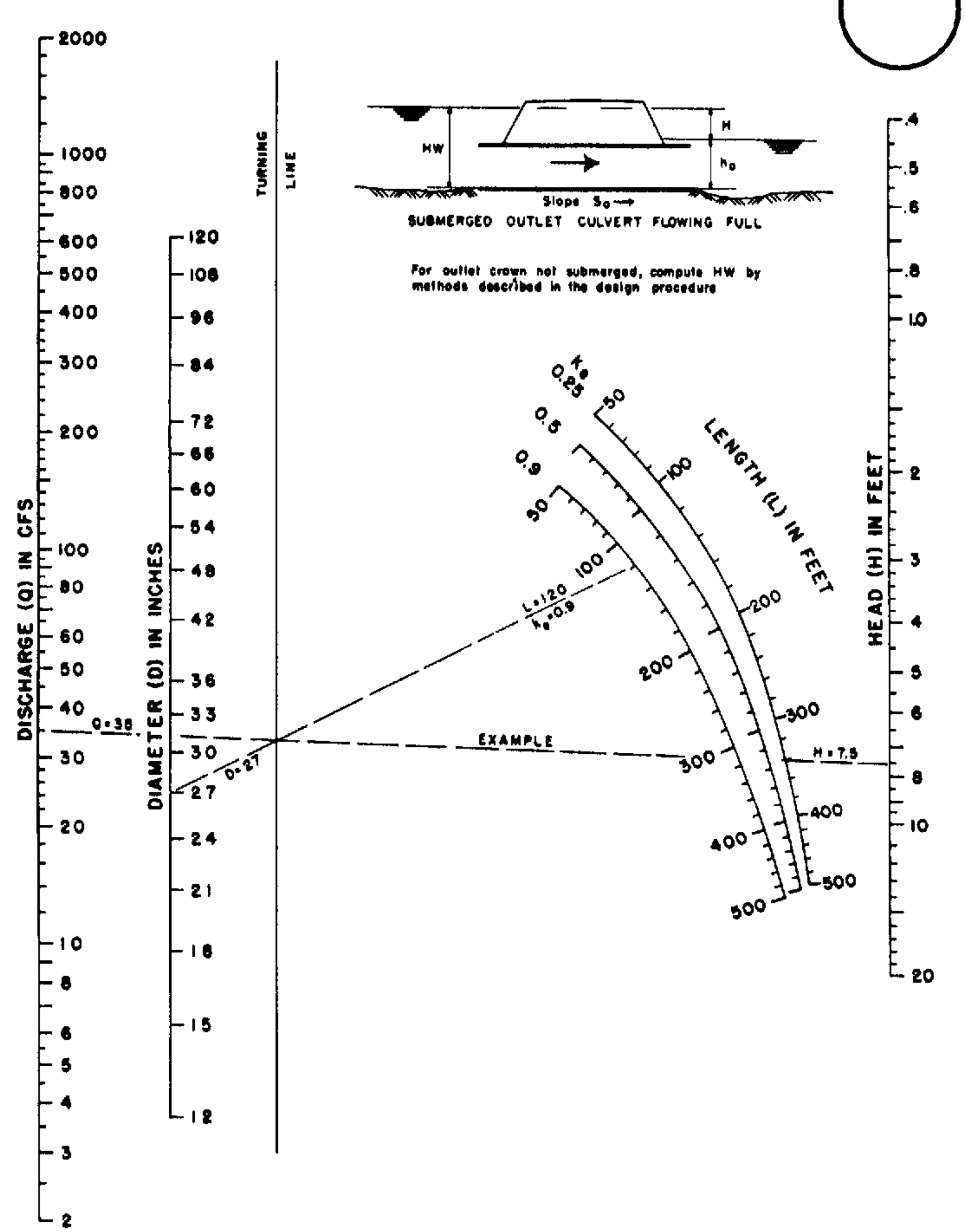
## CHART 6B




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

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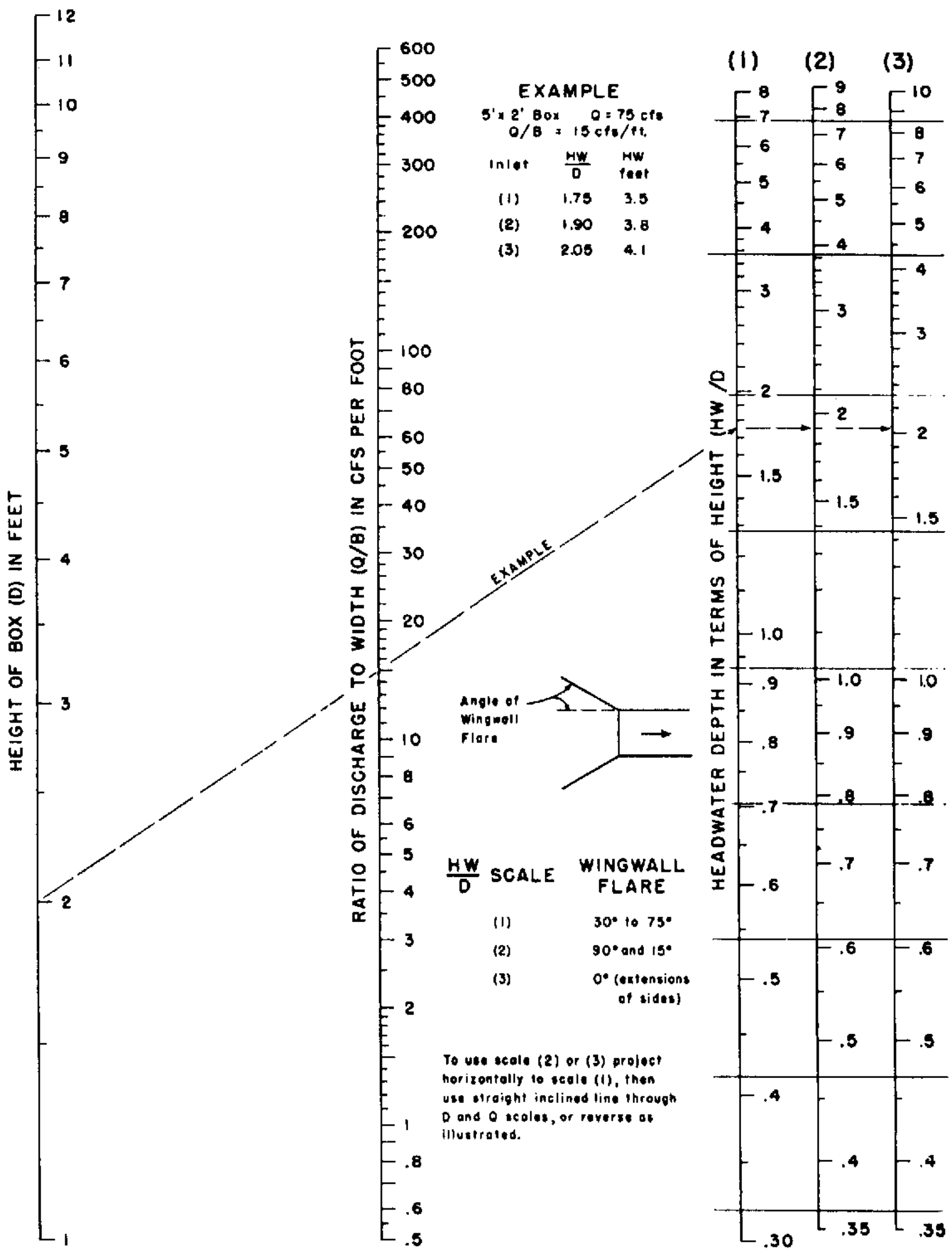
# CHART 8B




HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

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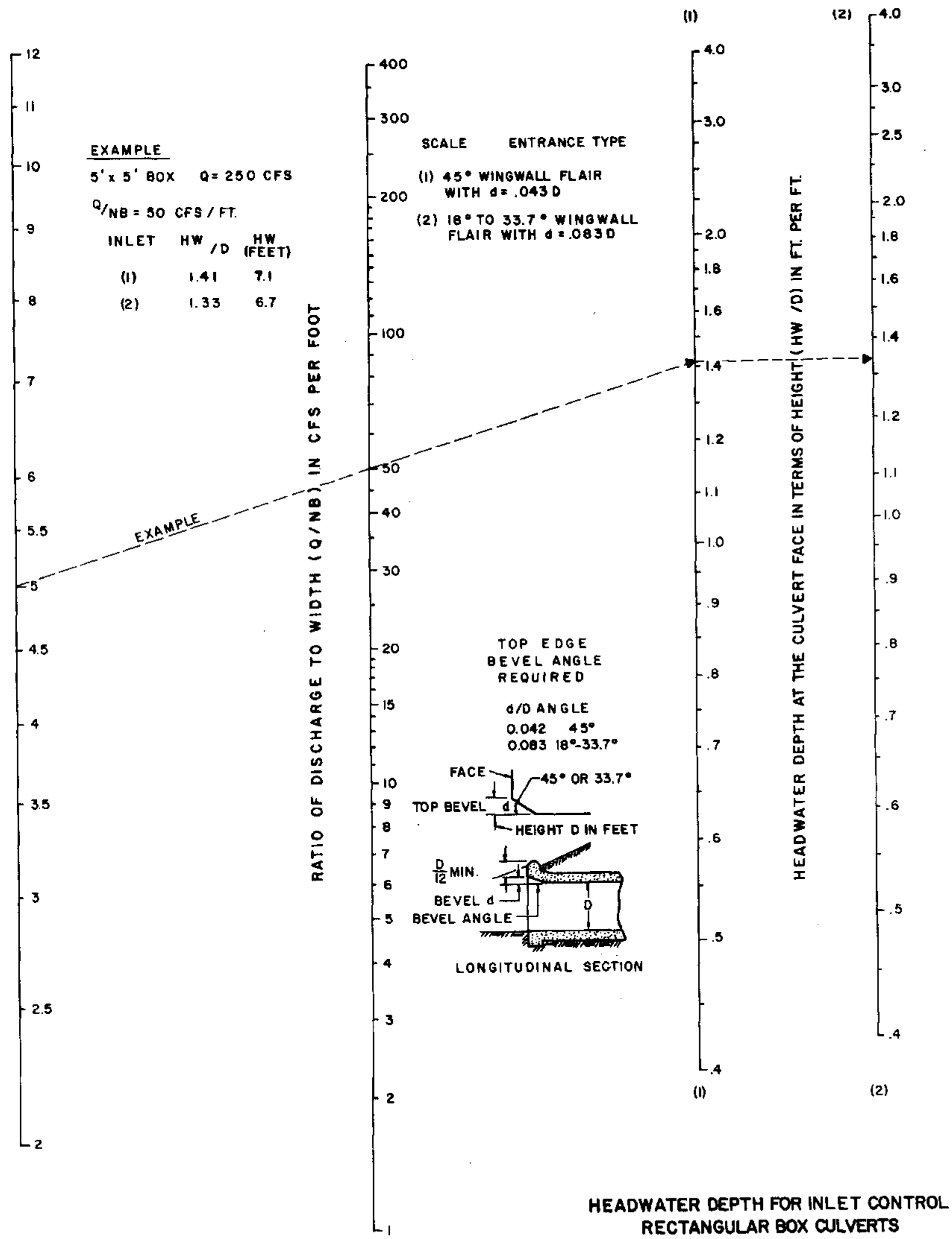
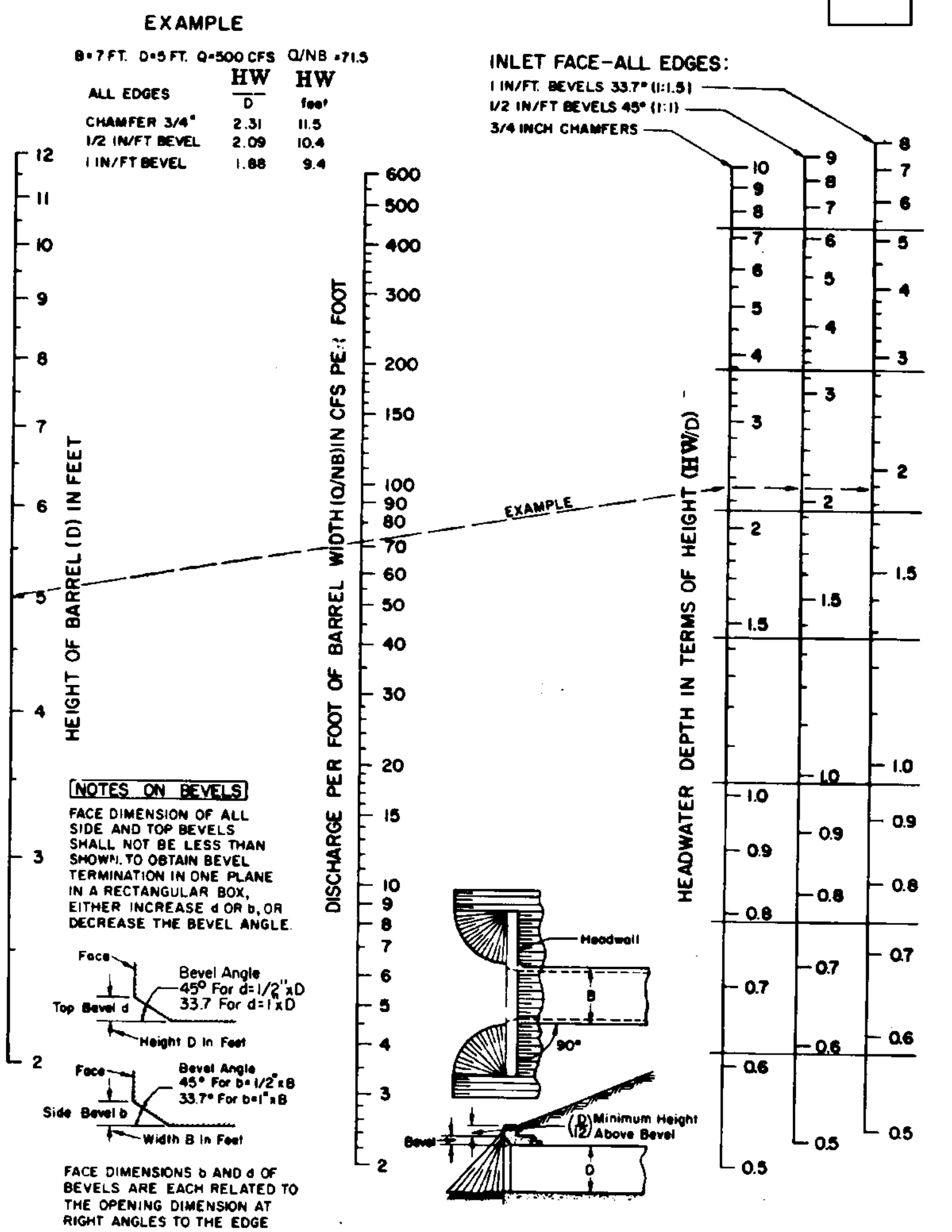
## CHART 9B




**HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET**

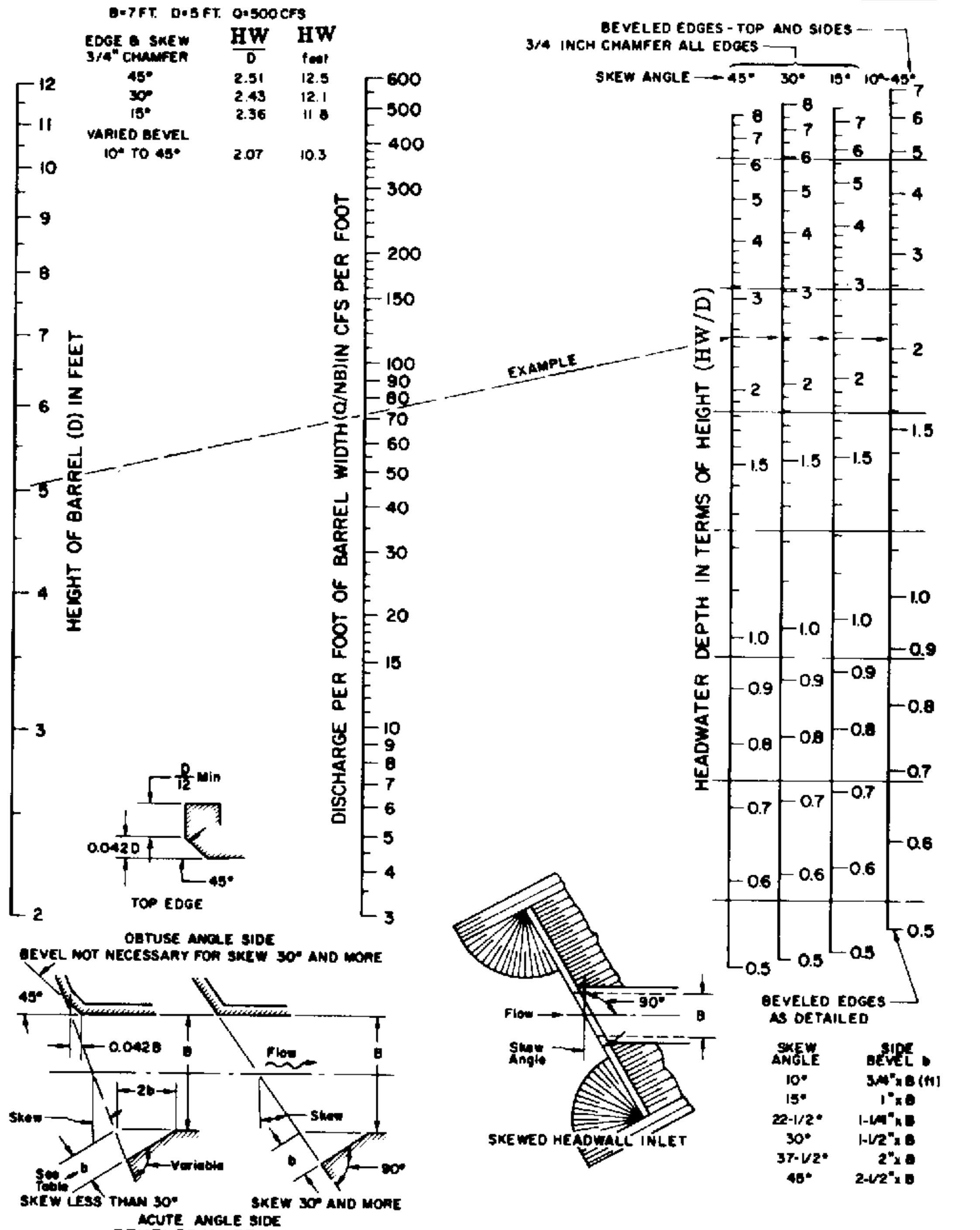
# CHART 10B




HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

# CHART 11B

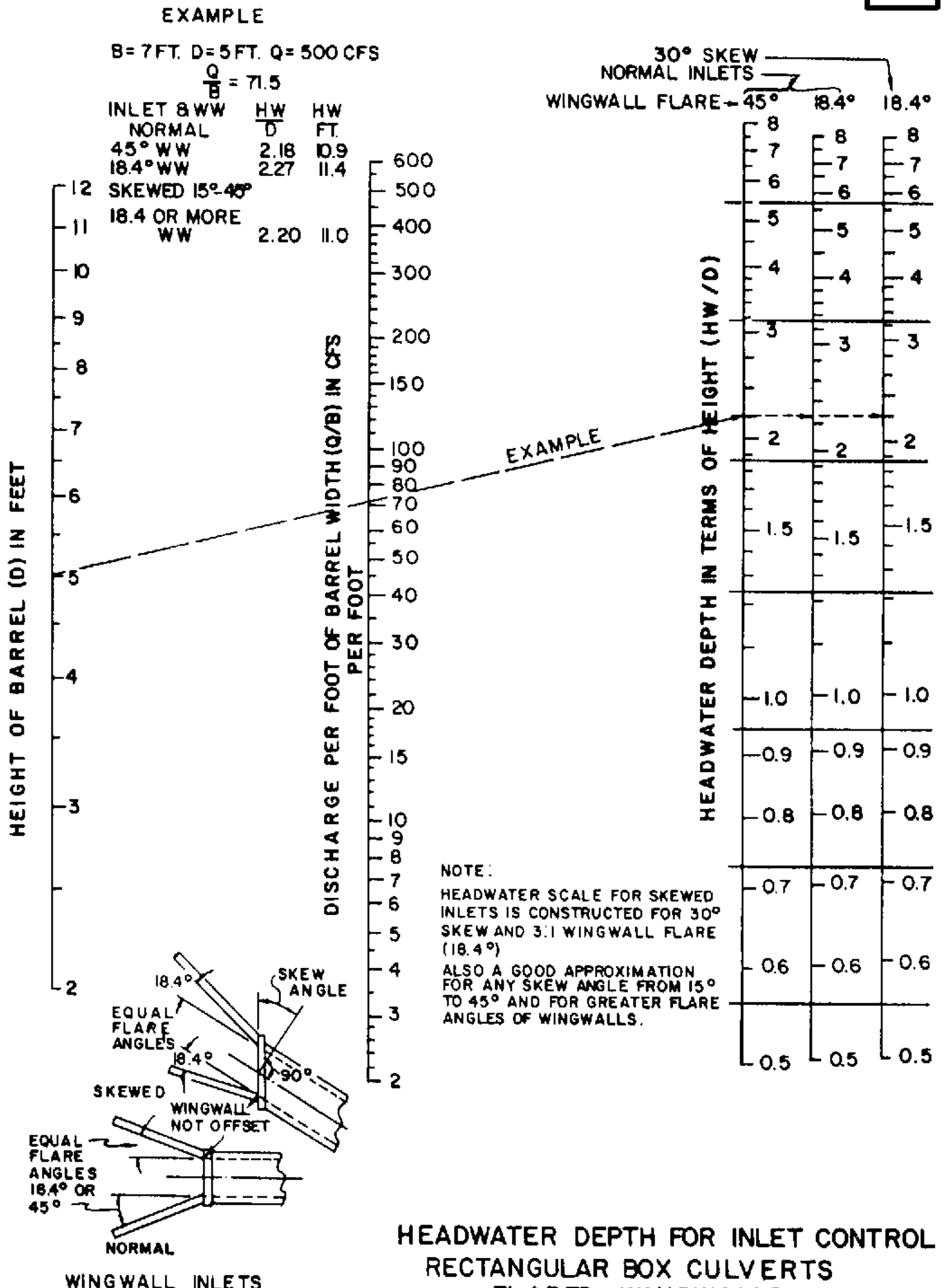



BEVELED INLET EDGES
DESIGNED FOR SAME CAPACITY AT ANY SKEW

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

## HEADWATER DEPTH FOR INLET CONTROL
## SINGLE BARREL BOX CULVERTS
## SKEWED HEADWALLS
## CHAMFERED OR BEVELED INLET EDGES

# CHART 12B




BUREAU OF PUBLIC ROADS
OFFICE OF R & D AUGUST 1968

HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS
NORMAL AND SKEWED INLETS
3/4" CHAMFER AT TOP OF OPENING

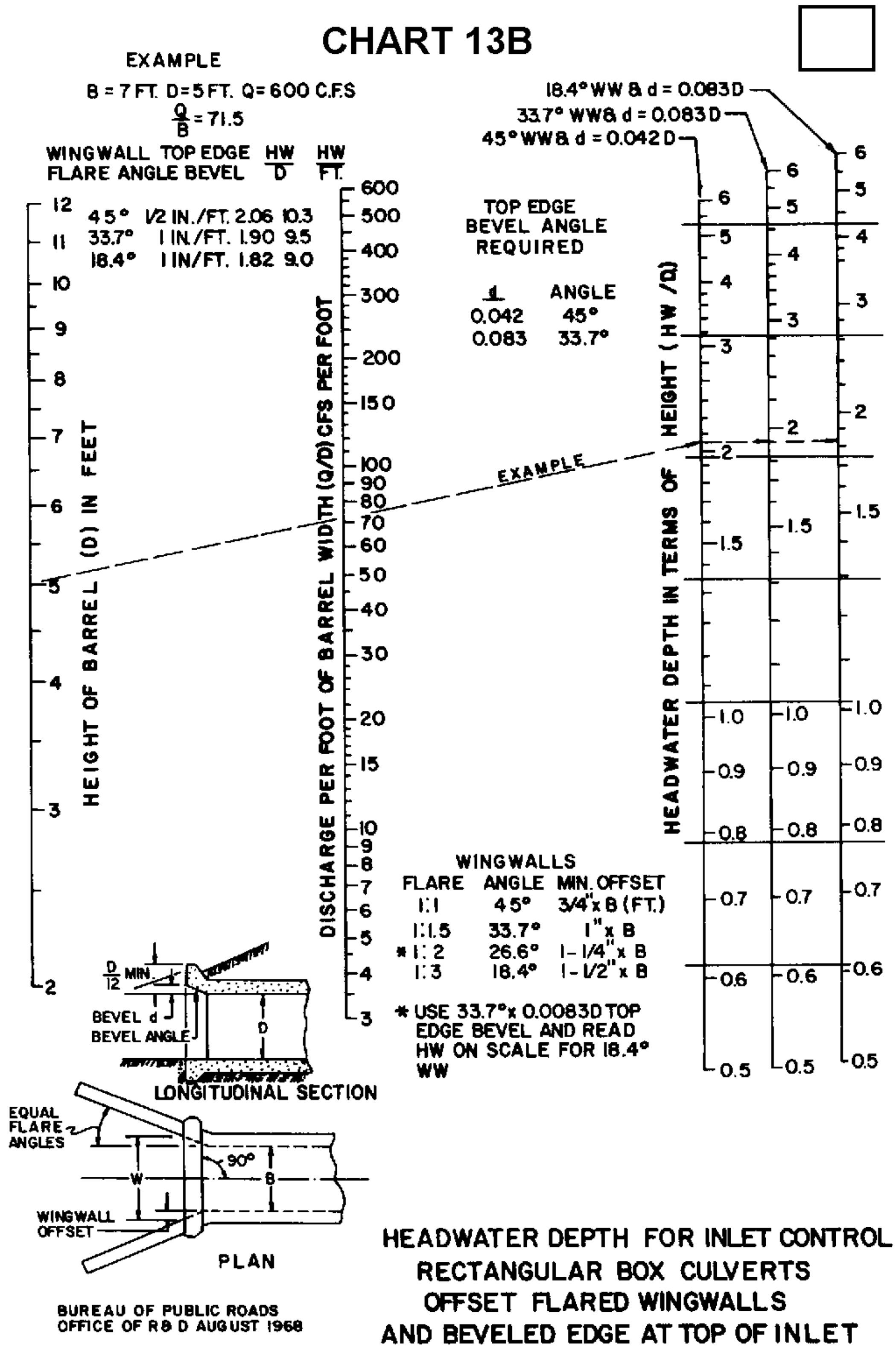
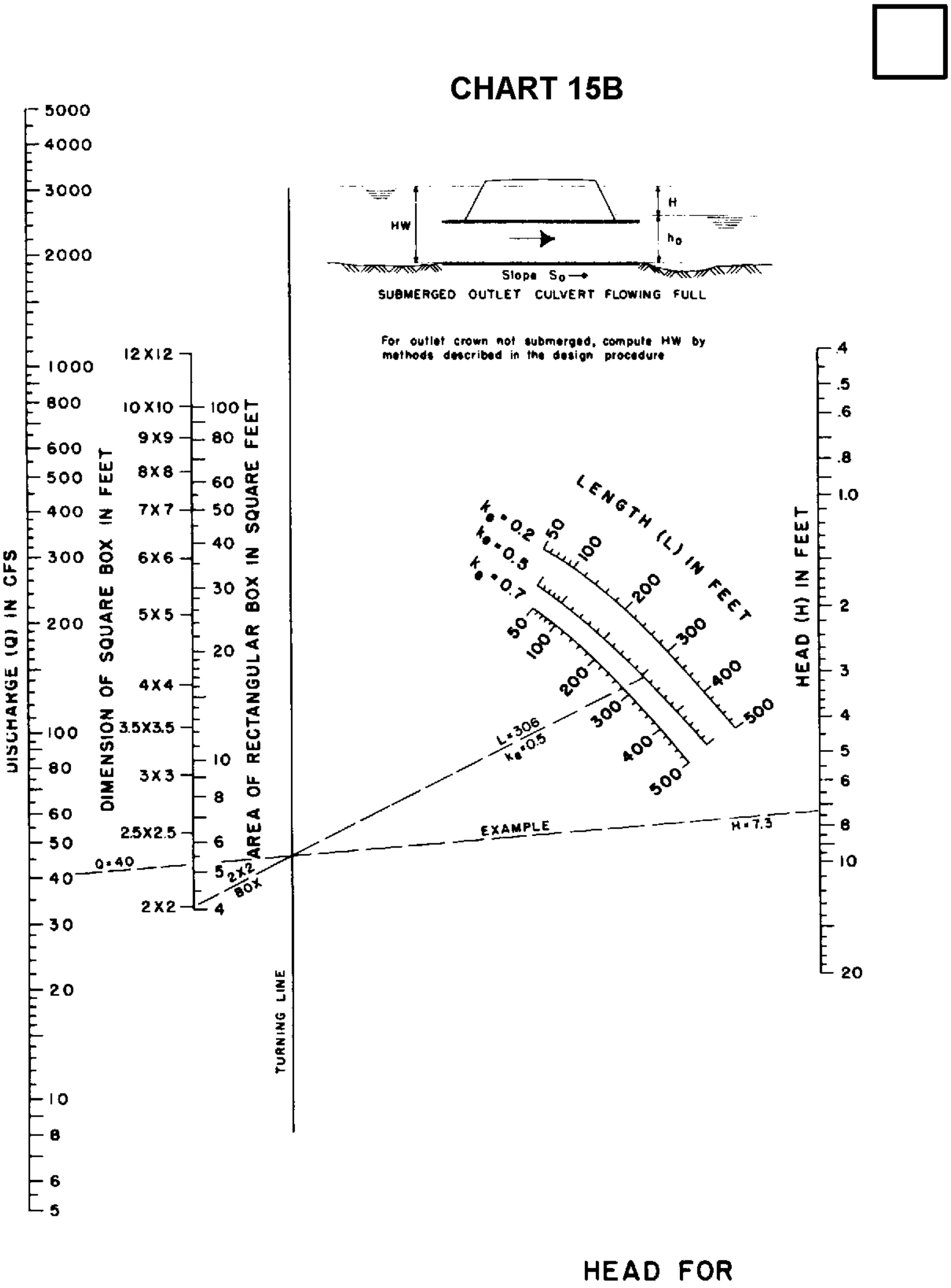


CHART 13B
EXAMPLE
B = 7 FT. D = 5 FT. Q = 600 C.F.S
Q/B = 71.5
WINGWALL FLARE ANGLE
TOP EDGE BEVEL
HW/D
HW FT.
45° 1/2 IN./FT. 2.06 10.3
33.7° 1 IN./FT. 1.90 9.5
18.4° 1 IN./FT. 1.82 9.0
HEIGHT OF BARREL (D) IN FEET
DISCHARGE PER FOOT OF BARREL WIDTH (Q/D) CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
18.4° WW & d = 0.083D
33.7° WW & d = 0.083D
45° WW & d = 0.042D
TOP EDGE BEVEL ANGLE REQUIRED
d ANGLE
0.042 45°
0.083 33.7°
EXAMPLE
WINGWALLS
FLARE ANGLE MIN. OFFSET
1:1 45° 3/4" x B (FT.)
1:1.5 33.7° 1" x B
* 1:2 26.6° 1-1/4" x B
1:3 18.4° 1-1/2" x B
* USE 33.7° x 0.0083D TOP EDGE BEVEL AND READ HW ON SCALE FOR 18.4° WW
D/12 MIN.
BEVEL d
BEVEL ANGLE
D
LONGITUDINAL SECTION
EQUAL FLARE ANGLES
90°
W
B
WINGWALL OFFSET
PLAN
BUREAU OF PUBLIC ROADS
OFFICE OF R & D AUGUST 1968
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
OFFSET FLARED WINGWALLS
AND BEVELED EDGE AT TOP OF INLET

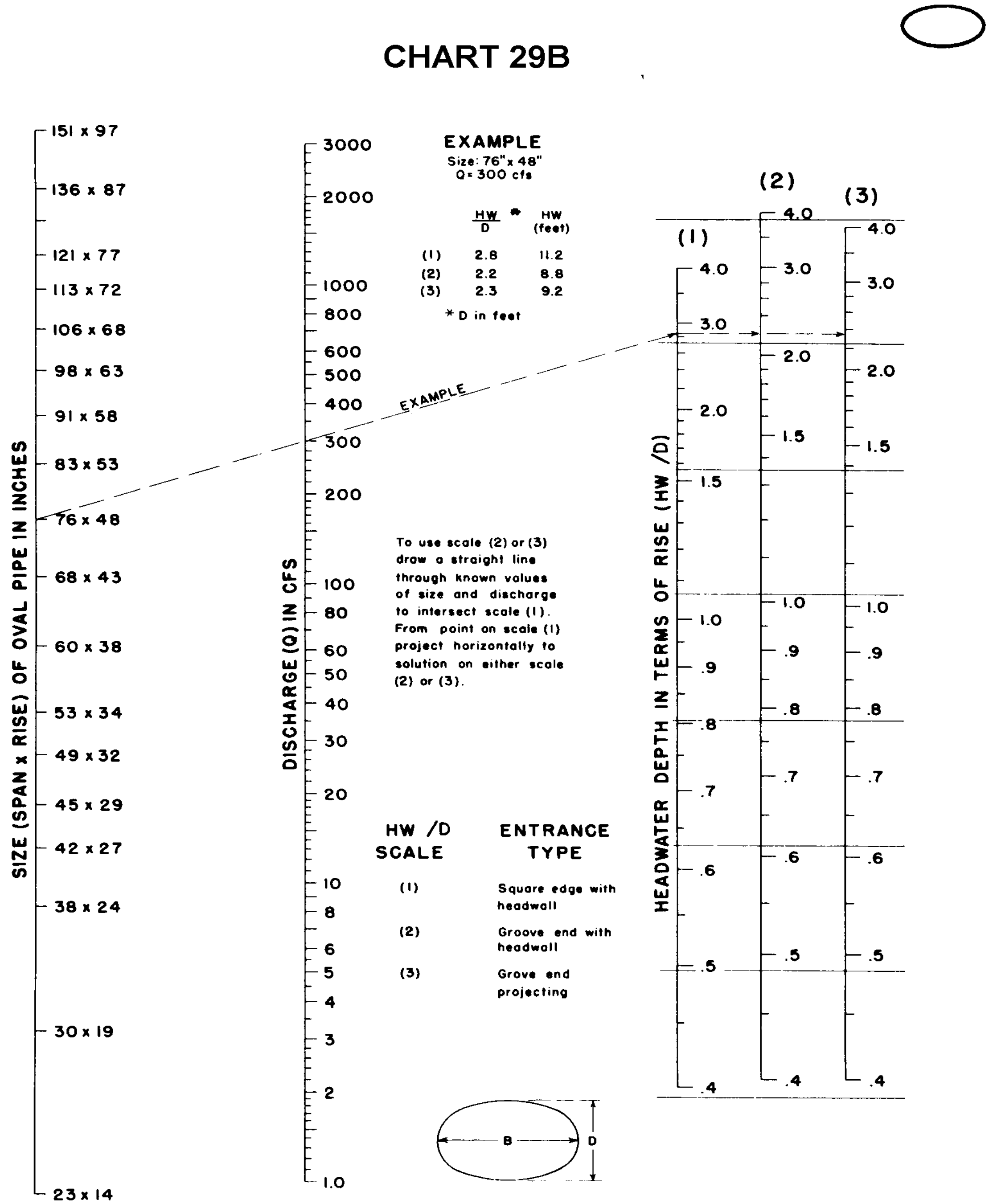
# CHART 15B




HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

AU OF PUBLIC ROADS JAN. 1963

## CHART 29B




HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

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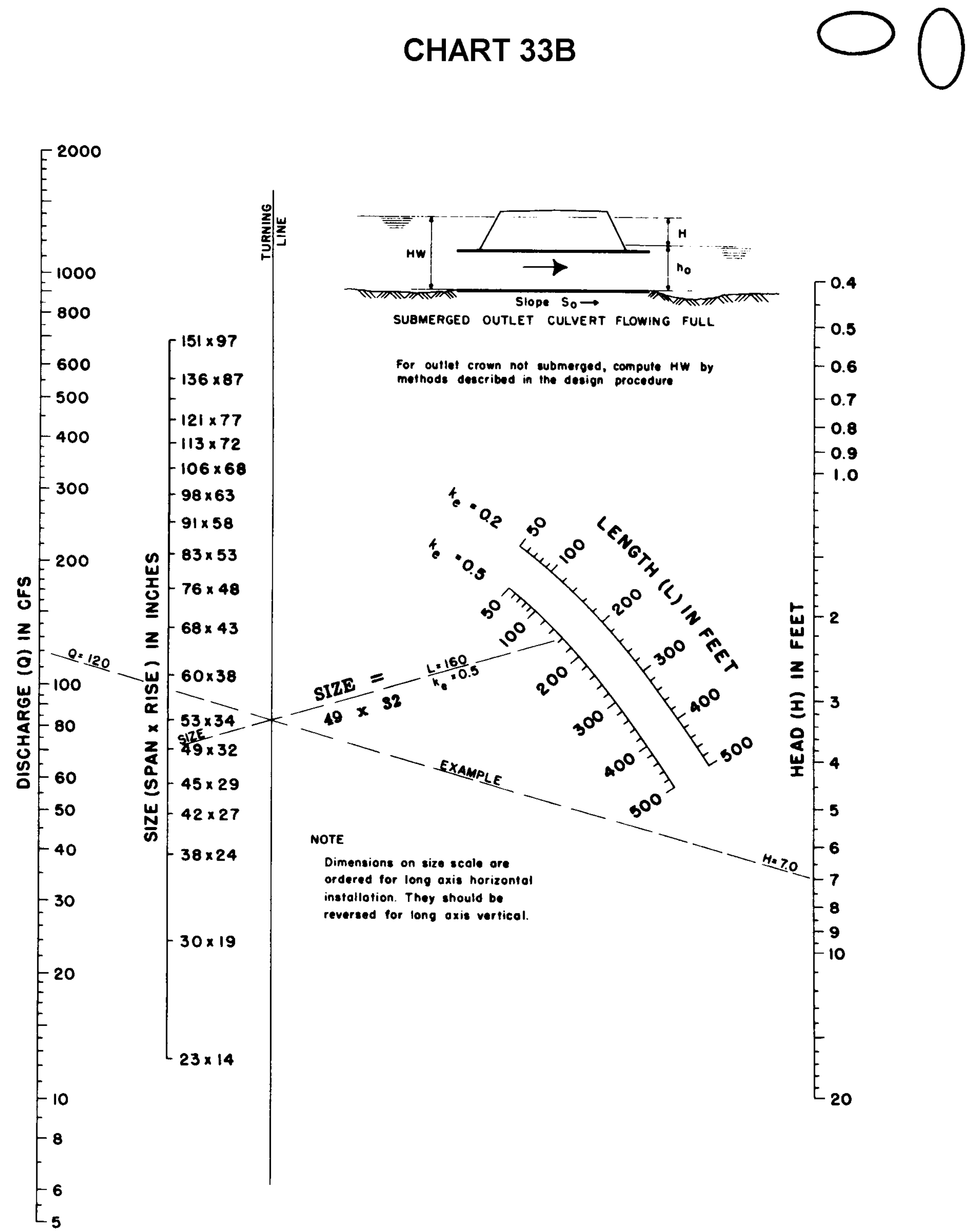
# CHART 33B




**HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL**

**n = 0.012**

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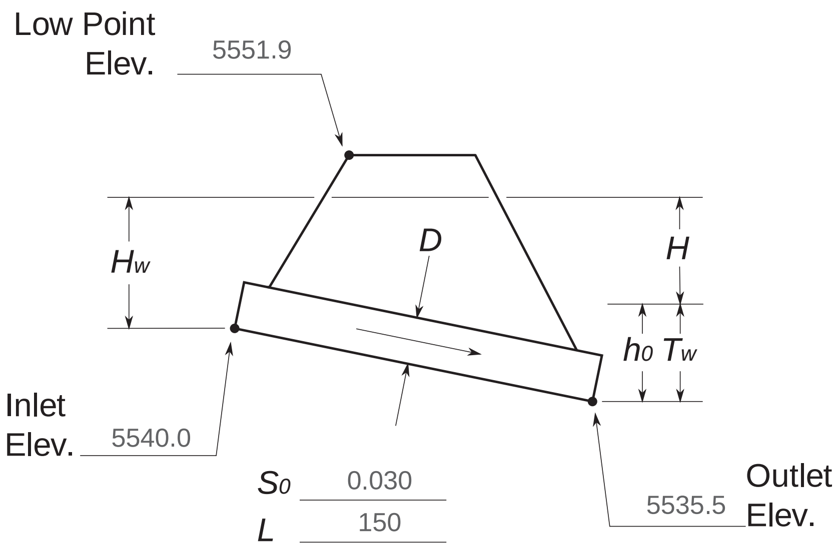
| **Figure 1000-10 Culvert Design Form Example** | Project: Example #19 | Consultant: Civil Engineering Inc | Date: 10/23/2015 |
|---|---|---|---|
| | Route/Highway: Main St | Engineer: Jane Doe | |
| | Mile Marker: 123 | Other: Culvert Replacement | |




### Culvert Data

| | |
|---|---|
| Size: 48" | $n$: 0.024 |
| Material: CMP | $Q_{Full}$: 135 |
| Inlet Style: Flared End Section | $V_{Full}$: 10.75 |
| $k_e$: 0.5 | V = Q/A |

### Outlet Control Equations

(1) $H_w = H + h_0 - LS_0$

(2) For $T_w < D, h_0 = \frac{d_c+D}{2}$ or $T_w$ (whichever is greater)

$T_w > D, h_0 = T_w$

(3) For Box Culvert: $d_c = 0.315\left(\frac{Q}{B}\right)^{2/3}$ (B = width of the box culvert)

| $Q$ | Inlet Control | | Outlet Control | | | | | | Design Curve | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $\frac{H_w}{D}$ | $H_w$ | $H$ | $T_w$ | $T_w < D$: $d_c$ | $T_w < D$: $h_0 = \frac{d_c + D}{2}$ | $T_w > D$: $h_0$ | $H_w$ | $H_w$ | Control Method | WSEL |
| Col 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| 73 | 1.0 | 4.0 | 2.2 | 1.5 | 2.6 | 3.3 | | 1.0 | 4.0 | Inlet | 5544.0 |
| 115 | 1.5 | 6.0 | 5.3 | 2.0 | 3.2 | 3.6 | | 4.4 | 6.0 | Inlet | 5546.0 |
| 145 | 2.0 | 8.0 | 8.4 | 2.5 | 3.6 | 3.8 | | 7.7 | 8.0 | Inlet | 5548.0 |
| 170 | 2.5 | 10.0 | 11.6 | 3.0 | 3.7 | 3.9 | | 11.0 | 11.0 | Outlet | 5551.0 |
| 195 | 3.0 | 12.0 | 15.3 | 3.5 | 3.8 | 3.9 | | 14.7 | 14.7 | Outlet | 5554.7 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

## Figure 1000-11 – Low Water Crossing Design Example

**Step 1** Calculating Existing 100-Year WSEL

| 100-Yr WSEL (ft.) | Depth (ft.) | Area (ft.) | Wetted Perimeter (ft.) | Hydraulic Radius (ft.) |
|---|---|---|---|---|
| 5404.0 | 4.0 | 65.0 | 38.59 | 1.685 |
| 5405.0 | 5.0 | 111.0 | 58.68 | 1.891 |
| *5406.0* | *6.0* | *177.0* | *78.78* | *2.247* |
| 5407.0 | 7.0 | 263.0 | 98.88 | 2.660 |
| 5408.0 | 8.0 | 369.0 | 119.0 | 3.101 |

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

$$1431\ cfs = \frac{1.49}{0.04}(177)(2.247)^{2/3}(0.016)^{1/2}$$

**Step 3** Calculating Proposed Road Grade Elevation

| Road Deck Elev (ft.) | Area (ft.) | Wetted Perimeter (ft.) | Hydraulic Radius (ft.) | Discharge (cfs) |
|---|---|---|---|---|
| 5403.0 | 224.0 | 96.40 | 2.324 | 1852 |
| 5403.5 | 213.5 | 96.35 | 2.216 | 1710 |
| 5404.0 | 198.0 | 96.30 | 2.056 | 1509 |
| *5404.1* | *194.3* | *96.29* | *2.018* | *1462* |
| 5404.2 | 190.4 | 96.28 | 1.978 | 1413 |
| 5404.5 | 177.5 | 96.25 | 1.844 | 1258 |
| 5405.0 | 152.0 | 96.20 | 1.580 | 971.6 |

$$1462\ cfs = \frac{1.49}{0.04}(194.3)(2.018)^{2/3}(0.016)^{1/2}$$

Channel Geometry (Not to Scale)