Chapter 10

CULVERTS

SOUTH DAKOTA DRAINAGE MANUAL

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Chapter 10
CULVERTS

10.1 INTRODUCTION

10.1.1 Definition

A culvert is defined as the following:

- A structure that can be designed hydraulically to take advantage of submergence to increase hydraulic capacity.
- A structure used to convey surface runoff through embankments.
- A structure, as distinguished from bridges, that is usually covered with embankment and is 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.
- A structure (bridge) designed hydraulically as a culvert is addressed in this Chapter, regardless of its span length.

10.1.2 Purpose

Chapter 10 provides design procedures for the hydraulic design of highway culverts that are based on FHWA Hydraulic Design Series No. 5 (HDS 5), *Hydraulic Design of Highway Culverts* (Reference (1)). The AASHTO *Highway Drainage Guidelines*, Chapter 4 (Reference (2)), provides an overview of highway culverts.

10.1.3 Concepts

The following concepts are important in culvert design:

1. Critical Depth. In channels with regular cross section, critical depth is the depth at which the specific energy of a given flow rate is at a minimum. For a given discharge and cross-section geometry, there is only one critical depth.
2. Crown. The crown is the inside top of the culvert.
3. Flow Type. USGS (Reference (3)) has established seven culvert flow types, which assist in determining the flow conditions at a particular culvert site. Diagrams of these flow types are provided in Section 10.4.

4. 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.

5. Improved Inlet. An improved inlet has an entrance geometry that decreases the flow contraction at the inlet and thus increases the capacity of culverts. These inlets are referred to as either side- or slope-tapered. The side-tapered inlet has a face wider than the culvert. The slope-tapered inlet has both a larger face and increased flow-line slope at the entrance. Beveled edges at the culvert face may also improve the hydraulic capacity of a culvert for both conventional and improved inlets.

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

7. Normal Depth. Normal depth occurs in a channel or culvert when the slope of the water surface and channel bottom is the same and the water depth remains constant. The discharge and velocity are constant throughout the reach. Normal flow will exist in a culvert operating on a constant slope provided that the culvert is sufficiently long.

8. Slope. A steep slope occurs where the normal depth is less than the critical depth. A mild slope occurs where the normal depth is greater than the critical depth.

9. Submerged. 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 1.2D, where D is the culvert diameter or barrel height.

10.1.4 Symbols

To provide consistency within this Chapter, the symbols given in Figure 10.1-A will be used. These symbols were selected because of their wide use in culvert publications.

Symbol	Definition	Units
A	Area of cross section of flow	sq ft
AHW	Allowable HW	ft
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	—
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/sec^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	Head loss at junction	ft
H_g	Head loss 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 indicates section)	ft
HW_i	Headwater depth above inlet invert	ft
HW_o	Headwater depth above the outlet invert	ft
HW_r	Upstream depth, measured above the roadway crest	ft
k_e	Entrance loss coefficient	—
L	Length of culvert	ft
n	Manning's roughness coefficient	—
P	Wetted perimeter	ft
Q	Rate of discharge	cfs
Q_d	Design discharge	cfs
Q_o	Overtopping flow	cfs
Q_r	Routed (reduced) peak flow	cfs
R	Hydraulic radius (A/P)	ft
S	Slope of culvert	ft/ft
S_o	Slope of streambed	ft/ft
TW	Tailwater depth above outlet invert of culvert	ft
V	Mean velocity of flow with barrel full	fps
V_d	Mean velocity in downstream channel	fps
V_o	Mean velocity of flow at culvert outlet	fps
V_u	Mean velocity in upstream channel	fps
γ	Unit weight of water	pcf
τ	Tractive force	psf

Figure 10.1-A — SYMBOLS AND DEFINITIONS

10.2 STANDARD PRACTICES

The following standard practices apply to culverts:

- 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 (see Section 7.6.2).
- Survey information should include topographic features, channel characteristics, aquatic life, high-water information, existing structures and other related site-specific information.
- 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 use (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 6 "Documentation of Hydraulic Studies".
- Where practical, some means should be provided for personnel and equipment access to facilitate maintenance.

10.3 DESIGN CRITERIA

10.3.1 Site Criteria

10.3.1.1 Structure Type Selection

Culverts are used:

- where bridges are not hydraulically required,
- where debris and ice potential are tolerable, and
- where more economical than a bridge (including guardrail and safety concerns).

Bridges are used:

- where culverts are impractical,
- where more economical than a culvert,
- to satisfy land-use requirements,
- to mitigate environmental harm caused by a culvert,
- to avoid floodway encroachments, and
- to accommodate ice and large debris.

10.3.1.2 Length and Slope

The culvert length and slope should be chosen to approximate existing topography and as practical:

- The culvert invert should be aligned with the channel bottom and the skew angle of the stream.

- The culvert ends should match the geometry of the roadway embankment.

10.3.1.3 Ice Buildup

Ice buildup should be mitigated as necessary by:

- assessing the flood damage potential resulting from a plugged culvert,
- considering increasing the size, and/or
- using extended middle walls (on multiple-cell box culverts).

10.3.1.4 Debris Control

Debris control should be designed using HEC 9 (Reference (4)) and should be considered:

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

A debris wall should be used for multiple-cell box culverts where debris/ice might be a problem. The wall is an extension of the interior wall at the inlet end that tapers at 2H:1V to the streambed. The height of the wall at the end should match that of the exterior walls.

10.3.2 Design Limitations

10.3.2.1 Allowable Design Headwater

The designer should determine if any of the following constraints or commitments will establish the allowable design headwater before applying the standard criteria:

- land use in watershed,
- potential property damage,
- FEMA 100-year flood requirements,
- debris impacts,
- historic high water,
- outlet velocity, and
- duration of flow.

Culverts should be sized so that the water elevation (associated with the design frequency; see Figure 7.6-A) should be at or below the following to prevent water from encroaching onto the roadway and to prevent overflow to a different drainage basin. If this cannot be achieved, consider overflow impacts to downstream (adjacent) culverts. The maximum allowable design headwater should not exceed the following:

- one foot below the low subgrade shoulder at the lowest point of the roadway within the drainage basin,
- top of ditch block,
- finished top of approach, and
- summit of ditch grade.

For intersecting roads, analyze the appropriate storm design frequency for the designated road classification as in Figure 7.6-A. The following also applies:

- Interstates, arterials and collectors: The maximum design headwater should meet the first criteria above.
- Local roads: The maximum design headwater should be at the edge of traveled way.

In addition to the specific allowable design headwater criteria above, the following guidelines provide the designer with general parameters for sizing culverts to limit excessive headwater and ponding based on different site/terrain conditions:

Terrain	Allowable Headwater Guidelines
Flat/Cropland	Diameter of pipe plus 1.0 ft
Rolling	1.5 times diameter of pipe
Mountainous	2 times diameter of pipe

10.3.2.2 Allowable Headwater for Temporary On-Site Traffic Diversions

The diversion should be designed for the design frequency determined using the risk rating procedure provided in Chapter 14 "Bridge Hydraulics," Appendix 14.B. Factors in the estimation process include average daily traffic (ADT), loss of life, property damage, alternative detour length, height above streambed, drainage area and traffic interruptions. Where practical, the traffic diversion profile grade should be low enough for overtopping at higher flood frequencies without creating excessive backwater. Where upstream insurable buildings could be affected, the traffic diversion and drainage structures should be sized to prevent an increase in the upstream Q_{100} water surface elevations over existing conditions. FEMA requirements should be met where applicable.

10.3.2.3 Review Headwater

The headwater for the review frequency (see Section 7.6.2.3) should:

- not overtop Interstate and NHS highways;
- not exceed a 1-ft increase over the existing 100-year flood elevation in the National Flood Insurance Program mapped floodplains or in the vicinity of insurable buildings;
- have a level of inundation that is tolerable to upstream property and roadway for the review discharge; and

- consider a duration or inundation that is tolerable to the upstream vegetation to avoid crop damage.

10.3.2.4 Tailwater Relationship (Channel)

- 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 9 "Roadside 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 the approximate hydraulic gradeline if the culvert outlet is operating with a free outfall.
- Use the headwater elevation of any nearby, downstream structure or restrictive channel if it is greater than the channel depth.

10.3.2.5 Tailwater Relationship (Confluence or Large-Water Body)

Where the culvert is located on a tributary that joins with a larger body of water immediately downstream:

- Use the downstream high-water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent).
- If statistically independent, use a likely combination resulting in the greater tailwater depth (worst-case scenario).

10.3.2.6 Maximum Outlet Velocity

The maximum velocity at the culvert exit should be consistent with the velocity in the natural channel or should be mitigated with:

- channel stabilization (see Chapter 15 "Bank Protection"), and/or
- energy dissipation (see Chapter 11 "Energy Dissipators").

See Section 10.4.6 for specific criteria.

10.3.2.7 Minimum Culvert Velocity

The minimum velocity in the culvert barrel should result in a tractive force ($\tau = \gamma dS$) greater than critical τ of the transported streambed material at low-flow rates (see also Section 9.4 for channel design practices and procedures):

- Use 3 fps when streambed material size is not known.
- If clogging is probable, consider the installation of a sediment trap or a size of culvert to facilitate cleaning.

10.3.2.8 Storage (Temporary or Permanent)

Storage will normally not be considered in culvert design for rural areas. If storage is being assumed upstream of the culvert, consideration should be given to:

- limiting the total area of flooding,
- limiting the average time that bankfull stage is exceeded for the design flood to 24 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 through the purchase of right-of-way or easement.

10.3.3 Culvert Design Features

10.3.3.1 Culvert Shape and Material Selection

The *SDDOT Standard Plates* are provided for those culvert types most often used by the Department. The *Standard Plates* include:

- reinforced circular concrete pipe,
- reinforced concrete pipe arch,
- circular corrugated metal pipes,
- corrugated metal pipe arch,
- precast single box culvert, and
- precast double box culvert.

The allowable culvert materials are based on installation location, service life and soil conditions.

The pipe culvert materials allowed for use in South Dakota highways include reinforced concrete (RCP), corrugated metal (CMP) and high density polyethylene (HDPE). The locations and soil conditions where each material is allowed are specified below.

The Materials and Surfacing Office will provide a soils report with a list of the corrosivity ratings established by the Natural Resources Conservation Service (NRCS). Any recommendation of culvert type to exclude from use, if any, will be included in the soils report.

Soil corrosivity has the most effect on the durability or service life of pipes in SD with CMP being the most susceptible. The NRCS rates of corrosivity and the allowable materials provided in the guidelines have been found by past experience to meet these service life requirements. The durability of CMP is also more susceptible to abrasion from flowing bed material than the other culvert types.

Figure 10.3-A — CULVERT MATERIAL TYPE SECTION AND SERVICE LIFE GUIDELINES

Pipe Culvert Installation Location	Service Life	Allowable Pipe Culvert Material
Mainline Cross Pipe – Interstate, US & State Highways	75 years	RCP
Intersecting Roads and Entrances with Paved Surfacing (AC or PCCP)	50 years	RCP, CMP [1] & HDPE [3]
Intersecting Roads and Entrances with Non-Paved Surfacing	25 years	RCP, CMP [2] & HDPE [3]
Storm Sewer	75 years	RCP
Critical Locations [4]	75 years	RCP

[1] CMP is allowed in soils with a low to moderate soil corrosivity rating.

[2] CMP is allowed in all soils, except where not recommended by the Materials and Surfacing Office.

[3] The maximum allowable diameter of HDPE pipe that can be installed is 36”. The end sections for HDPE pipe shall be metal, conform to the type of end section as shown in the plans, and be compatible with the HDPE pipe.

[4] Critical locations are considered to be culvert installation sites where premature replacement would involve inordinate expense, severe inconvenience or safety considerations, such as high fills.

CMP should be specified in the plans at locations where CMP is an allowable material and soils meet the allowable corrosivity rating.

HDPE pipe and RCP may be substituted for CMP in intersecting roads and entrances by the Contractor at no additional cost to the State. The designer will need to verify with the County or Township what pipe material substitutions are allowed for pipes installed under their roads that fall outside the State’s highway right-of-way width.

Flexible pipe (CMP and HDPE) are not recommended for use in highway mainline, storm sewer and critical locations based on the following disadvantages:

a. Customer Service Survey indicates that a smooth riding roadway is very important to our customers. The flexible pipe requires granular backfill which creates a potential conduit for water to enter the highway embankment and creates potential frost heave problems in shallow embankments. Considerations also need to be made for the availability and cost of the granular backfill.

b. Flexible pipes have less structural strength. Much of the pipe strength is dependent on the surrounding backfill support or allowable soil bearing pressure.

c. Installation requirements are more stringent and require more inspection.

d. More pipe alignment deflection tends to occur during backfilling due to the lighter weight and greater flexibility of these pipes.

e. The lighter weight and buoyancy of these materials create the potential for the pipe to float when inundated with water, particularly at the end sections. These pipes may require the ends to be weighted or more securely fastened to the ground, such as with concrete headwalls.

f. There are greater limitations on the fill height allowed over the pipe.

The major advantages for the use of concrete pipe are generally opposite of the above disadvantages for flexible pipe. These items are major factors in pipe culvert installations.

Where a reinforced concrete box culvert (RCBC) is used, SDDOT practice is to design both cast-in-place and precast options where applicable. The contractor is allowed to select the option unless the RCBC has a special end treatment that (in rare cases) must be cast-in-place. However, in rare instances, it may be acceptable to use a precast barrel with a cast-in-place end section with approval of the Bridge Design Engineer.

10.3.3.2 Culvert Cover Depth and Strength Requirements

Reinforced Concrete Pipe

Strength requirements for concrete pipe culverts are shown below in Figure 10.3-B in terms of maximum allowable cover for various diameters and classes of pipe. The minimum subgrade cover over concrete pipe shall be the greater of 12” or 1/8 the pipe inside rise. In rare cases where the subgrade cover cannot be obtained and only under rigid PCC pavements, the cover can be measured from the top of the rigid pavement as long as there is a minimum of 9” of compacted granular fill between top of pipe and bottom of pavement slab.

Class 2 pipe with normal backfill is the standard design. Any other combination of pipe class and backfill must be clearly identified on the plans.

For depths requiring greater than Class 5 pipe, special classes of pipe are available (Class 4000 D, 5000 D, and 6000 D). These sites will have to be designed on a site by site basis.

When class of pipe required is 4 or higher, step down the class of pipe as the cover decreases. Go as low as Class 2 only if the lengths at the end are more than 16’. As a guideline, do not step down from Class 3 to Class 2 and use only one or the other. Examples of typical pipe class stepping are 4-3 or 4-2 and 5-4-3 or 5-4-2.

Class of pipe under railroad tracks require special considerations. Typically, Class 5 pipe is used for a distance of 25’ to 30’ (8-10 m) either side of centerline of the tracks or to the ROW line, whichever is less. However, the Department’s Railroad engineer should be contacted for each specific case.

Figure 10.3-B — RC PIPE MAXIMUM ALLOWABLE DEPTH OF COVER

RCP Round				
Diam.	Class 2	Class 3	Class 4	Class 5
12”	10’	14’	22’	33’
18”	11’	15’	22’	33’
24”	10’	14’	22’	33’
30”	10’	14’	22’	33’
36”	10’	14’	21’	32’
42”	10’	14’	21’	32’
48”	10’	14’	21’	32’
54”	10’	13’	21’	32’
60”	9’	13’	20’	31’
66”	9’	13’	20’	31’
72”	9’	13’	20’	31’
78”	9’	13’	20’	31’
84”	9’	13’	20’	31’
90”	9’	13’	20’	31’
96”	9’	12’	20’	31’
102”	8’	12’	19’	31’
108”	8’	12’	19’	30’

RCP Arch				
Eq. Diam.	Class 2	Class 3	Class 4	Class 5
18”	8’	11’	17’	25’
24”	8’	11’	16’	25’
30”	8’	11’	17’	25’
36”	8’	11’	17’	26’
42”	8’	11’	17’	26’
48”	8’	11’	17’	26’
54”	8’	11’	17’	26’
60”	8’	11’	17’	26’
72”	8’	11’	17’	26’
84”	7’	11’	17’	26’
96”	7’	11’	17’	26’
108”	7’	10’	17’	26’
120”	7’	10’	16’	26’
132”	7’	10’	17’	26’

<u>Corrugated Metal Pipe</u>

The thickness or gauge & corrugation of the pipe varies with the height of cover above the top of the culvert. The following tables give the gauge & corrugation of corrugated metal pipes required under various heights of cover. Pipe size, gauge, corrugation, and maximum allowable depth of cover are shown in Figure 10.3-C and Figure 10.3-D.

The minimum subgrade cover over corrugated metal pipe required for normal highway live loads (H20 or H25) varies from 12 to 24 inches and depends on the type and size of pipe. See the following tables. In rare cases where the required subgrade cover cannot be obtained and only under rigid PCC pavements, the cover can be measured from the top of the rigid pavement.

The gauge & corrugation selected for each pipe, or segment of pipe, must be specified on the particular cross section, pipe estimate and the plan profile sheets of the construction plans.

Figure 10.3-C — CM PIPE MAXIMUM ALLOWABLE DEPTH OF COVER

	MAXIMUM DEPTH OF COVER (feet)												
	2-2/3 x ½ Corrugation			3 x 1 Corrugation					5 x 1 Corrugation				
Thickness, in.	0.064	0.079	0.109	0.064	0.079	0.109	0.138	0.168	0.064	0.079	0.109	0.138	0.168
Gauge	16	14	12	16	14	12	10	8	16	14	12	10	8
Diameter													
12”	213												
15”	170												
18”	142												
24”	106	133											
30”	85	106	149										
36”	71	88	124										
42”	60	76	106										
48”			93	61	76				54	68			
54”				54	68	95			48	60	84		
60”				48	61	85			43	54	76		
66”				44	55	78			39	49	69		
72”					51	71	92			45	63	81	
78”					47	66	84			41	58	75	
84”					43	61	78			38	54	70	
90”					40	57	73			36	50	65	
96”						53	69	84			47	61	75
102”						50	64	79			44	57	70
108”						47	61	75			42	54	66
114”						45	58	71			40	51	63
120”							55	67				49	60

CM pipes 96” & less shall use 12” minimum subgrade cover.
CM pipes greater than 96” shall use 18” minimum subgrade cover.

Figure 10.3-D — CM ARCH PIPE MAXIMUM ALLOWABLE DEPTH OF COVER

	MAXIMUM DEPTH OF COVER (feet)					
	2-2/3 x ½ Corrugation			3 x 1 & 5 x 1 Corrugation		
Thickness, in.	0.064	0.079	0.109	0.079	0.109	0.138
Gauge	16	14	12	14	12	10
Equiv. Diameter						
15”	6					
18”	6					
24”	6					
30”	6					
36”		6				
42”			6			
48”			6			
54”				10		
60”				10		
66”				10		
72”				8		
78”					8	
84”					8	
90”					8	
96”					8	
102”					8	
108”						8
114”						8
120”						8

CM Arch pipes 84” & less shall use 18” minimum subgrade cover.
CM Arch pipes greater than 84” shall use 24” minimum subgrade cover.

The maximum and minimum depth of cover for CM Arch Pipe is controlled by the allowable soil bearing pressure at the pipe corners. The above depths are based on a typical South Dakota allowable soil bearing pressure of 2000 psf. Increasing the pipe wall thickness or gauge will not increase the maximum allowable cover.

10.3.3.3 Culvert Size

For pipe culverts, typical sizes range from 18 in to 84 in, available in 6-in increments. For precast concrete box culverts, typical sizes are available from the industry.

The culvert size selected should be based on engineering and economic criteria related to site conditions. The following minimum sizes should be used to avoid construction, maintenance and clogging problems:

- Reinforced concrete box culverts: Minimum 3-ft height
- Cross culvert pipes: 24 in
- Culverts beneath major intersecting roads: 24 in
- Culverts beneath minor intersecting roads and approaches: 18 in

Cross culvert pipe that drains a median ditch under one set of lanes of a divided highway may be reduced to 18 inch if needed to obtain minimum subgrade cover requirements.

Cross pipe from drop inlet type structures, including median drains, should follow Chapter 12 Storm Drainage Systems criteria.

10.3.3.4 Multiple-Pipe Culverts

Multiple-pipe culverts may be preferable where fill height is limited. Where used, multiple-pipe culverts should fit within the natural dominant channel with minor widening of the channel to avoid conveyance loss through sediment deposition in one or more of the barrels. They should be avoided where:

- the approach flow is high velocity, particularly if supercritical. (These sites require either a single barrel or special inlet treatment to avoid adverse hydraulic jump effects);

- 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.

Figure 10.3-E (concrete pipe) and Figure 10.3-F (metal pipe) present the recommended spacing between multiple-pipe installations. These recommended pipe spacings may be reduced where channel width or other topographic conditions dictate. The minimum spacing without considering control density fill is based on 12 ft between edges of pipe end sections.

When the required pipe spacing is less than the minimum, the smallest allowable pipe spacing with controlled density fill up to the pipe haunches should be provided as shown in Figure 10.3-G. Pipe spacing with controlled density fill is based on either 0 ft or 1 ft between pipe end sections and depends on the type of end section. The 1 ft between pipe end sections is provided for end sections that do not flare and are the same width as the pipe barrel. The 0 ft between pipe end sections for CMP sloped and safety ends assumes the end sections are placed as close together as possible by overlapping the perpendicular flare of the end sections. The pipe spacing and quantity of controlled density fill per foot of pipe length is provided in Figures 10.3-E and Figure 10.3-F.

Controlled density fill between metal pipes will require the pipes to be anchored down to resist buoyancy forces and prevent them from floating up. When using controlled density fill for metal pipes, a plan note shall be included stating that anchoring methods are determined by the contractor and incidental to other pipe bid items.

10.3.3.5 Culvert Slopes

The culvert slope should generally match the connecting channel slope, if practical. Culvert slopes can be flattened to induce sedimentation and facilitate fish passage. A 0% slope can be used for an equalizer-type culvert in a lake or ponded area. Maximum culvert slopes are as follows:

- For reinforced concrete pipe: 6%
- For corrugated metal pipe: 8%

Where the maximum slopes are exceeded, downspouts should be used (see Section 10.3.3.8) if the flow can be accommodated in a 36-in pipe. For larger flows, consult the Bridge Hydraulic Engineer. To dissipate excessive outlet velocities due to steep culvert slopes, see Chapter 11 “Energy Dissipators”.

10.3.3.6 Pipe Length Measurement for Culverts

Pipe lengths are measured along the culvert flow line. To avoid the need for cutting sections, the design length should be in increments of 2 ft. Typical rounding should be as in the following example: When the measured length is between 80.6 ft and 82.5 ft, use a design length of 82 ft.

Pipe Type	End Section	Pipe Size	Center-to-Center Spacing of Multiple Pipe				Controlled Density Fill (CDF) (cu yd/ft)
			Recommended Without CDF	Minimum Without Considering CDF	With CDF		
			20 ft between pipe barrels	12 ft between pipe end sections	1 ft between pipe end sections	0 ft between pipe end sections	
RCP	Safety	18 in	22 ft	14 ft	2.9 ft		0.050
		24 in	23 ft	14 ft	3.5 ft		0.071
	Sloped	24 in	23 ft	14 ft	3.5 ft		0.071
		30 in	23 ft	15 ft	4.1 ft		0.096
		36 in	24 ft	16 ft	4.7 ft		0.124
		42 in	24 ft	16 ft	5.3 ft		0.154
		48 in	25 ft	17 ft	5.8 ft		0.179
		54 in	25 ft	17 ft	6.4 ft		0.215
		60 in	26 ft	18 ft	7.0 ft		0.254
	Flared	18 in	22 ft	15 ft		3.4 ft	0.067
		24 in	23 ft	16 ft		4.5 ft	0.117
		30 in	23 ft	18 ft		5.6 ft	0.181
		36 in	24 ft	19 ft		6.7 ft	0.259
		42 in	24 ft	19 ft		7.3 ft	0.312
		48 in	25 ft	20 ft		7.8 ft	0.358
		54 in	25 ft	20 ft		8.4 ft	0.416
		60 in	26 ft	21 ft		8.8 ft	0.454
		66 in	27 ft	21 ft		9.4 ft	0.516
		72 in	27 ft	22 ft		10.0 ft	0.580
		78 in	28 ft	23 ft		10.6 ft	0.648
		84 in	28 ft	23 ft		11.1 ft	0.703
		90 in	29 ft	24 ft		12.1 ft	0.842
	Sectional	96 in	30 ft	22 ft	10.5 ft		0.535
		108 in	31 ft	23 ft	11.7 ft		0.656
		120 in	32 ft	24 ft	12.8 ft		0.768
RCP ARCH	Safety	18 in	22 ft	15 ft	3.3 ft		0.038
	Sloped	24 in	23 ft	15 ft	3.9 ft		0.041
		30 in	24 ft	16 ft	4.6 ft		0.056
		36 in	24 ft	16 ft	5.4 ft		0.071
		42 in	25 ft	17 ft	6.0 ft		0.083
	Flared	18 in	22 ft	15 ft		3.4 ft	0.040
		24 in	23 ft	17 ft		4.6 ft	0.062
		30 in	24 ft	18 ft		5.7 ft	0.095
		36 in	24 ft	19 ft		6.8 ft	0.128
		42 in	25 ft	19 ft		7.3 ft	0.142
		48 in	26 ft	20 ft		7.8 ft	0.157
		54 in	26 ft	20 ft		8.4 ft	0.180
		60 in	27 ft	21 ft		9.0 ft	0.196
		72 in	29 ft	23 ft		11.2 ft	0.314
		84 in	30 ft	25 ft		13.3 ft	0.423
	Sectional	96 in	32 ft	24 ft	12.7 ft		0.287
		108 in	33 ft	25 ft	14.2 ft		0.332
		120 in	35 ft	27 ft	15.7 ft		0.395
		132 in	36 ft	28 ft	16.7 ft		0.428

Figure 10.3-E — SPACINGS FOR MULTIPLE CONCRETE PIPES (Wide Streams/Undefined Streams)

Pipe Type	End Section	Pipe Size	Center-to-Center Spacing of Multiple Pipe: Recommended Without CDF — 20 ft between pipe barrels	Minimum Without Considering CDF — 12 ft between pipe end sections	With CDF — 0 ft between pipe end sections	Controlled Density Fill (CDF) (cu yd/ft)
CMP	Safety & Sloped	18 in	22 ft	15 ft	2.7 ft	0.042
		24 in	22 ft	16 ft	3.2 ft	0.060
		30 in	23 ft	17 ft	4.0 ft	0.094
		36 in	23 ft	18 ft	4.5 ft	0.119
		42 in	24 ft	19 ft	5.3 ft	0.165
		48 in	24 ft	19 ft	5.8 ft	0.197
		54 in	25 ft	20 ft	6.3 ft	0.230
		60 in	25 ft	20 ft	6.8 ft	0.266
	Flared	18 in	22 ft	16 ft	4.3 ft	0.087
		24 in	22 ft	18 ft	5.7 ft	0.153
		30 in	23 ft	19 ft	7.0 ft	0.233
		36 in	23 ft	20 ft	8.3 ft	0.330
		42 in	24 ft	22 ft	9.7 ft	0.451
		48 in	24 ft	22 ft	10.5 ft	0.545
		54 in	25 ft	23 ft	11.5 ft	0.664
		60 in	25 ft	24 ft	12.5 ft	0.794
		66 in	26 ft	25 ft	13.0 ft	0.884
		72 in	26 ft	25 ft	13.5 ft	0.976
		78 in	27 ft	26 ft	14.0 ft	1.071
		84 in	27 ft	26 ft	14.5 ft	1.167
CMP ARCH	Safety & Sloped	18 in	22 ft	16 ft	2.9 ft	0.021
		24 in	22 ft	16 ft	3.5 ft	0.031
		30 in	23 ft	17 ft	4.4 ft	0.048
		36 in	24 ft	18 ft	5.0 ft	0.060
		42 in	24 ft	19 ft	5.9 ft	0.085
		48 in	25 ft	20 ft	6.6 ft	0.105
		54 in	25 ft	20 ft	7.2 ft	0.204
		60 in	26 ft	21 ft	7.8 ft	0.238
		72 in	27 ft	22 ft	8.8 ft	0.248
	Flared	18 in	22 ft	16 ft	4.2 ft	0.041
		24 in	22 ft	17 ft	5.5 ft	0.071
		30 in	23 ft	19 ft	6.7 ft	0.106
		36 in	24 ft	20 ft	8.3 ft	0.160
		42 in	24 ft	21 ft	9.3 ft	0.204
		48 in	25 ft	22 ft	10.2 ft	0.250
		54 in	25 ft	23 ft	11.5 ft	0.476
		60 in	26 ft	24 ft	12.5 ft	0.568
		66 in	26 ft	25 ft	13.5 ft	0.665
		72 in	27 ft	26 ft	14.1 ft	0.636

Figure 10.3-F — SPACINGS FOR MULTIPLE METAL PIPES
(Wide Streams/Undefined Streams)

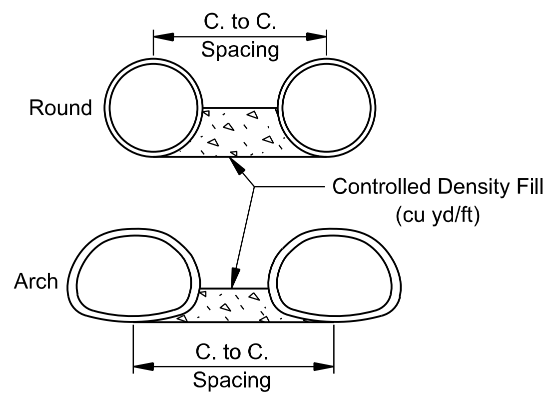


Figure 10.3-G — CONTROLLED DENSITY FILL LIMITS BETWEEN MULTIPLE PIPE

For a skewed installation, plot the culvert in its actual position and scale the required length. See Section 10.3.3.6 for additional length of pipe to add to skewed culvert installations.

End sections are not considered a part of the pipe length. They are measured and paid for by the number of end sections installed.

Culverts with 36 in and larger diameter or equivalent diameter should have the opening of the end section located outside the clear zone. Additional length of pipe may be needed to extend these culverts to meet this criteria. See Chapter 10 "Roadside Safety" of the *South Dakota Road Design Manual* for pipe sizes and end treatments allowed in relation to clear zones.

10.3.3.7 Skewed Installations

The degree of skew is measured as the angle between the culvert installation and a line perpendicular to the highway centerline. The degree of skew should be to the nearest 1 degree. A culvert skew angle is described in terms of which end is forward — left-hand forward (LHF) or right-hand forward (RHF). For example, if the left end of the culvert is ahead of the line perpendicular to the centerline and the angle is 15 degrees, the installation would be described as "15° LHF."

Where practical, culvert installations should be designed to conform as closely as possible to the natural drainage channels. Headwalls, if used, are parallel to the roadway, but skewed to the culvert. For pipe culverts, avoid excessive skews > 30 degrees.

Where a pipe culvert is skewed more than 5 degrees, prepare a special cross section of its placement site. The special cross section should contain the following information:

- pipe skew;
- pipe length; and
- inlet and outlet station, offset and elevation.

The measured culvert length must be increased by adding the length shown in Figure 10.3-H to each end of the pipe. This length should be added before the pipe length is rounded up to the nearest 2 ft. The additional length is required to provide a 3H:1V inslope to the top of the end of the end section from the point where the pipe originally hits the inslope along the pipe centerline.

Pipe Diameter or Equiv.	**End Section Type**	**Round**									**Arch**								
		Skew									**Skew**								
		5°	**10°**	**15°**	**20°**	**25°**	**30°**	**35°**	**40°**	**45°**	**5°**	**10°**	**15°**	**20°**	**25°**	**30°**	**35°**	**40°**	**45°**
18 in	Safety	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24 in	Sloped	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.9	1.7	0.0	0.0	0.0	0.0	0.0	0.3	0.8	1.3	2.0
30 in	Sloped	0.0	0.0	0.0	0.0	0.0	0.0	0.5	1.3	2.2	0.0	0.0	0.0	0.0	0.1	0.5	1.1	1.7	2.6
36 in	Flared	1.3	1.7	2.1	2.6	3.2	3.9	4.7	5.7	6.9	0.3	0.7	1.1	1.5	2.1	2.7	3.5	4.4	5.4
42 in	Flared	1.5	1.9	2.3	2.9	3.5	4.2	5.1	6.1	7.4	0.3	0.7	1.1	1.6	2.2	2.9	3.6	4.6	5.7
48 in	Flared	1.6	2.1	2.6	3.1	3.8	4.6	5.6	6.8	8.2	0.4	0.8	1.2	1.7	2.3	3.0	3.8	4.8	6.0
54 in	Flared	3.1	3.6	4.1	4.8	5.5	6.4	7.4	8.7	10.3	0.4	0.8	1.3	1.9	2.5	3.2	4.1	5.1	6.3
60 in	Flared	3.2	3.6	4.2	4.9	5.6	6.5	7.6	8.8	10.4	0.4	0.9	1.4	2.0	2.6	3.4	4.3	5.3	6.6
66 in	Flared	5.1	5.6	6.3	7.0	7.9	9.0	10.3	11.8	13.7									
72 in	Flared	4.7	5.3	6.0	6.8	7.7	8.8	10.1	11.7	13.7	3.0	3.6	4.3	5.0	5.9	6.9	8.1	9.5	11.2
78 in	Flared	5.4	6.0	6.7	7.6	8.6	9.8	11.3	13.1	15.3									
84 in	Flared	7.0	7.7	8.4	9.4	10.5	11.8	13.4	15.4	17.8	4.1	4.8	5.6	6.6	7.6	8.9	10.4	12.2	14.4
90 in	Flared	7.7	8.4	9.2	10.2	11.4	12.8	14.5	16.5	19.1									
96 in	Sloped	4.2	4.8	5.6	6.5	7.6	8.9	10.6	12.5	15.0	3.2	3.8	4.6	5.5	6.6	7.9	9.4	11.2	13.4
108 in	Sloped	3.5	4.2	5.1	6.2	7.5	9.0	10.9	13.3	16.2	3.7	4.5	5.4	6.4	7.6	9.1	10.8	12.9	15.5
120 in	Sloped	6.6	7.4	8.3	9.5	11.0	12.8	14.9	17.6	20.9	3.9	4.8	5.8	6.9	8.2	9.8	11.8	14.1	16.9

Figure 10.3-H — ADDITIONAL LENGTH NEEDED AT EACH END FOR SKEWED CONCRETE PIPE (ft)

10.3.3.8 Broken-Back Culverts

A broken-back culvert, which combines two or more 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. These culverts are used for grade control and to control outlet velocities. See Section 10.9 for an overview on the hydraulic design of a broken-back culvert.

10.3.3.9 Downspouts

The following presents general criteria for designing downspouts:

1. Cross pipe downspouts should be installed with RC pipe on the upstream end, followed by a concrete-to-metal pipe transition section, CM pipe elbow, straight CM pipe, CM pipe elbow, straight CM pipe and CM pipe end section. Approach pipe downspouts should be installed with all CM pipe.

2. Use a 1% grade for sizing the pipe on the upstream end. Review different gradients for alternative sizing of pipes.

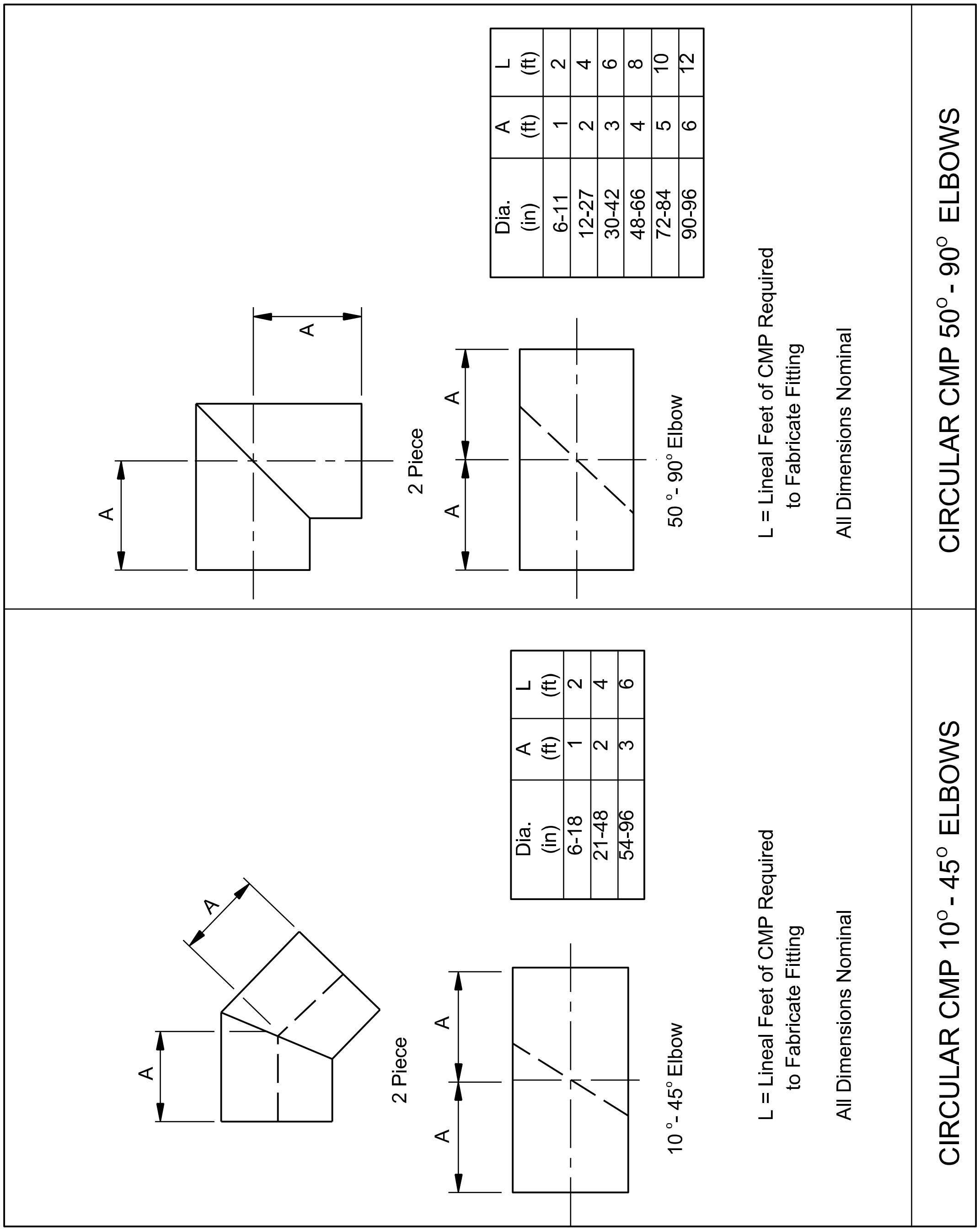
3. The recommended size of downspout installations should not exceed a 36-in diameter for constructability. Options include installing multiple pipe or installing the larger pipe on a steeper grade with energy dissipator sections at the outlet end of the pipe. If a larger pipe with energy dissipators is an option, this must be coordinated with the Bridge Hydraulic Engineer on a per-site basis. Larger than 36-in diameter pipe may be used for downspout applications depending on site conditions, constructability and cost comparisons. Constructability and site conditions should be reviewed with field staff during the design process for concurrence.

4. Elbows used should be with 2½° increments (1° increments in special cases).

5. The length of the CM pipe installed should not include the length of the CM pipe elbows. The CM pipe elbows should be paid for separately. See Figure 10.3-I for laying lengths of elbows. See Equation 10.1 in Section 10.4.5.6 for loss coefficients.

6. The length of the downstream end of downspout should be a minimum of 5 times the diameter of the designed pipe. Additional pipe length may be needed to dissipate the energy.

7. Maintain a minimum of 1 ft of cover beneath the subgrade and inslope. See Chapter 18 "Plans Assembly" of the *South Dakota Road Design Manual* for an illustration, required note and measurement.



Dia. (in)	A (ft)	L (ft)
6-11	1	2
12-27	2	4
30-42	3	6
48-66	4	8
72-84	5	10
90-96	6	12

Dia. (in)	A (ft)	L (ft)
6-18	1	2
21-48	2	4
54-96	3	6

Figure 10.3-I — LENGTHS OF ELBOWS

8. Consider erosion protection at the outlet end.

9. Thrust blocks, seepage collars and vent tubes should be considered. See Reference (5) for guidance on designing for thrust.

10.3.3.10 Median and Entrance Placement

A cross section of the placement site is prepared for each entrance or intersecting road pipe culvert. See Chapter 18 “Plans Assembly” of the *South Dakota Road Design Manual* for pipe illustrations.

Pipes should not be installed through permanent maintenance crossovers on the Interstate. On other divided highways with depressed medians, attempt to minimize the number of drainage pipes installed through median crossovers by placing them at a crest when practical. Median drainage approaching a crossover should be removed with a pipe across the mainline highway. This will provide for safer crossover inslopes.

10.3.3.11 End Treatment (Inlet or Outlet)

10.3.3.11.1 General

End treatment selection for culverts is based on a consideration of three factors:

- safety,
- hydraulic efficiency, and
- structural integrity.

Safety considerations are based on the potential consequences of a run-off-the-road vehicle impacting a culvert end section. In general, any culvert end treatment within the roadside clear zone should be made traversable or shielded by guardrail. Chapter 10 “Roadside Safety” of the *South Dakota Road Design Manual* presents SDDOT criteria for the safety treatment of culvert end sections.

Section 10.4 discusses hydraulic efficiency considerations for culvert end treatments.

10.3.3.11.2 Other SDDOT Practices

The following summarizes other SDDOT practices for the selection of culvert end treatments:

- Occasionally, it may be warranted to use headwalls on large-diameter metal pipe to take advantage of the improved efficiency of a bevel.

- RCBCs use wingwalls that conform to a 2H:1V fill slope normal to the roadway centerline.
- Do not use projecting end treatments.
- A sloped apron may be used in front of a culvert. It should have a 3H:1V slope, starting D/2 in front of the face and the headwater on the crest should match the headwater on the face.

Improved (or tapered) inlets:

- May be considered for culverts that will operate in inlet control.
- Should only be used if practical. They can increase the hydraulic performance of the culvert, but may also add to the total culvert cost. See Section 10.8.
- May be considered where fish passage is required without a slope-taper. See Section 10.3.4.6.

10.3.3.12 Weep Holes

If weep holes are used to relieve uplift pressure, they should be designed similar to underdrain systems.

10.3.4 Related Designs

10.3.4.1 Buoyancy Resistance

Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy resistance should be considered for all flexible culverts greater than a 48-in diameter. Factors affecting buoyancy include steepness of the culvert slope, depth of the potential headwater (debris blockage may increase), flatness of the upstream fill slope, height of the fill, large culvert skews or mitered ends.

10.3.4.2 Outlet Protection

Outlet protection (see Chapter 11 "Energy Dissipators") for the selected culvert design flood should be provided where the outlet scour hole depth computations indicate:

- the scour hole will undermine the culvert outlet and/or roadway embankment,
- 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.

10.3.4.3 Relief Opening

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

10.3.4.4 Land-Use Culverts

Consideration may be given to combining drainage culverts with other land-use requirements necessitating passage under a hi ghway (e.g., livestock, wildlife, pedestrian passage). The designer should consider the following:

- during the selected design flood, the land use is temporarily forfeited but available during lesser floods;
- 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;
- they should be sized to ensure that it can serve its intended land frequency use function up to and including a 2-year flood; and
- the height and width constraints should satisfy the hydraulic or land-use requirements, whichever is larger.

10.3.4.5 Irrigation Facilities

Unless legally abandoned, an irrigation structure should be required even if the irrigation canal or ditch is no longer used. The canal or ditch owner should approve the use of multiple-barrel culverts. Provision should be made to accommodate any water escaping the ditch to avoid a flood hazard. Irrigation facilities should be designed using Equation 10.6 in Section 10.4.5.7, which accounts for the entrance and exit velocity head, or HEC-RAS (see Section 18.2.3). The design should accommodate the water and flood right using the criteria below that yields the largest culvert size:

- constrain the headwater within the existing canal or ditch banks unless provision is made for overflow during high flows,
- provide freeboard to pass expected debris,
- ensure no increase in the velocity beyond what the unprotected ditch material or protection will sustain,
- avoid a flood hazard from 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.

10.3.4.6 Fish Passage

The Environmental Office determines when it is necessary to accommodate fish passage through a culvert. See Appendix 10.A for SDDOT guidelines. Where needed, SDDOT practice is to depress the culvert invert below the streambed. This practice is not a standard practice and should only be applied to proposed structures in streams identified during the environmental review process. A hydraulic design procedure for culverts with inverts below the streambed is provided in the final draft of HEC 26 (Reference (6)). HY-8 software includes buried inverts for common culvert shapes (see Section 18.2.3).

If fish passage cannot be accommodated by depressing the invert, the designer should consult Chapter 15 of the AASHTO *Model Drainage Manual* for hydraulic design guidance for alternative sill-type fishways (e.g., fish ladders) (see Reference (7)).

10.3.4.7 Pipe Culvert Rehabilitation

Existing pipes that are damaged or deteriorated to the point that their functionality has been significantly impacted should be considered for rehabilitation. See Appendix 10.B for SDDOT typical pipe culvert rehabilitation methods.

10.4 CULVERT HYDRAULIC DESIGN

10.4.1 Standard SDDOT Practice

SDDOT has adopted HDS 5 (Reference (1)) as its standard practice for the hydraulic design of culverts. The designer has the option of performing an analysis using the equations outlined in this Section, using the nomographs from HDS 5 or using a computer-aided design that is consistent with the equations provided in HDS 5 (see Section 18.2.3).

Culverts will be designed for a constant discharge that will normally be the peak discharge. This will yield a conservatively sized structure where temporary storage is available but not used.

10.4.2 General

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

- analyzing nonuniform 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, 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.

10.4.3 Culvert Design Practices

The design procedures in this Chapter use the following practices.

10.4.3.1 Control Section

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

10.4.3.2 Minimum Performance

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

10.4.4 Inlet Control

For inlet control, 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.

10.4.4.1 Headwater Factors

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.

Inlet edge configuration describes the entrance type. Inlet edge configurations include thin-edge projecting, mitered, square edges in a headwall, flared ends on box culverts and beveled edges.

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

10.4.4.2 Hydraulics

Three regions of flow are shown in Figure 10.4-A — unsubmerged, transition and submerged.

10.4.4.3 Unsubmerged

For headwater between the invert and the culvert crown, the entrance operates as a weir, which 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

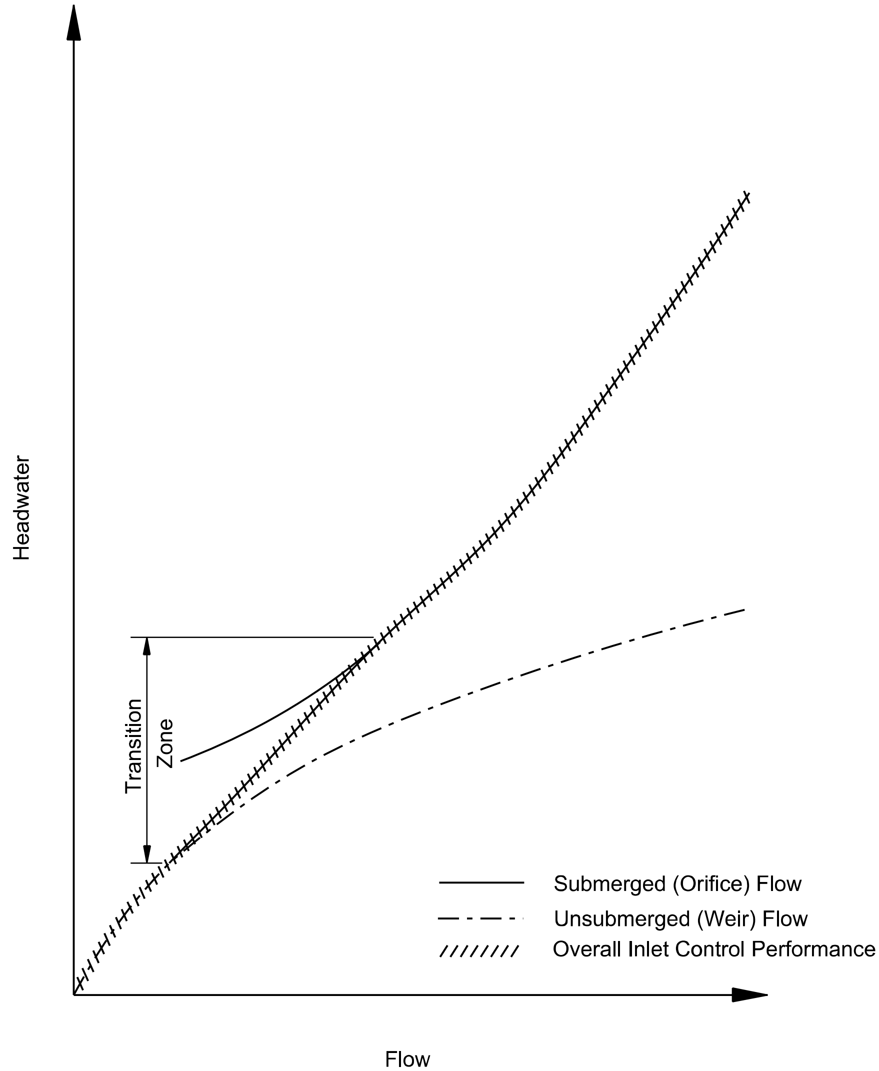


Figure 10.4-A — Inlet Control Flow Curves

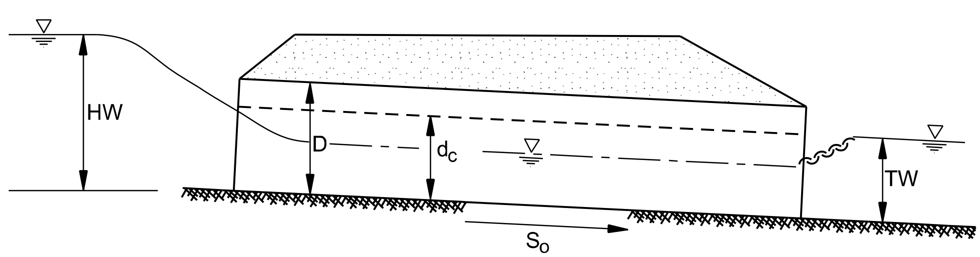


Figure 10.4-B — FLOW TYPE I

HDS 5 (Reference (1)) contains the equations that were developed from model test data; see Figure 10.4-B, Flow Type I.

10.4.4.4 Submerged

For headwaters above the inlet, the culvert operates as an orifice, which is an opening submerged on the upstream side and flowing freely on the downstream side that 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 (Reference (1)) contains flow equations that were developed from model test data. See Figure 10.4-C, Flow Type V.

10.4.4.5 Transition Zone

The transition zone is located between the unsubmerged and the 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.

10.4.4.6 Nomographs

The inlet control flow versus headwater curves that are established using the above procedure are the basis for constructing the inlet control design nomographs (see Section 10.10). Note that, in the inlet control nomographs, HW is measured to the total upstream energy grade line including the approach velocity head.

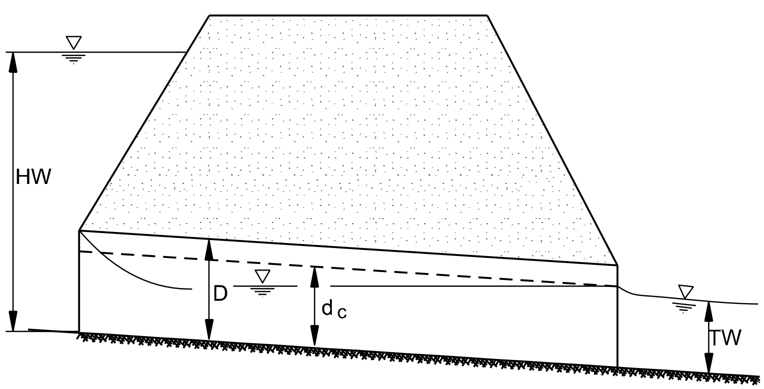


Figure 10.4-C — FLOW TYPE V

10.4.5 Outlet Control

Outlet control has depths and velocity that are subcritical. The control of the flow is at the downstream end of the culvert (the outlet). 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 of the barrel factors. All of the inlet control factors also influence culverts in outlet control.

10.4.5.1 Barrel Roughness

Barrel roughness is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient; i.e., the Manning's n value. Typical Manning's n values are presented in Figure 10.10-A.

10.4.5.2 Barrel Area

Barrel area is measured perpendicular to the flow.

10.4.5.3 Barrel Length

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

10.4.5.4 Barrel Slope

Barrel slope is the actual slope of the culvert barrel and is often the same as the natural stream slope. However, where the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope.

10.4.5.5 Tailwater Elevation

Tailwater is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation or field observations are used to define the tailwater elevation (see Sections 10.3.2.4 and 10.3.2.5).

10.4.5.6 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 an 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:

1. Losses: $H_L = H_e + H_f + H_v + H_b + H_j + H_g$ (Equation 10.1)

   where:

   H_L = total energy loss, ft
   H_e = entrance loss, ft
   H_f = friction losses, ft
   H_v = exit loss (velocity head), ft (equivalent to H_o; see Equation 10.4d)
   H_b = bend losses, ft (see HDS 5, Reference (1))
   H_j = losses at junctions, ft (see HDS 5)
   H_g = losses at grates, ft (see HDS 5)

2. Velocity: $V = Q/A$ (Equation 10.2)

   where:

   V = average barrel velocity, fps
   Q = flow rate, cfs
   A = cross sectional area of flow with the barrel full, sq ft

3. Velocity head: $H_v = V^2/2g$ (Equation 10.3)

   where:

   g = acceleration due to gravity, 32.2 ft/sec^2

4. Entrance loss: $H_e = k_e (V^2/2g)$ (Equation 10.4a)

   where:

   k_e = entrance loss coefficient; see Figures 10.10-B and Figure 10.10-C.

5. Friction loss: $H_f = [(29n^2L)/R^{1.33}] [V^2/2g]$ (Equation 10.4b)

   where:

   n = Manning's roughness coefficient (see Figure 10.10-A)
   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

6. Exit loss: $H_o = 1.0\,[(V^2/2g) - (V_d^2/2g)]$ (Equation 10.4b)

where:

V_d = channel velocity downstream of the culvert, fps (usually neglected; see Equation 10.4d).

$H_o = H_v = V^2/2g$ (Equation 10.4d)

Equation 10.4d is the standard option in HY-8. If the designer chooses the Utah State University (USU) Method (which is presented in HY-8), the following equation will be used: $H_o = (V - V_d)^2/2g$. This equation was formulated for applications like irrigation channels where a small amount of energy is lost in the transition back to the channel.

7. Barrel losses: $H = H_e + H_o + H_f$ (Equation 10.5)

$$H = [1 + k_e + (19.63n^2L/R^{1.33})]\,[V^2/2g]$$

10.4.5.7 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 in Figure 10.4-D, the following relationship results:

$$HW_o + \left(V_u^2/2g\right) = TW + \left(V_d^2/2g\right) + H_L \quad \text{(Equation 10.6)}$$

where:

HW_o = headwater depth above the outlet invert, ft
V_u = approach velocity, fps
TW = tailwater depth above the outlet invert, ft
V_d = downstream velocity, fps
H_L = sum of all losses (Equation 10.1)

Note: This equation is only true if TW is higher than critical depth at the outlet.

10.4.5.8 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.

10.4.5.9 Nomographs (Full Flow)

The nomographs were developed assuming that the culvert barrel is flowing full and:

- $TW \geq D$, Flow Type IV (see Figure 10.4-D); or $d_c \geq D$, Flow Type VI (see Figure 10.4-E).
- V_u is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert.
- V_d is small and its velocity head can be neglected.
- Equation 10.6 becomes:

$$HW = TW + H - S_oL \qquad \text{(Equation 10.7)}$$

where:

HW = depth from the inlet invert to the energy grade line, ft
H = the value read from the nomographs (Equation 10.5), ft
S_oL = drop from inlet to outlet invert, ft

10.4.5.10 Nomographs (Partial Full Flow)

Equations 10.1 through 10.7 were developed for full barrel flow. The equations also apply to the flow situations that are effectively full-flow conditions, if $TW < d_c$; see Figure 10.4-F.

Backwater calculations may be required that 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.

10.4.5.11 Nomographs (Partial Full Flow) — 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 of the way between critical depth and the top of the barrel, or $(d_c + D)/2$ above the outlet invert. The 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

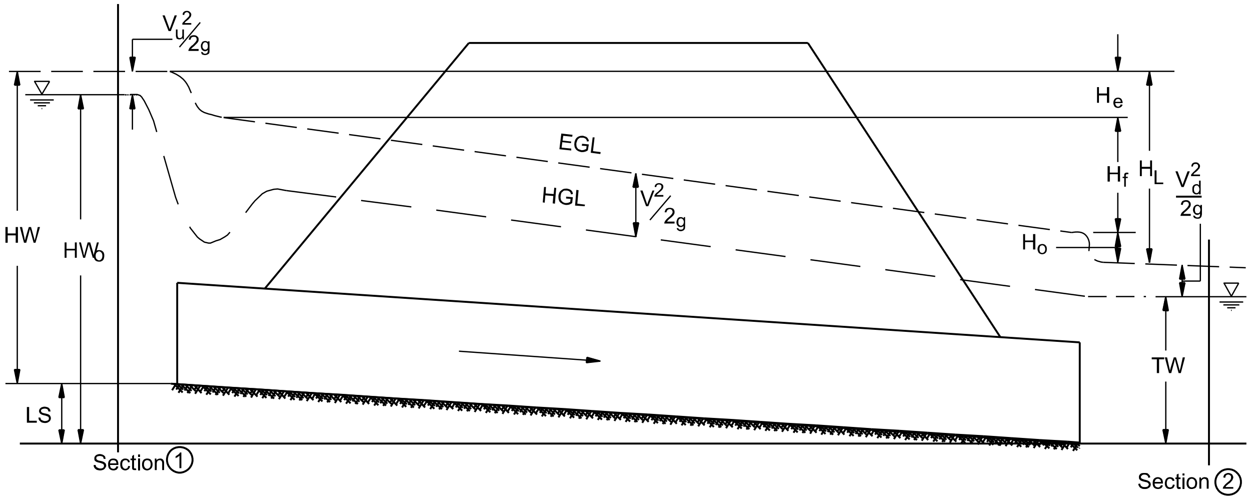


Figure 10.4-D — FLOW TYPE IV

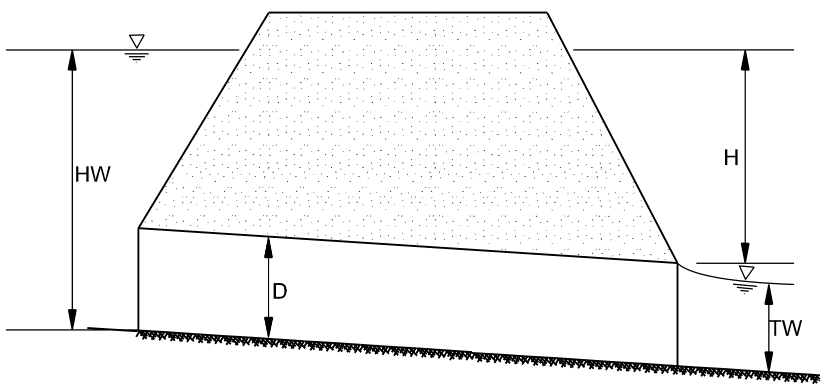


Figure 10.4-E — FLOW TYPE VI

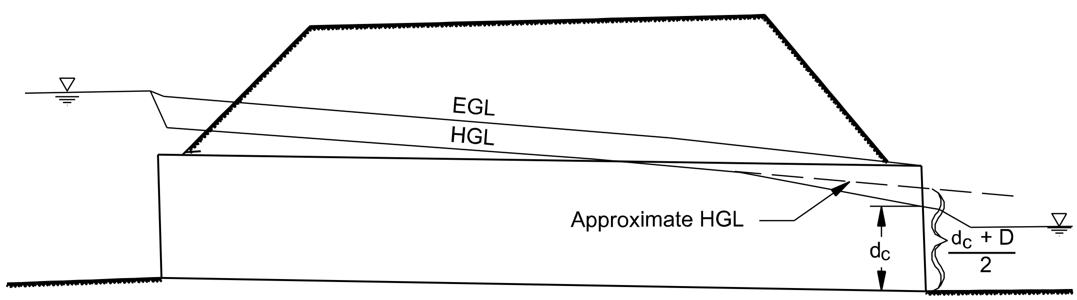


Figure 10.4-F— FLOW TYPE VII

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:

$$HW = h_o + H - S_oL \quad \text{(Equation 10.8)}$$

where:

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

Adequate results are obtained down to a HW = 0.75D. For lower headwaters, backwater calculations are required. See Figure 10.4-G if TW < d_c and Figure 10.4-H if TW > d_c.

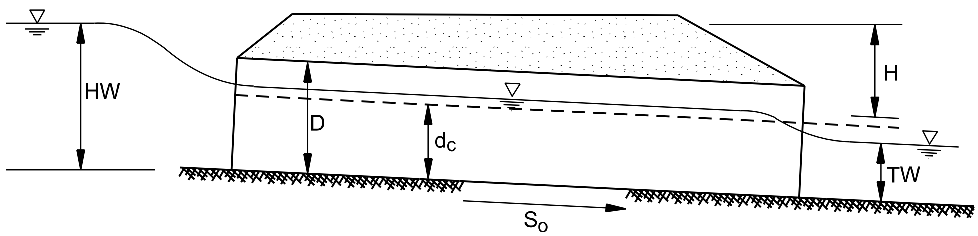


Figure 10.4-G — FLOW TYPE II, TW < d_c

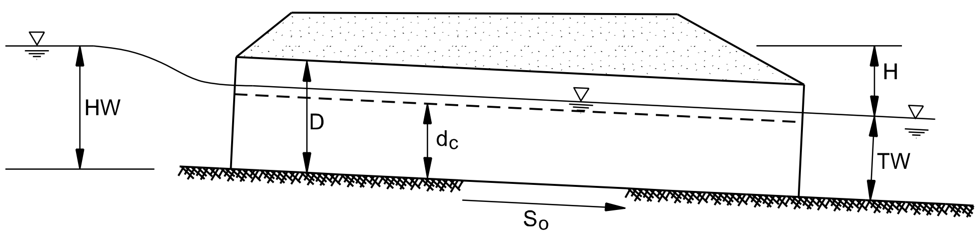


Figure 10.4-H — FLOW TYPE III, TW > d_c

10.4.6 Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually result in outlet velocities that are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion if they are substantially higher than the natural stream velocity. The following is general guidance (see also Figure 7.6-A on design frequency):

- If V_o > 6 fps, consider erosion protection.
- If V_o > 10 fps, consider gabion protection.
- If V_o > 15 fps, consider energy dissipation (see Chapter 11 "Energy Dissipators").

If the guidance above indicates that protection may be required, compare the shear stress caused by the outlet velocity to the shear stress that the downstream channel can resist. If the shear stress generated by the outlet velocity does not exceed the permissible shear stress of the material in the downstream channel, protection is not required.

10.4.6.1 Inlet Control

The velocity is calculated from Equation 10.2 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 at the exit the depth and flow area.
- 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 from design aids in hydraulic publications (e.g., HDS 3, Reference (8)).

10.4.6.2 Outlet Control

The velocity is calculated from Equation 10.2 after determining the outlet depth. The cross sectional area of the flow is defined by the geometry of the outlet and either culvert 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.
- The total barrel area is used where the tailwater or critical depth exceeds the top of the barrel.

10.4.7 Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad-crested weir. Flow coefficients for flow overtopping roadway embankments are found in HDS 1 (Reference (9)) and are included on Figure 10.10-S. The equation is:

$$Q_r = C_d L HW_r^{1.5} \quad \text{(Equation 10.9)}$$

where:

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

Overtopping information should be computed and provided on the plans for all new and sliplined mainline pipe culverts that have diameters larger than 24" (equivalent round for arch). The size requirements for sliplined pipe culverts are based on the existing pipe size. Pipe culvert extensions typically do not require overtopping information unless significant changes are being made to the hydraulic conditions of site conditions warrant it.

The overtop elevation is the elevation where pipe culvert backwater will begin to flow to a different drainage basin. The overtop elevation is the lower of the following possibilities, which may or may not be the same as the pipe culvert allowable design headwater elevation:

1. Centerline finished elevation (lowest elevation in the area)
2. Top of ditch block
3. Finished top of approach
4. Summit of ditch grade

Overtopping information should be documented on the overtopping form (Figure 10.4-I) or other applicable culvert design forms.

The flow and headwater frequencies provided on the overtopping form are typically 25, 50 and 100 years, however, other frequencies may be needed if the design frequency is less than 25 year.

If the overtop flow/headwater is greater than the 100 year then record the 100 year flow and headwater on the overtopping form under "No Overtop".

If the overtop flow/headwater is less than the 100 year then determine the overtop flow and year according to the following and record on the overtopping form under "Overtop".

Typical pipe culvert hydraulic analysis will compute the overtop flow (HY-8 software) from which the overtop year can be determined. The overtop year is determined by interpolation from a plot of discharge vs. recurrence interval (frequency) on log graph paper (Figure 10.4-J). The standard frequency flows immediately above and below the overtop flow are plotted on Figure 10.4-J and connected with a line. These standard frequency flows are typically 25 and 50 year or 50 and 100 year. The overtop year is determined where the overtop flow intersects the plotted line.

Pipe culvert hydraulic analysis that do not compute the overtop flow (CDS software) can use either of the following two methods to determine the overtop year and flow. The preferred method from this type of hydraulic analysis would be to adjust the flow in the analysis until the culvert headwater matches the overtop elevation. This provides the overtop flow that would be used to determine the overtop year by following the procedure in the preceding paragraph. The other method uses standard frequency headwaters immediately above and below the overtop headwater as computed in the hydraulic analysis assuming no overtopping. These standard frequency headwaters are typically 25 and 50 year or 50 and 100 year. The overtop year is determined by linear interpolation of the overtop headwater between the above mentioned standard frequency headwaters. The standard frequency flows immediately above and below the overtop flow are then plotted on Figure 11-2 and connected with a line. These standard frequency flows are typically 25 and 50 year or 50 and 100 year. The overtop flow is determined where the overtop year intersects the plotted line.

The overtop frequency for all pipe culverts should be equal to or greater than the design frequency. Pipe culverts that do not meet this criteria should be redesigned or have a justification approved and documented.

Overtopping Summary

The pipe culvert overtopping information should be provided on the profile sheets of the plans with the following formats based on the size of the drainage area and the overtopping flood frequency.

Sites with drainage areas less than 1000 acres use the following formats.

For Overtop Less than 100 yr.

Q yr. = ______cfs
EL. = ______

For Overtop Greater than 100 yr.

Q 100 = ______cfs
EL. = ______

Sites with drainage areas 1000 acres and more use the following format.

The examples also show how to handle sites with optional structures. When there are several optional structures for a site, the worst case data should be shown on the plan sheet. In the case of 25 yr and 100 yr. high water elevations, the highest one from the several options should be shown. If there is overtopping at the crossing, use the lowest overtopping frequency.

Normal crossing without overtopping.
Q25 = 210 cfs
EL. = 73.2
Q100 = 395 cfs
EL. = 74.1

In this case another option had a Q100 high water elevation of 74.0

Crossing with overtopping.

Q25 = 210 cfs
EL. = 73.2
Q_{OT} = Q65 = 300 cfs
EL. = 73.7
Q100 = 395 cfs
EL. = 73.9

In this case another option had an overtopping frequency of Q_{OT} = Q70 = 310 cfs, and the EL. (100 Yr) is with overtopping.

Project: Designer: Date: Sheet____ of ____

Station	Pipe Size	Acres	O.T. Elev	Inlet Elev	O.T. Head Water	25 Year		50 Year		100 Year		Overtop		No Overtop	
						H.W.	Q	H.W.	Q	H.W.	Q	Year	Q	Q100	ELEV

Figure 10.4-I — OVERTOPPING PIPE FORM

Project ______________________ Station ______________________ Drainage Area _______________

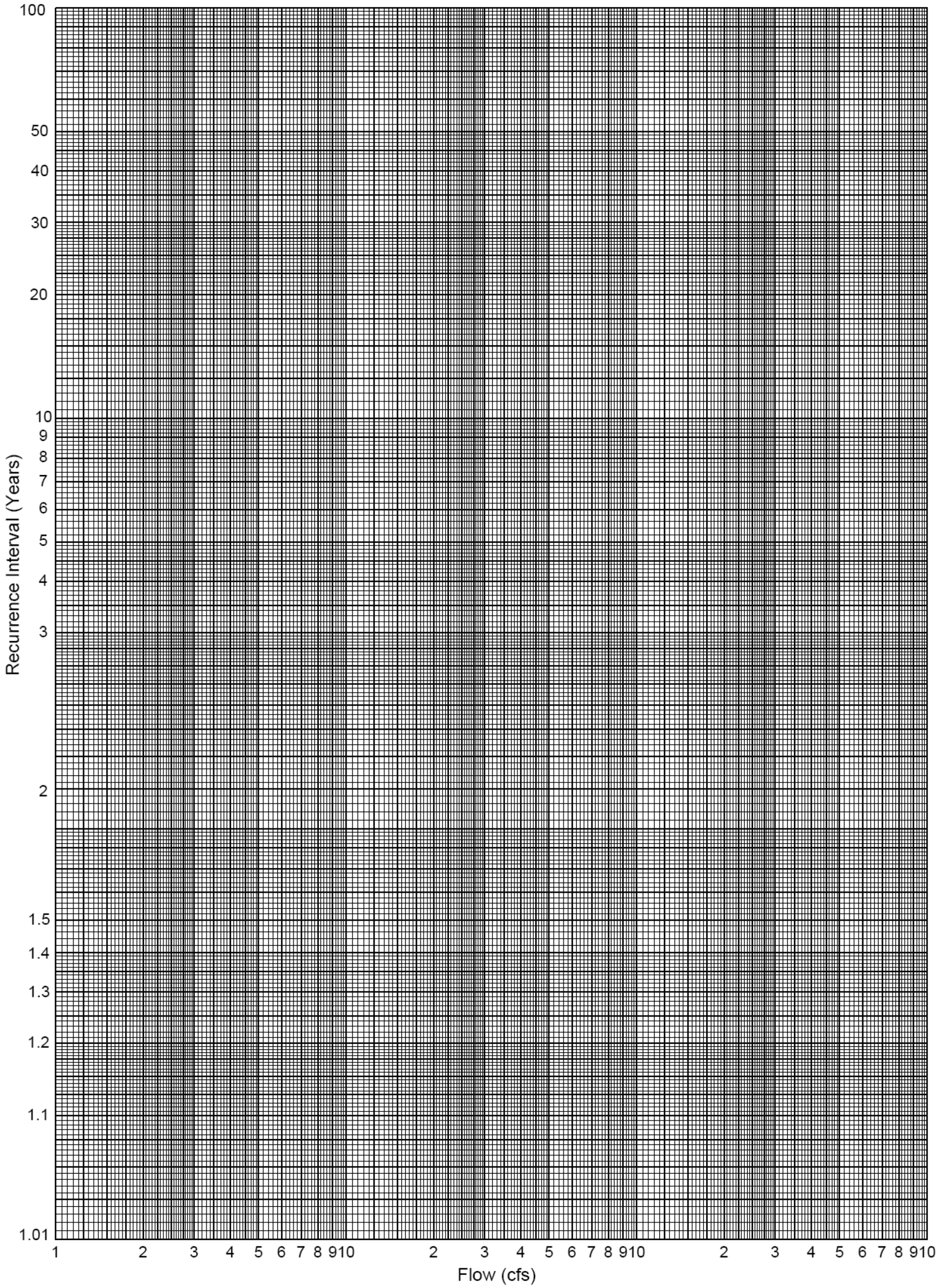


Figure 10.4-J — LOG GRAPH PAPER

10.4.7.1 Crest Length

The crest length is difficult to determine where the crest is defined by a roadway sag vertical curve:

- Recommend subdividing into a series of segments. The flow over each segment is 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.

10.4.7.2 Total Flow

Total flow is calculated for a given upstream water surface elevation:

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

10.4.8 Performance Curves

Performance curves are plots of flow rate versus headwater depth or elevation, velocity or outlet scour. Performance curves should be developed for all culverts for evaluating the hydraulic capacity of a culvert for various headwaters, outlet velocities and scour depths. These curves will display the consequence of high-flow rates at the site and provide a basis for evaluating flood hazards.

The culvert performance curve consists of the controlling portions of the individual performance curves for each of the following control sections (see Figure 10.4-K).

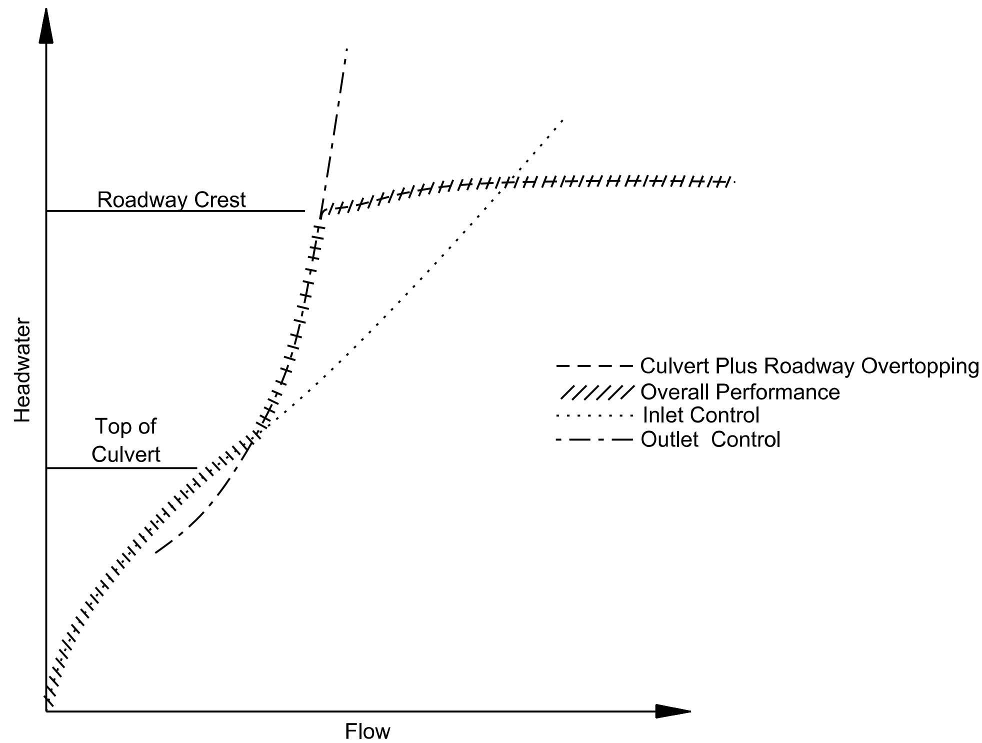


Figure 10.4-K — OVERALL PERFORMANCE CURVE

Inlet

The inlet performance curve is developed using the inlet control nomographs (see Section 10.10).

Outlet

The outlet performance curve is developed using Equations 10.1 through 10.8, the outlet control nomographs or backwater calculations.

Roadway

The roadway performance curve is developed using Equation 10.9.

<u>Overall</u>

The overall performance curve is the sum of the flow through the culvert and the flow across the roadway. The curve 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 should be calculated.

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

3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and Equation 10.9 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 10.4-K.

10.5 DESIGN PROCEDURE AND DOCUMENTATION

10.5.1 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 13 "Storage Facilities". The designer should be familiar with all the equations in Section 10.4 before using these procedures. Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe or costly structure. If the proposed culvert drains more than 1000 acres, the designer should perform the assessments required for the Draft Hydraulic Design Report before designing the culvert (see Section 10.5.2).

The computation form has been provided in Section 10.10 (Figure 10.10-T) 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 Chapter 5 "Data Collection" — The minimum data are:

- USGS, site and location maps;
- embankment cross section;
- roadway profile;
- photographs;
- field visit (assessment of sediment, debris); and
- design data at nearby structures.

b. Studies by other agencies including:

- small dams — NRCS, USACE, BLM;
- canals — NRCS, USACE, USBR;
- floodplain — FEMA, NRCS, USACE, USGS, NOAA; and
- storm drain — local or private.

c. Environmental constraints including:

- commitments contained in environmental documents,
- fish migration, and
- wildlife passage.

d. Design criteria:

- review Section 10.3 for applicable criteria, and
- 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 9 "Roadside Channels" and Chapter 15 "Bank Protection".

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

Step 4 Summarize Data on Design Form

a. See Culvert Design Form, Figure 10.10-T.
b. Data from Steps 1 through 3.

Step 5 Select Design Alternative

a. See Section 10.3.3 "Culvert Design Features".
b. Choose culvert material, shape, size and entrance type.

Step 6 Select Design Discharge (Q_d)

a. See Section 10.3.2 "Design Limitations".
b. Determine flood frequency from Figure 7.6-A.
c. Determine Q from discharge-frequency plot (Step 2).
d. Divide Q by the number of barrels. *Note: For circular shapes, this assumption is good for center-to-center spacings that are 1.5D or greater.*

Step 7 Determine Inlet Control Headwater Depth (HW_i)

a. Use the applicable inlet control nomograph in Section 10.10. The minor effect that skew has on headwater for RCBCs can only be determined with Figures 10.10-N and Figure 10.10-O (i.e., this is not included in HY8).

b. Locate the size or height on the scale.

c. Locate the discharge:

- For a circular shape, use discharge.
- For a box shape, use Q per foot of width.

d. Locate HW/D ratio:

- Use a straight edge.

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

- Mark the first HW/D scale. Extend a horizontal line to the desired scale and read HW/D and note on the Culvert Design Form in Figure 10.10-T.

d. Calculate headwater depth (HW_i):

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

- Neglecting the approach velocity, HW_i = HW. *Note: This is SDDOT practice*.

- Including the approach velocity, HW_i = HW – (approach velocity head).

Step 8 <u>Determine Outlet Control Headwater Depth at Inlet (HW_{oi})</u>

a. Calculate the tailwater depth (TW) using the design flow rate and normal depth (single section) or using a water surface profile.

b. Calculate culvert critical depth (d_c) using the appropriate chart in Section 10.10:

- Locate flow rate and read d_c.

- d_c cannot exceed D.

- If $d_c > 0.9D$, consult Reference (10) for a more accurate d_c, if needed, because curves are truncated where they converge.

c. Calculate $(d_c + D)/2$.

d. Determine (h_o):

- h_o = the larger of TW or (d_c + D)/2.

e. Determine (k_e):

- Entrance loss coefficient from Figures 10.10-B and Figure 10.10-C.

f. Determine losses through the culvert barrel (H):

- Use applicable nomograph in Section 10.10 or Equation 10.5 or 10.6 if outside range.

- Locate appropriate k_e scale:

  + locate culvert length (L) or (L_1),

  + use (L) if Manning's n matches the n value of the culvert, and

  + use (L_1) to adjust for a different culvert n value:

$$L_1 = L(n_1/n)^2 \qquad \text{(Equation 10.10)}$$

where:

L_1	=	adjusted culvert length, ft
L	=	actual culvert length, ft
n_1	=	desired Manning n value
n	=	Manning n value from Figure 10.10-A

- Mark point on turning line:

  + use a straight edge, and
  + connect size with the length.

- Read (H):

  + use a straight edge,
  + connect Q and turning point, and
  + read (H) on Head Loss scale.

g. Calculate outlet control headwater (HW_{oi}):

- Use Equation 10.11 if V_u and V_d are neglected (SDDOT practice):

  $$HW_{oi} = H + h_o - S_oL \qquad \text{(Equation 10.11)}$$

- Use Equations 10.1, 10.4c and 10.6 to include V_u and V_d.

- If HW_{oi} is less than 1.2D and control is outlet control, the barrel may flow partly full:

  + if the headwater depth falls below 0.75D, the approximate nomograph method should not be used and the approximate method of using the greater of tailwater or (d_c + D)/2 may not be applicable; and

  + backwater calculations should be used to determine the headwater.

Step 9 Determine Controlling Headwater (HW_c)

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

b. $HW_c = HW_i$; if $HW_i > HW_{oi}$.

Step 10 Compute Discharge Over the Roadway (Q_r)

a. Calculate depth above the roadway (HW_r):

- $HW_r = HW_c - HW_{ov}$.
- HW_{ov} = height of road above inlet invert.

b. If $HW_r \leq 0$, $Q_r = 0$
If $HW_r > 0$, determine C_d from Figure 10.10-S

c. Determine length of roadway crest (L).

d. Calculate Q_r using Equation 10.12:

- $$Q_r = C_d\, L\, HW_r^{1.5} \qquad \text{(Equation 10.12)}$$

Step 11 Compute Total Discharge (Q_t)

a. $Q_t = Q_d + Q_r$ (Equation 10.13)

Step 12 Calculate Outlet Depth and Velocity (V_o)

If inlet control is the controlling headwater:

a. Calculate flow depth at culvert exit:

- use culvert normal depth (d_n), or
- use water surface profile within culvert.

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:

- use culvert (d_c) if d_c > TW.
- use (TW) if culvert d_c < TW < D.
- use (D) if D < TW or d_c > D

b. Calculate flow area (A).

c. Calculate exit velocity (V_o) = Q/A.

Step 13 Review Results

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

- the barrel must have adequate cover,
- the length should be close to the approximate length,
- the headwalls and wingwalls must fit site,
- the allowable headwater should not be exceeded,

- the allowable overtopping flood frequency should not be exceeded, and

- the allowable outlet velocity should 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 (Q_o = overtopping flow):

- Q_{100}, if $Q_o \leq Q_{100}$.
- Q_{500}, if $Q_o > Q_{100}$.
- Q_{max}, if no overtopping is possible
  (Q_{max} = largest flood that can be estimated).

Step 15 Special Considerations

Consider the following options:

- flood routing if a large upstream headwater pool exists (see Section 10.7);

- tapered inlets if culvert is in inlet control and has limited available headwater (see Section 10.8);

- broken-back culverts (see Section 10.9);

- energy dissipation if V_o is larger than the normal V in the downstream channel (see Chapter 11 "Energy Dissipators");

- debris control storage for sites with sediment concerns (e.g., alluvial fans) or with other debris concerns (see HEC 9, Reference (4)); and

- related designs (e.g., buoyancy resistance, relief openings, land-use culverts, irrigation facilities, fish passage, thrust blocks) (see Section 10.3.4).

10.5.2 Documentation

Section 6.3.8.2 presents the information that should be documented in the Hydraulic Design Report for culverts. In addition, where the drainage area is greater than 1000 acres, the following applies to documentation requirements:

I. *Draft Hydraulic Design Report*

In addition to the items discussed in Section 6.3.8.2, the Report should include the following, as appropriate:

A. Site-Specific Hydraulic Performance Criteria (Section 14.2)

B. Risk Assessment (Section 14.2.2.5)

- Floodplain land use
- Environmentally sensitive areas (e.g., fisheries, wetlands)

C. Stream Stability Assessment

- Level I qualitative analysis (Section 14.4.2.1)
- Geomorphic factors (Figure 14.4-A) and hydraulic factors (Figure 14.4-B) that affect stream stability
- Identification of existing bed or bank instability (HEC 20, Reference (11))

D. Hydrologic Computations

- Discharges for specified frequencies (Section 14.2.2)
- Discharge and frequency for historical flood that complements the high-water marks used for calibration

E. Hydraulic Computations

- Computational method (Section 14.5.3.2)
- Computer model selection (Section 18.2.3)
- Hydraulic performance for existing conditions
- Hydraulic performance of proposed designs
- Scour computations, if appropriate (Section 14.6)

II. *Final Hydraulic Design Report*

In addition to the items already included in the Draft Hydraulic Design Report and the items discussed in Section 6.3.8.2, the Final Hydraulic Design Report should include the following, as appropriate:

- Risk analysis documentation, if applicable (Section 14.2.2.5).
- Erosion protection details (Section 15.6 and Section 15.7).
- Countermeasure design details (Section 15.9).
- Scour computations (Section 14.6).

- Scour countermeasure design details (Section 14.6.9).
- Scour monitoring plan or instrumentation, if applicable (Section 14.6.9).

10.6 DESIGN EXAMPLE (USING NOMOGRAPHS)

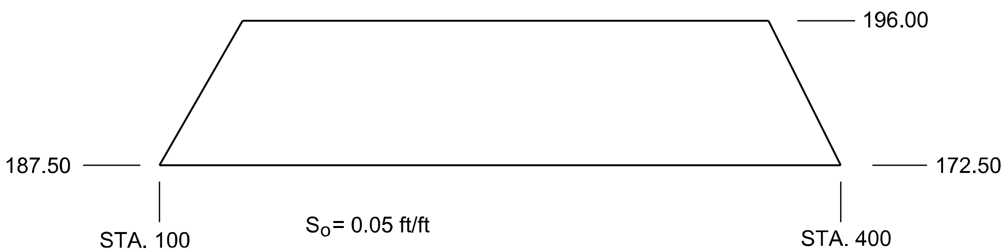
The following example problem follows the design procedure steps described in Section 10.5 and is documented on the computation sheet in Figure 10.6-A. Designers should document Steps 1 through 3 in the project file and summarize the results on a computation sheet or computer output.

Note: The example shown is for a 50-year frequency, which would be typical for an Interstate project.

Step 1 Assemble Site Data and Project File

a. Site survey project file contains:

- USGS, site and location maps;
- roadway profile; and
- embankment cross section.



Site visit notes indicate:

- no sediment or debris problems, and
- no nearby structures.

b. Studies by other agencies — none.

c. Environmental and risk assessment show:

- no buildings near floodplain,
- no sensitive floodplain values,
- no FEMA involvement, and
- convenient detours exist.

d. Design criteria:

- 50-year frequency for design, and
- 100-year frequency for review.

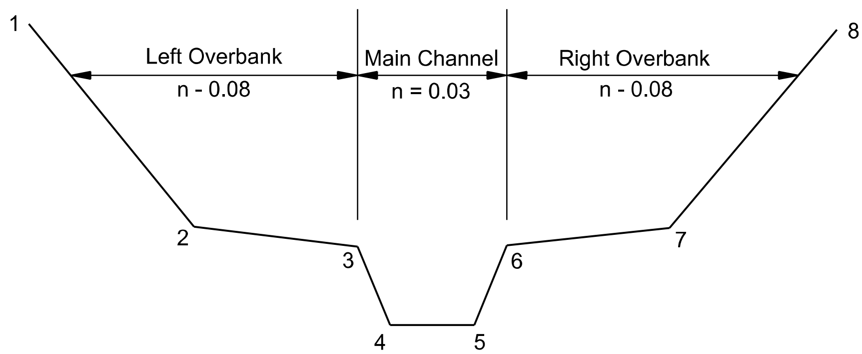
Step 2 Determine Hydrology

USGS regression equations yield:

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

Step 3 Design Downstream Channel

Cross section of channel (slope = 0.05 ft/ft):



Point	Station, ft	Elevation, ft
1	12	180.0
2	22	175.0
3	32	174.5
4	34	172.5
5	39	172.5
6	41	174.5
7	51	175.0
8	61	180.0

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

Q (cfs)	TW (ft)	V (ft/s)
100	1.4	11.1
200	2.1	13.7
300	2.5	16.0
400	2.8	17.5
500	3.1	18.8

Note: Above table is based on an iterative process using Manning's equation.

PROJECT: Example Problem

STATION: 9+00

SHEET 1 **OF** 1

CULVERT DESIGN FORM

DESIGNER/DATE PLT / ______

REVIEWER/DATE ______ / ______

HYDROLOGICAL DATA

SEE ADDL SHTS

☒ **METHOD:** USGS

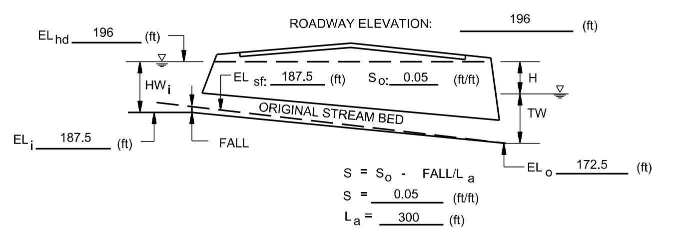
☐ **DRAINAGE AREA:** ______ ☐ **STREAM SLOPE:** ______

☐ **CHANNEL SHAPE:** ______

☐ **ROUTING:** ______ ☐ **OTHER:** ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW(ft)
50	400	2.8
100	500	3.1



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
RCBC - 7x6 - Bevel	400	57	1.27	7.6	0	195.1	2.8	4.7	5.4	5.4	.2	2.8	180.7	195.1	32	195.1'<196' OK
	500	72	1.56	9.4	0	196.9	3.1	5.4	5.7	5.7	.2	4.3	182.2	196.9	34	>196' Calc Qr
Performance Curve	100	14	.46	2.8	0	190.3										
	200	29	.78	4.7	0	192.2										
	300	43	1.01	6.1	0	193.6										

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

(3) FALL=HW_i - (EL_{hd}-EL_{sf}): FALL 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) $H=[1+k_e+(29n^2L)/R^{1.33}]V^2/2g$

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

SUBSCRIPT DEFINITIONS:

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

COMMENTS / DISCUSSION:

Use 500cfs in culvert:

$Q_r=C_dLWH_r^{1.5}$=3.03(200')(.9)$^{1.5}$=520cfs

Q_T=500+520=1020cfs

CULVERT BARREL SELECTED:

SIZE: 7 X 6

SHAPE: RCBC

MATERIAL: ______ n .012

ENTRANCE: Beveled

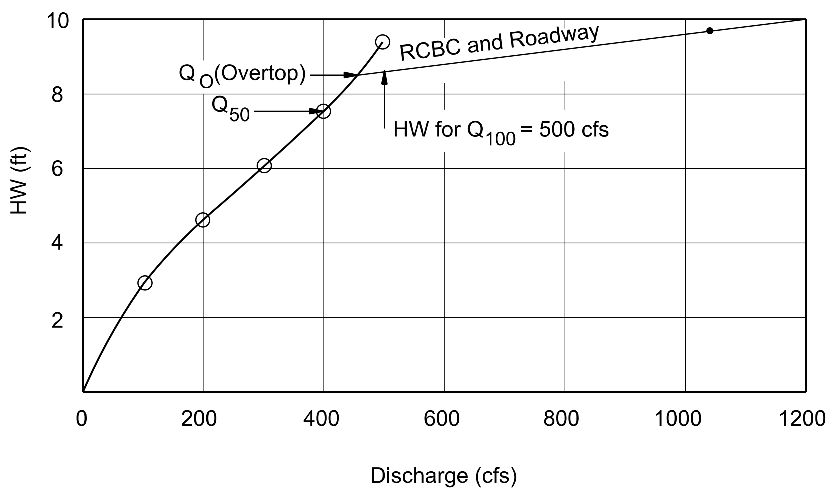


Figure 10.6-A — CULVERT DESIGN FORM AND PERFORMANCE CURVE FOR DESIGN EXAMPLE

Step 4 Summarize Data on Design Form

See Figure 10.6-A.

Step 5 Select Design Alternative

Shape — box
Material — concrete
Size — 7 ft × 6 ft
Entrance — beveled

Step 6 Select Design Discharge

$Q_d = Q_{50} = 400$ cfs

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use inlet control nomograph – HDS 5 Chart 10 (Figure 10.10-M)

a. D = 6 ft

b. Q/B = 400/7 = 57

c. HW/D = 1.33 for ¾-in chamfer
HW/D = 1.27 for 45° bevel

d. HW_i = (HW/D)D = (1.27)6 = 7.6 ft (Neglect the approach velocity)

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

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

b. d_c = 4.7 ft from HDS 5 Chart 14 (Figure 10.10-Q)

c. $(d_c + D)/2 = (4.7 + 6)/2 = 5.4$ ft

d. h_o = the larger of TW or $(d_c + D)/2$
$h_o = (d_c + D)/2 = 5.4$ ft

e. k_e = 0.2 from HDS 5 Table 2 (Figure 10.10-B)

f. Determine (H) from HDS 5 Chart 15 (Figure 10.10-R):

- k_e scale = 0.2
- culvert length (L) = 300 ft
- n = 0.012 (same as in Figure 10.10-A)
- area = 42 sq ft
- H = 2.8 ft

g. $HW_{oi} = H + h_o – S_oL = 2.8 + 5.4 – (0.05)300 = – 6.8$ ft

Because HW_{oi} is less than 1.2D, the barrel will not flow full at the design discharge, which is the conservative assumption used for hand or nomograph solutions. The assumption is extremely conservative in this case because a negative HW_{oi} indicates that the required HW is below the streambed. A computer solution shows that Flow Type V (see Figure 10.4-C) is present in the barrel at the design discharge.

Step 9 Determine Controlling Headwater (HW_c)

a. $HW_c = HW_i = 7.6$ ft $> HW_{oi} = -6.8$ ft

b. The culvert is in inlet control.

Step 10 Compute Discharge Over Roadway (Q_r)

a. Calculate depth above the roadway:

$HW_{ov} = 196 - 187.5 = 8.5$

$HW_r = HW_c - HW_{ov} = 7.6 - 8.5 = -0.9$ ft

b. If $HW_r \leq 0$, $Q_r = 0$

Step 11 Compute Total Discharge (Q_t)

$Q_t = Q_d + Q_r = 400 \text{ cfs} + 0 = 400 \text{ cfs}$

Step 12 Calculate Outlet Depth and Velocity (V_o)

INLET CONTROL:

a. Calculate normal depth (d_n):

$$Q = (1.486/n)A\,R^{2/3}\,S^{1/2} = 400 \text{ cfs}$$
$$400 = (123.8)(7)(d_n)[7(d_n)/(7 + 2d_n)]^{2/3}(0.05)^{0.5}$$
$$14.4 = (7)(d_n)[7(d_n)/(7 + 2d_n)]^{2/3}$$

try $d_n = 2.0$ ft, $16.5 > 14.4$

use $d_n = 1.8$ ft, $14.1 \approx 14.4$

b. $A = (1.8)7 = 12.6$ sq ft

c. $V_o = Q/A = 400/12.6 = 31.7$ fps

Step 13 Review Results

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

- barrel has (8.5 – 6) = 2.5 ft of cover;
- L = 300 ft is OK, because inlet control;
- headwalls and wingwalls fit site;
- allowable headwater (8.5 ft) > 7.6 ft is OK; and
- overtopping flood frequency > 50-year.

Step 14 Plot Performance Curve

Use Q_{100} for the upper limit. Steps 6 through 12 should be repeated for each discharge used to plot the performance curve. These computations are provided on the computation form in Figure 10.6-A.

Step 15 Special Considerations

Consider the following options:

- Flood Routing. Because a small upstream headwater pool exists, routing is not feasible.

- Tapered Inlets. Culvert is in inlet control and has limited available headwater.

- Broken-Back Culvert. No break in slope is needed.

- Energy Dissipation. Because V_o = 31.7 fps > 18.0 fps in the downstream channel, review options in Chapter 11 "Energy Dissipators".

- Debris Control. The site has no sediment or other debris problems.

- Related Designs. These considerations do no apply, and fish passage is not an issue.

Step 16 Documentation

Include the computation sheet in the project file.

10.7 FLOOD ROUTING CULVERT DESIGN

10.7.1 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, the culvert analysis will be conservative. Normal SDDOT practice is to only consider storage in urban areas where storm water management practices are in use or in rural areas where permanent storage is provided upstream. Flood routing can be used in other cases where storage is available and there is a need to reduce the culvert size. Flood routing should be considered for sites where routing has been occurring at the current highway crossing. If routing is not continued for such cases, there may be an increased flow rate downstream.

10.7.2 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 (Chapter 5 "Data Collection") and the hydrology analysis completed to include estimating a hydrograph (Chapter 7 "Hydrology"). Once this essential information is available, the culvert can be designed.

Flood routing through a culvert can be time consuming. Software should be used for flood routing so that alternative culverts can be evaluated that satisfy the given criteria (design). See Section 18.2.3.

Before attempting to design a culvert to take advantage of storage, the designer should review the culvert storage routing design process included in HDS 5, Chapter 5 (Reference (1)). This review 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.

10.8 TAPERED INLETS

10.8.1 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. Therefore, 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.

SDDOT practice is to only use a tapered inlet where a traditional culvert entrance proves to be hydraulically inadequate for the site. On some steeper slopes, a tapered inlet may be advantageous to reduce the size of the culvert. Cost savings can be realized on large culverts by making the barrel more efficient. A disadvantage of tapered inlets is potential debris clogging in the culvert.

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 (see HDS 5, Reference (1)).

10.8.2 Side-Tapered Inlets

The side-tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the side walls (Figure 10.8-A). 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 10.8-A, 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 10.8-B 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

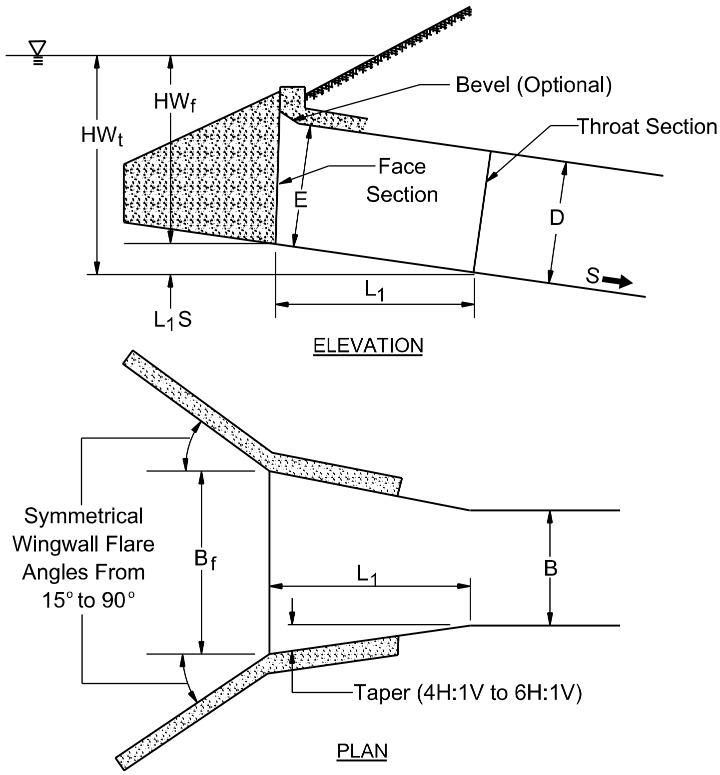


Figure 10.8-A — SIDE-TAPERED INLET

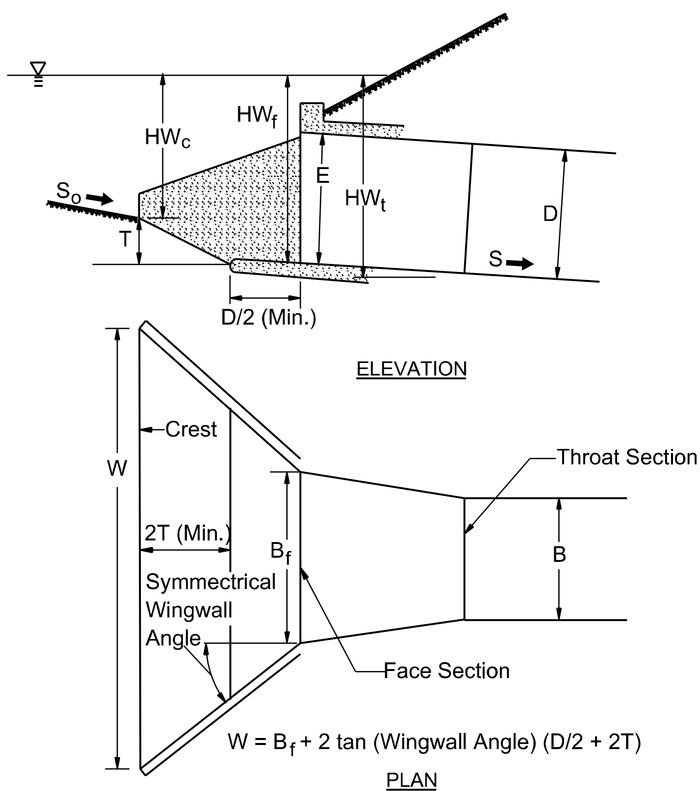


Figure 10.8-B — SIDE-TAPERED INLET WITH UPSTREAM DEPRESSION CONTAINED BETWEEN WINGWALLS

upstream crest where the slope of the depression meets the streambed should be checked to ensure 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 (throat section) should control at the design discharge.

10.8.3 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 10.8-C). 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 section are determined by the design procedures in HDS 5 (Reference (1)). 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.

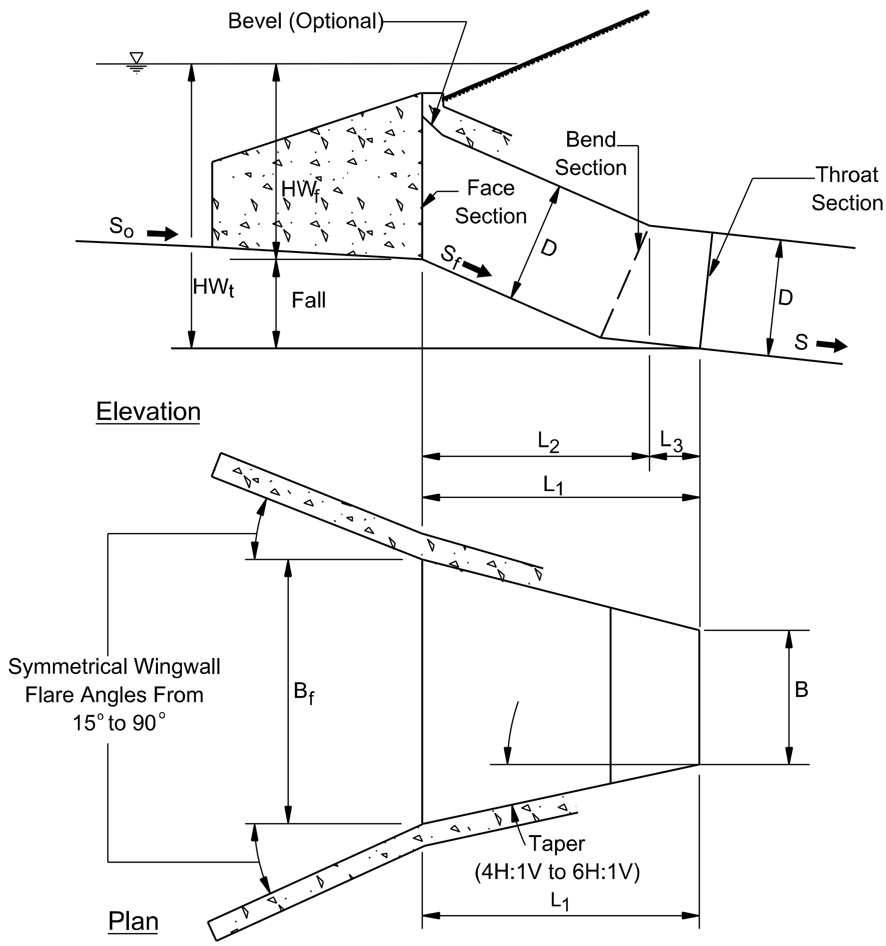


Figure 10.8-C — SLOPE-TAPERED INLET WITH VERTICAL FACE

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; therefore, the face sections of these inlets are larger than the face sections of equivalent depressed side-tapered inlets. The required face size can be reduced by the use of bevels or other favorable edge configurations. The vertical face slope-tapered inlet design is shown in Figure 10.8-C.

The slope-tapered inlet is the most complex inlet improvement discussed 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.

10.8.4 Hydraulic Design

10.8.4.1 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, and then select the segment of each curve, which defines the minimum overall culvert performance (see Figure 10.8-D). Typically, only the face and throat control curves are calculated.

10.8.4.1.1 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.

10.8.4.1.2 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

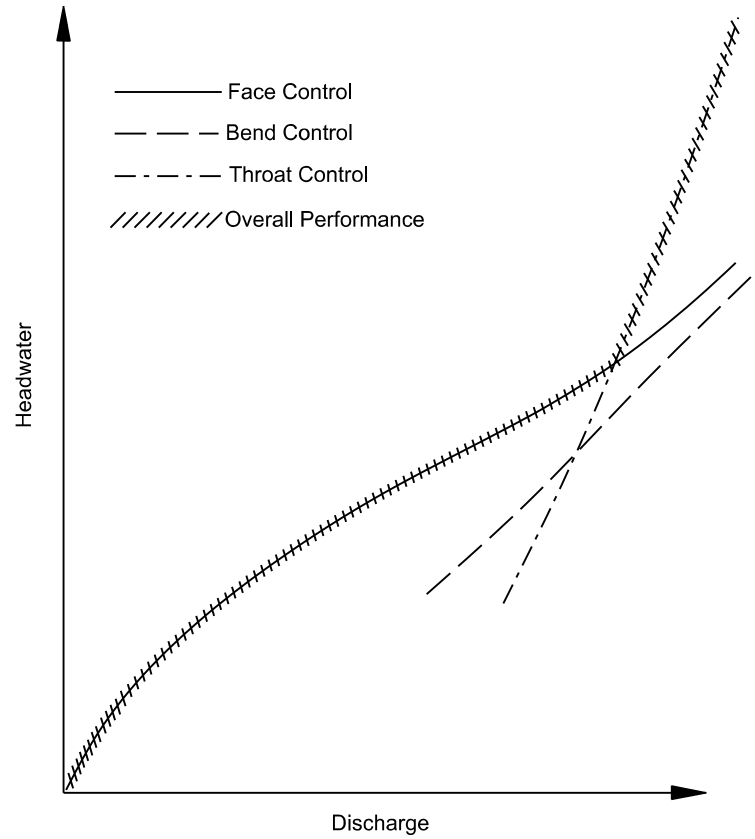


Figure 10.8-D — INLET CONTROL PERFORMANCE CURVES (Schematic)

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 in this *Manual* are followed. However, the bend will contribute to the inlet losses that are included in the inlet loss coefficient, k_e.

10.8.4.2 Outlet Control

When a culvert with a tapered inlet performs in outlet control, the hydraulic analysis is the same as described in Section 10.4 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.

10.8.5 Design Methods

Tapered inlet design begins with the selection of the culvert barrel size, shape and material. The calculations are performed using the Culvert Design Form (see Figure 10.10-U). The design nomographs contained in Section 10.10 are used to design the tapered inlet. 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 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. Some 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 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 10.8-E, 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.

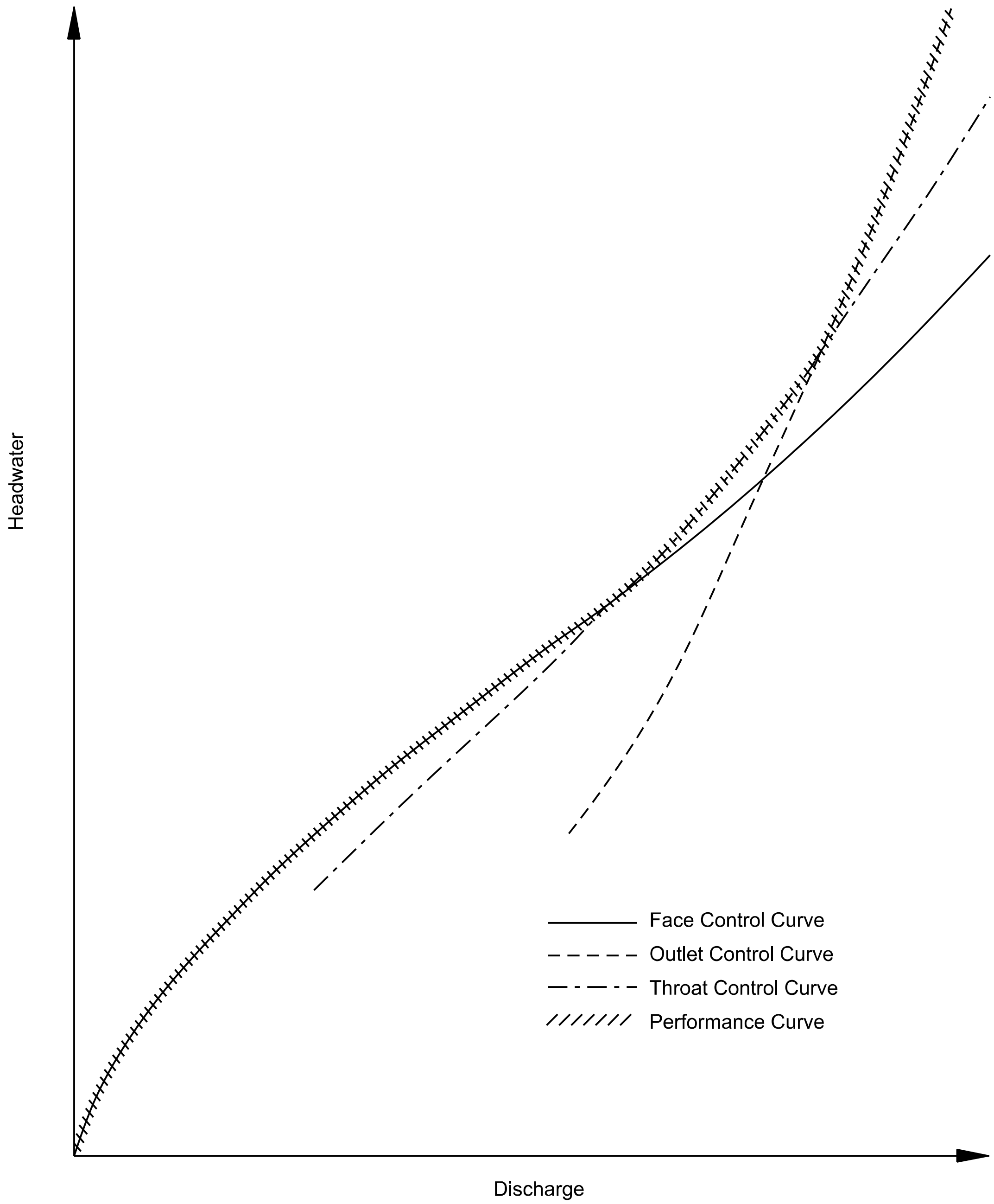


Figure 10.8-E — CULVERT OVERALL PERFORMANCE CURVE (Schematic)

10.9 BROKEN-BACK CULVERTS

10.9.1 Introduction

The broken-back culvert design procedure is based on material taken from the *TxDOT Hydraulic Design Manual*, 2004. SDDOT practice is to consider a broken-back culvert as an alternative method for grade control or energy dissipators (see Chapter 11 "Energy Dissipators").

10.9.2 Broken-Back Culvert Guidelines

One potential mechanism for creating a hydraulic jump is the broken-back configuration, two types of which are depicted in Figure 10.9-A and Figure 10.9-B. 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 (see Figure 10.9-A and Figure 10.9-B) 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.

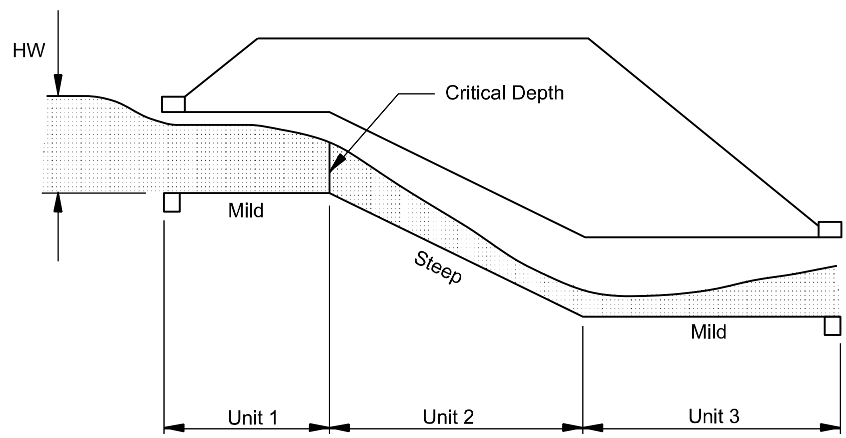


Figure 10.9-A — THREE-UNIT BROKEN-BACK CULVERT

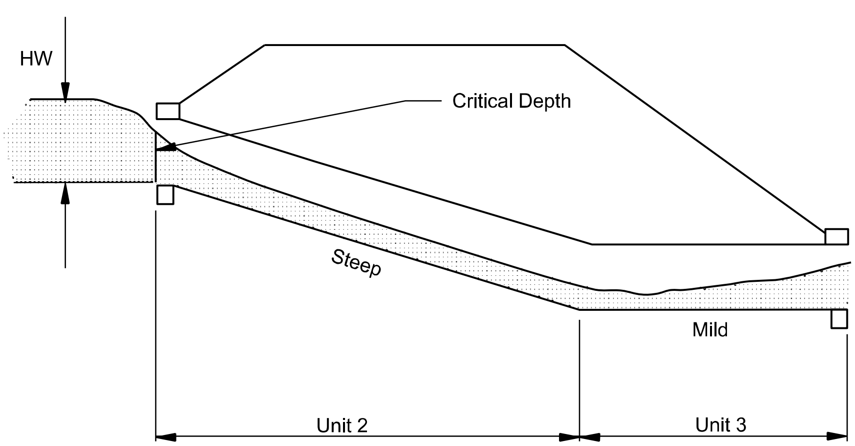


Figure 10.9-B — TWO-UNIT BROKEN-BACK CULVERT

10.9.3 Broken-Back Design Procedure

The design of a broken-back culvert is not especially difficult, but certain provisions must be made or circumstances found so that the primary intent of reducing velocity at the outlet is realized. Figure 10.9-C outlines the broken-back culvert design process. The hydraulics of circular and rectangular culverts can be determined using the Broken-back Culvert Analysis Program (BCAP) software from the Nebraska Department of Roads. (see Chapter 18 "Hydraulic Software"). The design of associated energy dissipators is contained in HEC 14, Chapter 7 (Reference (12)).

Step	Action
Step 1	Establish a flow-line profile.
Step 2	Size the culvert.
Step 3	Begin to calculate a supercritical profile.
Step 4	Complete profile calculations.
Step 5	Consider hydraulic jump cautions.

Figure 10.9-C — BROKEN-BACK CULVERT DESIGN PROCESS

10.10 DESIGN AIDS

Section 10.10 presents several tables, figures and forms required for the hydraulic design of culverts. These include:

1. Manning's n Values for Culverts. Figure 10.10-A provides ranges of n values that have been determined in the laboratory and from SDDOT design practice. This practice provides that alternative materials can be substituted for a given structure that is designed to have either a smooth or a corrugated interior.

2. Entrance Loss Coefficients. See Figure 10.10-B for generic values. Figure 10.10-C presents k_e values specifically for SDDOT pipe culvert end sections. For SDDOT entrance loss coefficients (k_e) for box culverts, refer to FHWA-HRT-06-138.

3. Culvert Nomographs. T he following culvert nomographs for circular and rectangular shapes are included; see HDS 5, Reference (1) for other culvert nomographs. The HDS 5 Calculator (available from FHWA) allows the designer to use the nomographs electronically :

   - Figure 10.10-D "Headwater Depth for Concrete Pipe Culverts with Inlet Control,"
   - Figure 10.10-E "Headwater Depth for C. M. Pipe Culverts with Inlet Control,"
   - Figure 10.10-F "Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control,"
   - Figure 10.10-G "Critical Depth (Circular Pipe),"
   - Figure 10.10-H "Head for Concrete Pipe Culverts Flowing Full (n = 0.012),"
   - Figure 10.10-I "Head for Standard C. M. Pipe Culverts Flowing Full (n = 0.024),"
   - Figure 10.10-J "Head for Structural Plate Corrugated Metal Pipe Culverts Flowing Full (n = 0.0328 to 0.0302),"
   - Figure 10.10-K "Headwater Depth for Box Culverts with Inlet Control,"
   - Figure 10.10-L "Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls 18° to 33.7° and 45° with Beveled Edge at Top of Inlet,"
   - Figure 10.10-M "Headwater Depth for Inlet Control, Rectangular Box Culverts, 90° Headwall, Chamfered or Beveled Inlet Edges,"

- Figure 10.10-N "Headwater Depth for Inlet Control, Single Barrel Box Culverts, Skewed Headwalls, Chamfered or Beveled Inlet Edges,"

- Figure 10.10-O "Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls, Normal and Skewed Inlet Edges, ¾″ Chamfer at Top of Opening,"

- Figure 10.10-P "Headwater Depth for Inlet Control, Rectangular Box Culverts, Offset Flared Wingwalls and Beveled Edge at Top of Inlet,"

- Figure 10.10-Q "Critical Depth (Rectangular Section),"

- Figure 10.10-R "Head for Concrete Box Culverts Flowing Full (n = 0.012)," and

- Figure 10.10-S "Discharge Coefficients for Roadway Overtopping."

4. Design Forms. The following Design Forms are presented for hand calculations for the hydraulic design of culverts:

   - Figure 10.10-T "Culvert Design Form," which is the standard form used for culverts. The procedure in Section 10.5 is based on this Form.

   - Figure 10.10-U "Side/Slope Tapered Design Form," which is used for the tapered inlets discussed in Section 10.8.

Type of Conduit	Wall Description	Manning's n Laboratory[1]	SDDOT Design
Concrete Pipe	Smooth	0.010-0.011	0.012
Concrete Boxes	Smooth	0.012-0.015	0.012
Spiral Rib Metal Pipe	Smooth walls	0.012-0.013	0.012
Corrugated Metal Pipe, Pipe-Arch and Box (Annular and Helical corrugations — See Figure B-3; Manning's n varies with barrel size (See HDS 5, Reference (1))	2⅔ × ½ in Annular	0.022-0.027	0.024
	2⅔ × ½ in Helical	0.011-0.023	0.024
	6 × 1 in Helical	0.022-0.025	0.024
	5 × 1 in	0.025-0.026	0.024
	3 × 1 in	0.027-0.028	0.024
	6 × 2 in Structural Plate	0.033-0.035	0.035
	9 × 2½ in Structural Plate	0.033-0.037	0.035
Corrugated Polyethylene	Smooth	0.009-0.015	0.012
Corrugated Polyethylene	Corrugated	0.018-0.025	0.024
Polyvinyl Chloride (PVC)	Smooth	0.009-0.011	0.012

[1] Source: HDS 5, Reference (1).

Figure 10.10-A — MANNING'S n VALUES FOR CULVERTS

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

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

* "End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both *inlet* and *outlet* control. Some end sections, incorporating a *closed* taper in their design, have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Figure 10.10-B — ENTRANCE LOSS COEFFICIENTS
(Outlet Control, Full or Partly Full)

SDDOT Plate Number	Type	k_e	HY8 k_e[1]
450.10	Reinforced Concrete Pipe Flared Ends	0.5	0.5
450.11	Reinforced Concrete Pipe Arch Flared Ends	0.5	0.5
450.12	R.C.P. Safety Ends With or Without Bars	0.7	0.7[2]
450.13	Reinforced Concrete Pipe Sloped Ends	0.7	0.7[2]
450.14	R.C.P. Sloped Ends With or Without Tie Bars	0.7	0.7
450.15	R.C.P. Sloped End Bar Assemblies	0.7	0.7
450.16	Reinforced Concrete Pipe Sectional Ends	0.5	0.5
450.17	Reinforced Concrete Pipe Arch Sectional Ends	0.5	0.5
450.35	Corrugated Metal Pipe Flared Ends	0.5	0.5
450.36	Corrugated Metal Pipe Arch Flared Ends	0.5	0.5
450.37	Corrugated Metal Pipe Sloped Ends	0.7	0.7
450.38	Corrugated Metal Pipe Safety Ends	0.7	0.7
NA	Corrugated Metal Pipe with thin edge projecting from fill (no headwall or end section)	0.9	0.9

[1] *To match entrance loss coefficients (k_e) with the HY-8 k_e values, open HY-8 help and select the following options: Analysis, Headwater Computations, Inlet Control, Polynomial Coefficients. The outlet control k_e values are included in the tables.*

[2] *Because HY-8 does not contain this edge treatment for RCP Arch, select a corrugated steel material and change the n value to 0.012.*

Note: For SDDOT entrance loss coefficients for box culverts, refer to FHWA-HRT-06-138.

Figure 10.10-C — SDDOT ENTRANCE LOSS COEFFICIENTS FOR PIPE CULVERTS (Outlet Control, Full or Partly Full)

CHART 1B

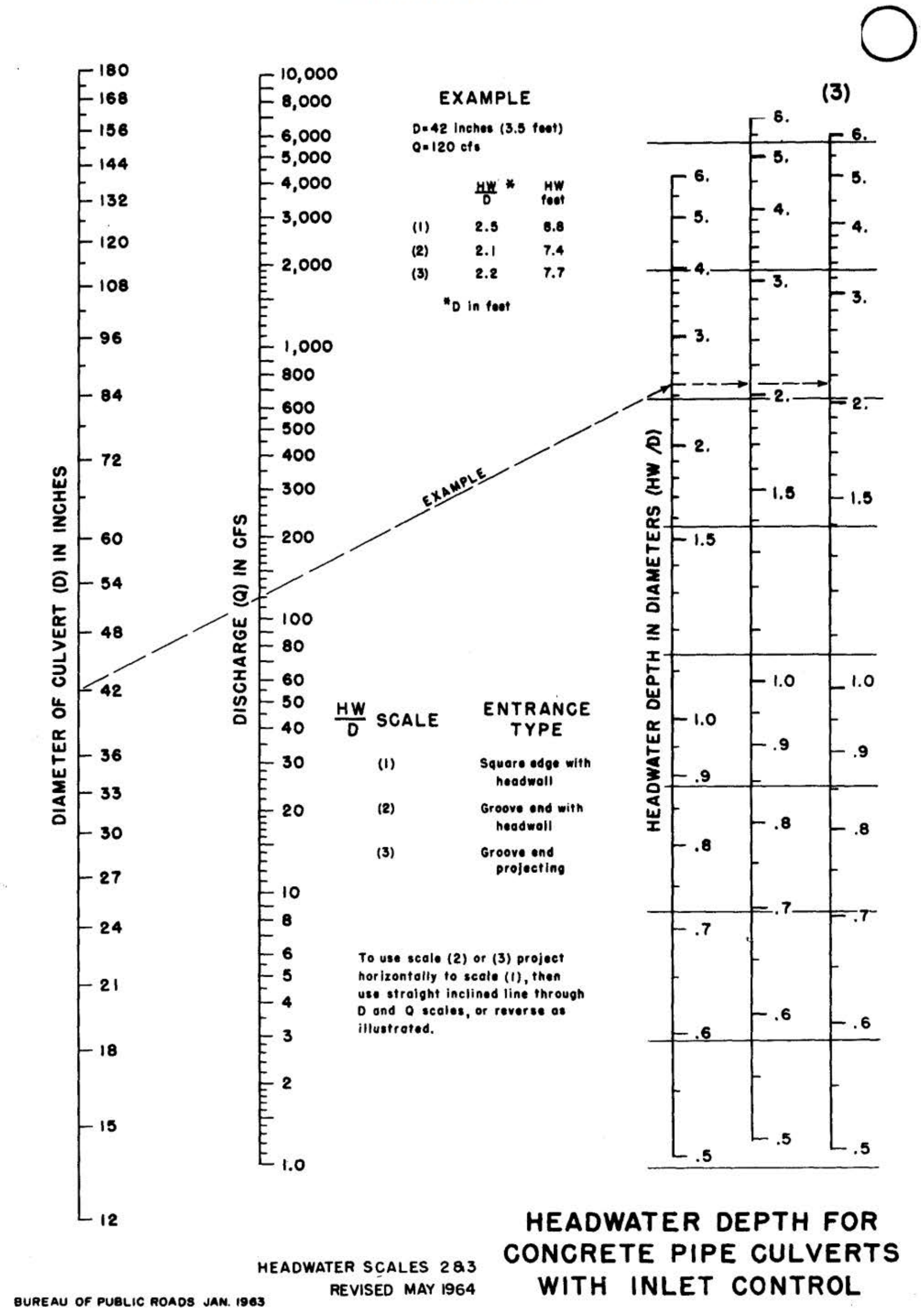


Figure 10.10-D

CHART 2B

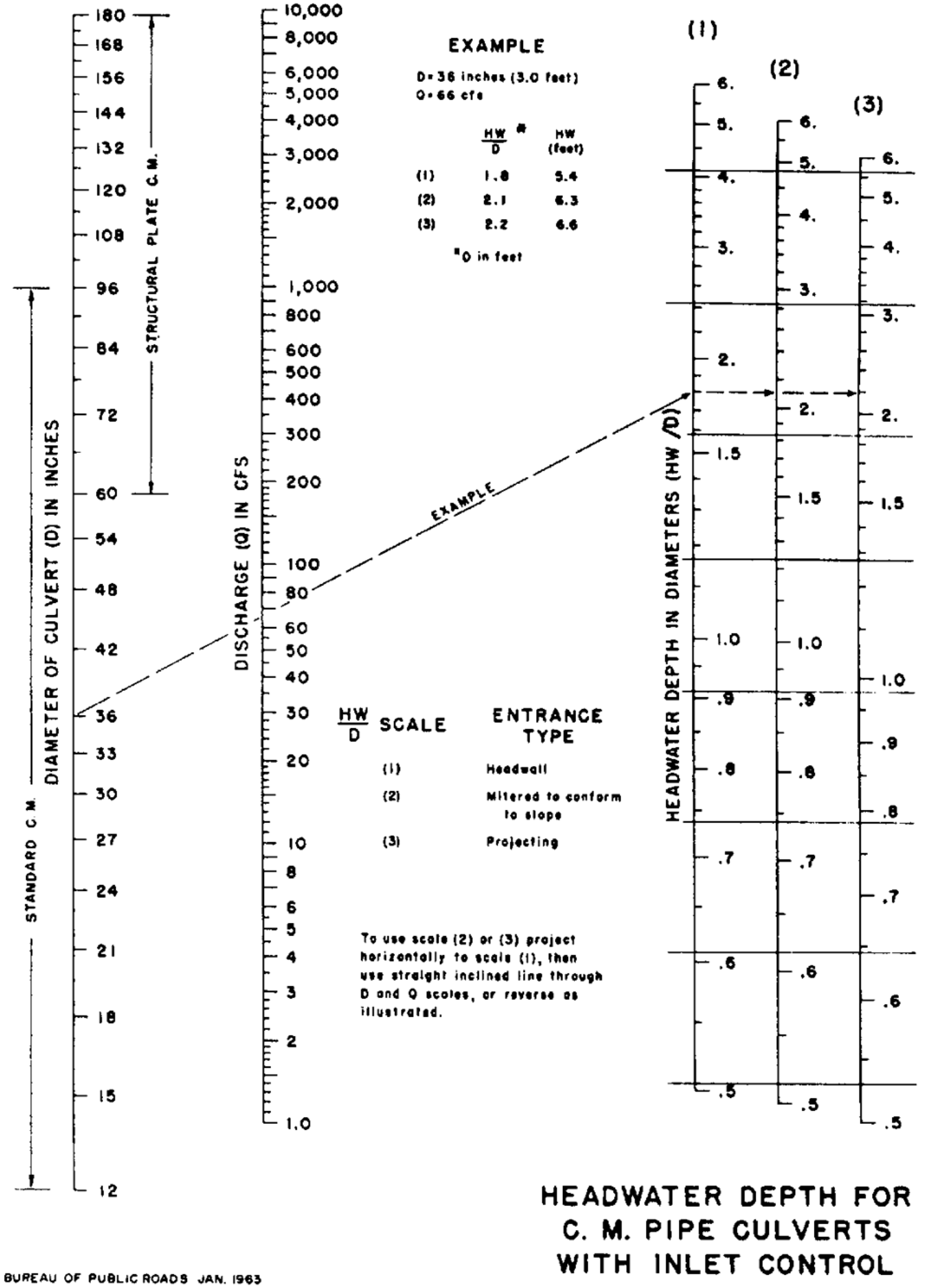
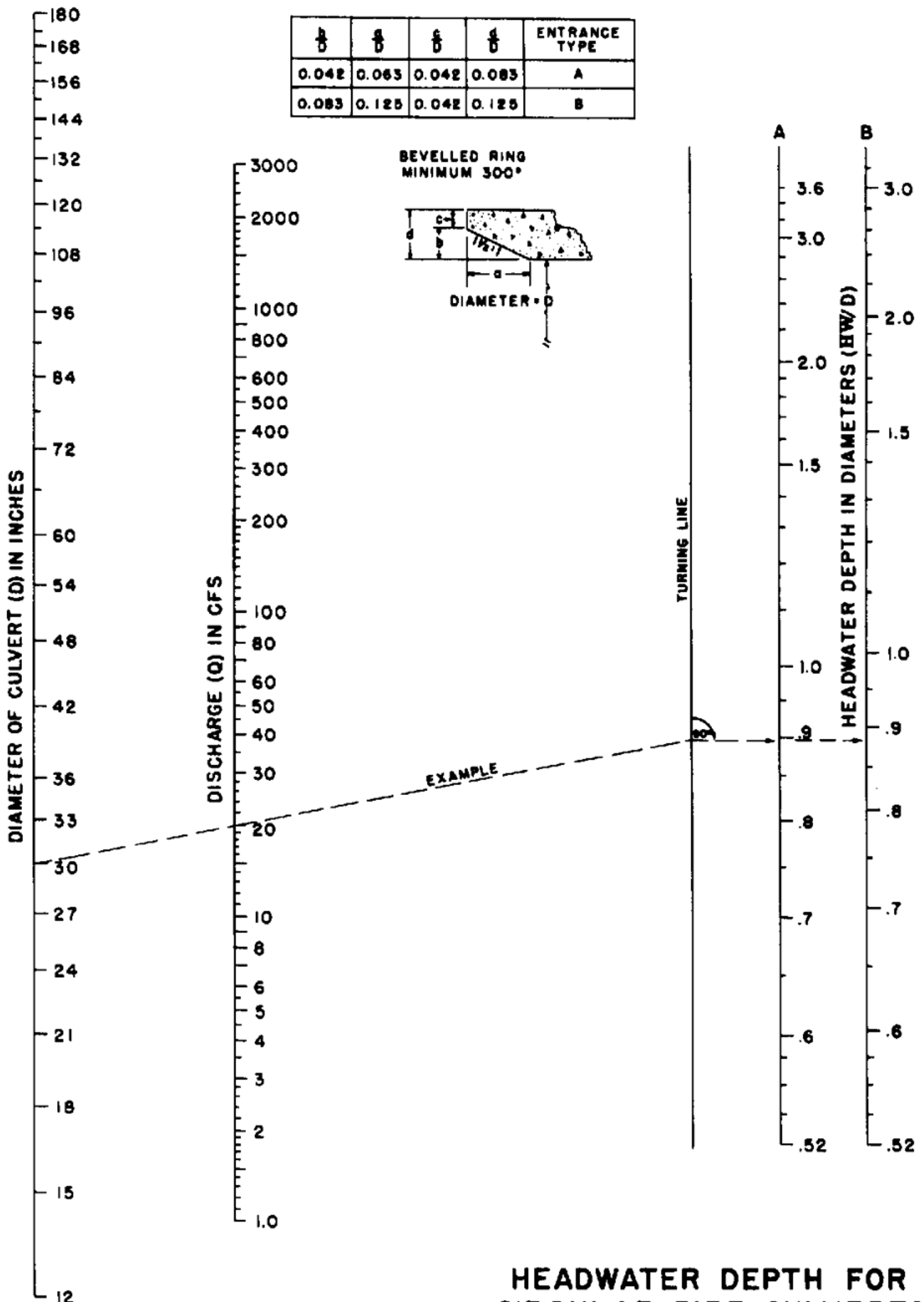


Figure 10.10-E

CHART 3B

b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B

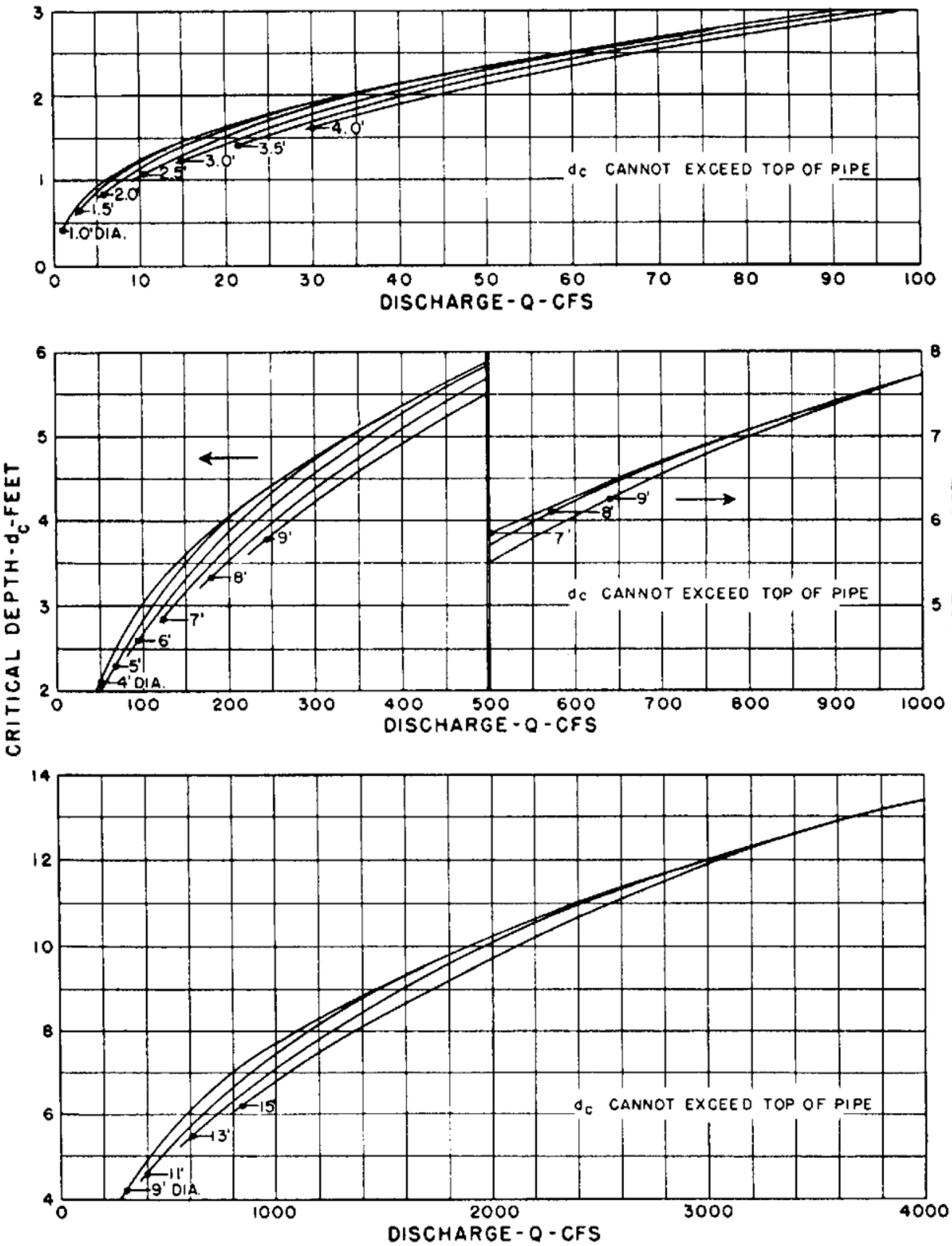


FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

Figure 10.10-F

CHART 4B

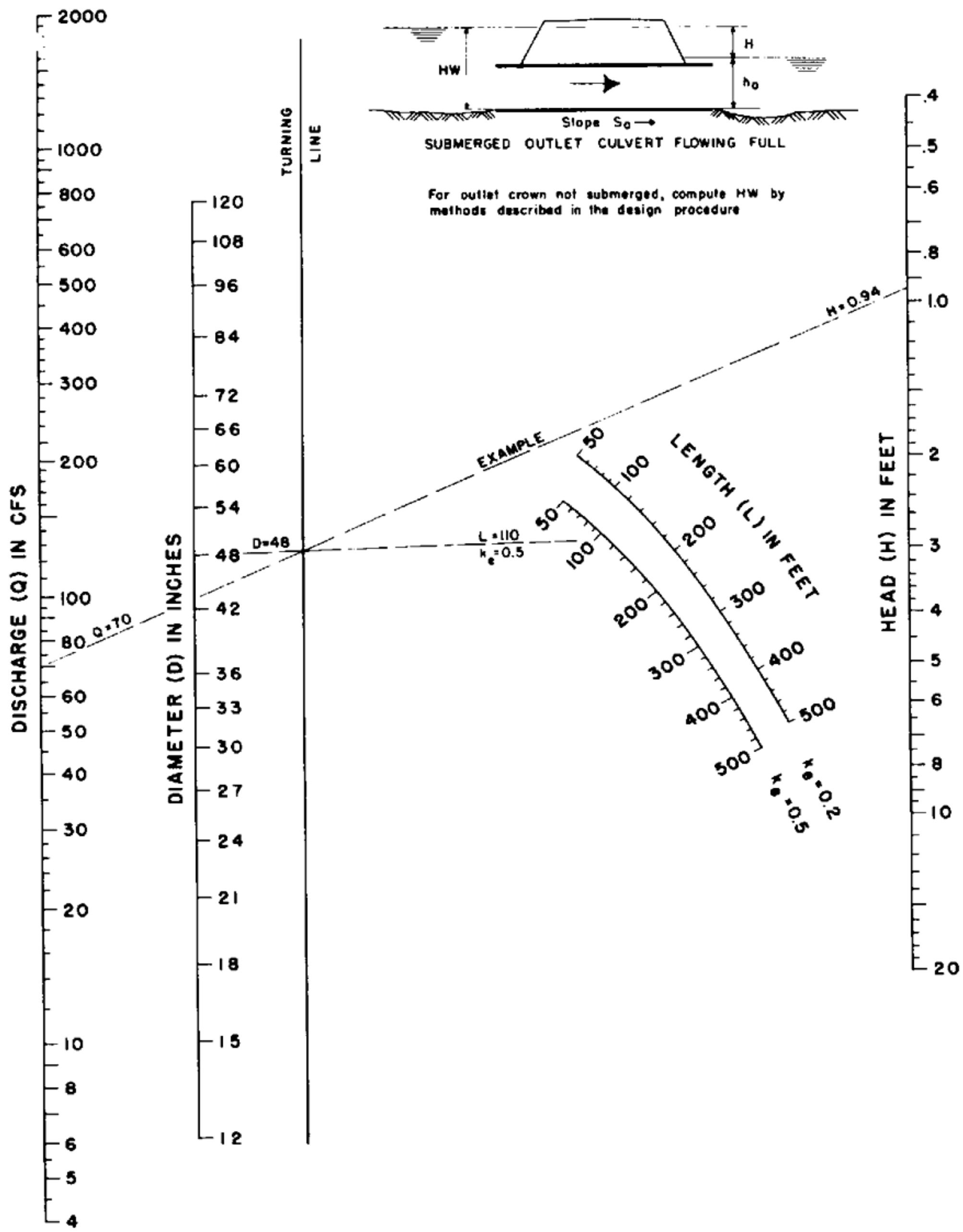


BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

Figure 10.10-G

CHART 5B



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

Figure 10.10-H

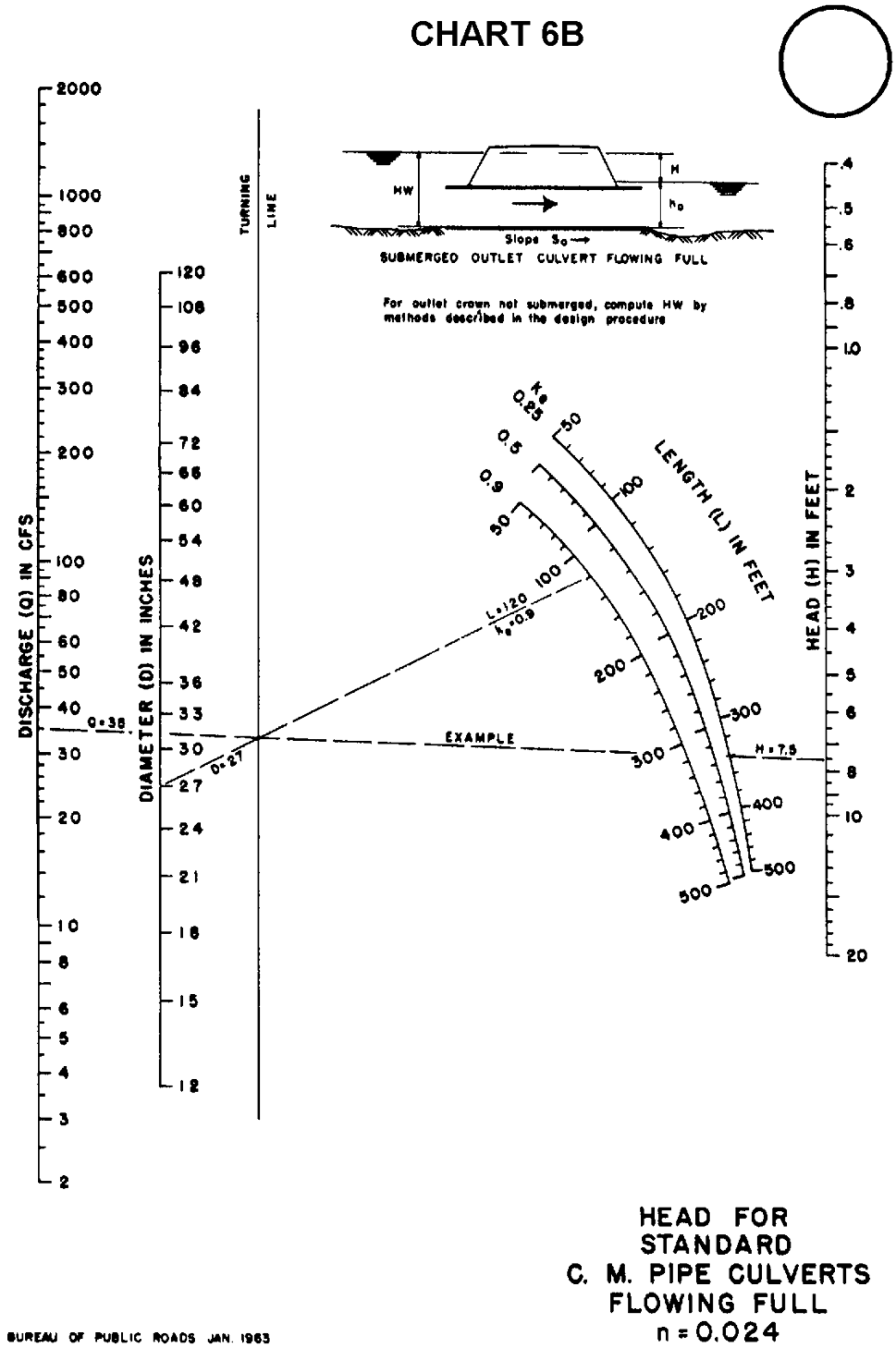


Figure 10.10-I

CHART 7B

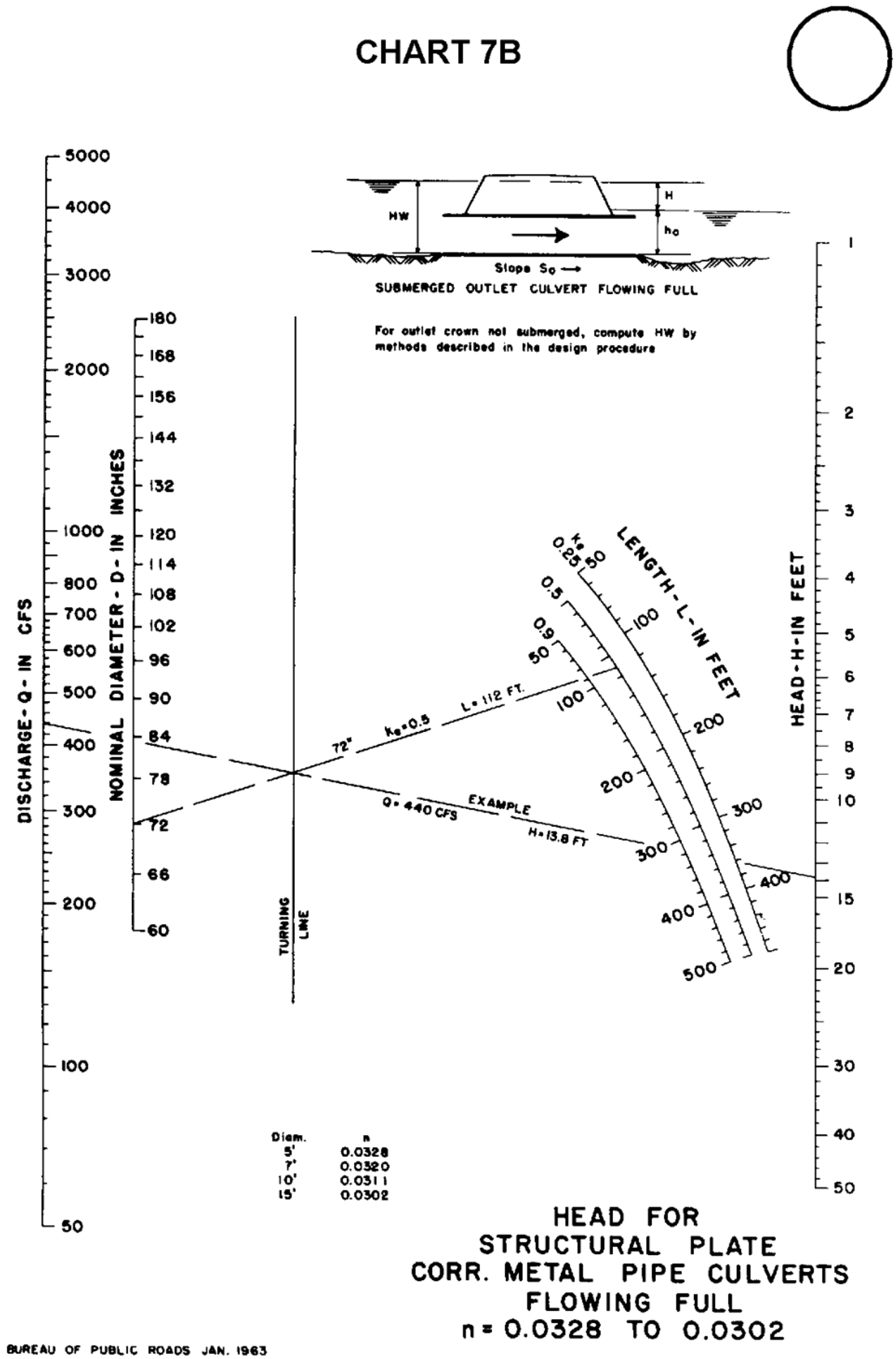


Figure 10.10-J

CHART 8B

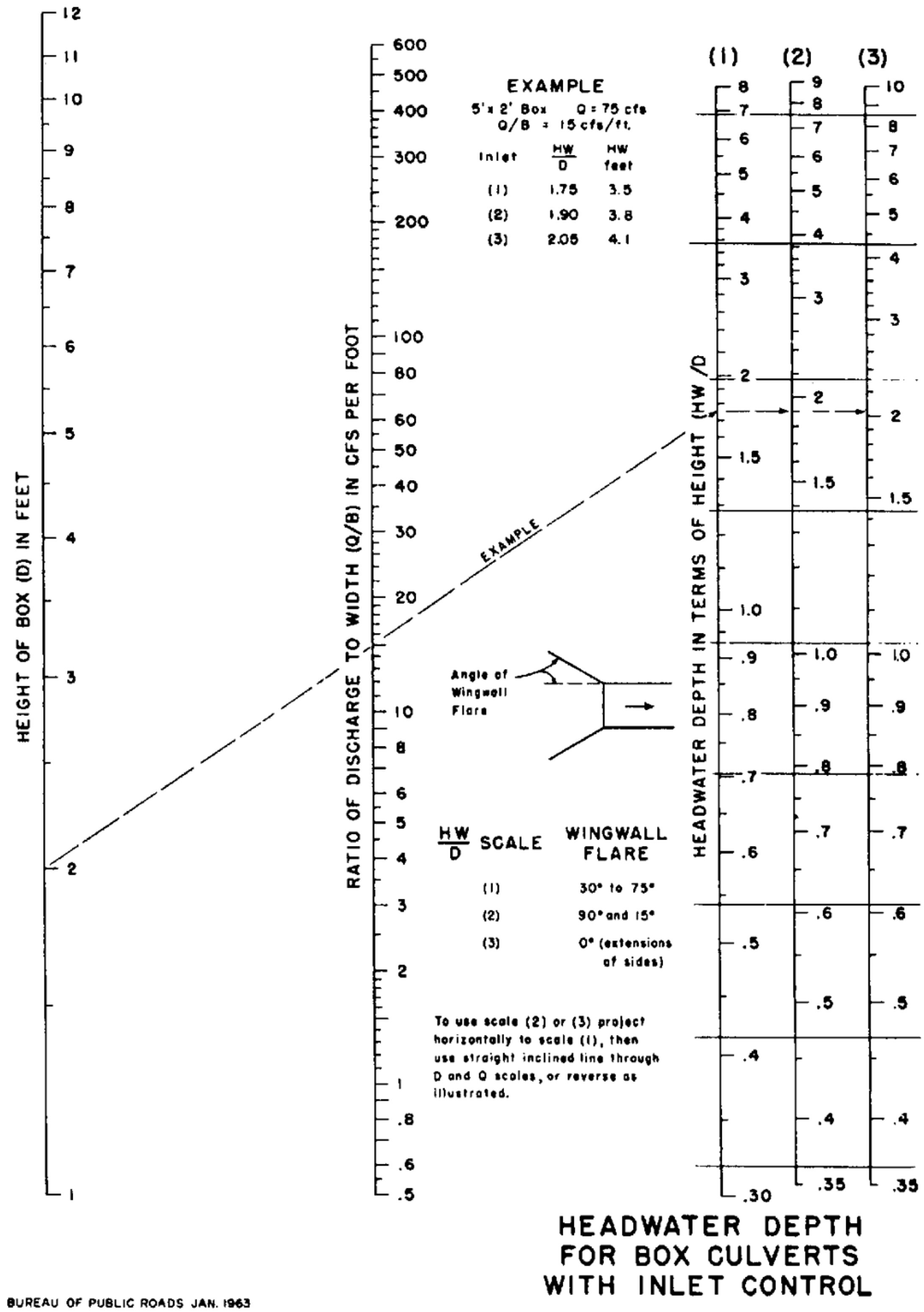


Figure 10.10-K

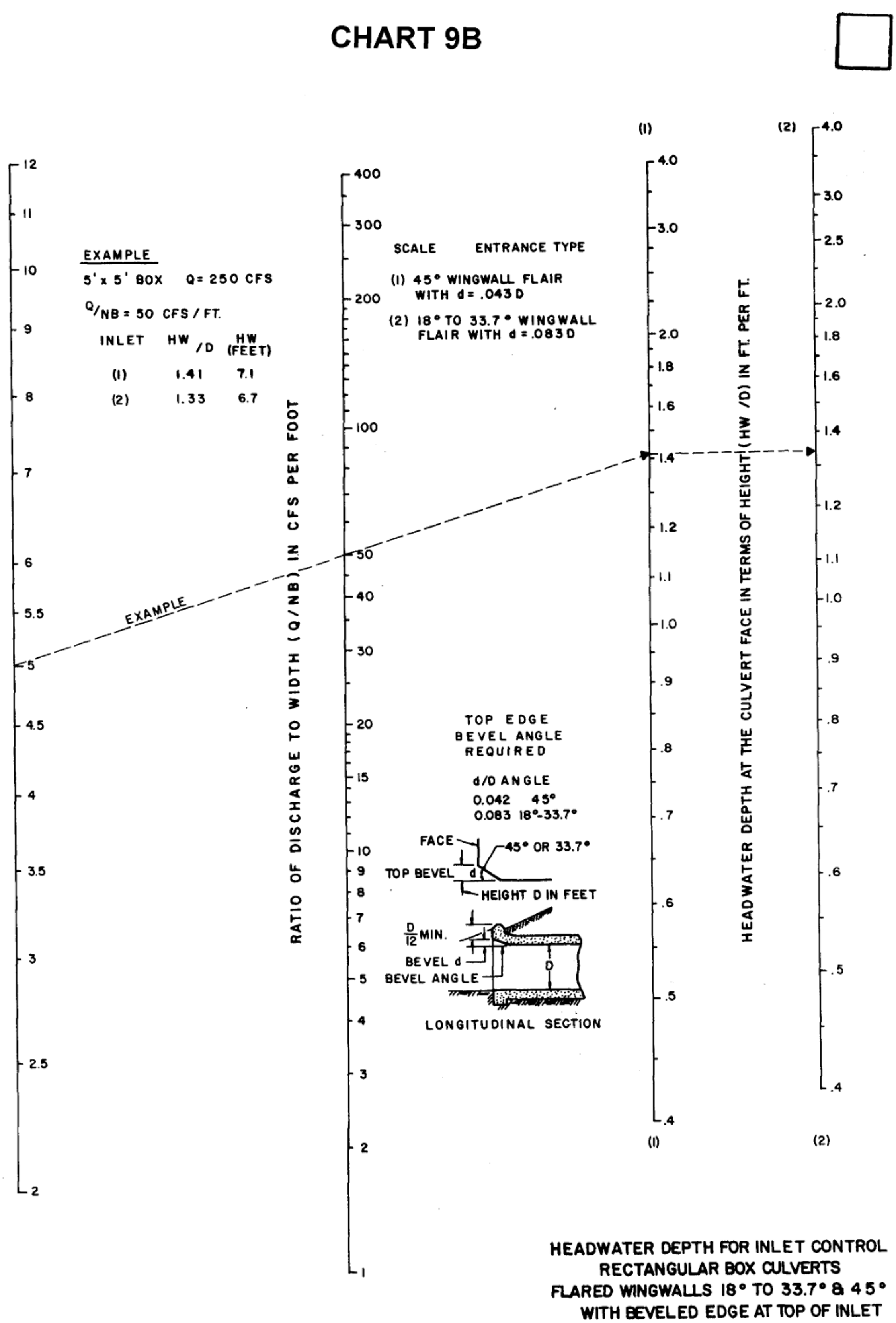
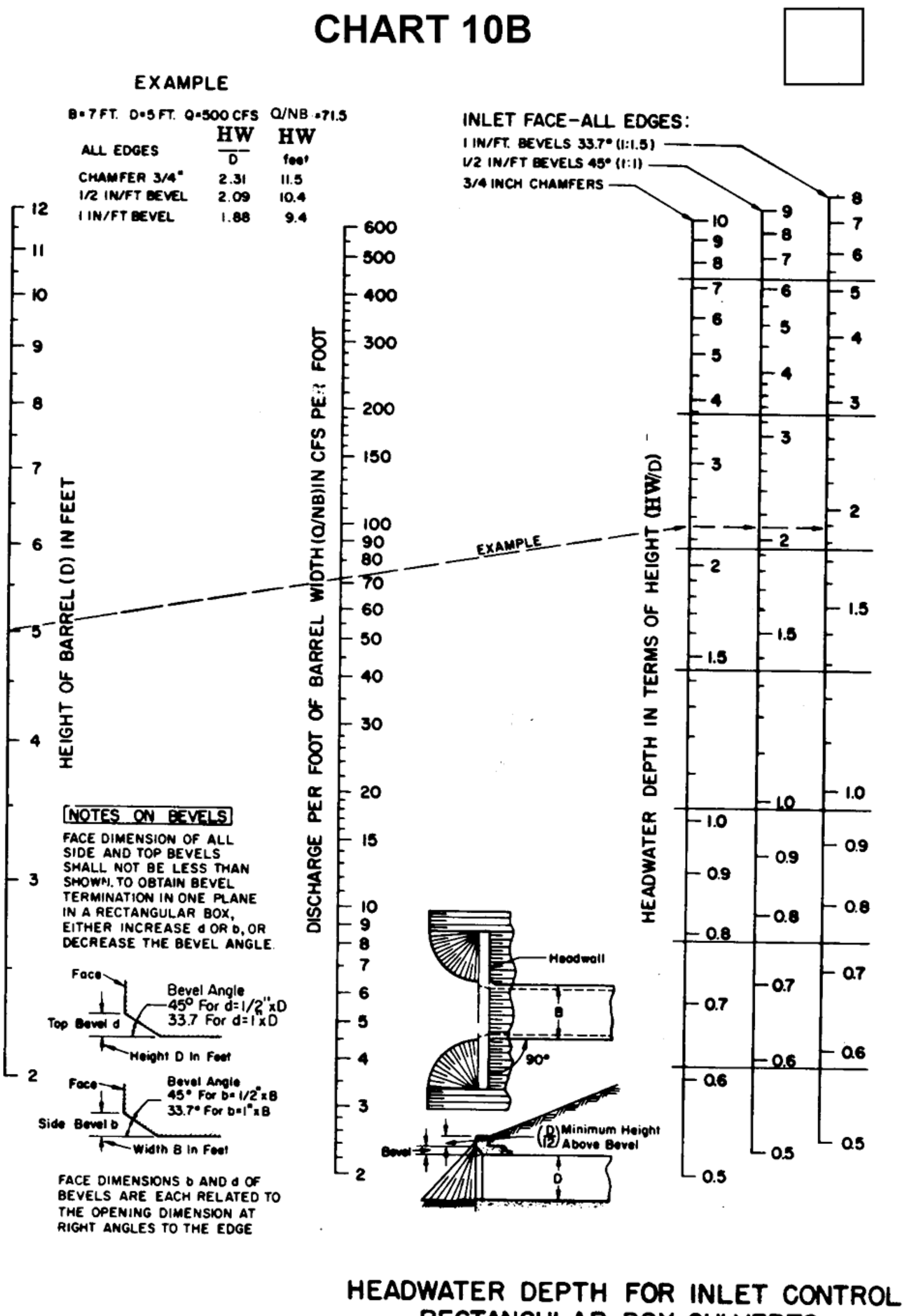


Figure 10.10-L



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

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

Figure 10.10-M

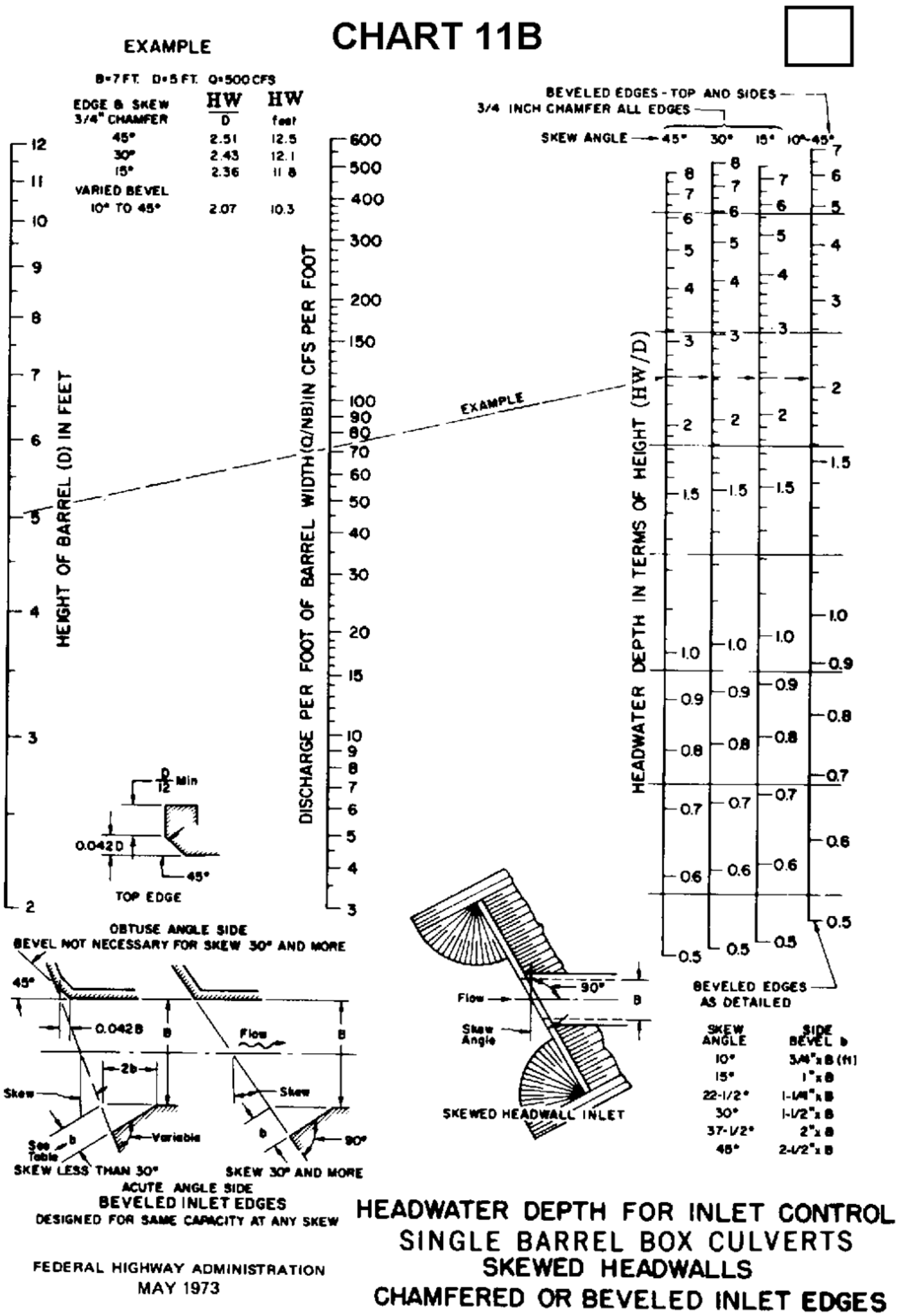


Figure 10.10-N

CHART 12B

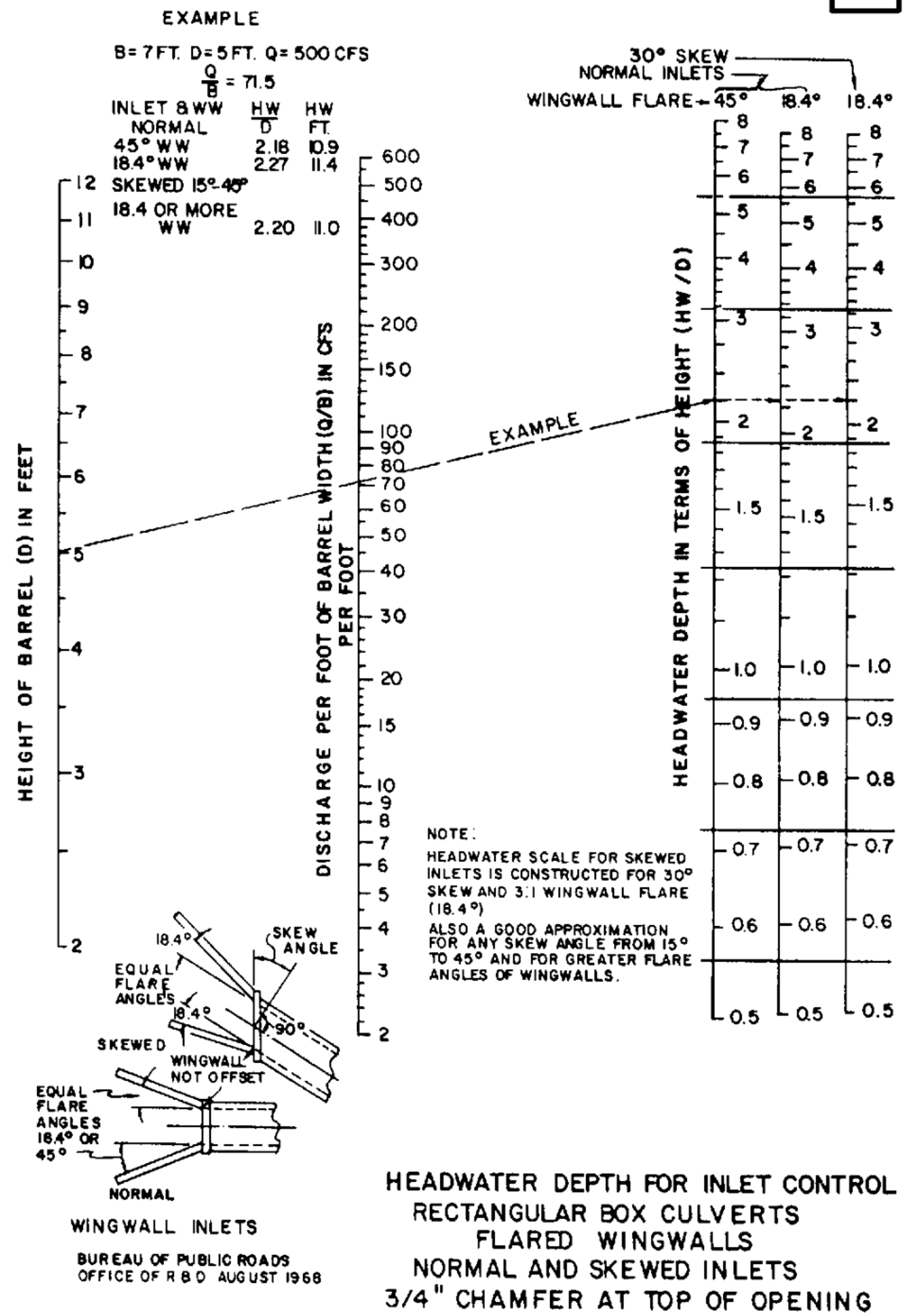


Figure 10.10-O

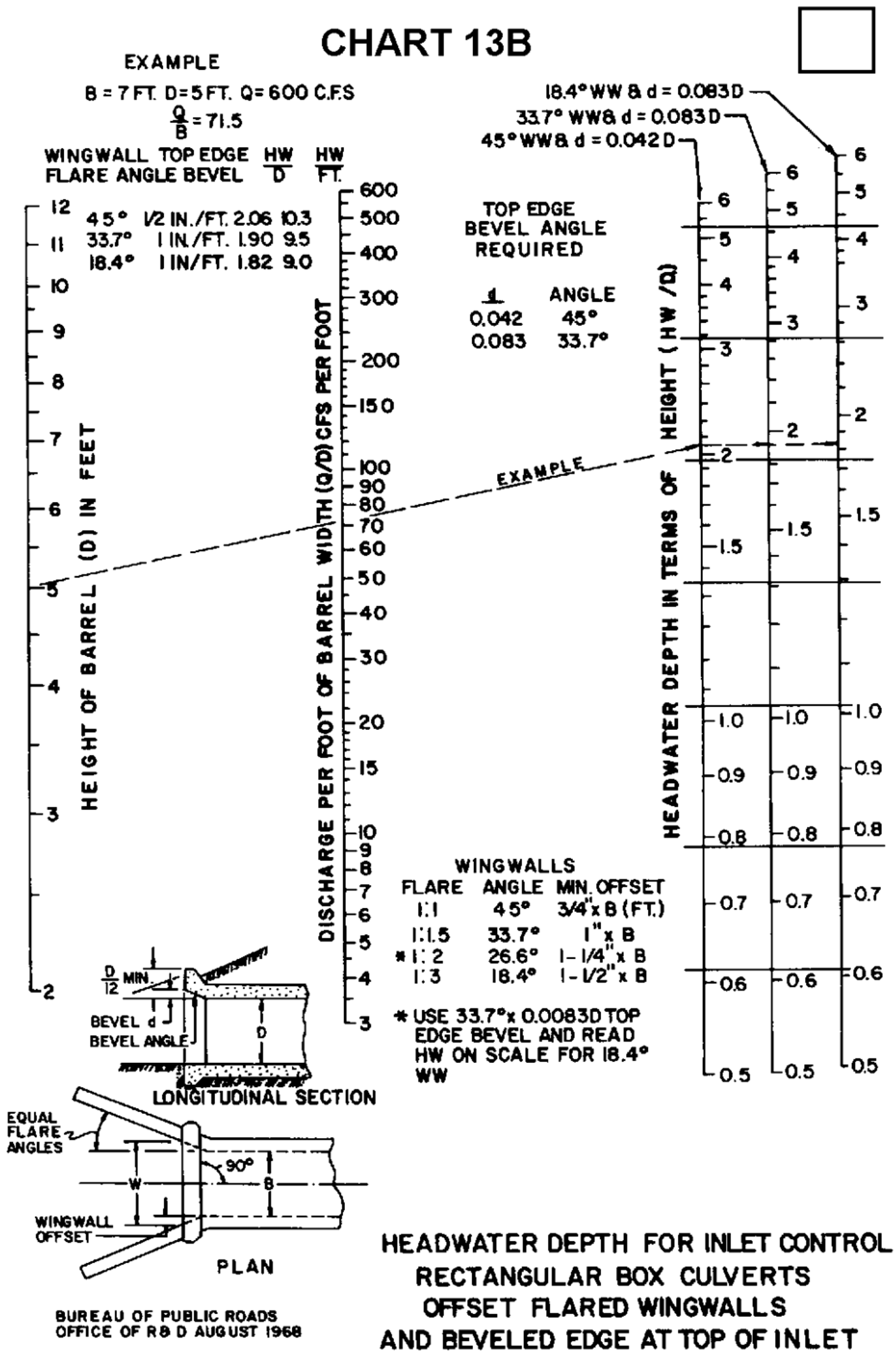


Figure 10.10-P

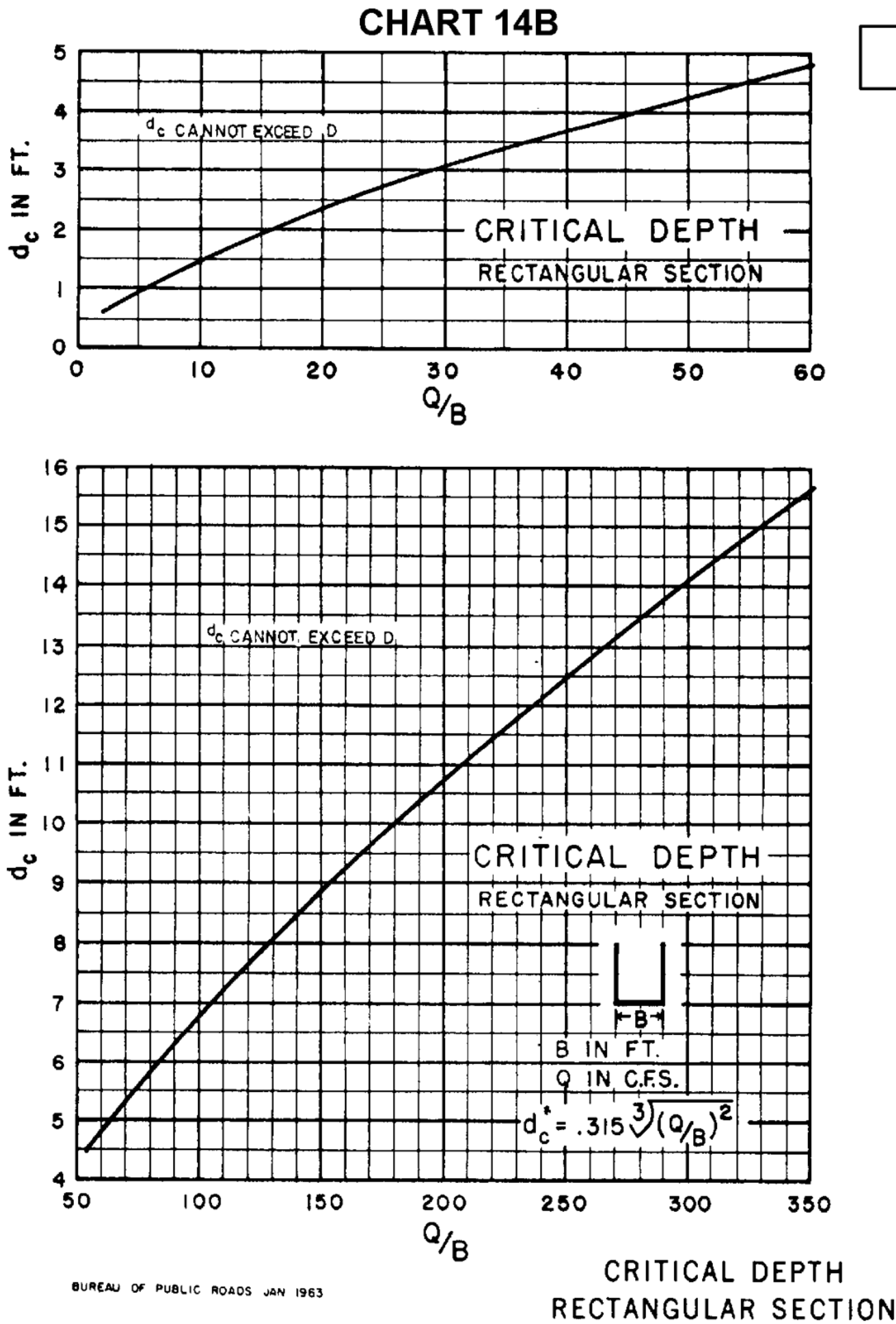


Figure 10.10-Q

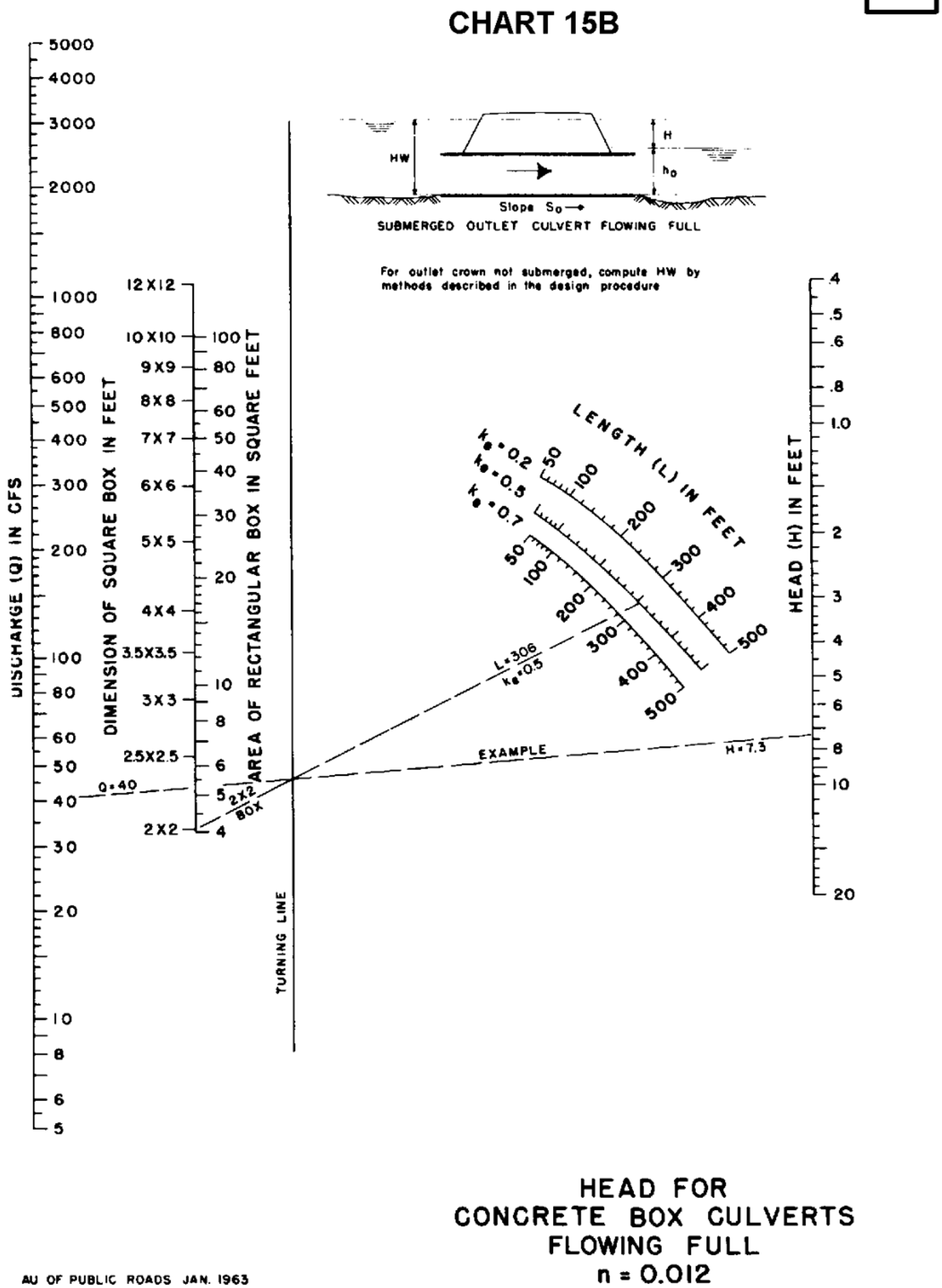
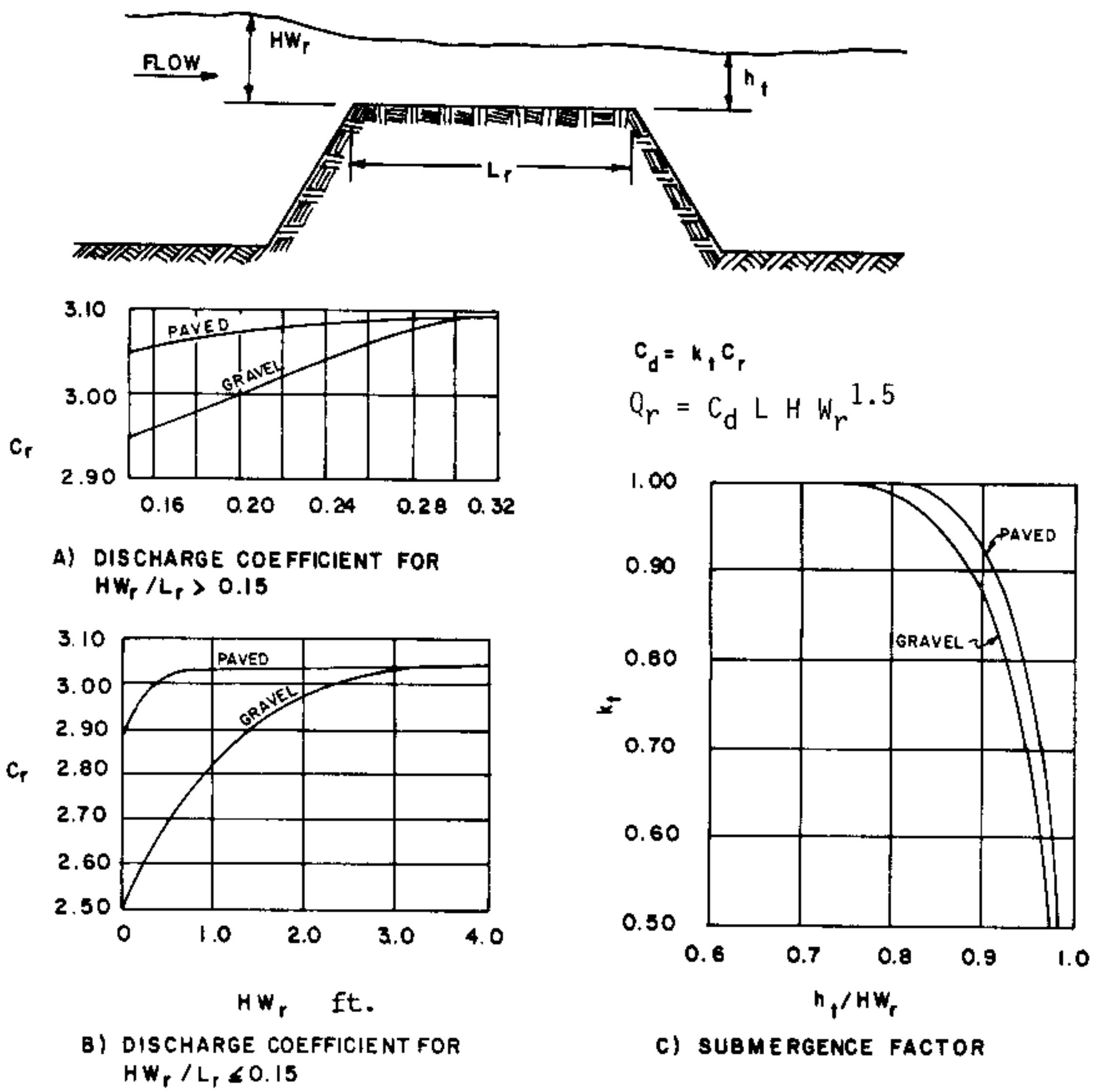


Figure 10.10-R

CHART 60B



English Discharge Coefficients for Roadway Overtopping

Figure 10.10-S

PROJECT: ____________________

____

STATION: ____________________

SHEET ______ OF ______

CULVERT DESIGN FORM

DESIGNER / DATE: ______ OF ______

REVIEWER / DATE: ______ OF ______

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD: ____________________

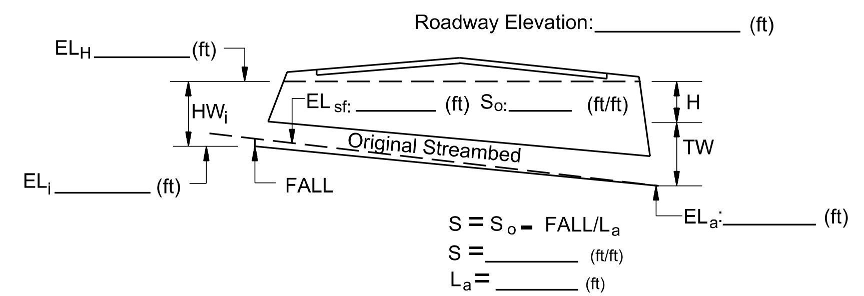
☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______

☐ 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)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL 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) $H = [1 + k_e + (19.63\ n^2 L)/R^{1.33}]\ V^2/2g$

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

SUSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____________________

SHAPE: ____________________

MATERIAL: ______ n ______

ENTRANCE: ____________________

Figure 10.10-T — CULVERT DESIGN FORM

TAPERED INLET DESIGN FORM

PROJECT: ______________________________

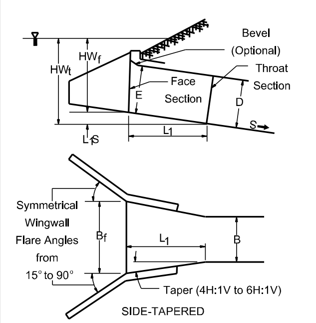
STATION: ______________ SHEET _____ OF _____

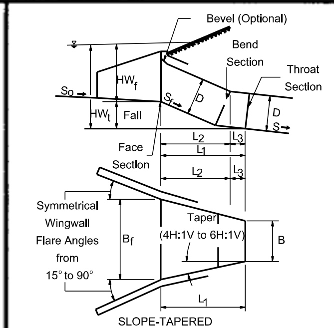
DESIGNER/DATE: __________ / ________

REVIEWER/DATE: __________ / ________

DESIGN DATA:

Q _____ = ______ (cfs) ; EL_{hi} __________ (ft)
EL. THROAT INVERT ____________ (ft)
EL. STREAM BED AT FACE ___________ (ft)
FALL _________ (ft) TAPER _________ : 1 (4H:1V TO 6H:1V)
STREAM SLOPE, S_o = ______________ (ft/ft)
SLOPE OF BARREL, S = ________________ (ft/ft)
S_f ____________ : 1 (2H:1V TO 3H:1V)
BARREL SHAPE AND MATERIAL : __________________
N = ______________ , B = _____________ , D = _____________
INLET EDGE DESCRIPTION ___________________________





COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f/E (3)	Q/B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)

(1) SIDE-TAPERED : EL. FACE INVERT = EL. THROAT INVERT (FIRST ESTIMATE)
SLOPE-TAPERED : EL. FACE INVERT = EL. STREAM BED AT FACE
(2) $HW_f = EL_{hi}$ - EL. FACE INVERT
(3) $1.1\,D \geq E \geq D$
(4) FROM DESIGN CHARTS
(5) MIN. $B_f = Q/(Q/B_f)$
(6) MIN. $L_3 = 0.5$ NB
(7) L_2 = (EL. FACE INVERT - EL. THROAT INVERT) S_f
(8) CHECK $L_2 = ((B_f - NB)/2) * TAPER - L_3$
(9) IF (8) > (7), ADJ. $L_3 = ((B_f - NB)/2) * TAPER - L_2$
(10) IF (7) > (8), ADJ. TAPER = $(L_2 + L_3)/((B_f - NB)/2)$
(11) SIDE-TAPERED : $L = ((B_f - NB)/2) * TAPER$
SLOPE-TAPERED : $L_1 = L_2 + L_3$
(12) $HW_c = EL_{hi}$ - EL. CREST INVERT
(13) MIN. W = $K_u Q/HW_c^{1.5}$
WHERE K_u=0.64 FOR SI AND 0.35 FOR US CUSTOMARY

SELECTED DESIGN

B_f __________
L_1 __________
L_2 __________
L_3 __________
BEVELS ANGLE ____ °
b = ___ in , d = ___ in
TAPER ________ : 1V
S_f = ________ : 1V

Figure 10.10-U — SIDE/SLOPE TAPERED DESIGN FORM

10.11 REFERENCES

(1) FHWA, *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5, FHWA-NHI-01-020, Washington, DC, 2001 (revised 2005).

(2) AASHTO, *Highway Drainage Guidelines, 4th Edition,* Chapter 4 "Hydraulic Design of Highway Culverts," Technical Committee on Hydrology and H ydraulics, American Association of State Highway and Transportation Officials, Washington, DC, 2005.

(3) Bodhaine, G. L., "Measurement of Peak Discharge at Culverts by Indirect Methods," Chapter A3, Book 3 in *Techniques of Water-Resources Investigations of the United States Geological Survey*, United States Government Printing Office, Washington, DC, 1968, fifth printing 1988.

(4) FHWA, *Debris-Control Structures Evaluation and Countermeasures*, Hydraulic Engineering Circular No. 9, FHWA-IF-04-016, Federal Highway Administration, US Department of Transportation, Washington, DC, 2005.

(5) US Bureau of Reclamation, *Design of Small Canal Structures*, Denver, CO, 1960, 1978.

(6) FHWA, *Aquatic Organism Passage Design Guidelines For Roadway Culverts*, Hydraulic Engineering Circular No. 26 (Draft Final), Federal Highway Administration, US Department of Transportation, Washington, DC, 2009.

(7) AASHTO, *Model Drainage Manual,* Technical Committee on Hydrology and Hydraulics, American Association of State Highway and Transportation Officials, Washington, DC, 2005.

(8) FHWA, *Design Charts for Open-Channel Flow*, Hydraulic Design Series No. 3, FHWA-EPD-86-102, 1961.

(9) FHWA, *Hydraulics of Bridge Waterways*, Hydraulic Design Series No. 1, FHWA-EPD-86-101, Federal Highway Administration, US Department of Transportation, Washington, DC, 1978.

(10) King, H. W., Brater, E.F., James Lindell, Wei, C., *Handbook of Hydraulics, 7th Edition*, McGraw-Hill Book Company, 1996.

(11) FHWA, *Stream Stability at Highway Structures*, Hydraulic Engineering Circular No. 20, 3rd Edition, FHWA-NHI-01-002, 2001.

(12) FHWA, *Hydraulic Design of Energy Dissipators for Culverts and Channels,* Hydraulic Engineering Circular No. 14, FHWA-NHI-06-086, Federal Highway Administration, US Department of Transportation, Washington, DC, 2006.

Appendix 10.A

FISH PASSAGE GUIDELINES

The Appendix applies to all culvert projects impacting the Topeka Shiner or other fishery resources.

10.A.1 INTRODUCTION

Environmental regulations, principally the *Endangered Species Act* and *Clean Water Act*, require that culverts be designed in a manner that does not impact fish movement. This Appendix provides guidelines for fish passage design at culvert stream crossings.

10.A.2 FISH PASSAGE REQUIREMENTS

Culverts can cause significant declines in the occupied range of fishes if they act as a barrier to movement. Culverts can become barriers if normal stream processes (e.g., sediment transport) are altered. Fish passage will be addressed by designing culverts to accommodate normal channel forming processes within the culvert. This can be accomplished by setting the floor of the box culvert at or below the adjustment profile line of the stream and by using culverts that are larger than the bankfull stream channel. Culverts should be designed based on typical hydraulic criteria; however, these guidelines should be used to verify that fish passage will not be impacted.

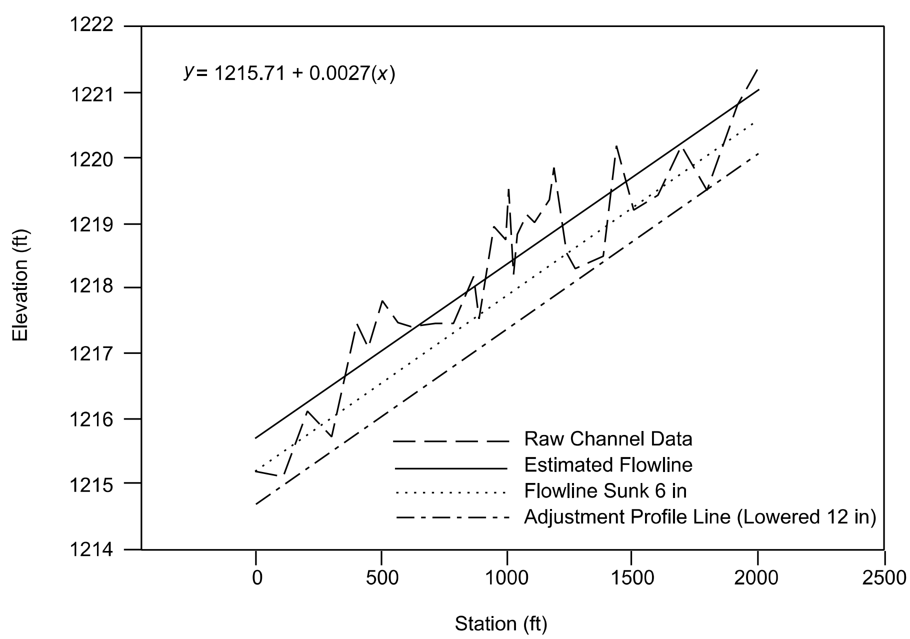
10.A.3 IDENTIFICATION OF FISH PASSAGE CULVERTS

Fish passage culverts will be identified by the applicable SDDOT Office in the initial work request provided to designers or design consultants. Critical review of fish passage designs will be provided to the designer by the SDDOT Environmental Office after submission of the preliminary hydraulic datasheet, survey data and site layout. The designer will develop all data needed for critical review of the fish passage design in the hydraulic design layout provided. It is intended that fish passage issues be resolved prior to the TS&L inspection.

10.A.4 IDENTIFICATION OF THE STREAM FLOW LINE AND ADJUSTMENT POTENTIAL LINE

Culverts are required to be sunk below the stream flow line to allow the development of natural channel features within the culvert and to prevent outlet perching leading to restricted fish movement. The extent that culverts are sunk below the stream flow line will be dependent on channel roughness and slope. To determine the appropriate elevation for culverts, the designer must identify the stream flow line and the adjustment profile line. The stream flow line is defined as the least squares linear regression model of the stream thalweg (i.e., deep point of the channel) survey points. The adjustment profile line is defined as the flowline created by connecting the deepest portion of the pools as shown on the channel longitudinal profile.

The flowline of the culvert should be lowered such that it is at or below the adjustment profile line of the stream channel. In most cases, culverts sunk 1 ft below the stream flow line will meet fish passage requirements. Culverts sunk 6 in below the stream flow line will fail to meet fish passage requirements in many cases. In some cases, culverts may be required to be sunk more than 1 ft below the stream flowline (see Figure 10.A-A).



Note: *Stream longitudinal profile, channel flowline estimated by least squares linear regression and the hypothetical location of the channel flowline if lowered 6 in and 12 in. The linear equation for the estimated channel flowline is given in the upper left corner of the figure. To meet fish passage standards, the culvert will be required to be sunk a minimum of 12 in below the estimated channel flowline.*

Figure 10.A-A

10.A.5 IDENTIFICATION OF BANKFULL WIDTH AND CULVERT WIDTH CRITERIA

Culvert width should to be at least 1.2 times the bankfull channel width unless special circumstances dictate otherwise. The bankfull channel can be generally defined as the Q_2 stream channel or the elevation at which streamflow spills into the floodplain, whichever is less. In most cases, culverts will be sized much greater than the bankfull channel based solely on hydraulic criteria. In some rare cases, culverts may constrict the bankfull channel, especially if the culvert is designed for a very low flood recurrence frequency or the culvert is being placed in a watershed with a very large drainage area (i.e., > 100 sq mi). It is recommended that designers check the relationship between culvert width and bankfull width prior to submitting a preliminary hydraulic design. If it appears that a culvert is slightly less than or close to the minimum culvert width, it is strongly encouraged that the designer contact the SDDOT Environmental Office. At the request of a consultant designer, SDDOT will determine the minimum culvert width.

The bankfull channel width may be estimated with topographical contours generated from project survey data, hydrologic models, surveyed channel cross sections or empirical field measurements of the bankfull channel. The bankfull channel width should be measured away from the roadway/stream interface, and should exclude areas where the channel appears to be unstable or where dams or dugouts predominate the area. In most cases, the existing structure will have had caused some degree of stream bank erosion. Bankfull channel width measurements taken near an existing structure will typically result in an overestimation of the true bankfull channel width (see Figure 10.A-B and Figure 10.A-C).

Note: The red arrows indicate the approximate bankfull width. Note that the stream channel immediately adjacent to the box culvert is much wider than the true bankfull width.

Figure 10.A-B

Note: The red arrows indicate the approximate bankfull width. Note that the stream channel immediately adjacent to the box culvert is much wider than the true bankfull width.

Figure 10.A-C

In some special cases, an exemption to the minimum culvert width will be allowed. Exemptions will only be allowed if strong evidence is available to suggest that fish passage will not be adversely impacted due to the width of the culvert.

Appendix 10.B

PIPE CULVERT REHABILITATION

10.B.1 INTRODUCTION

Existing pipes that are damaged or deteriorated to the point that their functionality has been significantly impacted should be considered for rehabilitation. SDDOT typical pipe culvert rehabilitation methods are provided below in Figure 10.B-A.

10.B.2 PIPE CULVERT INSPECTION

Pipe inspection should be performed to determine the condition of the existing pipe and the need for rehabilitation. The pipe inspection information will be used to determine what method of pipe rehabilitation should be used. The interior of the pipe will need to be inspected. Personnel should enter pipe that are safe and large enough to crawl through. Small or unsafe pipe interiors can be inspected using a powerful spot light from both ends or a remote video camera.

Pipe inspections should review all aspects of the pipe, roadway and upstream and downstream channels for indicators of pipe deficiencies. Some of the many pipe features to inspect include: extent of corrosion, width of cracks in pipe, depth of concrete spalling, exposed reinforcement, pipe deformation, pipe misalignment, pipe joint condition and seperation, end section separation, soil infiltration, material flowing through pipe, pipe and channel sedimentation, pipe plugging, upsteam ponding, channel erosion, road settlement, pavement cracking, holes in inslope, and embankment voids around pipe.

10.B.2 PIPE CULVERT REHABILITATION METHODS

Deficiencies encountered on most SDDOT pipe culverts include corrosion of metal pipes and joint failure and end section separation of concrete pipe. The following pipe rehabilitation methods are provided to repair these deficiencies and others.

Figure 10.B-A — SDDOT TYPICAL CULVERT REHABILITATION METHODS

Pipe Rehabilitation Method	Existing Pipe Type	Trenchless
Replacement - Open Trench	All Types	No
Replacement - Bore & Jack	All Types	Yes
Sliplining	All Types	Yes
Internal Joint Sealing	Concrete	Yes
Invert Paving with Concrete	Metal	Yes
Reset Ends	All Types	Yes

The above replacement methods involve installation of new pipe while the other methods rehabilitate existing pipe in place.

The need for a trenchless repair is a major consideration for pipe rehabilitations. The major site conditions favoring trenchless pipe rehabilitations over open trench replacement are high fill, high traffic volumes, and/or long detour routes. Some of the issues to consider in determining what pipe rehabilitation method to use are as follows.

- Existing pipe deficiencies: Use a method that will repair the pipe deficiencies.

- Cost: Compare costs between the acceptable methods for correcting the pipe deficiencies. This is often costs between a trenchless method and open trench replacement. Generally costs for trenchless methods include pipe cleaning, pipe, pipe installation and grout or joint seals, whereas open trench replacement includes traffic control, pavement removal, excavation, pipe removal, pipe, pipe installation, backfill, pavement installation and erosion control.

- Traffic impacts: Consider the volume of traffic, availability and lengths of detours and delays that may be caused.

- Height of Fill: Trenchless methods should have more consideration at high fill locations.

- Hydraulic capacity: A hydraulic analysis of the proposed pipe rehabilitation should be done by a Hydraulics Engineer to ensure the new upstream water surface elevation meets allowable headwater design criteria. This may require obtaining survey as requested. Special attention should be made where there are upstream buildings. Generally a hydraulic analysis is not required for joint repairs or resetting ends.

- Outlet velocity: The hydraulic analysis by a Hydraulics Engineer shall determine the new outlet velocity and if additional outlet protection is required. Sliplining pipe will decrease Manning's n value and decrease flow area, which will typically increase outlet velocities.

- Structural Strength: The pipe rehabilitation may need to provide pipe structural strength where the existing pipe has severe deterioration. A Structural Engineer (Office of Bridge Design) should determine if the proposed repair will provide sufficient strength.

- Soil Corrosivity: Obtain recommendations from SDDOT Material & Surfacing Office for pipe materials that will be acceptable for the soil corrosivity at each location.

- Bends in pipe: Sliplining should not be used where the existing pipe has an unaccessable bend. Small angle bends can be sliplined. Boring and jacking pipe is installed in straight lines and would not be used where a bend is required.

- Railroad: Boring and jacking pipe is typically used when new pipe for highway purposes is required under a railroad.

10.B.2.1 Bore and Jack Pipe

Boring and jacking pipe typically involves excavating a circular bore hole with a rotating auger type cutting head just ahead of pipe being pushed or jacked through the hole. Hydraulic jacks on a rail system push the auger and pipe through the hole. A drive pit is created for the jacks and rail to be placed in and to provide resistive force for the jacks. The pipe is concrete designed by the Supplier to resist jacking forces and is a minimum Class 5 concrete. The bore hole is just slightly larger than the pipe and the remaining void outside the pipe is filled with grout. Spoils from the excavation are augered back through the pipe to the drive pit. Boring and jacking pipe usually requires the existing pipe to be plugged with controlled density fill to prevent embankment settlement from future collapse of the existing pipe.

10.B.2.2 Sliplining Pipe

Sliplining pipe involves pushing or pulling a new pipe into the existing pipe as a liner and grouting the void between the liner and existing pipe. The liner pipe shall be high density polyethylene (HDPE) or PVC pipe with a smooth interior surface. The largest diameter liner pipe that will fit into the existing pipe should be used to maximize flow capacity. The liner pipe should be joined into a continuous length with joints that are adequate for pushing or pulling the liner pipe though the existing pipe. The joints should not create an increase in the outside diameter of the liner pipe to allow more smooth insertion of the liner. Liner joints are typically threaded, snap together, gasket or heat fusion welded. Pressure grouting shall be done to ensure all the voids are filled between the liner pipe and existing pipe including all breaks or holes in and around the existing pipe. Prior to sliplining, the existing pipe shall be cleaned of obstructions and solids that will prevent the insertion of the liner.

The diameter of liner to use for the existing pipe size is located in Figure 10.B-B. Larger size liners are available. If the type of liner pipe is not known, use closed profile HDPE for design. Flow capacity of the liner can be computed using a Manning's n value of 0.009.

Sliplining pipe material types and requirements:
1. Closed profile HDPE – Meets ASTM F894 and ASTM D3350 with cell classification 345464C. Minimum pipe stiffness shall be 46 psi in accordance with ASTM D2412. "Culvert Renew" pipe product or similar meets this.
2. Solid wall HDPE – Meets ASTM F714 (SDR 32.5) and ASTM D3350 with cell classification 345464C. "Snap-Tite" pipe product or similar meets this.
3. PVC – Meets ASTM F949 with minimum pipe stiffness of 46 psi. Contech's A2 Liner Pipe product or similar meets this. These types of pipe are available in 12" to 36" diameters.
4. Spirally Wound PVC – Meets ASTM F949 with minimum pipe stiffness of 46 psi. Rib Loc's Ribline product or similar meets this. This is generally a higher cost product usually best for

small access locations, like in a manhole. It requires special equipment by the supplier to construct and install the liner on site.

Figure 10.B-B — SLIPLINING PIPE DIMENSIONS

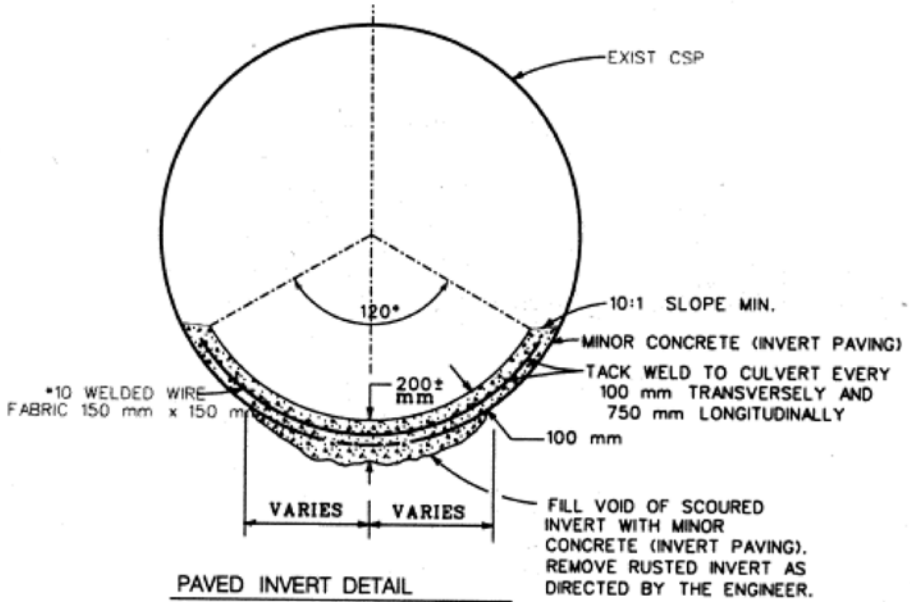
Existing Pipe I.D. (in.)	Closed Profile HDPE O.D. (in.)	Closed Profile HDPE I.D. (in.)	Solid Wall HDPE O.D. (in.)	Solid Wall HDPE I.D. (in.)	PVC O.D. (in.)	PVC I.D. (in.)	Spirally Wound PVC O.D. (in.)	Spirally Wound PVC I.D. (in.)
24	20.24	18.00	22.00	20.65	22.60	20.70	20.45	20.00
30	27.06	24.00	28.00	26.29	25.60	23.50	27.45	27.00
36	33.82	30.00	32.00	30.03	32.20	29.50	32.79	32.00
42	40.65	36.00	40.00	36.95	38.70	35.50	38.79	38.00
48	45.20	40.00	42.00	39.42			42.79	42.00
54	47.47	42.00	48.00	44.33			48.79	48.00
60			54.00	50.68			54.79	54.00

10.B.2.3 Internal Joint Sealing

Internal pipe joint sealing systems may be used where the pipe is large enough for human entry and the external hydraulic pressure is low. One method for sealing joints internally includes a flexible rubber sleeve held in place over the joint with stainless steel expansion bands. Prior to installing the seal, the joint and area receiving the seal should be cleaned and filled with mortar to provide a smooth non-porous surface. This rubber sleeve method requires a round pipe and minor settlement between pipe sections.

10.B.2.43 Invert Paving with Concrete

This method can be used to rehabilitate corroded and severely deteriorated inverts of CMP by paving with reinforced concrete. If abrasion is a significant factor the concrete should have the lowest practicable water/cement ratios and the aggregate source should be harder material than the streambed load with a high durability index. Consideration should be given for using a higher strength concrete of 6000 psi or higher. Where there is significant loss of pipe invert, it is necessary to tie the concrete to the more structurally sound portions of the pipe wall to transfer pipe walls compressive thrust into the invert slab. Angle iron or shear connector studs can be welded to the pipe wall to provide this bond between concrete and pipe wall. Paving thickness ranges from 3 to 6 inches with welded wire fabric reinforcement. Paving limits typically range from 90 to 120 degrees for the internal angle. A typical detail for invert paving for situations with minimal loss of invert is shown below. Design for this method should be provided by the Office of Bridge Design.



Additional References for Pipe Culvert Rehabilition

There is a c onsiderable amount of references available with information on the above mentioned pipe culvert rehabilitation methods and others. The following are some standard references.

Caltrans Supplement to FHWA Culvert Repair Practices Manual:
http://www.dot.ca.gov/hq/oppd/dib/dib83-01.htm

FHWA – Culvert Repair Practices Manual Volume 1:
http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=36&id=94

FHWA – Culvert Repair Practices Manual Volume 2:
www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=37&id=90

FHWA Central Federal Lands Highway Division – Culvert Pipe Liner Guide and Specifications:
http://www.cflhd.gov/programs/techDevelopment/hydraulics/culvert-pipe-liner/