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CHAPTER

4

HYDRAULIC DESIGN OF OPEN CHANNELS, CULVERTS, BRIDGES, AND DETENTION STRUCTURES

Section 4.1 Open Channels, Bridges, and Detention Structure Design Overview

4.1.1 Storm Water System Design

4.1.1.1 Introduction

Storm water system design is an integral component of both site and overall storm water management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on storm water runoff.

Storm water collection systems must be designed to provide adequate surface drainage while at the same time meeting other storm water management goals such as water quality, streambank protection, habitat protection, and groundwater recharge.

4.1.1.2 System Components

The storm water system components consist of all the *integrated* site design practices and storm water controls utilized on the site. Three considerations largely shape the design of the storm water systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove storm water from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, storm water ponds and wetlands, and small underground pipe systems which collect storm water runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger storm water systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The storm water (major) system consists of natural waterways, open channels, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The storm water system includes not only the trunk line system that receives the water, but also the natural overland relief which functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This chapter is intended to provide design criteria and guidance on several on-site flood control system components, including culverts (Section 4.2), bridges (Section 4.3), vegetated and lined open channels (Section 4.4), storage design (Section 4.5), outlet structures (Section 4.6), and energy dissipation devices for outlet protection (Section 4.7). The rest of this section covers important considerations to keep in mind in the planning and design of storm water drainage facilities.

4.1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).

C. Check potential drainage outlets and methods, including:
   1. On-site (structural control, receiving water)
   2. Off-site (highway, storm drain, receiving water, regional control)
   3. Natural drainage system (swales)
   4. Existing drainage system (drain pipe)

D. Analyze areas for probable location of drainage structures and facilities.

E. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall storm water management system and plan.

4.1.2 Key Issues in Storm Water System Design

4.1.2.1 Introduction

The traditional design of storm water systems has been to collect and convey storm water runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Storm water systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural storm water controls to mitigate the major storm water impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in storm water system design.

4.1.2.2 General Design Considerations

- Storm water systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or storm water systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or storm water (minor) systems.
- It is important to ensure that the combined on-site flood control system and major storm water system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor storm water systems and/or major storm water structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of storm water systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major storm water system design capacity.

4.1.2.3 Culverts

- Culverts can serve double duty as flow retarding structures in grass channel design. Care should be taken to design them as storage control structures if depths exceed several feet, and to ensure safety during flows.
- Improved entrance designs can absorb considerable slope and energy for steeper sloped designs, thus helping to protect channels.

4.1.2.4 Bridges

- Bridges enable streams to maintain flow conveyance.
- Bridges are usually designed so that they are not submerged.
- Bridges may be vulnerable to failure from flood-related causes.
- Flow velocities through bridge openings should not cause scour within the bridge opening or in the stream reaches adjacent to the bridge.

4.1.2.5 Open Channels

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.
- Channels can be designed with natural meanders improving both aesthetics and pollution removal through increased contact time.
- Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress and super elevation.

- Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 4.1-1 illustrates a compound section that carries the 2-year and 100-year flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 1:12 slope to ensure drainage.

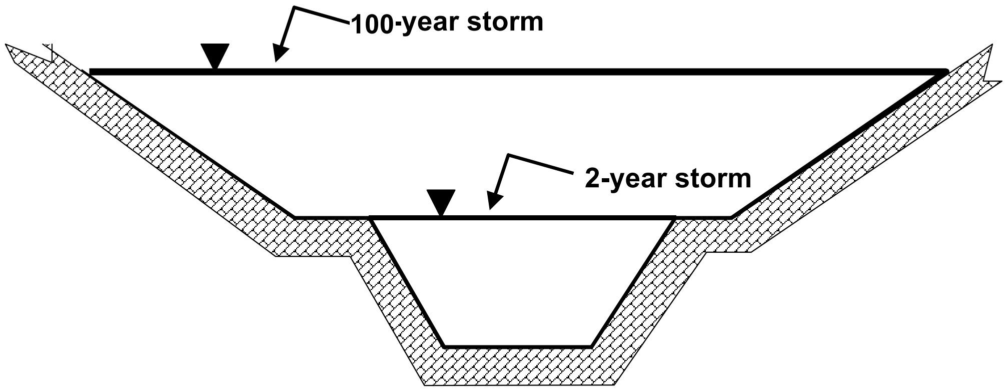

Figure 4.1-1 Compound Channel

- Flow control structures can be placed in the channels to increase residence time. Higher flows should be calculated using a channel slope from the top of the cross piece to the next one if it is significantly different from the channel bottom for normal depth calculations. Channel slope stability can also be ensured through the use of grade control structures that can serve as pollution reduction enhancements if they are set above the channel bottom. Regular maintenance is necessary to remove sediment and keep the channels from aggrading and losing capacity for larger flows.

4.1.2.6 Storage Design

- Storm water storage within a storm water system is essential to providing the extended detention of flows for water quality treatment and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection.
- Runoff storage can be provided within an on-site flood control system through the use of structural storm water controls and/or nonstructural features.
- Storm water storage can be provided by detention, extended detention, or retention.
- Storage facilities may be provided on-site, or as regional facilities designed to manage storm water runoff from multiple projects.

4.1.2.7 Outlet Structures

- Outlet structures provide the critical function of the regulation of flow for structural storm water controls.
- Outlet structures may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.
- Smaller, more protected outlet structures should be used for water quality and streambank protection flows.
- Large flows, such as flood flows, are typically handled through a broad crested weir, a riser with different sized openings, a drop inlet structure, or a spillway through an embankment.

4.1.2.8 Energy Dissipators

- Energy dissipators should be designed to return flows to non-eroding velocities to protect downstream channels.
- Care must be taken during construction that design criteria are followed exactly. The designs presented in this Manual have been carefully developed through model and full-scale tests. Each part of the criteria is important to their proper function.

4.1.3 Design Storm Recommendations

Listed below are the design storm recommendations for various storm water drainage system components to be designed and constructed in accordance with the minimum storm water management standards. Some jurisdictions may require that the design of storm water conveyance systems be sized for other storm frequencies. Please consult your local review authority to determine the local requirements. It is recommended that the full build-out conditions be used to calculate flows for the design storm frequencies below.

Storm Drainage Systems

Including storm drainage systems and pipes that do not convey runoff under public roadways, sometimes called lateral closed systems.

- 10- to 25-year design storm (for pipe and culvert design)
- 10- to 25-year design storm (for inlet design)
- 100-year design storm (for sumped inlets, unless overflow facilities are provided)

Roadway Culvert Design

Cross drainage facilities that transport runoff under roadways.

- 25- to 100-year design storm, or in accordance with TxDOT requirements, whichever is more stringent. (Criteria to be taken into consideration when selecting design flow include: roadway type, tailwater or depth of flow over road, structures, and property subject to flooding, emergency access, and road replacement costs.)

Bridge Design

Cross drainage facilities with a span of 20 feet or larger.

- 100-year design storm

Open Channel Design

Open channels include all channels, swales, etc.

- 100-year design storm

Channels may be designed with multiple stages (e.g., a low flow channel section containing the 2-year to 5-year flows, and a high flow section that contains the design discharge) to improve stability and better mimic natural channel dimensions. Where flow easements can be obtained and structures kept clear, overbank areas may also be designed as part of a conveyance system wherein floodplain areas are designed for storage and/or conveyance of larger storms.

Energy Dissipation Design

Includes all outlet protection facilities. All drainage system outlets, whether for closed conduits, culverts, bridges, open channels, or storage facilities, should provide energy dissipation to protect the receiving conveyance element from erosion.

- 100-year design storm (major systems)

Check Storm

Used to estimate the runoff that is routed through the drainage system and storm water management facilities to determine the effects on the facilities, adjacent property, floodplain encroachment, and downstream areas.

- 100-year design storm

References

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

Prince George's County, MD, 1999. Low-Impact Development Design Strategies, An Integrated Design Approach.

The Dewberry Companies, 2002, 2nd Edition, Land Development Handbook, McGraw-Hill Companies, Inc., New York, NY.

American Society of Civil Engineers, 1993, Design and Construction of Urban Stormwater Management Systems, Manual and Report No. 77.

Larry W. Mays, Editor, 2001, Stormwater Collection Systems Design Handbook, McGraw-Hill Companies, Inc., New York, NY.

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Section 4.2
Culvert Design

4.2.1 Overview

A *culvert* is a short, closed (covered) conduit that conveys storm water runoff under an embankment or away from the street right-of-way. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows.

The hydraulic and structural designs of a culvert must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

4.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 4.2-1 will be used. These symbols were selected because of their wide use.

Table 4.2-1 Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel depth	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
H_f	Depth of pool or head, above the face section of invert	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	-
L	Length of culvert	ft
N	Number of barrels	-
Q	Rate of discharge	cfs
S	Slope of culvert	ft/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

4.2.3 Design Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable.

4.2.3.1 Frequency Flood

See Section 4.1 or the local review authority for design storm requirements for the sizing of culverts.

The 100-year frequency storm shall be routed through all culverts to be sure building structures (e.g., houses, commercial buildings) are not flooded or increased damage does not occur to the highway or adjacent property for this design event.

4.2.3.2 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. The maximum allowable velocity for corrugated metal pipe is 15 feet per second. There is no specified maximum allowable velocity for reinforced concrete pipe, but outlet protection shall be provided where discharge velocities will cause erosion problems. To ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 feet per second, for the 1-year flow, when the culvert is flowing partially full is required

4.2.3.3 Buoyancy Protection

Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.

4.2.3.4 Length and Slope

The culvert length and slope should be chosen to approximate existing topography and, to the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment. The maximum slope using concrete pipe is 10% and for CMP is 14% before pipe-restraining methods must be taken. Maximum vertical distance from throat of intake to flowline in a drainage structure is 10 feet. Drops greater than 4 feet will require additional structural design.

4.2.3.5 Debris Control

In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled *Debris Control Structures* be consulted.

4.2.3.6 Headwater Limitations

Headwater is water above the culvert invert at the entrance end of the culvert. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway and is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The following criteria related to headwater should be considered:

- The *allowable headwater* is the depth of water that can be ponded at the upstream end of the culvert during the design flood, which will be limited by one or more of the following constraints or conditions:
    1. Headwater be non-damaging to upstream property
    2. Ponding depth be no greater than the low point in the road grade unless overflow has been allowed by the roadway design or at the applicable design criteria, such as the 5-year or 10-year flood level.

3. Ponding depth be no greater than the elevation where flow diverts around the culvert
4. Elevations established to delineate floodplain zoning
5. 18-inch (or applicable) freeboard requirements

- The headwater should be checked for the 100-year flood to ensure compliance with flood plain management criteria and for most facilities the culvert should be sized to maintain flood-free conditions on major thoroughfares with 18-inch freeboard at the low-point of the road.
- The maximum acceptable outlet velocity should be identified (see subsection 4.4.3).
- Either the headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided where these velocities are exceeded.
- In general, the constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.
- Other site-specific design considerations should be addressed as required.

4.2.3.7 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions. The following conditions must be considered:

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.
- For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined. See Section 4.4, *Open Channel Design*.
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body may establish the culvert tailwater.

4.2.3.8 Storage

If storage is being assumed or will occur upstream of the culvert, refer to subsection 4.2.4.6 regarding storage routing as part of the culvert design.

4.2.3.9 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 4.2-2.

4.2.3.10 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all metal culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall.

This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

4.2.3.11 Wingwalls and Aprons

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

4.2.3.12 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.

4.2.3.13 Material Selection

Reinforced concrete pipe (RCP), pre-cast and cast in place concrete boxes are recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP and fully coated corrugated metal pipe can be used in all other cases. High-density polyethylene (HDPE) pipe may also be used as specified in the municipal regulations. Table 4.2-3 gives recommended Manning's n values for different materials.

4.2.3.14 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

4.2.3.15 Culvert Sizes

The minimum allowable pipe diameter shall be 18 inches.

4.2.3.16 Weep Holes

Weep holes are sometimes used to relieve uplift pressure on headwalls and concrete rip-rap. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels through the fill embankment. The filter materials should be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

Table 4.2-2 Inlet Coefficients

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

1 Although laboratory tests have not been completed on K_e values for High-Density Polyethylene (HDPE) pipes, the K_e values for corrugated metal pipes are recommended for HDPE pipes.

* Note: "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.

Source: HDS No. 5, 2001

Table 4.2-3 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corrugations	2 2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 2001

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

4.2.3.17 Outlet Protection

See Section 4.5 for information on the design of outlet protection.

4.2.3.18 Erosion and Sediment Control

Erosion and sediment control shall be in accordance with the latest approved Soil Erosion and Sediment Control Ordinance for the municipality.

4.2.3.19 Environmental Considerations

Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.

4.2.3.20 Safety Considerations

Roadside safety should be considered for culverts crossing under roadways. Guardrails or safety end treatments may be needed to enhance safety at culvert crossings. The AASHTO roadside design guide should be consulted for culvert designs under and adjacent to roadways.

4.2.4 Design Procedures

4.2.4.1 Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth:

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

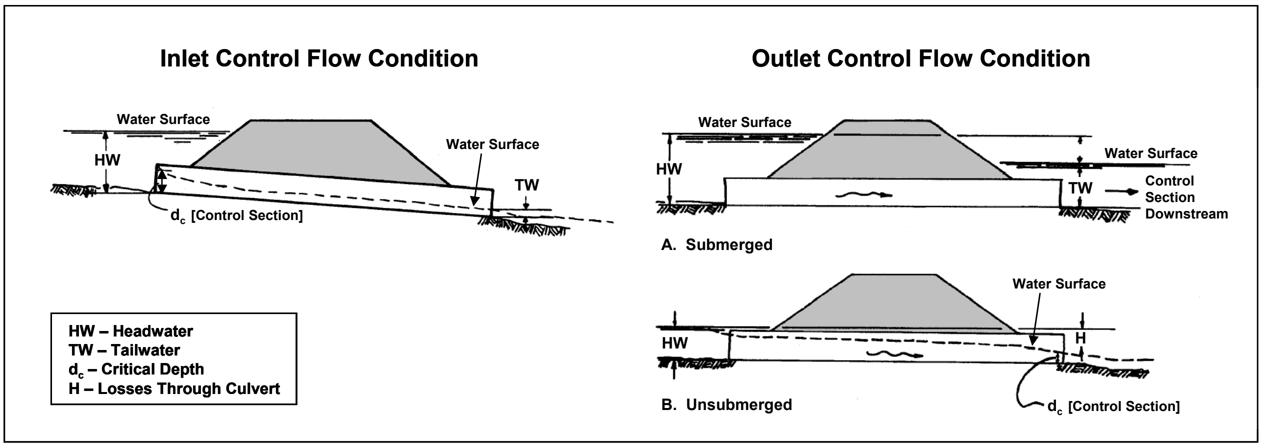


Figure 4.2-1 Culvert Flow Conditions
(Adapted from: HDS-5, 2001)

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001.

4.2.4.2 Procedures

The culvert design process includes the following basic stages:

1. Define the location, orientation, shape, and material for the culvert to be designed. In many instances, consider more than single shape and material.
2. With consideration of the site data, establish allowable outlet velocity and maximum allowable depth of barrel.
3. Based on upon subject discharges, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (culvert hydraulic length, entrance conditions, and conduit shape and material).
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.
6. Treat any excessive outlet velocity separately from headwater.

There are three procedures for designing culverts: inlet control design equations, manual use of inlet and outlet control nomographs, and the use computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

4.2.4.3 Inlet Control Design Equations

This section contains explanations of the equations and methods used to develop the design charts in HDS No. 5, where those equations and methods are not fully described in the main text. The following topics are discussed: the design equations for the unsubmerged and submerged inlet control nomographs, the dimensionless design curves for culvert shapes and sizes without nomographs, and the dimensionless critical depth charts for long span culverts and corrugated metal box culverts.

Inlet Control Nomograph Equations: The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. Of these, the first and fourth through seventh reports dealt with the hydraulics of pipe and box culvert entrances, with and without tapered inlets (4, 7, to 10). These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs (56 and 57).

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

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

Table 4.2-4 contains the unsubmerged and submerged inlet control design equations. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with tow correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs.

The constants and the corresponding equation form are given in Table 4.2-5. Table 4.2-5 is arranged in the same order as the design nomographs in section 4.2.4.4, and provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

The equations may be used to develop design curves for any conduit shape or size. Careful examination of the equation constants for a given form of equation reveals that there is very little difference between the constants for a given inlet configuration. Therefore, given the necessary conduit geometry for a new shape from the manufacturer, a similar shape is chosen from Table 4.2-5, and the constants are used to develop new design curves. The curves may be quasi-dimensionless, in terms of $Q/AD^{0.5}$ and HW_i/D, or dimensional, in terms of Q and HW_i for a particular conduit size. To make the curves truly dimensionless, $Q/AD^{0.5}$ must be divided by $g^{0.5}$, but this results in small decimal numbers. Note that coefficients for rectangular (Box) shapes should not be used for nonrectangular (circular, arch, pipe-arch, etc.) shapes and vice-versa. A constant slope value of 2 percent (0.02) is usually selected for the development of design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent (except for mitered inlets).

Table 4.2-4 Inlet Control Design Equations

Unsubmerged*		
Form (1)	$\frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{K_u Q}{AD^{0.5}}\right)^M - 0.5S$ ***	**(4.2.1)**
Form (2)	$\frac{HW_i}{D} = K\left(\frac{K_u Q}{AD^{0.5}}\right)^M$	**(4.2.2)**
Submerged**		
	$\frac{HW_i}{D} = c\left(\frac{K_u Q}{AD^{0.5}}\right)^2 + Y - 0.5S$ ***	**(4.2.3)**
Definitions		
HW_i	Headwater depth above inlet control section invert, m (ft)	
D	Interior height of culvert barrel, m (ft)	
H_c	Specific head at critical depth ($d_c + V_c^2/2g$), m^2 (ft^2)	
Q	Discharge, m^3/s (ft^3/s)	
A	Full cross sectional area of culvert barrel, m^2 (ft^2)	
S	Culvert barrel slope, m/m (ft/ft)	
K, M, c, Y	Constants from Table 4.2-5	
K_u	1.811 SI (1.0 English)	
* Equations 4.2.1 and 4.2.2 (unsubmerged) apply to about $Q/AD^{0.5}$ = 1.93 (3.5 English)		
** Equation 4.2.3 (submerged) above applies to about $Q/AD^{0.5}$ = 2.21 (4.0 English)		
*** For mitered inlets use +0.7 S instead of -0.5 S as the slope correction factor.		

Table 4.2-5 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
1	Circular Concrete	1	Square edge w/ headwall	1	.0098	2.0	.0398	.67	56/57
		2	Groove end w/ headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69	56/57
		2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7° bevels		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30° to 75° wingwall flares	1	.026	1.0	.0347	.81	56
		2	90° and 15° wingwall flares		.061	.75	.0400	.80	56
		3	0° wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45° wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18° to 33.7° wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90° headwall w/ 3/4” chamfers	2	.515	.667	.0375	.79	8
		2	90° headwall w/ 45° bevels		.495	.667	.0314	.82	
		3	90° headwall w/ 33.7° bevels		.486	.667	.0252	.865	

Table 4.2-5 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References*
11	Rectangular Box	1	3/4" chamfers; 45° skewed headwall	2	.545	.667	.04505	.73	8
		2	3/4" chamfers; 30° skewed headwall		.533	.667	.0425	.705	
		3	3/4" chamfers; 15° skewed headwall		.522	.667	.0402	.68	
		4	45° bevels; 10°-45° skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4" chamfers	1	45° non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4° non-offset wingwall flares		.493	.667	.0361	.806	
		3	18.4° non-offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71	
13	Rectangular Box Top Bevels	1	45° wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7° wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4° wingwall flares - offset		.493	.667	.0227	.887	
16-19	CM Boxes	2	90° headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	
29	Horizontal Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
30	Vertical Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch 18" Corner Radius CM	1	90° headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch 18" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
		2	No Bevels		.0088	2.0	.0368	.68	
		3	33.7° Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch 31" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
			No Bevels		.0088	2.0	.0368	.68	
			33.7° Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Thin wall projecting		.0340	1.5	.0496	.57	
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.90	3
		2	Rough tapered inlet throat		.519	.64	.0210	.90	
56	Elliptical Inlet Face	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
		2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75	
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular Concrete	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
		2	Side tapered-more favorable edges		.56	.667	.0378	.87	
59	Rectangular Concrete	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
			Slope tapered-more favorable edges		.50	.667	.0378	.71	

* These references are cited in FHWA, 2001, HYD-5. They can be accessed at the Federal Highway Administration web site: www.fhwa.dot.gov/bridge/hydpub.htm.

4.2.4.4 Nomographs

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figures 4.2-2(a) and (b) show examples of an inlet control and outlet control nomographs for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001, Second Edition.

This section presents design guidance for culverts originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

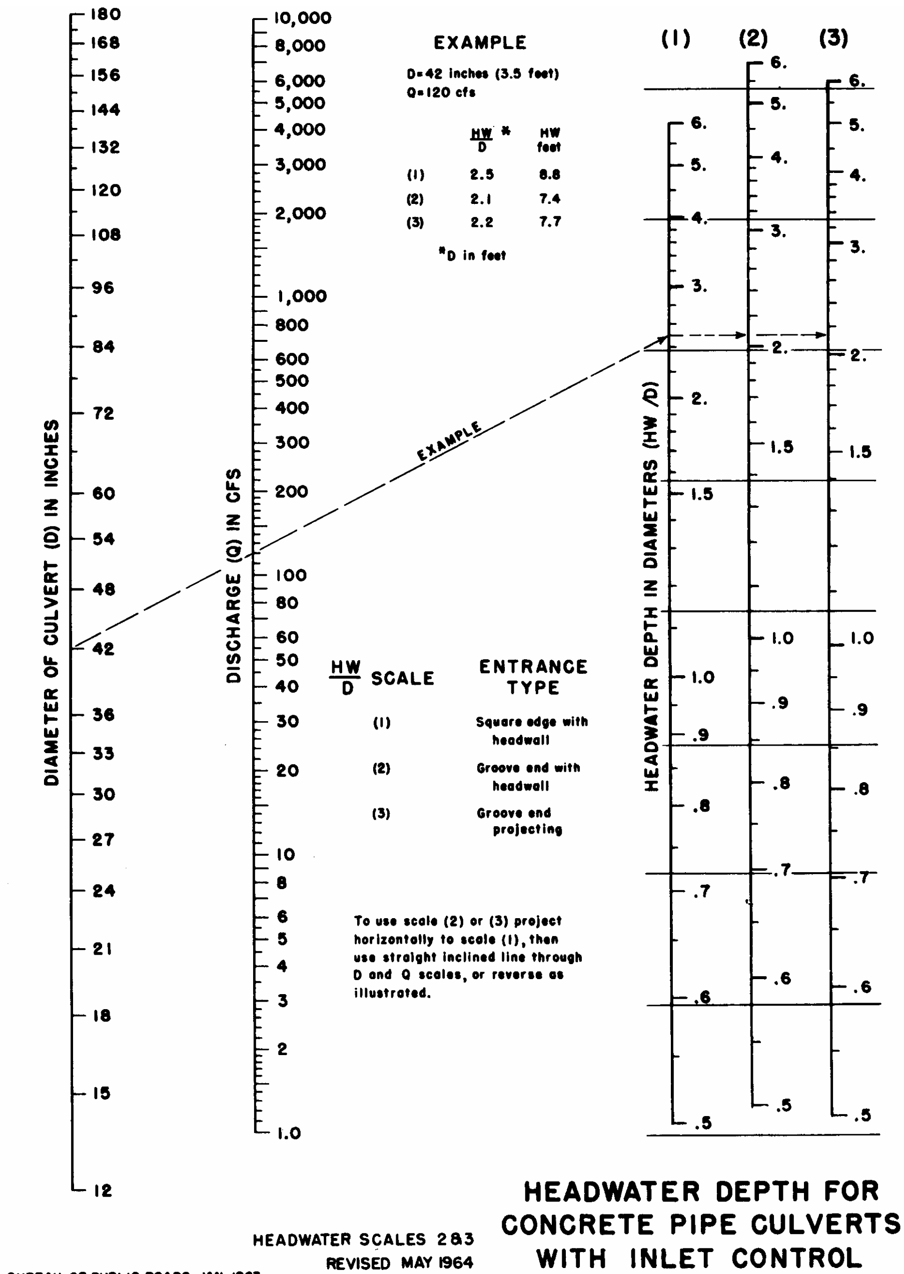


Figure 4.2-2(a) Headwater Depth for Concrete Pipe Culvert with Inlet Control

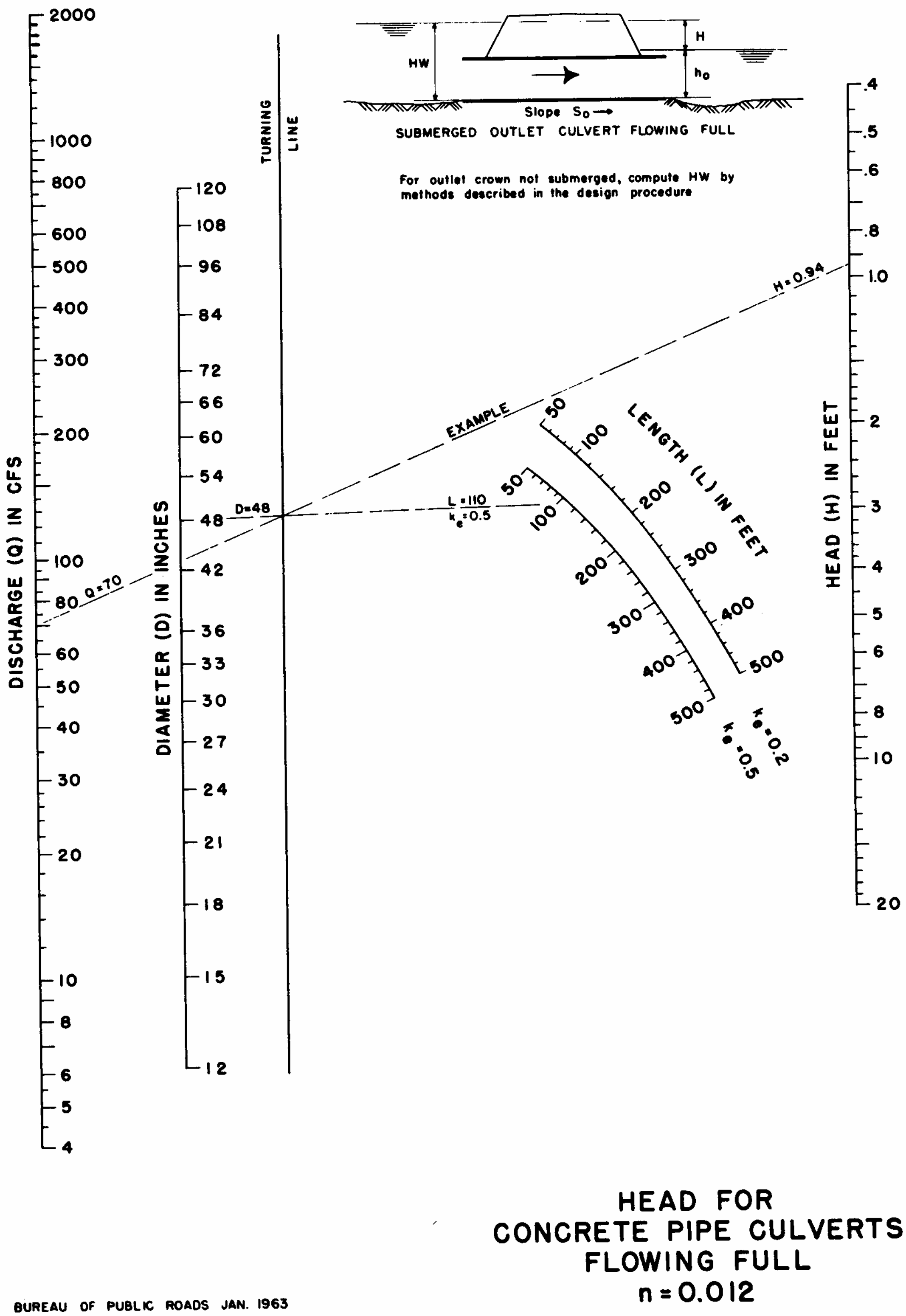


Figure 4.2-2(b) Head for Concrete Pipe Culverts Flowing Full

4.2.4.5 Design Procedure

The following design procedure requires the use of inlet and outlet nomographs.

Step 1 List design data:

- Q = discharge (cfs)
- L = culvert length (ft)
- S = culvert slope (ft/ft)
- TW = tailwater depth (ft)
- V = velocity for trial diameter (ft/s)
- K_e = inlet loss coefficient
- HW = allowable headwater depth for the design storm (ft)

Step 2 Determine trial culvert size by assuming a trial velocity of 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

Step 3 Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

$$\mathbf{HW = H + h_o - LS} \qquad \mathbf{(4.2.4)}$$

where:

- h_o = ½ (critical depth + D), or tailwater depth, whichever is greater
- L = culvert length
- S = culvert slope

Step 4 Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

Step 5 Calculate exit velocity and if erosion problems might be expected, refer to Section 4.7 for appropriate energy dissipation designs. Energy dissipation designs may affect the outlet hydraulics of the culvert.

4.2.4.6 Performance Curves - Roadway Overtopping

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the use of computer programs.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step 1 Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The 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.

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

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 4.2.5 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d L(HW)^{1.5}} \qquad \mathbf{(4.2.5)}$$

where:

Q = overtopping flow rate (ft^3/s)

C_d = overtopping discharge coefficient

L = length of roadway (ft)

HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 4.2-3 on the next page for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 44 in HDS No. 5, 2001.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

4.2.4.7 Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See subsection 4.2.7 and Section 2.2 for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, 2001, Federal Highway Administration, pages 123 - 142.

Note: Storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.

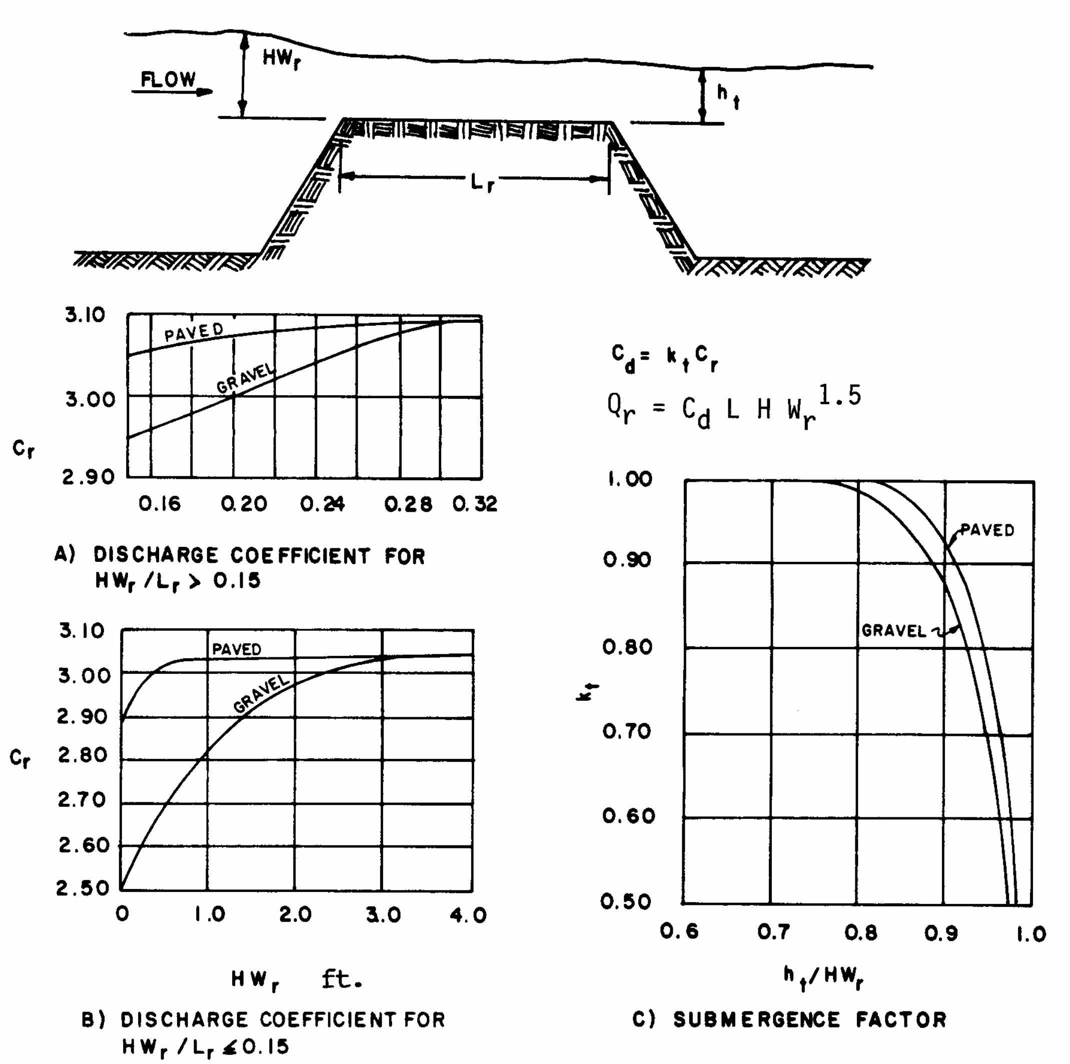


Figure 4.2-3 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 2001)

4.2.5 Culvert Design Example

4.2.5.1 Introduction

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

4.2.5.2 Example

Size a culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

4.2.5.3 Example Data

<u>Input Data</u>

Discharge for 2-yr flood = 35 cfs

Discharge for 25-yr flood = 70 cfs

Allowable H_w for 25-yr discharge = 5.25 ft

Length of culvert = 100 ft

Natural channel invert elevations - inlet = 15.50 ft, outlet = 14.30 ft

Culvert slope = 0.012 ft/ft

Tailwater depth for 25-yr discharge = 3.5 ft

Tailwater depth is the normal depth in downstream channel

Entrance type = Groove end with headwall

4.2.5.4 Computations

1. Assume a culvert velocity of 5 ft/s. Required flow area = 70 cfs/5 ft/s = 14 ft^2 (for the 25-yr recurrence flood).
2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = $(3.14D^2)/4$ or D = $(\text{Area times } 4/3.14)^{0.5}$. Therefore: D = $((14 \text{ sq ft} \times 4)/3.14)^{0.5} \times 12$ in/ft) = 50.7 in
3. A grooved end concrete culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 4.2-2(a)), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.
4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft, which is less than the allowable headwater of 5.25 ft. Since 3.72 ft is considerably less than 5.25 try a small culvert.
5. Using the same procedures outlined in steps 4 and 5 the following results were obtained.

   42-inch culvert – HW = 4.13 ft

   36-inch culvert – HW = 5.04 ft

   Select a 36-inch culvert to check for outlet control.
6. The culvert is checked for outlet control by using Figure 4.2-2(b).

   With an entrance loss coefficient K_e of 0.20, a culvert length of 100 ft, and a pipe diameter of 36 in., an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: $HW = H + h_o - LS$

   Compute h_o

   $h_o = T_w$ or ½ (critical depth in culvert + D), whichever is greater.

   h_o = 3.5 ft or h_o = ½ (2.7 + 3.0) = 2.85 ft

   Note: critical depth is obtained from Figure 3.2-18(b).

   Therefore: h_o = 3.5 ft

   The headwater depth for outlet control is:

   $HW = H + h_o - LS = 2.8 + 3.5 - (100) \times (0.012) = 5.10$ ft
7. Since HW for outlet control (5.10 ft) is greater than the HW for inlet control (5.04 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.

8. Estimate outlet exit velocity. Since this culvert is an outlet control and discharges into an open channel downstream with tailwater above culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the exit velocity will be:

   $Q = VA$

   Therefore: $V = 70 / (3.14(3.0)^2)/4 = 9.9$ ft/s

   With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See Section 4.7 (*Energy Dissipation Design)*.

9. Check for minimum velocity using the 2-year flow of 35 cfs.

   Therefore: $V = 35 / (3.14(3.0)^2/4 = 5.0$ ft/s > minimum of 2.5 - OK

10. The 100-year flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

Figure 4.2-4 provides a convenient form to organize culvert design calculations.

PROJECT: ____________	STATION: ____________	CULVERT DESIGN FORM
____________	SHEET ______ OF ______	DESIGNER / DATE: ______ OF ______ REVIEWER / DATE: ______ OF ______

HYDROLOGICAL DATA

SEE ADD'L SHTS.

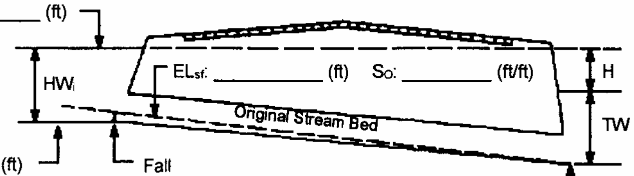
- ☐ METHOD: ____________
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ____________
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: ____________ (ft)

EL_{hi}: ______ (ft)



EL_i: ______ (ft)

EL_o: ______ (ft)

$S = S_o - Fall/L_o$

$S =$ ______ (ft/ft)

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

(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 4.2-4 Culvert Design Calculation Form

(Source: HDS No. 5, 2001)

4.2.6 Design Procedures for Beveled-Edged Inlets

4.2.6.1 Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Design Series No. 5 entitled, Hydraulic Design of Highway Culverts.

4.2.6.2 Design Figures

Four inlet control figures for culverts with beveled edges are found in Appendix D of HDS No. 5.

Chart	Page	Use for
3	D-3A & B	circular pipe culverts with beveled rings
10	D-10A & B	90^o headwalls (same for 90^o wingwalls)
11	D-11A & B	skewed headwalls
112	D-12A & B	wingwalls with flare angles of 18 to 45 degrees

The following symbols are used in these figures:

B – Width of culvert barrel or diameter of pipe culvert

D – Height of box culvert or diameter of pipe culvert

H_f – Depth of pool or head, above the face section of invert

N – Number of barrels

Q – Design discharge

4.2.6.3 Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 in subsection 4.2.8 apply only to bevels having either a 33^o angle (1.5:1) or a 45^o angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 0.5 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft. For example, the minimum bevel dimensions for an 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches

Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches

For a 1.5:1 bevel computations would result in d = 6 and b = 8 inches.

4.2.6.4 Design Figure Limits

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft by 8 ft box culvert:

Top Bevel is proportioned based on the height of 8 feet, which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

Side Bevel is proportioned based on the clear width of 16 feet, which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

4.2.6.5 Multibarrel Installations

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

4.2.6.6 Skewed Inlets

It is recommended that Chart 11 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Skewed inlets (at an angle with the centerline of the stream) should be avoided whenever possible and should not be used with side- or slope-tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

4.2.7 Flood Routing and Culvert Design

4.2.7.1 Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is accepted without flood routing, then costly over-design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- The total area of flooding,
- The average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

4.2.7.2 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 and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that a computer program be used to perform routing calculations; however, an engineer should be familiar with the culvert flood routing design process.

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

Step 1 A trial culvert(s) is selected

Step 2 A trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected

Step 3 Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

Step 4 The hydraulic findings are compared to the selected site criteria

Step 5 If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

4.2.7.3 Comprehensive Design Guidance

Comprehensive design discussions and guidance may be found in the Federal Highway Administration, National Design Series No. 5, document entitled Hydraulic Design of Highway Culverts, Second Edition, published in 2001. This document is available from the National Technical Information Service as Item Number PB2003102411*DL. (http://www.ntis.gov/search.htm) Search for this document using the Item Number.

References

American Association of State Highway and Transportation Officials, 1982. Highway Drainage Guidelines.

American Association of State Highway and Transportation Officials, 1998. Model Drainage Manual.

Debo, Thomas N. and Andrew J. Reese, 1995. Municipal Storm Water Management. Lewis Publishers.

Federal Highway Administration, 1978. Hydraulics of Bridge Waterways. Hydraulic Design Series No. 1.

Federal Highway Administration, 2001. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5. (Downloadable from the Internet http://www.ntis.gov/search.htm, Item Number PB2003102411*DL)

Federal Highway Administration, 1971. Debris-Control Structures. Hydraulic Engineering Circular No. 9.

Federal Highway Administration, 1987. HY8 Culvert Analysis Microcomputer Program Applications Guide. Hydraulic Microcomputer Program HY8.

Federal Highway Administration, 2001. Urban Drainage Design Manual, Second Edition. Hydraulic Engineering Circular No. 22.

Federal Highway Administration Internet Site for Publications, http://www.fhwa.dot.gov/bridge/hypub.htm#hds.

HYDRAIN Culvert Computer Program (HY8). Available from McTrans Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.

HYDRAIN for Windows available at GKY and Associates, Inc., Springfield, Virginia (http://www.gky.com/HFW/.)

U. S. Department of Interior, 1983. Design of Small Canal Structures.

Section 4.3
Bridge Design

4.3.1 Overview

Bridges enable streams to maintain flow conveyance and to sustain aquatic life. They are important and expensive highway hydraulic structures vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing must be recognized and considered during the development, construction, and maintenance phases.

This section addresses structures designed hydraulically as bridges, regardless of length. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the floodplains.

The following subsections present considerations related to the hydraulics of bridges. It is generally excerpted from Chapter 9 of the Texas Department of Transportation (TxDOT) Hydraulics Design Manual dated March 2004.

4.3.1.1 Bridge Hydraulics Considerations

When beginning analysis for a cross-drainage facility, the flood frequency and stage-discharge curves should first be established, as well as the type of cross-drain facility. The choice is usually between a bridge and a culvert. Bridges are usually chosen if the discharge is significant or if the stream to be crossed is large in extent. Both types of facilities should be evaluated and a choice made based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

4.3.1.2 Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves:

- Determining the backwater associated with each alternative profile and waterway opening(s)
- Determining the effects on flow distribution and velocities
- Estimating scour potential

The hydraulic design of a bridge over a waterway involves the following such that the risks associated with backwater and increased velocities are not excessive:

- Establishing a location
- Bridge length
- Orientation
- Roadway and bridge profiles

A hydrologic and hydraulic analysis is recommended for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely affect the floodplain, even if no structural modifications are necessary. Typically, this should include the following:

- An estimate of peak discharge (sometimes complete runoff hydrographs)
- Existing and proposed condition water surface profiles for design and check flood conditions
- Consideration of the potential for stream stability problems and scour potential.

4.3.1.3 Freeboard

Navigational clearance and other reasons notwithstanding, the low chord elevation is defined as the sum of the design normal water surface elevation (high water) and a *freeboard*. For on system TxDOT bridges, TxDOT recommends a minimum freeboard of 2 ft to allow for passage of floating debris and to provide a safety factor for design flood flow. Higher freeboards may be appropriate over streams that are prone to heavy debris loads, such as large tree limbs, and to accommodate other clearance needs. Other constraints may make lower freeboards desirable, but the low chord should not impinge on the design high water. Generally, for off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced, unless the results of a risk assessment indicate a different structure is the most beneficial option.

4.3.1.4 Roadway/Bridge Profile

A bridge is integrated into both the stream and the roadway and must be fully compatible with both. Therefore, the alignment of the roadway and the bridge are the same between the ends of the bridge. Hydraulically, the complete bridge profile can be any part of the structure that stream flow can strike or impact in its movement downstream. If the stream gets high enough to inundate the structure, then all parts of the roadway and the bridge become part of the complete bridge profile.

For TxDOT design, the roadway must not be inundated by the design flood, but inundation by the 100-year flood is allowed. Unless the route is an emergency escape route, it is often desirable to allow floods in excess of the design flood to overtop the road. This helps minimize both the backwater and the required length of structure.

Several vertical alignment alternatives are available for consideration, depending on site topography, traffic requirements, and flood damage potential. The alternatives range from crossings that are designed to overtop frequently to crossings that are designed to rarely or never overtop.

4.3.1.5 Crossing Profile

The horizontal alignment of a highway at a stream crossing should be taken into consideration when selecting the design and location of the waterway opening as well as the crossing profile. Every effort should be made to align the highway so that the crossing will be normal to the stream flow direction (highway centerline perpendicular to the streamline).

Often, this is not possible because of the highway or stream configuration. When a skewed structure is necessary, it should be ensured that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow.

Bent caps should be oriented as near to the skew of the streamlines at flood stage as possible. Headers should be skewed to minimize eddy-causing obstructions. A relief opening may be provided to reduce the likelihood of trapped flow and minimize the amount of flow that would have to travel up against the general direction of flow along the embankment.

4.3.1.6 Single Versus Multiple Openings

For a single structure, the flow will find its way to an opening until the roadway is overtopped. If two or more structures have flow area available, after accumulating a head, the flow will divide and proceed to the structures offering the least resistance. The point of division is called a stagnation point.

In usual practice, the TxDOT recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft. If flow distances along the embankment are greater than recommended, an additional relief structure or opening should be considered. A possible alternative to the provision of an additional structure is a guide bank (spur dike) to control the turbulence at the header. Also, natural vegetation between the toe of slope and the right-of-way line is useful in controlling flow along the embankment. Therefore, special efforts should be made to preserve any natural vegetation in such a situation.

4.3.1.7 Factors Affecting Bridge Length

The discussions of bridge design assume normal cross sections and lengths (perpendicular to flow at flood stage). Usually one-dimensional flow is assumed, and cross sections and lengths are considered 90° to the direction of stream flow at flood stage.

If the crossing is skewed to the stream flow at flood stage, all cross sections and lengths should be normalized before proceeding with the bridge length design. If the skew is severe and the floodplain is wide, the analysis may need to be adjusted to offset the effects of elevation changes within the same cross section.

The following examples illustrate various factors that can cause a bridge opening to be larger than that required by hydraulic design.

- Bank protection may be placed in a certain location due to local soil instability or a high bank.
- Bridge costs may be cheaper than embankment costs.
- A highway profile grade line might dictate an excessive freeboard allowance. For sloping abutments, a higher freeboard will result in a longer bridge.
- High potential for meander to migrate, or other channel instabilities may warrant a longer opening.

4.3.2 Symbols and Definitions

The hydraulics of bridge openings are basically the same as those of open channel flow. Therefore, the symbols and definitions are essentially the same as those of Section 4.4.2 presented in Table 4.4-1. There are other definitions unique to bridges which are presented here. They are defined in the TxDOT Hydraulic Design Manual.

4.3.2.1 Flow Zones and Energy Losses

Figure 4.3-1 shows a plan of typical cross section locations that establish three flow zones that should be considered when estimating the effects of bridge openings.

Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. No detailed guidance is available, but a distance equal to about four times the length of the average embankment constriction is reasonable for most situations. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 4.3-2.

Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally, the bridge opening is obtained by superimposing the bridge geometry on cross sections 2 and 3.

Zone 3 represents an area from the upstream face of the bridge to a distance upstream where the contraction of flow must occur. A distance upstream of the bridge equal to the length of the average embankment constriction is a reasonable approximation of the location at which contraction begins. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the "approach" cross section.

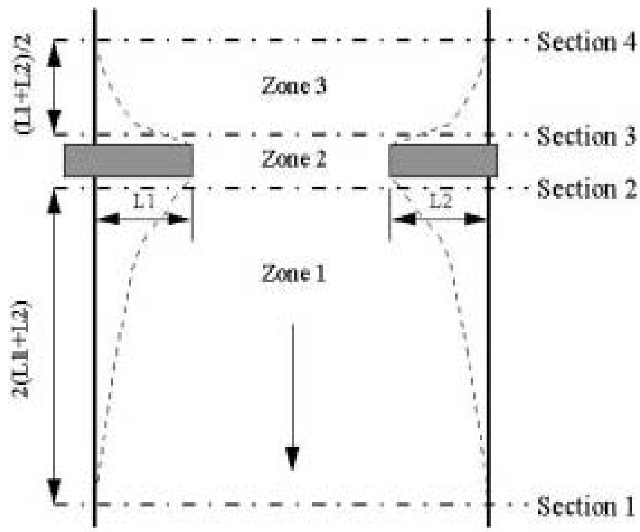


Figure 4.3-1 Flow Zones at Bridges
(TxDOT Hydraulic Design Manual)

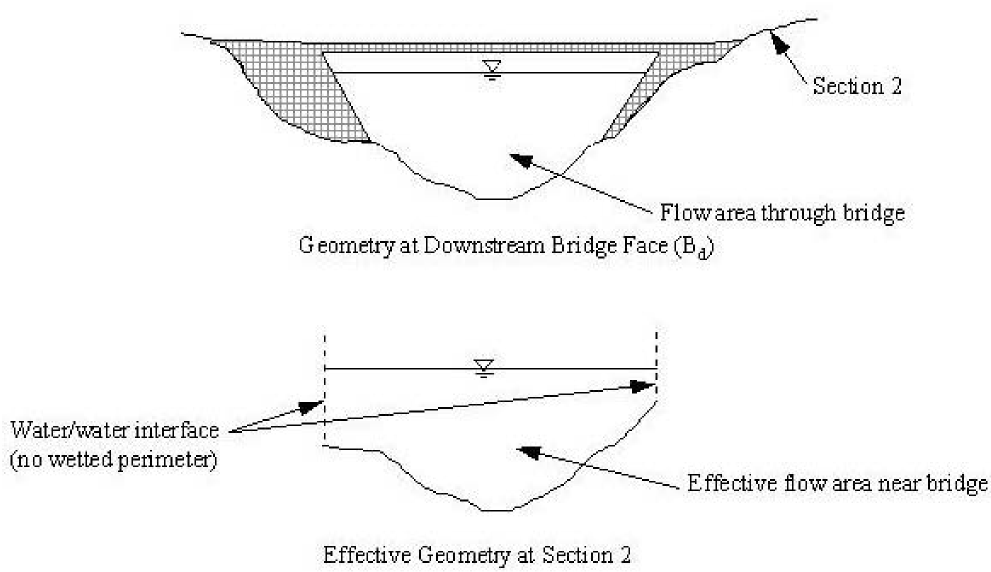


Figure 4.3-2 Effective Geometry for Bridge (Section 2 shown, Section 3 similar)
(TxDOT Hydraulic Design Manual)

4.3.2.2 Bridge Flow Class

The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow.

Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. (This condition should exist for the design frequency of all new on-system bridges.) Low flow is divided into categories as described in the "Low Flow Classes" table below Type I is the most common in Texas, although severe constrictions compared to the flow conditions could result in Types IIA and IIB. Type III is likely to be limited to steep hills and mountainous regions.

Low Flow Class	Description
I	Subcritical flow through all Zones
IIA	Subcritical flow through Zones 1 and 3; flow through critical depth in Zone 2
IIB	Subcritical flow through Zone 3; flow through critical depth in Zone 2, hydraulic jump in Zone 1
III	Supercritical flow through all Zones

High flow refers to conditions in which the water surface impinges on the bridge superstructure:

- When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate.
- When the tailwater submerges the low chord but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the tailwater overtops the roadway, neither sluice gate flow nor orifice flow is reasonable, and the flow is either weir flow or open flow.

4.3.3 Design Criteria

The design of a bridge should take into account many different engineering and technical aspects at the bridge site and adjacent areas. The following design criteria should be considered for all bridge designs as applicable. See the Local Criteria section of this manual for additional considerations.

4.3.3.1 Frequency Flood

Design discharges chosen by TxDOT for bridges vary with the functional classification and structure type. For major river crossings, a return period of 50 years is recommended. Flow small bridges, the recommended return period is 25 years. In all cases the check flood is for the 100-year return period.

4.3.3.2 Freeboard

Typical freeboard, the length between the computed design water surface and the low chord, is two feet. In urban settings, it may be prudent to use the 100-year fully-developed discharge to check the bridge design. The 100-year flood discharge, assuming blockage of outlet works, with 6" of freeboard. Some municipalities may specify different design storms and freeboard requirements.

4.3.3.3 Loss Coefficients

The contraction and expansion of water through the bridge opening creates hydraulic losses. These losses are accounted for through the use of loss coefficients. Table 4.3-1 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 4.3-1 Recommended Loss Coefficients for Bridges		
Transition Type	Contraction (K_c)	Expansion (K_e)
No losses computed	0.0	0.0
Gradual transition	0.1	0.3
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

4.3.4 Design Procedures

The following is a general bridge hydraulic design procedure.

1. Determine the most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.
2. Determine design discharge from hydrologic studies or available data (City, Federal Emergency Management Agency (FEMA), US Army Corp of Engineers (USACE), TxDOT, or similar sources).
3. If available, obtain effective FEMA hydraulic backwater model. It is assumed that if a bridge is required instead of a culvert, the drainage area would exceed one square mile and could already be included in a FEMA study. If an effective FEMA model or other model is not available, a basic hydrologic model and backwater analysis for the stream must be prepared. The HEC-RAS computer model is routinely used to compute backwater water surface profiles.
4. Using USACE or FEMA guidelines, compute or duplicate an existing conditions water surface profile for the design storm(s). Compute a profile for the fully-developed watershed to use as a baseline for design of a new bridge/roadway crossing.
5. Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for preliminary sizing, use a normal-depth design procedure, or simply estimate a required trapezoidal opening.
6. Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard and any channelization upstream or downstream to transition the floodwaters through the proposed structure.
7. Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade, etc.). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.
8. Review the velocities and determine erosion control requirements downstream, through the structure, and upstream.
9. Finalize the design size and erosion control features, based on comparing the proposed model with the existing conditions profiles, impacts on other properties, FEMA guidelines, and Local Criteria.
10. Exceptions/Other Issues
    A. Conditional Letter of Map Amendment (CLOMR) may be needed for new crossings of streams studied by FEMA.
    B. If applicable, coordinate with USACE Regulatory Permit requirements.
    C. Evaluate the project with respect to iSWM policy regarding downstream impacts.
    D. Design should be for fully developed watershed conditions. If the available discharges are from FEMA existing conditions hydrology, the following options are available: (1) obtain new hydrology, (2) extrapolate fully-developed from existing data, or (3) variance from the local jurisdiction on design discharges
    E. Freeboard criteria may require an unusually expensive bridge or impracticable roadway elevation. A reasonable variance in criteria from the local jurisdiction may be available.

References

Texas Department of Transportation, March 2004. Hydraulic Design Manual. Available at http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd/@Generic__BookView.

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CHAPTER

4

Section 4.4
Open Channel Design

4.4.1 Overview

4.4.1.1 Introduction

Open channel systems and their design are an integral part of storm water drainage design, particularly for development sites utilizing better site design practices and open channel structural controls. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, stone riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design criteria and methods, including the use of channel design nomographs.

4.4.1.2 Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

<u>Vegetative Linings</u> – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see Section 1.4 and Chapter 3 for more details on using enhanced swales and grass channels for water quality purposes).

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities
- Standing or continuously flowing water
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of healthy vegetation.

If low flows are prevalent, a hard lined pilot channel may be needed, and it should be wide enough to accommodate maintenance equipment. Whether to allow pilot channels should be included in the local criteria section.

<u>Flexible Linings</u> – Rock riprap, including rubble and gabion baskets, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities

that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

4.4.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.4-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.4-1 Symbols and Definitions

Symbol	Definition	Units
α	Energy coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
delta d	Super-elevation of the water surface profile	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration of gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
w	Stone weight	lbs
y_c	Critical depth	ft

y_n	Normal depth	ft
z	Critical flow section factor	-

4.4.3 Design Criteria

4.4.3.1 General Criteria

The following criteria should be followed for open channel design:

- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
- Channel side slopes shall be stable throughout the entire length and the side slope shall depend on the channel material. A maximum of 2:1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum side slope of 3:1.
- Trapezoidal or parabolic cross sections are preferred over triangular shapes.
- For vegetative channels, design stability should be determined using low vegetative retardance conditions (Class D) and for design capacity higher vegetative retardance conditions (Class C) should be used.
- For vegetative channels, flow velocities within the channel should not exceed the maximum permissible velocities given in Tables 4.4-2 and 4.4-3.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.
- Open channel drainage systems are typically sized to handle a 25-year design storm. The 100-year design storm should be routed through the channel system to determine if the 100-year plus applicable building elevation restrictions are exceeded, structures are flooded, or flood damages increased.
- HEC-RAS is typically used to confirm the water surface profiles in open channels, and due to the complexity of hydraulic elements in roadside ditches (open channels), it should be used to evaluate water surface profiles in roadside ditches as well.

4.4.3.2 Velocity Limitations

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Recommended maximum velocity values for selected lining categories are presented in Table 4.4-2. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 4.4-3. Vegetative lining calculations are presented in Section 4.4.7 and stone riprap procedures are presented in Section 4.4.8.

Typically, local design limits the velocity to 15 fps in concrete lined channels. For gabions typical design velocities range from 10 fps for 6-inch mattresses up to 15 fps for 1-foot mattresses. Some manufacturers indicate that velocities of 20 fps are allowable for basket installations. No specific velocity limit is appropriate for rock riprap. The design of stable riprap lining depends upon the intersection of the velocity (local boundary shear) and the size and gradation of the riprap material. In general, velocity limitations should be set by the local jurisdiction.

4.4.4 Manning's n Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and stone riprap linings are given in Table 4.4-4. Recommended values for vegetative linings should be determined using Figure 4.4-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 4.4-6). Figure 4.4-1 is used iteratively as described in Section 4.4.6. Recommended Manning's values for natural channels that are either excavated or dredged, and natural are given in Table 4.4-2. For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959. Some of these values are given in Table 4.4-2 below.

Table 4.4-2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
MINOR NATURAL STREAMS		
Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	3 to 6
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	3 to 6
3. Some weeds, light brush on banks	0.035	3 to 6
4. Some weeds, heavy brush on banks	0.050	3 to 6
5. Some weeds, dense willows on banks	0.060	3 to 6
For trees within channels with branches submerged at high stage, increase above values by	0.010	
Irregular section with pools, slight channel meander, increase above values by	0.010	
Floodplain – Pasture		
1. Short grass	0.030	3 to 6
2. Tall grass	0.035	3 to 6
Floodplain – Cultivated Areas		
1. No crop	0.030	3 to 6
2. Mature row crops	0.035	3 to 6
3. Mature field crops	0.040	3 to 6
Floodplain – Uncleared		
1. Heavy weeds scattered brush	0.050	3 to 6
2. Wooded	0.120	3 to 6
MAJOR NATURAL STREAMS		
Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or	Range from 0.028 to 0.060	3 to 6

Table 4.4-2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
vegetation on banks. Values of "n" for larger streams of mostly regular sections, with no boulders or brush		
UNLINED VEGETATED CHANNELS		
Clays (Bermuda Grass)	0.035	5 to 6
Sandy and Silty Soils (Bermuda Grass)	0.035	3 to 5
UNLINED NON-VEGETATED CHANNELS		
Sandy Soils	0.030	1.5 to 2.5
Silts	0.030	0.7 to 1.5
Sandy Silts	0.030	2.5 to 3.0
Clays	0.030	3.0 to 5.0
Coarse Gravels	0.030	5.0 to 6.0
Shale	0.030	6.0 to 10.0
Rock	0.025	15

Table 4.4-3 Maximum Velocities for Vegetative Channel Linings

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda grass	0-5	6
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	5-10 >10	5 4
Grass mixture	0-5[1] 5-10	4 3
Sericea lespedeza, Weeping lovegrass, Alfalfa	0-5[4]	3
Annuals[5]	0-5	3
Sod		4
Lapped sod		5

[1] Do not use on slopes steeper than 10% except for side-slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stands can be maintained.
[3] Mixtures of Tall Fescue, Bahia, and/or Bermuda
[4] Do not use on slopes steeper than 5% except for side-slope in combination channel.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Source: Manual for Erosion and Sediment Control in Georgia, 1996

Table 4.4-4 Manning's Roughness Coefficients (n) for Artificial Channels				
		Depth Ranges		
Category	Lining Type	0-0.5 ft	0.5-2.0 ft	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	–	0.078	0.040
Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.				
*Some "temporary" linings become permanent when buried.				

Source: HEC-15, 1988.

When designing open channels, the usual choice of Manning's roughness coefficients may be found in Table 4.4-5. The local jurisdiction may choose to vary from these values.

Table 4.4-5 Manning's Roughness Coefficients for Design		
Lining Type	Manning's n	Comments
Grass Lined	0.035	Use for velocity check.
	0.050	Use for channel capacity check (freeboard check)
Concrete Lined	0.015	
Gabions	0.030	
Rock Riprap	0.040	$n = 0.0395 d_{50}^{1/6}$ where d_{50} is the stone size of which 50% of the sample is smaller
Grouted Riprap	0.028	FWHA

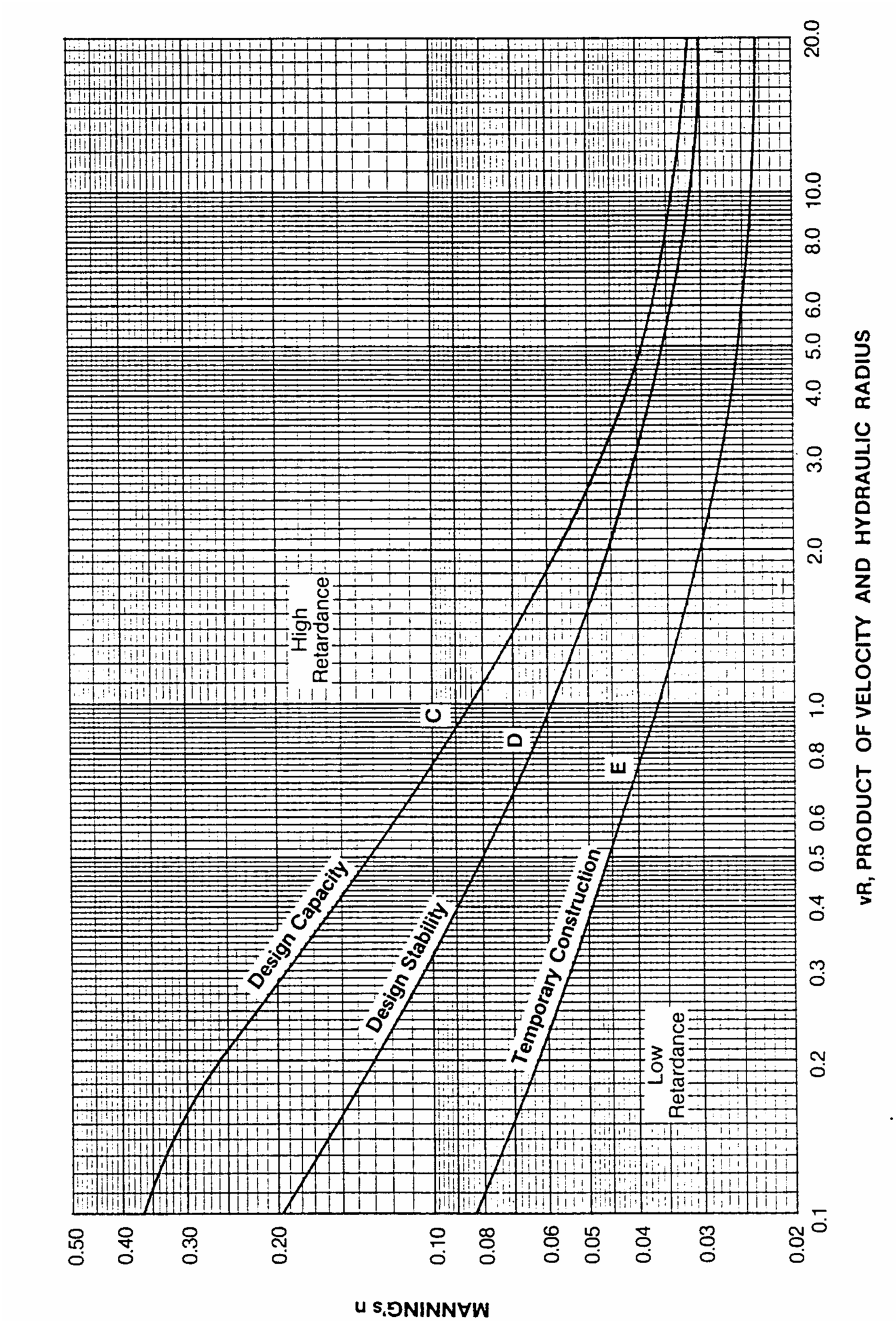


Figure 4.4-1 Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 4.4-6	Classification of Vegetal Covers as to Degrees of Retardance	
Retardance	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36")
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture	
	Little bluestem, bluestem, blue gamma other short and long stem Midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24")
	Laspedeza sericea	Good stand, not woody, tall (average 19")
	Alfalfa	Good stand, uncut (average 11")
	Weeping lovegrass	Good stand, unmowed (average 13")
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13")
C	Crabgrass	Fair stand, uncut (10 – 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture:	
	summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 – 8 ")
	Centipede grass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 – 12")
D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3 – 6")
	Grass-legume mixture:	
	fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 – 5")
	Lespedeza serices	After cutting to 2" (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.
Source: HEC-15, 1988

4.4.5 Uniform Flow Calculations

4.4.5.1 Design Charts

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, trapezoidal, and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. Examples of these charts and instructions for their use are given in subsection 4.4.12.

4.4.5.2 Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$\mathbf{v = (1.49/n)\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(4.4.1)}$$

$$\mathbf{Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(4.4.2)}$$

$$\mathbf{S = [Q_n/(1.49\, A\, R^{2/3})]^2} \qquad \mathbf{(4.4.3)}$$

where:

v = average channel velocity (ft/s)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
A = cross-sectional area (ft^2)
R = hydraulic radius A/P (ft)
P = wetted perimeter (ft)
S = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

For a more comprehensive discussion of open channel theory and design, see the reference USACE, 1991/1994.

4.4.5.3 Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

4.4.5.4 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from equation 4.4.2. The slope can be calculated using equation 4.4.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 4.4-2 and 4.4-3. Figure 4.4-2 provides a general solution for the velocity form of Manning's Equation, while Figure 4.4-3 provides a solution of Manning's Equation for trapezoidal channels.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 4.4-2:

Step 1 Determine open channel data, including slope in ft/ft, hydraulic radius in ft, and Manning's n value.

Step 2 Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

Step 3 Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

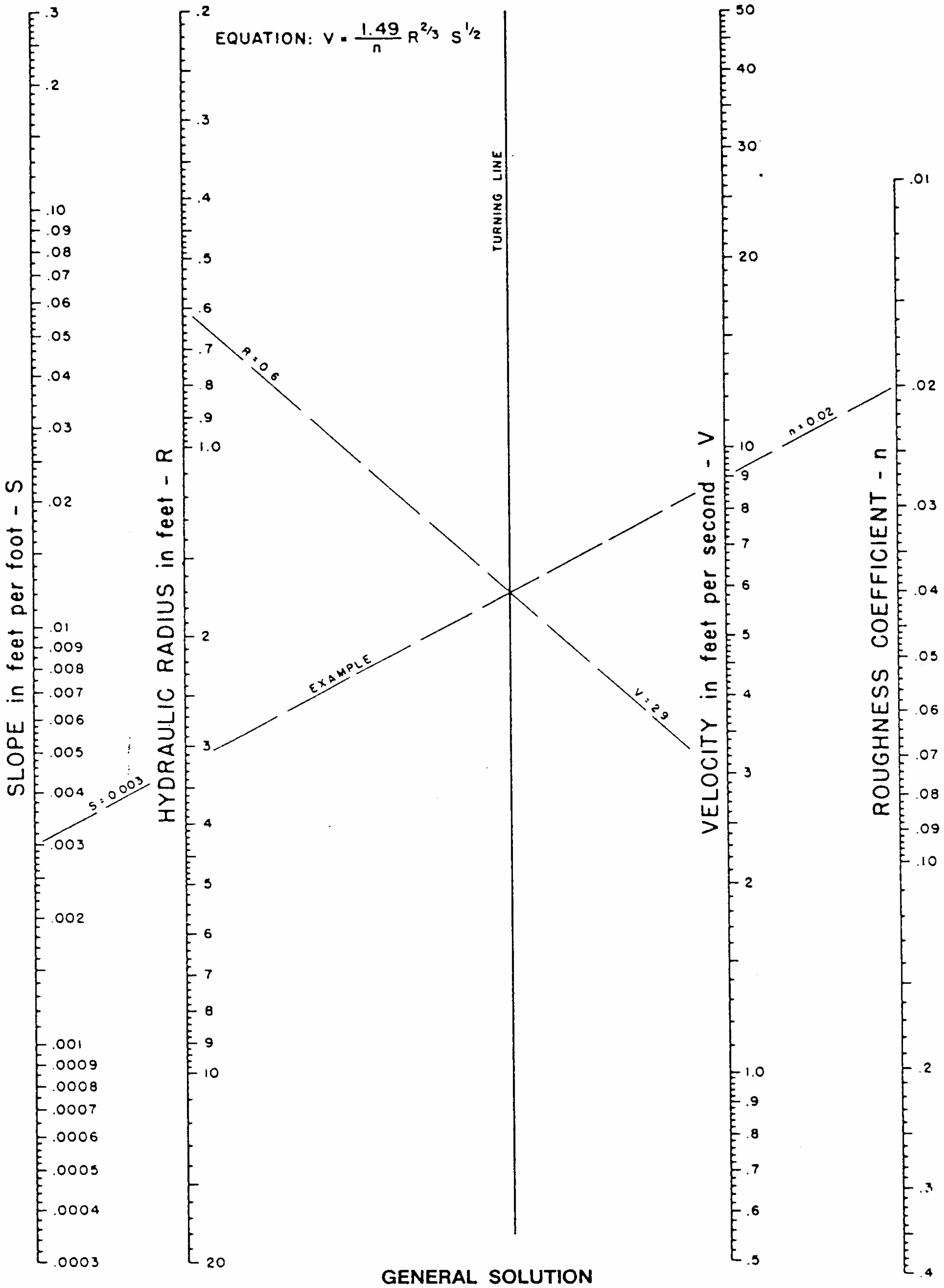
Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 4.4-3 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

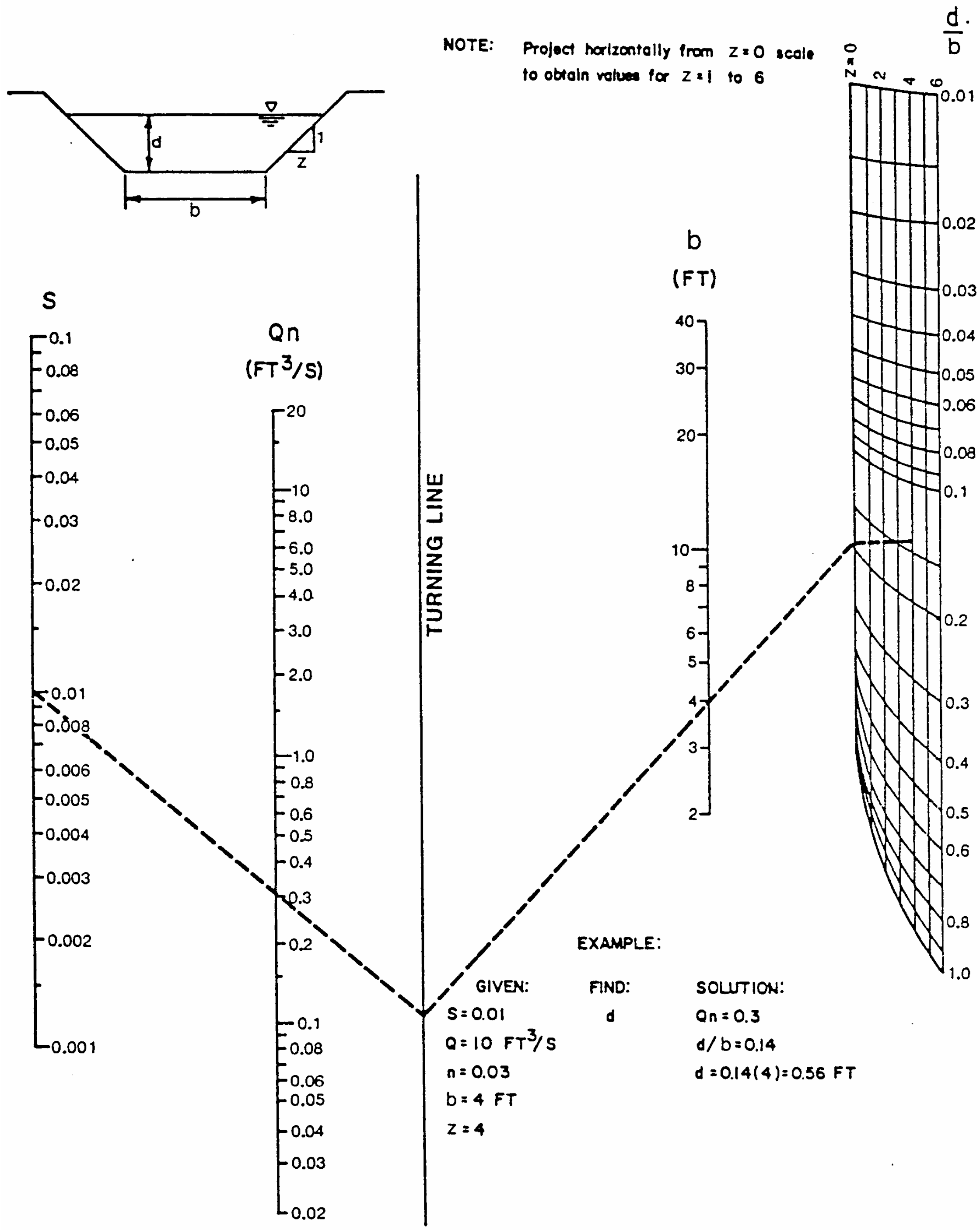
1. Given Q, find d.
   a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.
   b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.
   c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.
   d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.
2. Given d, find Q

   Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.
   Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.
   Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.
   Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 4.4-2 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 4.4-3 Solution of Manning's Equation for Trapezoidal Channels

4.4.5.5 Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as:

$$\mathbf{AR^{2/3} = (Qn)/(1.49\ S^{1/2})} \qquad \mathbf{(4.4.4)}$$

where:

A = cross-sectional area (ft)

R = hydraulic radius (ft)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

S = slope of the energy grade line (ft/ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of equation 4.4.4 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 4.4-4 for trapezoidal channels. Computer programs are also available for these calculations.

Step 1 Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

Step 2 Calculate the trapezoidal conveyance factor using the equation:

$$\mathbf{K_T = (Qn)/(b^{8/3}S^{1/2})} \qquad \mathbf{(4.4.5)}$$

where:

K_T = trapezoidal open channel conveyance factor

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

b = bottom width (ft)

S = slope of the energy grade line (ft/ft)

Step 3 Enter the x-axis of Figure 4.4-4 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

Step 4 From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

Step 5 Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

Note: If bends are considered, refer to equation 4.4.11

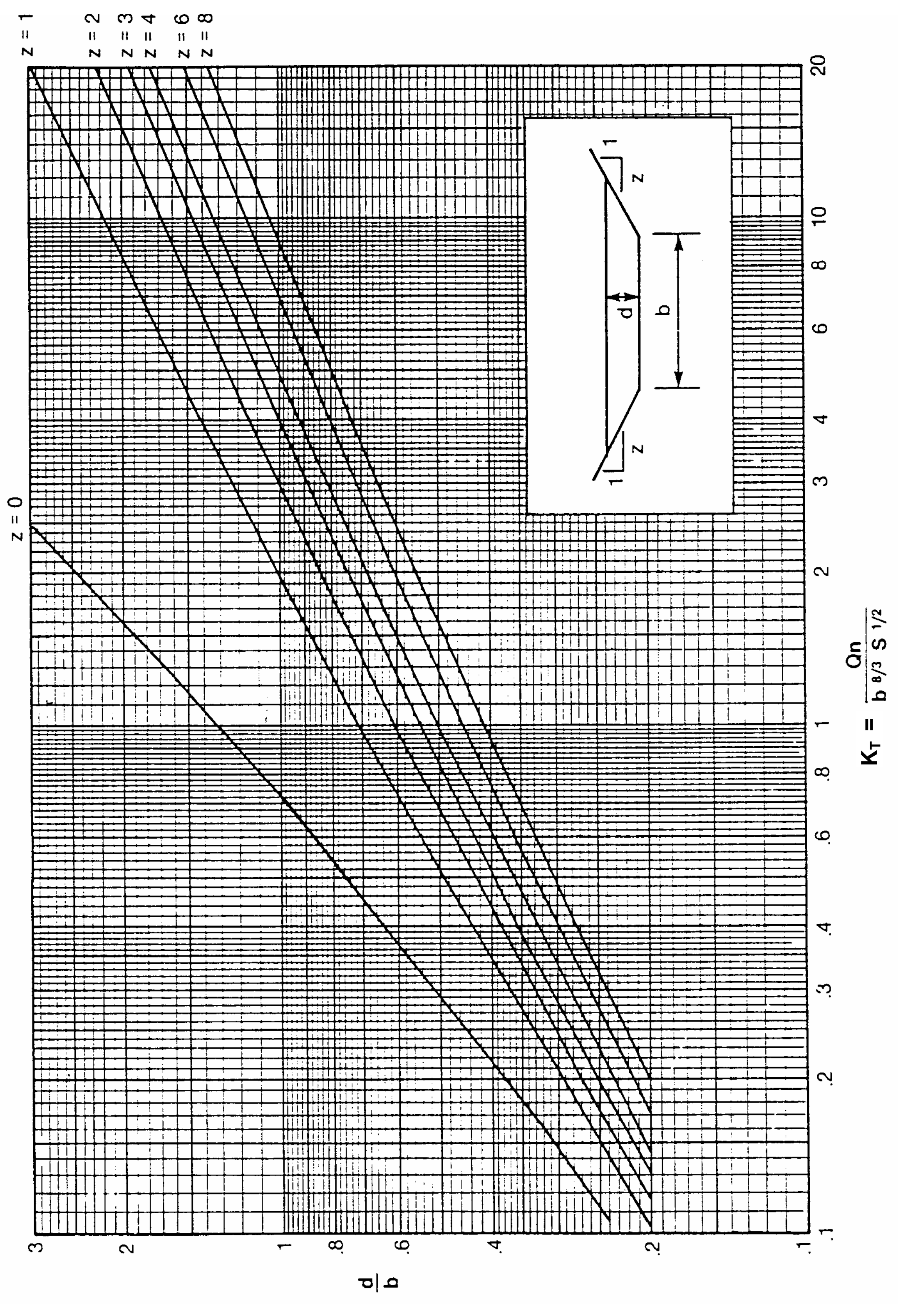


Figure 4.4-4 Trapezoidal Channel Capacity Chart
(Source: Nashville Storm Water Management Manual, 1988)

4.4.6 Critical Flow Calculations

4.4.6.1 Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities. Hydraulic jumps are possible under these conditions and consideration should be given to stabilizing the channel.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\mathbf{Q^2/g = A^3/T} \qquad \mathbf{(4.4.6)}$$

where:

Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/sec^2)
A = cross-sectional area (ft^2)
T = top width of water surface (ft)

Note: A trial and error procedure is needed to solve equation 4.4-6.

4.4.6.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 4.4-7) or section factors (as presented in Figure 4.4-5) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$\mathbf{Z = Q/(g^{0.5})} \qquad \mathbf{(4.4.7)}$$

where:

Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.3 ft/sec^2)

The following guidelines are given for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
3. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
4. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
5. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

Note: The head is the height of water above any point, plane, or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant ($V^2/2g$).

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/(gA/T)^{0.5} \quad (4.4.8)$$

where:

Fr = Froude number (dimensionless)
v = velocity of flow (ft/s)
g = acceleration of gravity (32.2 ft/sec^2)
A = cross-sectional area of flow (ft^2)
T = top width of flow (ft)

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 4.4-7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

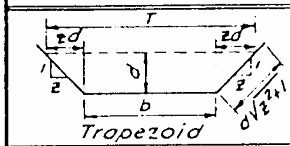
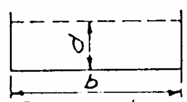
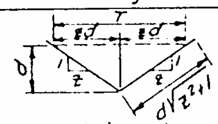
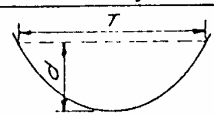
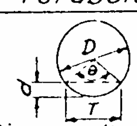
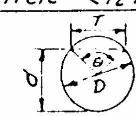
Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	N/A
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	N/A

where:

d_c = critical depth (ft)
Q = design discharge (cfs)
g = acceleration due to gravity (32.3 ft/s^2)
b = bottom width of channel (ft)
z = side slopes of a channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)

[1] See Figure 4.4-5 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

If the water surface profile in a channel transitions from supercritical flow to subcritical flow, a hydraulic jump must occur. The location of the hydraulic jump and its sequent depth are critical to proper design of free flow conveyances. To determine the location of a hydraulic jump, the standard step method is used to compute the water surface profile and specific force (momentum principle) and specific energy relationships are used. For computational methods refer to Chow, 1959, TxDOT, 2002, and Mays, 1999. The HEC-RAS computer program can be used to compute water surface profiles for both subcritical and supercritical flow regimes.

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ 1/	$\frac{2dT^2}{3T^2+8d^2}$ 1/	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
 Circle - < ½ full 2/	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
 Circle - > ½ full 3/	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

1/ Satisfactory approximation for the interval $0 < \frac{d}{T} \leqq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

2/ $\theta = 4\sin^{-1}\sqrt{d/D}$
3/ $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

Figure 4.4-5 Open Channel Geometric Relationships for Various Cross Sections

4.4.7 Vegetative Design

4.4.7.1 Introduction

A two-part procedure is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 4.4-6. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 4.4-6. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10%, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

4.4.7.2 Design Stability

The following are the steps for design stability calculations:

Step 1 Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

Step 2 Use Table 4.4-3 to assign a maximum velocity, v_m based on vegetation type and slope range.

Step 3 Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 4.4-1. Use retardance Class D for permanent vegetation and E for temporary construction.

Step 4 Calculate the hydraulic radius using the equation:

$$\mathbf{R = (vR)/v_m} \qquad \mathbf{(4.4.9)}$$

where:

R = hydraulic radius of flow (ft)

vR = value obtained from Figure 4.4-1 in Step 3

v_m = maximum velocity from Step 2 (ft/s)

Step 5 Use the following form of Manning's Equation to calculate the value of vR:

$$\mathbf{vR = (1.49\ R^{5/3}\ S^{1/2})/n} \qquad \mathbf{(4.4.10)}$$

where:

vR = calculated value of vR product

R = hydraulic radius value from Step 4 (ft)

S = channel bottom slope (ft/ft)

n = Manning's n value assumed in Step 3

Step 6 Compare the vR product value obtained in Step 5 to the value obtained from Figure 4.4-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

Step 7 For trapezoidal channels, find the flow depth using Figures 4.4-3 or 4.4-4, as described in Section 4.4.4.4. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 4.4.4.5.

Step 8 If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 4.4-6. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

4.4.7.3 Design Capacity

The following are the steps for design capacity calculations:

Step 1 Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 4.4-5 for equations).

Step 2 Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

Step 3 Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

Step 4 Use Figure 4.4-1 to find a Manning's n value for retardance Class C based on the vR value from Step 3.

Step 5 Use Manning's Equation (equation 4.4.1) or Figure 4.4-2 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

Step 6 Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

Step 7 Add an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

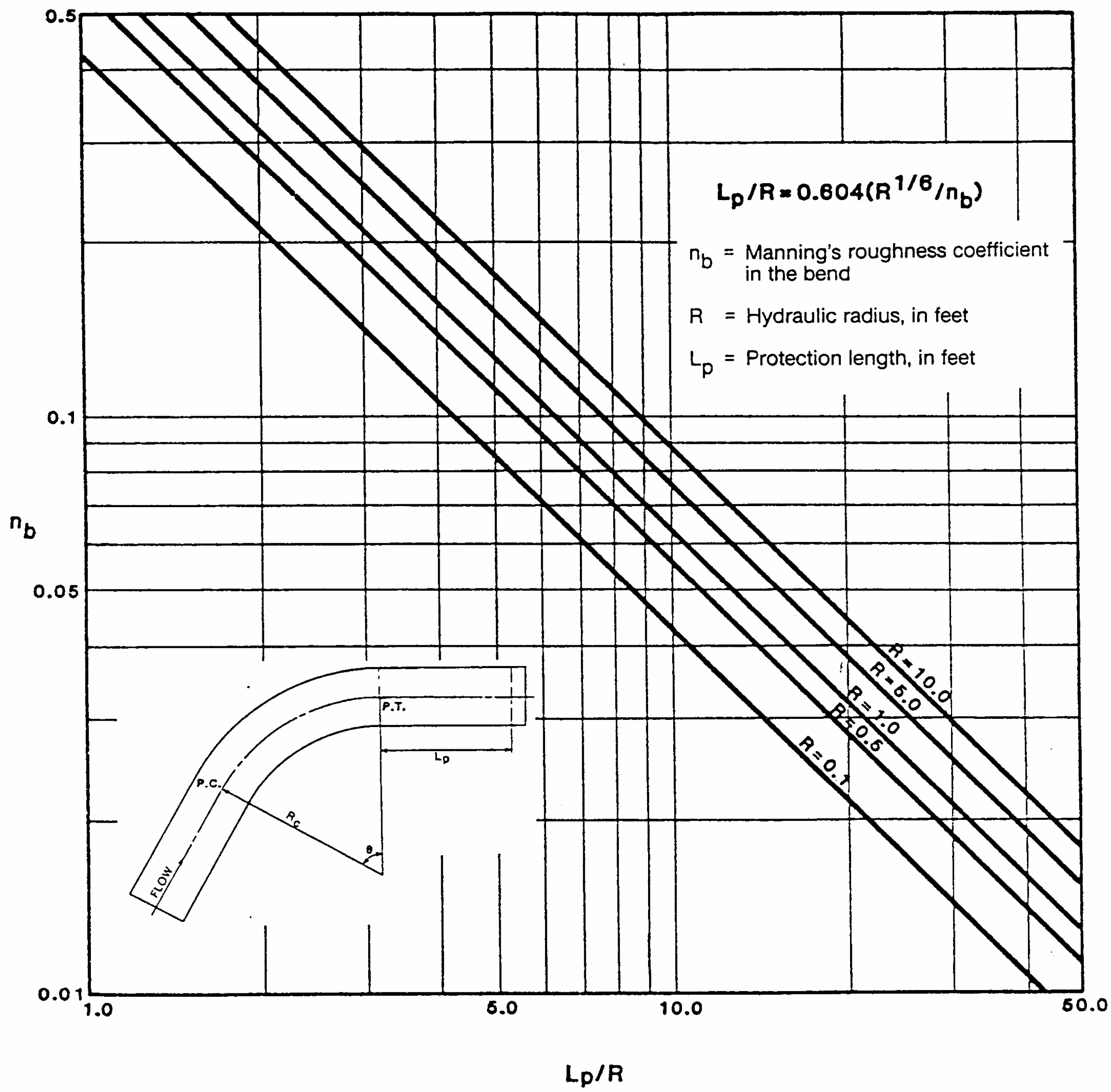
Step 8 If bends are considered, calculate super-elevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2T)/(gR_c) \qquad (4.4.11)$$

where:

Δd = super-elevation of the water surface profile due to the bend (ft)
v = average velocity from Step 6 (ft/s)
T = top width of flow (ft)
g = acceleration of gravity (32.2 ft/sec^2)
R_c = mean radius of the bend (ft)

Note: Add freeboard consistent with the calculated Δd.



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 4.4-6 Protection Length, Lp, Downstream of Channel Bend

4.4.8 Stone Riprap Design

4.4.8.1 Introduction

A number of agencies and researchers have studied and developed empirical equations to estimate the required size of rock riprap to resist various hydraulic conditions, including the U.S. Army Corps of Engineers (USACE), Natural Resource Conservation Service (NRCS), and the Federal Highway Administration (FHWA). As with all empirical equations based on the results of laboratory experiments, they must be used with an understanding of the range of data on which they are based.

Sections 4.4.8.2 and 4.4.8.3 give design guidance for designing stone riprap for open channels. Section 4.4.8.4 gives design guidance for designing stone riprap for culvert outfall protection. Section 4.7.4 gives additional guidance on the design of riprap aprons for erosion protection at outfalls, and Section 4.7.5 gives guidance on the design of riprap basins for energy dissipation.

4.4.8.2 Method #1: Maynord & Reese

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

- Minimum riprap thickness equal to d_{100}
- The value of d_{85}/d_{15} less than 4.6
- Froude number less than 1.2
- Side slopes up to 2:1
- A safety factor of 1.2
- Maximum velocity less than 18 feet per second
- If significant turbulence is caused by boundary irregularities, such as vertical drops, obstructions, or structures, this procedure is not applicable.

Procedure

Following are the steps in the procedure for riprap design using the method by Maynord & Reese:

Step 1 Determine the average velocity in the main channel for the design condition. Manning's n values for riprap can be calculated from the equation:

$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \qquad \textbf{(4.4.12)}$$

where:

n = Manning's roughness coefficient for stone riprap

d_{50} = diameter of stone for which 50%, by weight, of the gradation is finer (ft)

Step 2 If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 4.4-7 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

Step 3 If the specific weight of the stone varies significantly from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 4.4-8.

Step 4 Determine the required minimum d_{30} value from Figure 4.4-9, or from the equation:

$$\mathbf{d_{30}/D = 0.193\ Fr^{2.5}} \qquad \textbf{(4.4.13)}$$

where:

- d_{30} = diameter of stone for which 30%, by weight, of the gradation is finer (ft)
- D = depth of flow above stone (ft)
- Fr = Froude number (see equation 4.4.8), dimensionless
- v = mean velocity above the stone (ft/s)
- g = acceleration of gravity (32.2 ft/sec)

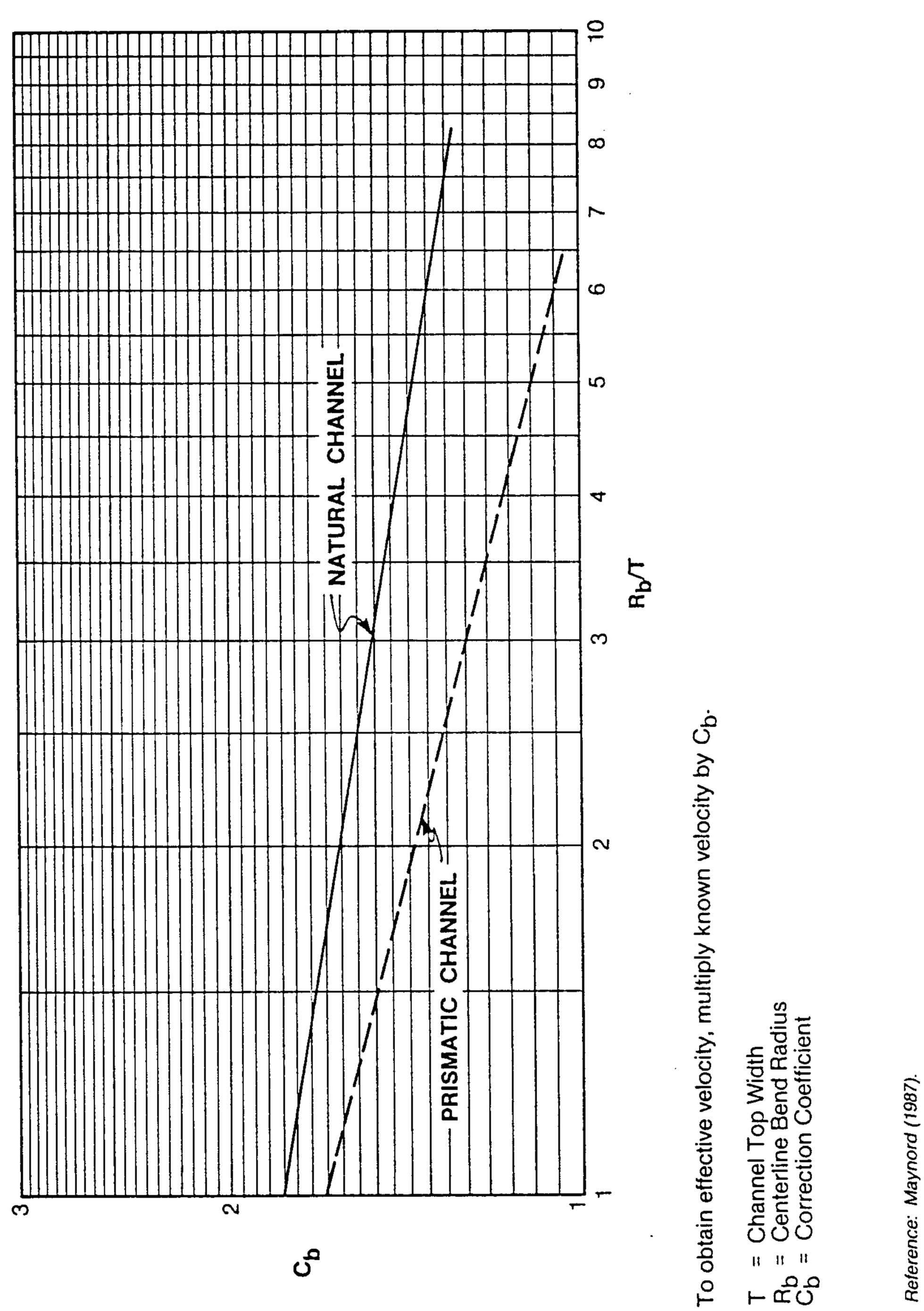


Figure 4.4-7 Riprap Lining Bend Correction Coefficient

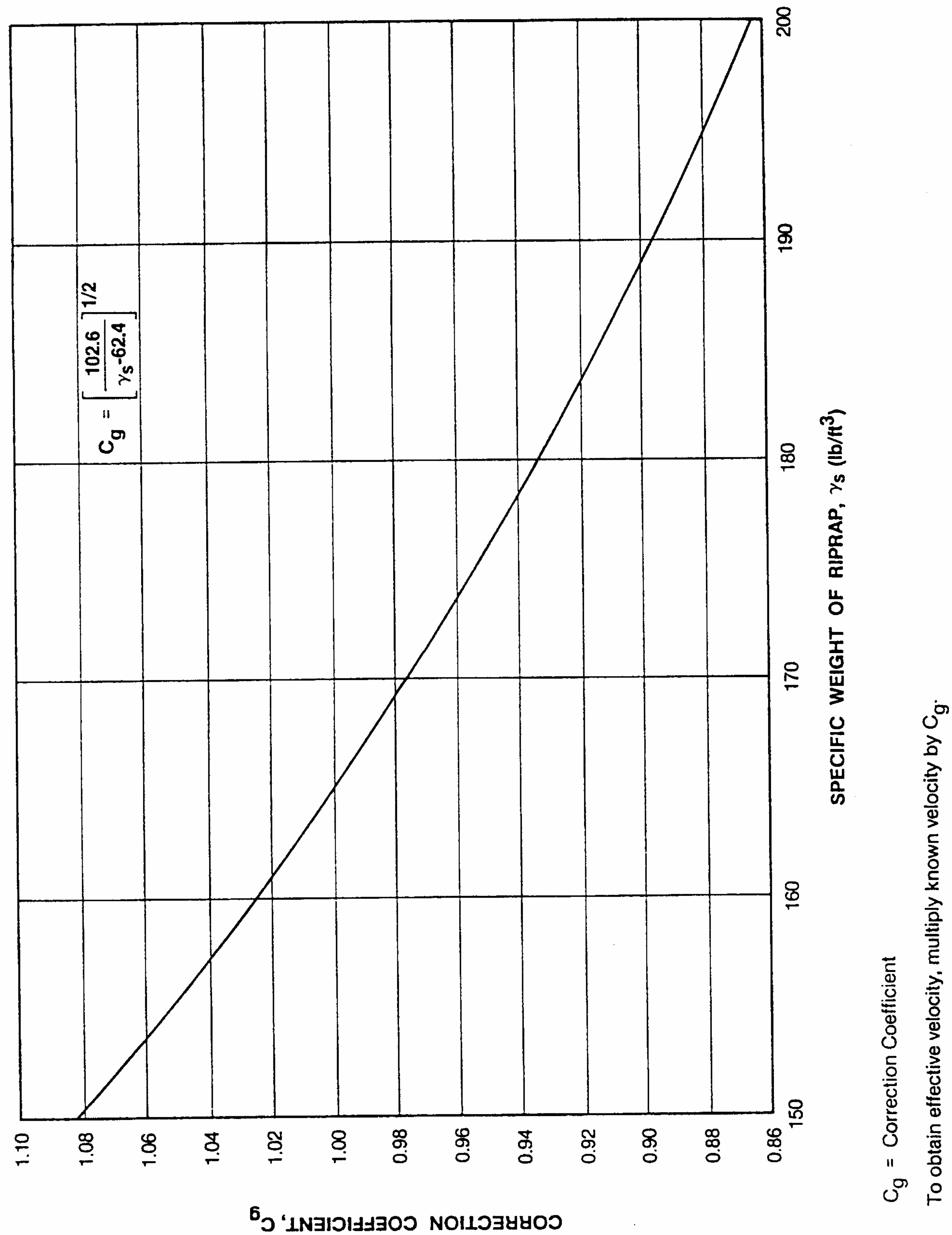
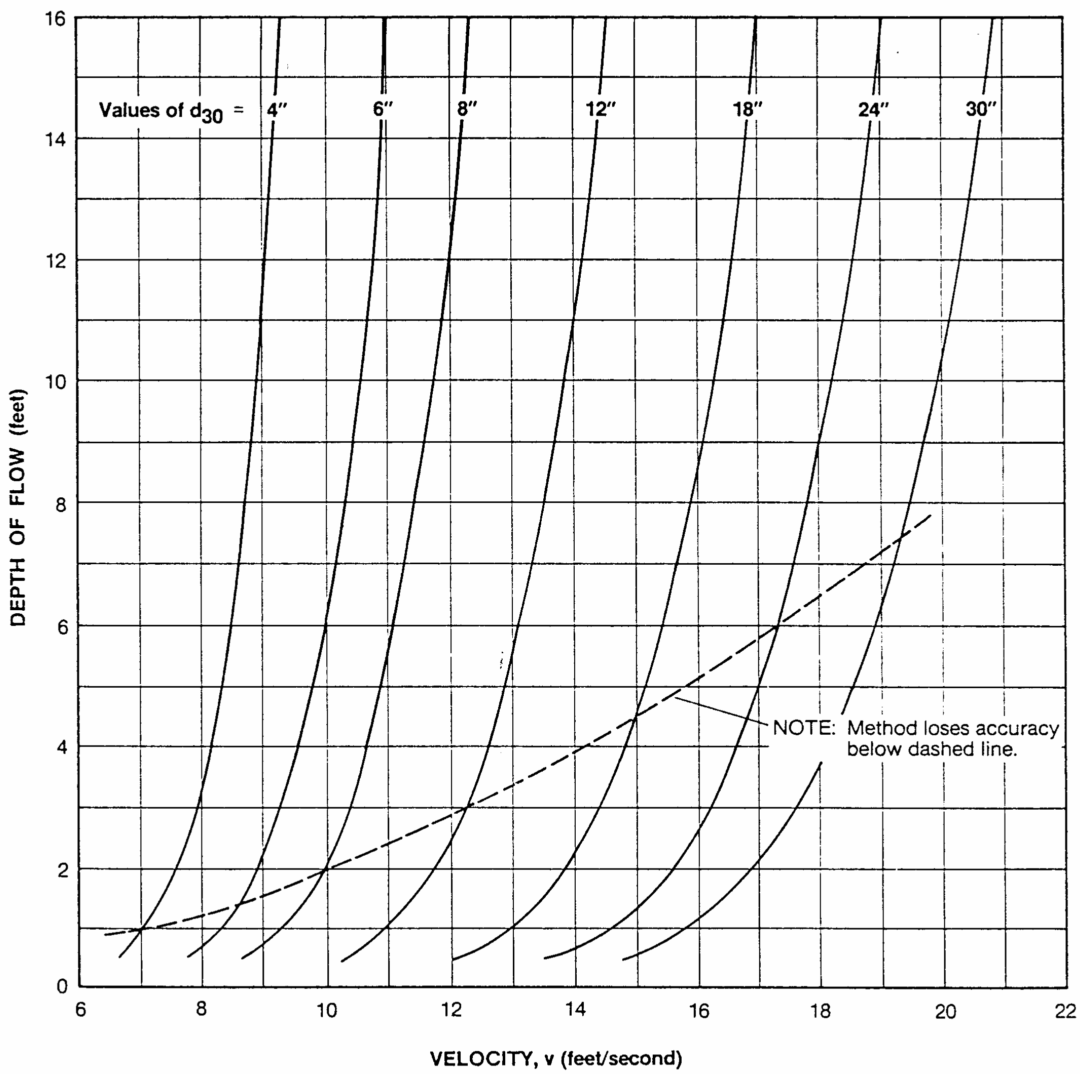


Figure 4.4-8 Riprap Lining Specific Weight Correction Coefficient
(Source: Nashville Storm Water Management Manual, 1988)



Reference: Reese (1988).

Figure 4.4-9 Riprap Lining d_{30} Stone Size – Function of Mean Velocity and Depth

Step 5 Determine available riprap gradations. A well graded riprap is required. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$\mathbf{W = 0.5236\ SW_s\ d^3} \quad \mathbf{(4.4.14)}$$

where:

W = stone weight (lbs)

d = selected stone diameter (ft)

SW_s = specific weight of stone (lbs/ft^3)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50% for underwater placement.

Step 6 If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 4.4-10 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

Step 7 Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

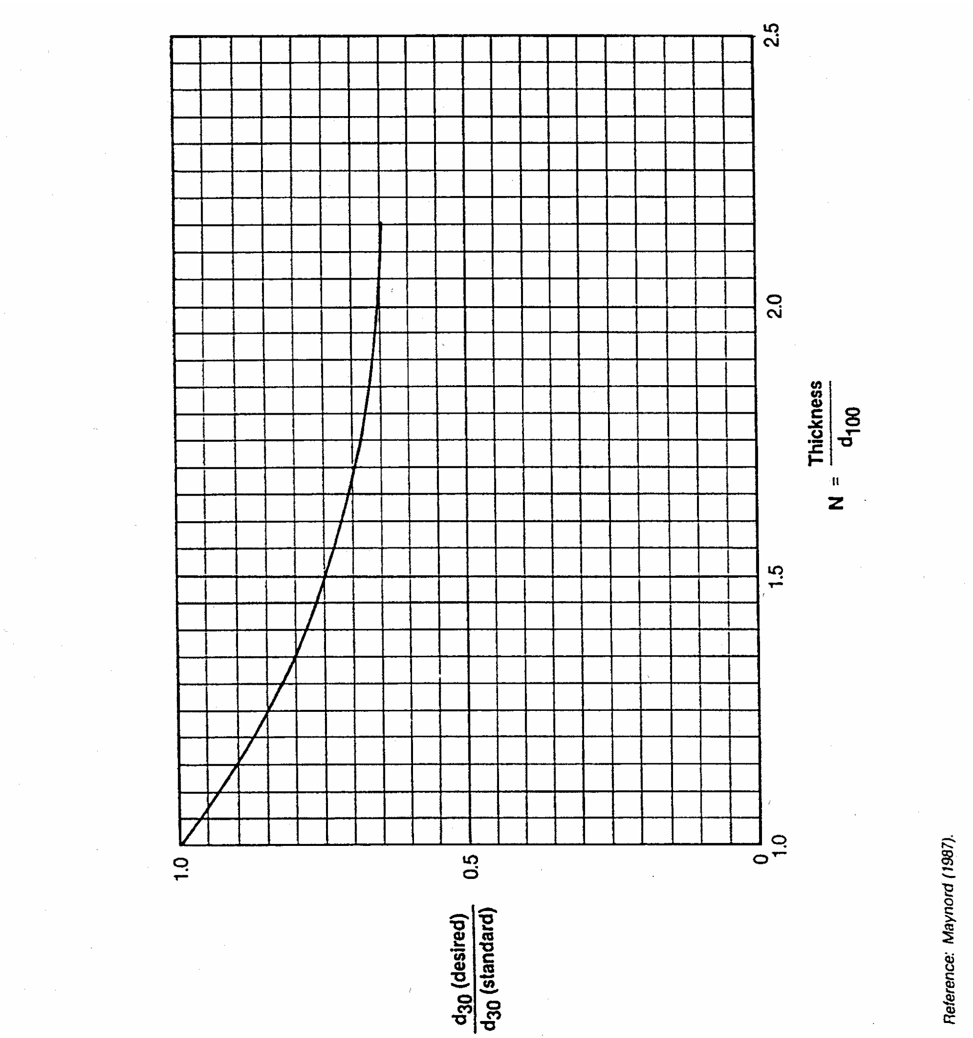


Figure 4.4-10 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3
(Source: Maynord, 1987)

4.4.8.3 Method #2: Gregory

The following procedure is based on excerpts from a paper prepared by Garry Gregory (June, 1987) and has been widely used in the Dallas-Fort Worth area for riprap design.

Procedure

Following are the steps in the procedure for riprap design using the method by Gregory:

Step 1 Calculate the boundary shear (tractive stress or tractive force) by:

$$\mathbf{T_o = \Upsilon_w RS} \qquad \textbf{(4.4.15)}$$

where:

- T_o = average tractive stress on channel bottom (lb/ft^2)
- Υ_w = unit weight of water (62.4 lb/ft^3)
- R = hydraulic radius of channel (ft)
- S = slope of energy gradient (ft/ft)

$$\mathbf{T_o' = T_o((1\text{-}\sin^2\Phi)/\sin^2\Theta)^{0.5}} \qquad \textbf{(4.4.16)}$$

where:

- T_o' = average tractive stress on channel side slopes (lb/ft^2)
- Φ = angle of side slope with the horizontal
- Θ = angle of repose of riprap (approximately 40°)

The greater value of T_o or T_o' governs.

Step 2 Determine the tractive stress in a bend in the channel by:

$$\mathbf{T_b = 3.15T(r/w)^{-0.5}} \qquad \textbf{(4.4.17)}$$

where:

- T_b = local tractive stress in the bend (lb/ft^2)
- T = the greater of T_o or T_o' from equations 4.4.15 and 4.4.16
- r = center-line radius of the bend (ft)
- w = water surface width at upstream end of bend (ft)

Step 3 Determine D_{50} size of riprap stone (size at which 50% of the gradation is finer weight) from:

$$\mathbf{D_{50} = T/0.04(\Upsilon_s\text{-}\Upsilon_w)} \qquad \textbf{(4.4.18)}$$

where:

- D_{50} = required average size of riprap stone (ft)
- T = the greater of T_o or T_o' from equations 4.4.15 and 4.4.16
- Υ_s = saturated surface dry (SSD) unit weight of stone (lb/ft^3)
- Υ_w = unit weight of water (62.4 lb/ft^3)

Step 4 Select minimum riprap thickness required from grain size curves in Figures 4.4-11 through 4.4-16. Select from smaller side of band at 50% finer gradation.

Step 5 Select riprap gradations table (Figures 4.4-17 and 4.4-18) based on riprap thickness selected in Step 4.

Step 6 Select bedding thickness from grain size curves in Figures 4.4-11 through 4.4-16, which was used to select the riprap thickness in Step 4. Note: The bedding thicknesses included in Figures 4.4-11 through 4.4-16 are based on using a properly designed geotextile underneath the bedding. If a geotextile is not used, the bedding thickness must be increased to a minimum of 9 inches for 24 inch and 30 inch riprap and a minimum of 12 inches for the 36 inch riprap.

Step 7 To provide stability in the riprap layer the riprap gradations should meet the following criteria for GRADATION INDEX:

$$\textbf{GRADATION INDEX: } [D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5 \qquad \textbf{(4.4.19)}$$

where: D_{85}, D_{50}, and D_{15} are the riprap grain sizes (mm) of which 85%, 50%, and 15% respectively are finer by weight.

Step 8 To provide stability of the bedding layer the bedding should meet the following filter criteria with respect to the riprap:

$$D_{15}/d_{85} < 5 < D_{15}/d_{15} < 40 \qquad \textbf{(4.4.20)}$$

$$D_{50}/d_{50} < 40 \qquad \textbf{(4.4.21)}$$

where: D refers to riprap sizes, and d refers to bedding sizes, both in mm.

Step 9 The geotextile underneath the bedding should be designed as a filter to the soil.

Step 10 Typical riprap design sections are shown in Figures 4.4-19 and 4.4-20, from the USACE publication EM1110-2-160.

4.4.8.4 Culvert Outfall Protection

The following procedure is used to design riprap for protection at culvert outfalls.

Step 1 Determine D_{50} size of riprap determined from:

$$D_{50} = \sqrt{V}/[1.8\sqrt{(2g(\Upsilon_s-\Upsilon_w)/\Upsilon_w)}] \qquad \textbf{(4.4.22)}$$

Step 2 Select riprap and bedding from Figures 4.4-11 through 4.4-16 using D_{50} from equation 4.4.22.

Step 3 Select gradations from tables in Figures 4.4-17 and 4.4-18

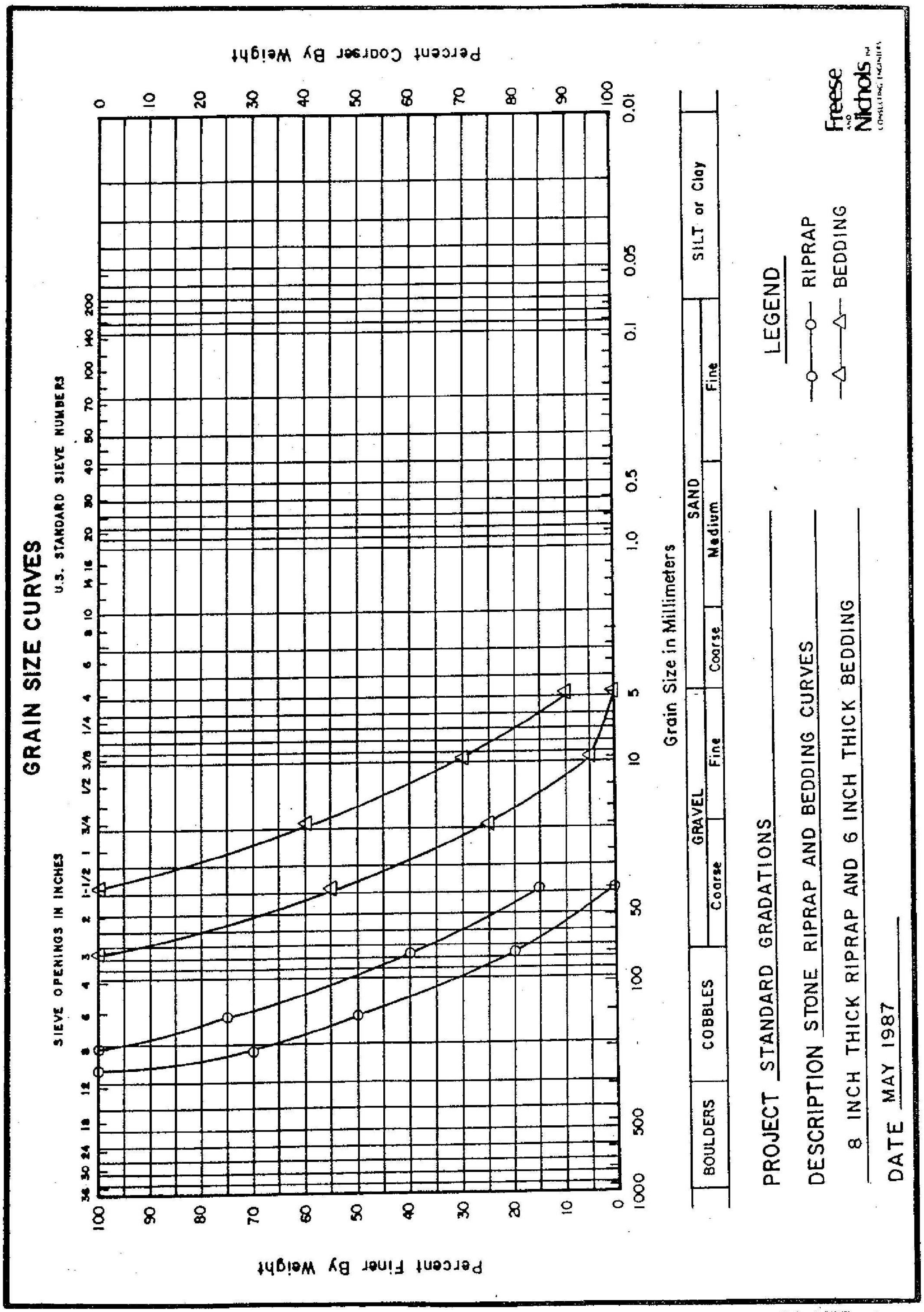


Figure 4.4-11 Grain Size Curve for 8" Riprap and 6" Bedding

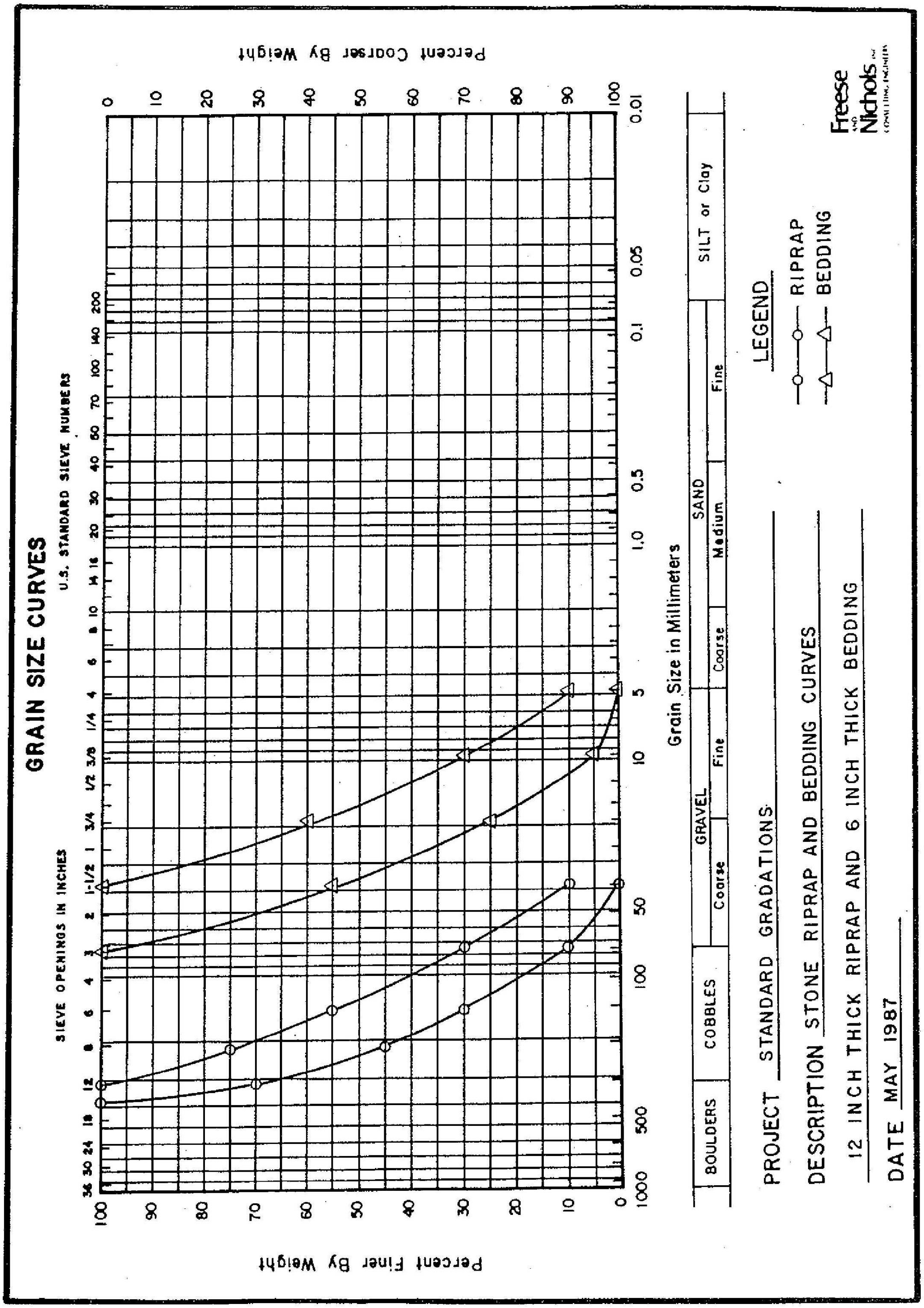


Figure 4.4-12 Grain Size Curve for 12" Riprap and 6" Bedding

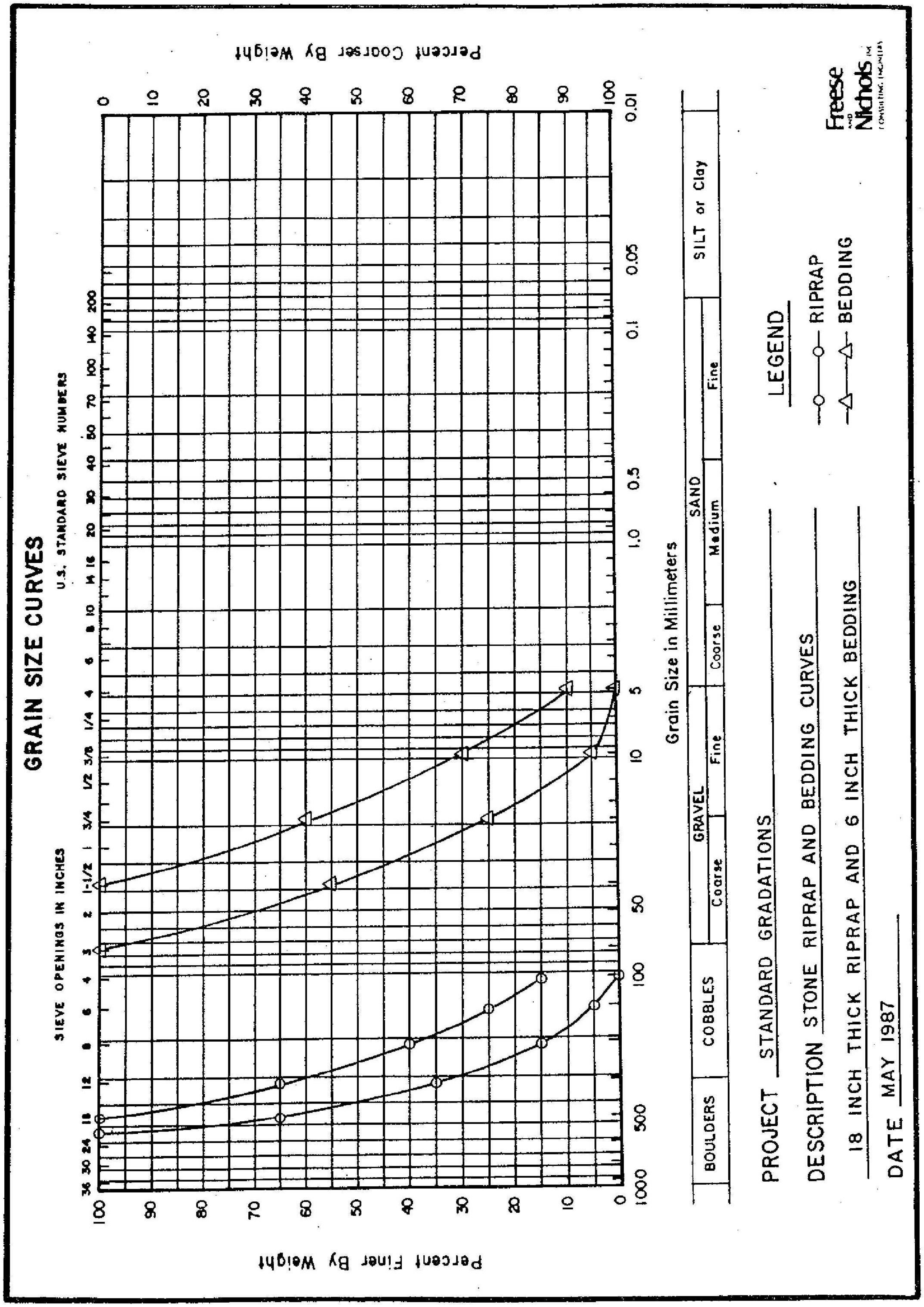


Figure 4.4-13 Grain Size Curve for 18" Riprap and 6" Bedding

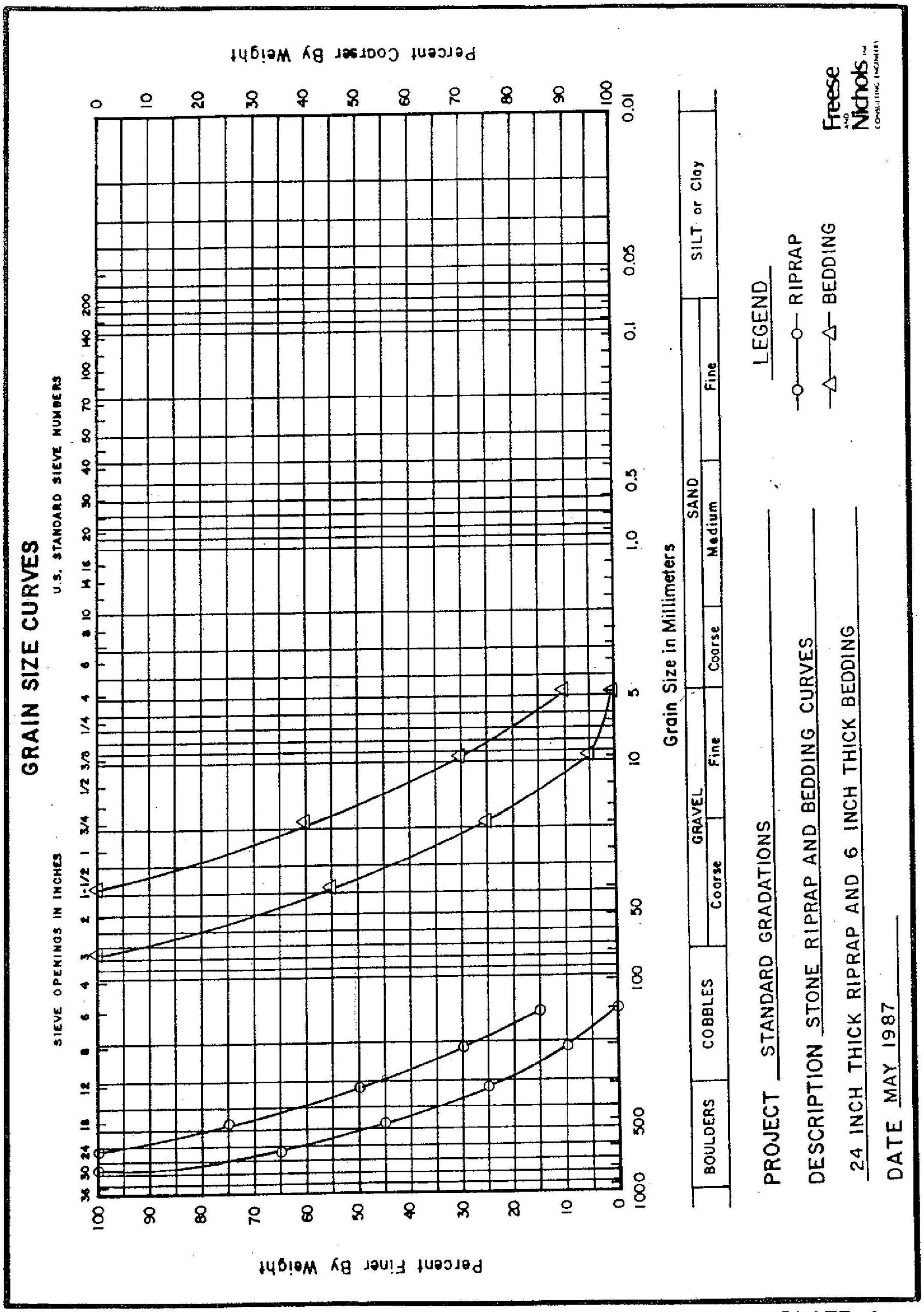


Figure 4.4-14 Grain Size Curve for 24" Riprap and 6" Bedding

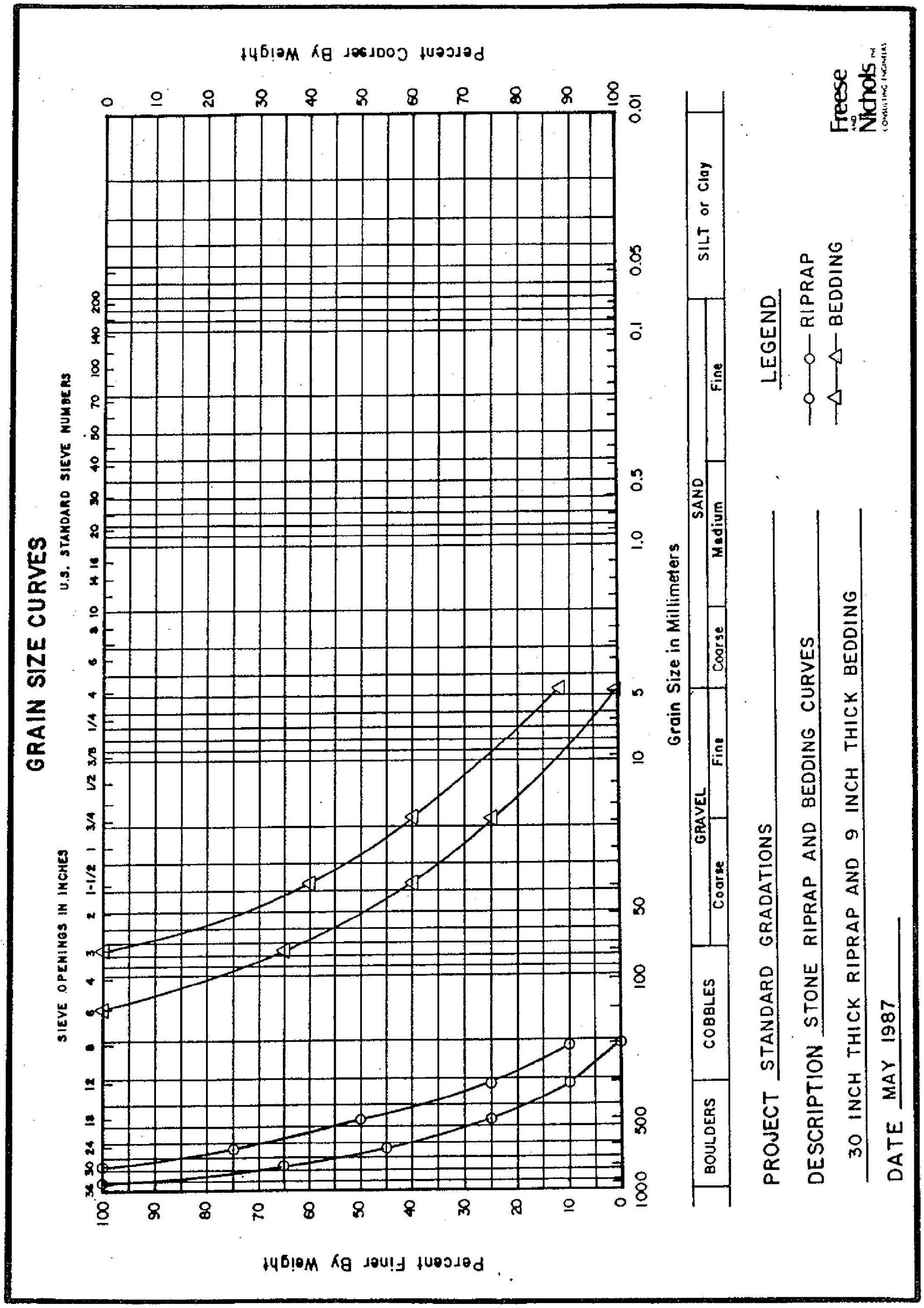


Figure 4.4-15 Grain Size Curve for 30” Riprap and 9” Bedding

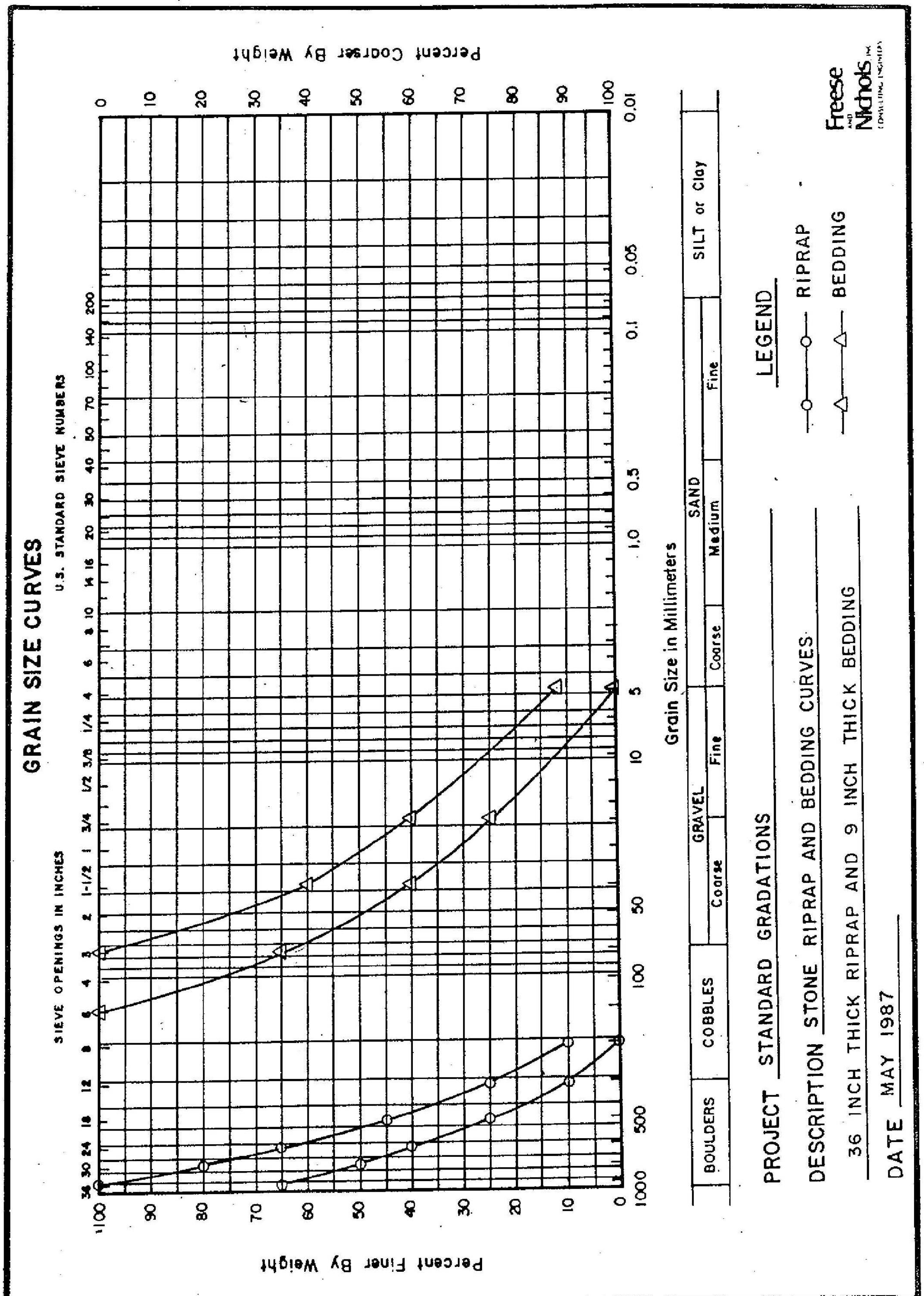


Figure 4.4-16 Grain Size Curve for 36" Riprap and 9" Bedding

RIPRAP GRADATIONS — 12" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
15 INCH	100
12 INCH	70 – 100
8 INCH	45 – 75
6 INCH	30 – 55
3 INCH	10 – 30
1-1/2 INCH	0 – 10

RIPRAP GRADATIONS — 8" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
10 INCH	100
8 INCH	70 – 100
6 INCH	50 – 75
3 INCH	20 – 40
1-1/2 INCH	0 – 15

BEDDING GRADATIONS — 6" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
3 INCH	100
1-1/2 INCH	55 – 100
3/4 INCH	25 – 60
3/8 INCH	5 – 30
No. 4	0 – 10

BEDDING GRADATIONS — 9" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
6 INCH	100
3 INCH	65 – 100
1-1/2 INCH	40 – 60
3/4 INCH	25 – 40
No. 4	0 – 12

Figure 4.4-17 Riprap Gradation Tables for 6", 8", 9", and 12" Thickness of Riprap

RIPRAP GRADATIONS

36" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
44 INCH	100
36 INCH	65 – 100
30 INCH	50 – 80
18 INCH	25 – 45
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS

30" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
36 INCH	100
30 INCH	65 – 100
24 INCH	45 – 75
18 INCH	25 – 50
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS

24" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65 – 100
18 INCH	45 – 75
12 INCH	25 – 50
8 INCH	10 – 30
6 INCH	0 – 15

RIPRAP GRADATIONS

18" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
21 INCH	100
18 INCH	65 – 100
12 INCH	35 – 65
8 INCH	15 – 40
6 INCH	5 – 25
4 INCH	0 – 15

Figure 4.4-18 Riprap Gradation Tables for 18", 24", 30", and 36" Thickness of Riprap

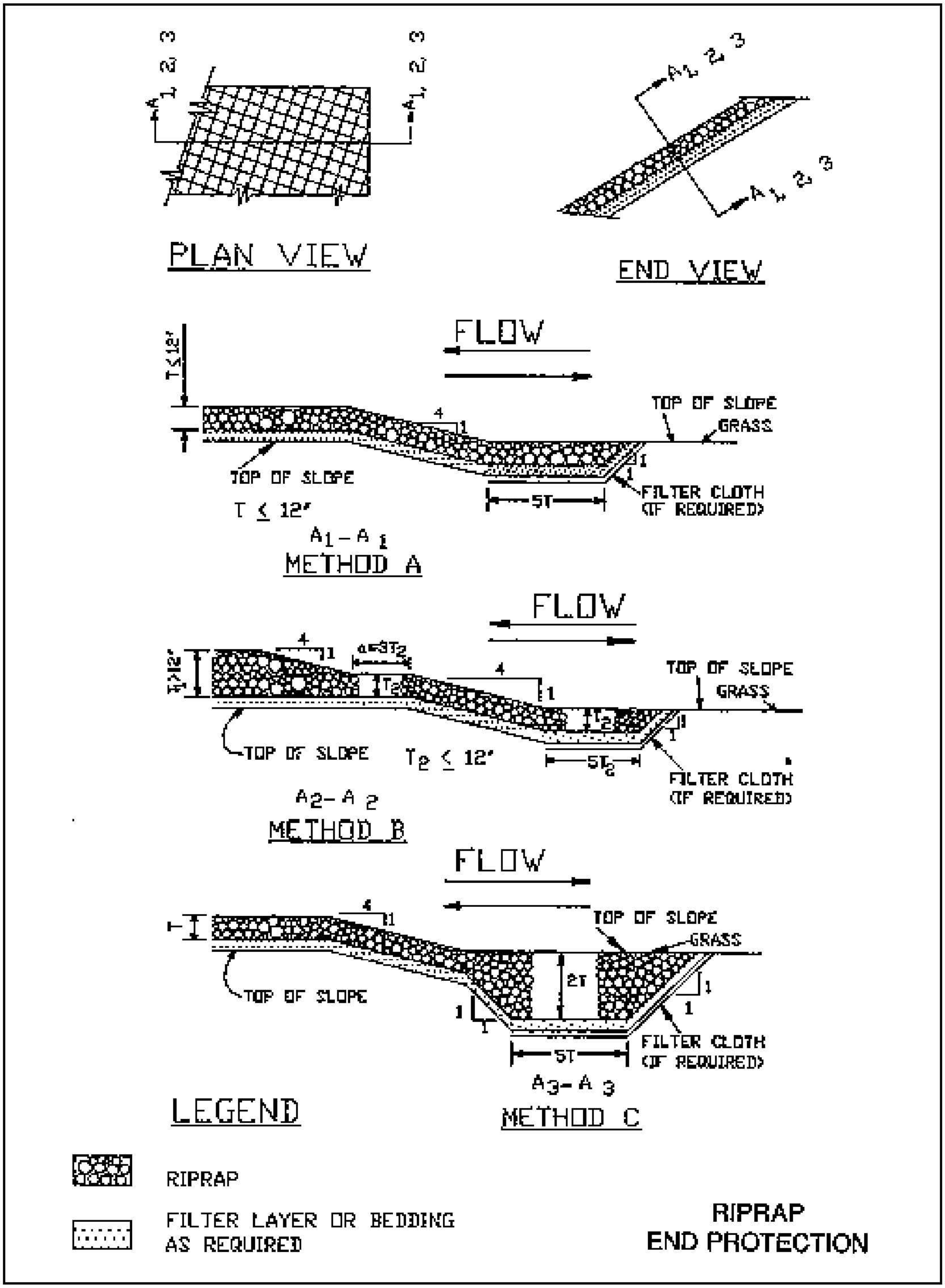


Figure 4.4-19 Typical Riprap Design Cross Sections

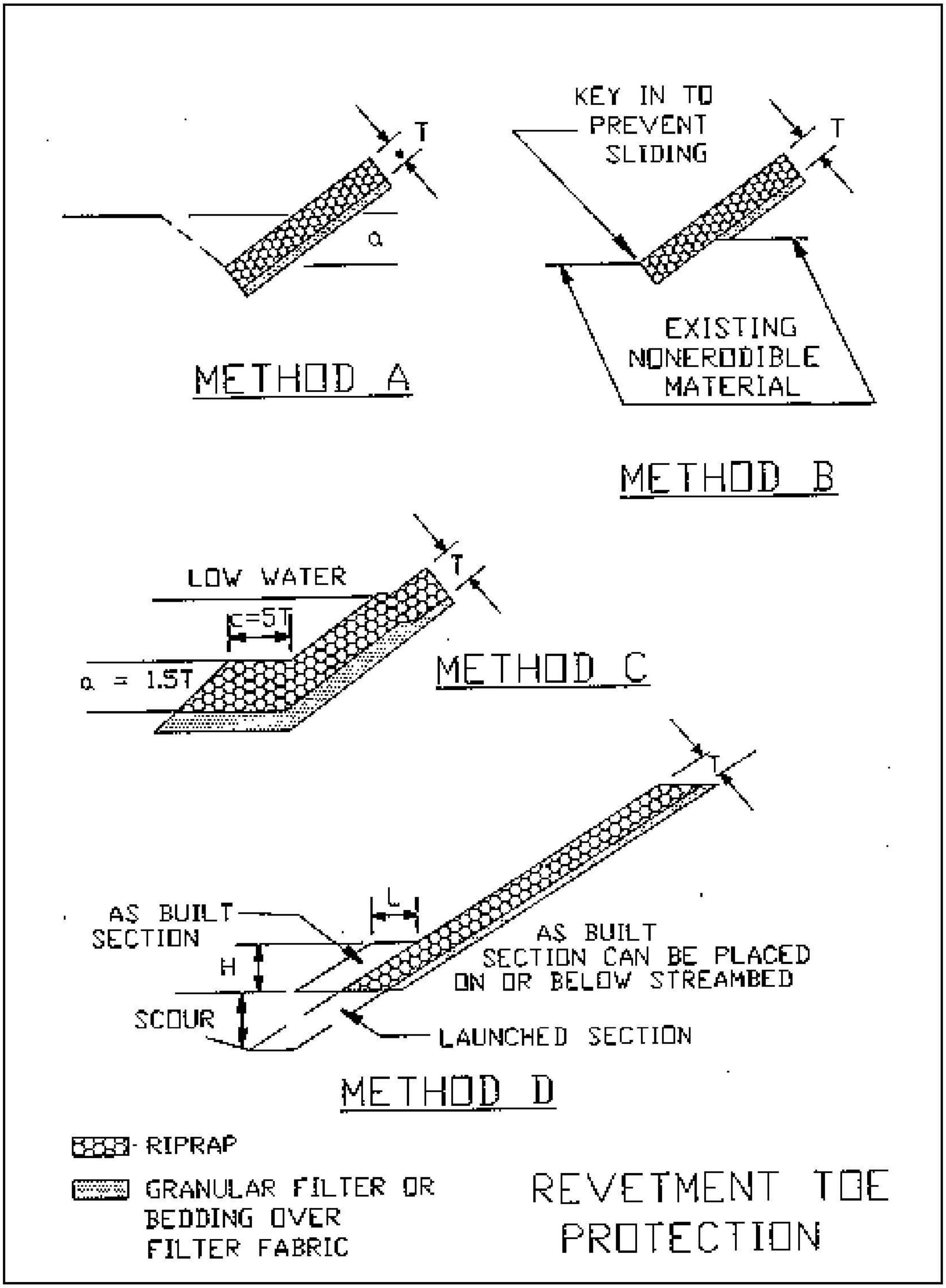


Figure 4.4-20 Typical Riprap Design for Revetment Toe Protection

4.4.9 Gabion Design

4.4.9.1 Introduction

Gabions come in three basic forms, the gabion basket, gabion mattress, and sack gabion. All three types consist of wire mesh baskets filled with cobble or small boulder material. The fill normally consists of rock material but other materials such as bricks have been used to fill the baskets. The baskets are used to maintain stability and to protect streambanks and beds.

The difference between a gabion basket and a gabion mattress is the thickness and the aerial extent of the basket. A sack gabion is, as the name implies, a mesh sack that is filled with rock material. The benefit of gabions is that they can be filled with rocks that would individually be too small to withstand the erosive forces of the stream. The gabion mattress is shallower (0.5 to 1.5 ft) than the basket and is designed to protect the bed or banks of a stream against erosion.

Gabion baskets are normally much thicker (about 1.5 to 3 ft) and cover a much smaller area. They are used to protect banks where mattresses are not adequate or are used to stabilize slopes (Figure 4.4-21), construct drop structures, pipe outlet structures, or nearly any other application where soil must be protected from the erosive forces of water. References to gabions in this manual refer generally to both mattresses and baskets. Sack gabions are rarely used in the United States and are not discussed.

Figure 4.4-21 Gabion Baskets Installed for Slope Stabilization along a Stream

Gabion baskets can be made from either welded or woven wire mesh. The wire is normally galvanized to reduce corrosion but may be coated with plastic or other material to prevent corrosion and/or damage to the wire mesh containing the rock fill. New materials such as Tensar, a heavy-duty polymer plastic material, have been used in some applications in place of the wire mesh. If the wire baskets break, either through corrosion, vandalism, or damage from debris or bed load, the rock fill in the basket can be lost and the protective value of the method endangered. Gabions are often used where available rock size is too small to withstand the erosive and tractive forces present at a project site. The available stone size may be too small due to the cost of transporting larger stone from remote sites, or the desire to have a project with a smoother appearance than obtained from riprap or other methods. Gabions also require about one third the thickness of material when compared to riprap designs. Riprap is often preferred, however, due to the low labor requirements for its placement.

The science behind gabions is fairly well established, with numerous manufacturers providing design methodology and guidance for their gabion products. Dr. Stephen T. Maynord of the U.S. Army Corps of Engineers Research and Development Center in Vicksburg, Mississippi, has also conducted research to develop design guidance for the installation of gabions. Two general methods are typically used to determine the stability of gabion baskets in stream channels, the critical shear stress calculation and the critical velocity calculation. A software package known as CHANLPRO has been developed by Dr. Maynord (Maynord et al. 1998).

Manufacturers have generated extensive debate regarding the use and durability of welded wire baskets versus woven wire baskets in project design and construction. Project results seem to indicate that performance is satisfactory for both types of mesh.

The rocks contained within the gabions provide substrates for a wide variety of aquatic organisms. Organisms that have adapted to living on and within the rocks have an excellent home, but vegetation may be difficult to establish unless the voids in the rocks contained within the baskets are filled with soil or a planting bed mixture.

If large woody vegetation is allowed to grow in the gabions, there is a risk that the baskets will break when the large woody vegetation is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The possibility of damage must be weighed against the desirability of vegetation on the area protected by gabions and the stability of the large woody vegetation. If large woody vegetation is kept out of the baskets, grasses and other desirable vegetation types may be established and provide a more aesthetic and ecologically desirable project than gabions alone.

4.4.9.2 Design

Primary design considerations for gabions and mattresses are: 1) foundation stability; 2) sustained velocity and shear-stress thresholds that the gabions must withstand; and 3) toe and flank protection. The base layer of gabions should be placed below the expected maximum scour depth. Alternatively, the toe can be protected with mattresses that will fall into any scoured areas without compromising the stability of the bank or bed protection portion of the project. If bank protection does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

The use of a filter fabric behind or under the gabion baskets to prevent the movement of soil material through the gabion baskets is an extremely important part of the design process. This migration of soil through the baskets can cause undermining of the supporting soil structure and failure of the gabion baskets and mattresses.

Primary Design Considerations

The major consideration in the design of gabion structures is the expected velocity at the gabion face. The gabion must be designed to withstand the force of the water in the stream.

Since gabion mattresses are much shallower and more subject to movement than gabion baskets, care should be taken to design the mattresses such that they can withstand the forces applied to them by the water. However, mattresses have been used in application where very high velocities are present and have performed well. But, projects using gabion mattresses should be carefully designed.

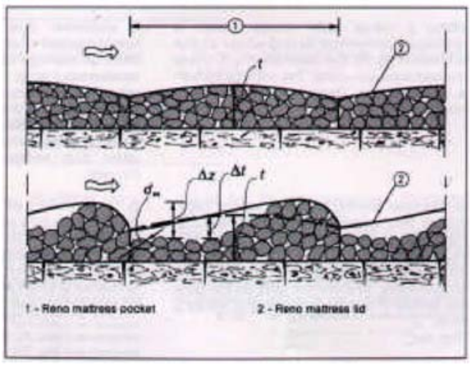
The median stone size for a gabion mattress can be determined from the following equation:

$$\mathbf{d_m = S_f C_s C_v d[(\Upsilon_w/(\Upsilon_s - \Upsilon_w))^{0.5}(V/\sqrt{(gdK_1)})]^{2.5}} \qquad \mathbf{(4.4.23)}$$

where:

d_m	=	average rock diameter in gabions (ft)
S_f	=	safety factor (1.1 minimum)
C_s	=	stability coefficient (usually 0.1)
C_v	=	velocity distribution coefficient = 1.283-0.2log(r/w) (minimum of 1.0) and equals 1.25 at end of dikes and concrete channels
r	=	center-line bend radius of main channel flow (ft)
w	=	water surface width of main channel (ft)
d	=	local flow depth at V (ft)
g	=	acceleration due to gravity (32.2 ft/s^2)
V	=	depth-averaged velocity (ft/s)
K_1	=	side slope correction factor (Table 4.4-8)
Υ_w	=	unit weight of water (62.4 lb/ft^3)
Υ_s	=	unit weight of stone (lb/ft^3)

Table 4.4-8 Values of K_1 for various Side Slopes to be used in Equation 4.4-23	
Side Slope	**K_1**
1V : 1H	0.46
1V : 1.5H	0.71
1V : 2H	0.88
1V : 3H	0.98
<1V : 4H	1.0



Equation 4.4-23 was developed to design stone size such that the movement of filler stone in the mattresses is prevented. This eliminates deformation that can occur when stone sizes are not large enough to withstand the forces of the water. The result of mattresses deformation is stress on the basket wire and increases in resistance to flow and the likelihood of basket failure. The upper portion of Figure 4.4-22 shows an undeformed gabion, while the lower portion shows how gabions deform under high-velocity conditions. Maccaferri Gabions gives guidance on sizing stone and allowable velocities for gabion baskets and mattresses, shown in Table 4.4-9.

Figure 4.4-22 Gabion Mattress Showing Deformation of Mattress Pockets under High Velocities

Table 4.4-9 Stone Sizes and Allowable Velocities for Gabions					
Type	**Thickness (ft)**	**Filling Stone Range**	**D_{50}**	**Critical Velocity**	**Limit Velocity**
Mattress	0.5	3 – 4”	3.4”	11.5	13.8
	0.5	3 – 6”	4.3”	13.8	14.8
	0.75	3 – 4”	3.4”	14.8	16
	0.75	3 – 6”	4.7”	14.8	20
	1.0	3 – 5”	4”	13.6	18
	1.0	4 – 6”	5”	16.4	21
Basket	1.5	4 – 8”	6”	19	24.9
	1.5	5 – 10”	7.5”	21	26.2

When the data in Table 4.4-9 are compared to Equation 4.4-22, if V = 11.5, C_s = 0.1, C_v = 1.0, K_1 = 0.71, ,γ_s = 150 lb/ft^3 and S_f = 1.1, the local flow depth must be on the order of 25 ft in order to arrive at the stone diameter of 3.4 in. shown in Table 4.4-9. Designers should use Equation 4.4-23 to take the depth of flow into account. Table 4.4-9 does, however, give some general guidelines for fill sizes and is a quick reference for maximum allowable velocities.

Maccaferri also gives guidance on the stability of gabions in terms of shear stress limits. The following equation gives the shear for the bed of the channel as:

$$\tau_b = \Upsilon_w Sd \qquad (4.4.24)$$

where S = bed or water surface slope through the reach (ft/ft)

The bank shear is generally taken as 75 percent of the bed shear, i.e.,

$$\tau_m = 0.75\ \tau_m \qquad (4.4.25)$$

These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\Upsilon_s - \Upsilon_w)d_m \qquad (4.4.26)$$

with critical shear stress for the banks given as:

$$\tau_s = \tau_c \sqrt{(1-(\sin^2\Theta/0.4304))} \qquad (4.4.27)$$

where Θ = angle of the bank rotated up from horizontal.

A design is acceptable if $\tau_b < \tau_c$ and $\tau_m < \tau_s$. If either $\tau_b > \tau_c$ or $\tau_m > \tau_s$ then a check must be made to see if they are less than 120 percent of τ_b and τ_s. If the values are less than 120 percent of τ_b and τ_s the gabions will not be subject to more than what Maccaferri defines as "acceptable" deformation. However it is recommended that the stone size be increased to limit deformation if possible.

Research has indicated that stone in the gabion mattress should be sized such that the largest stone diameter is not more than about two times the diameter of the smallest stone diameter and the mattress should be at least twice the depth of the largest stone size. The size range should, however, vary by about a factor of two to ensure proper packing of the stone material into the gabions. Since the mattresses normally come in discrete sizes, i.e. 0.5, 1.0, and 1.5 ft in depth, normal practice is to size the stone and then select the basket depth that is deep enough to be at least two times the largest stone diameter. The smallest stone should also be sized such that it cannot pass through the wire mesh.

Stability of Underlying Bed and Bank Material

Another critical consideration is the stability of the gabion foundation. This includes both geotechnical stability and the resistance of the soil under the gabions to the erosive forces of the water moving through the gabions. If there is any question regarding the stability of the foundation, i.e. possibility of rotational failures, slip failures, etc., a qualified geotechnical engineer should be consulted prior to and during the design of the bank/channel protection. Several manufacturers give guidance on how to check for geotechnical failure.

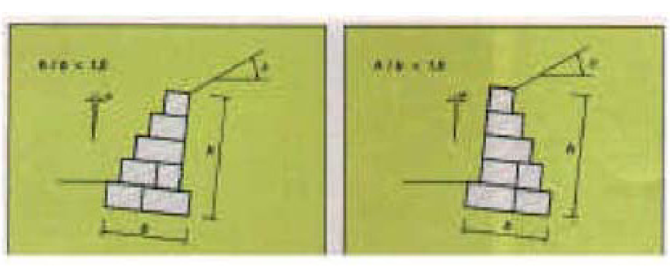


Figure 4.4-23 Front-step and Rear-step Gabion Layout

One of the critical factors in determining stability is the velocity of the water that passes through the gabions and reaches the soil behind the gabion. The water velocity under the filter fabric, i.e. water that moves through the gabions and filter fabric, is estimated to be one-fourth to one-half of the velocity at the mattress/filter interface.

The velocity at the mattress/filter interface, V_b, is estimated to be

$$V_b = 1.486\sqrt{S}(d_m/2)^{2/3}/n_f \qquad (4.4.28)$$

where n_f = 0.02 for filter fabric, 0.022 for gravel filter material

If the underlying soil material is not stable, additional filter material must be installed under the gabions to ensure soil stability.

The limit for velocity on the soil is different for each type of soil. The limit for cohesive soils is obtained from a chart, while maximum allowable velocities for other soil types are obtained by calculating V_e, the maximum velocity allowable at the soil interface, and comparing it to V" the residual velocity on the bed, i.e. under the gabion mattress and under the filter fabric. V_e for loose soils is equal to $16.1d^2$ while V_f is calculated by:

$$\mathbf{V_f = 1.486S\sqrt{V_a}(d_m/2)^{2/3}/n_f} \qquad \textbf{(4.4.29)}$$

where V_a = average channel velocity (ft/s)

If V_f is larger than two to four times V_e, a gravel filter is required to further reduce the water velocity at the soil interface under the gabions until V_f is in an acceptable range. To check for the acceptability of the filter use the average gravel size for d_m in equation 4.4-28. If the velocity V_f is still too high, the gravel size should be reduced to obtain an acceptable value for V_f.

Other Design Considerations

It may be possible to combine gabions with less harsh methods of bank protection on the upper bank and still achieve the desired result of a stable channel. Provisions for large woody vegetation and a more aesthetically pleasing project may also be used on the upper banks or within the gabions. However, the stability of vegetation or other upper bank protection should be carefully analyzed to ensure stability of the upper bank area. A failure in the upper bank region can adversely affect gabion stability and lead to project failure.

4.4.10 Uniform Flow – Example Problems

Example 1 -- Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ft. Solve using Figure 4.4-2:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.
2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/s from the velocity scale.

Example 2 -- Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 ft/ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

1. From Table 4.4-3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.
2. Assume an n value of 0.035 and find the value of vR from Figure 4.4-1, vR = 5.4
3. Use equation 4.4.9 to calculate the value of R: R = 5.4/4 = 1.35 ft
4. Use equation 4.4.10 to calculate the value of vR:
   $vR = [1.49\ (1.35)^{5/3}\ (0.015)^{1/2}]/0.035 = 8.60$
5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n Value	vR (Figure 4.4-1)	R (eq. 4.4.9)	vR (eq. 4.4.10)
0.035	5.40	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

6. Use Figure 4.4-3 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   Qn = (50) (0.040) = 2.0, S = 0.015
   For b = 10 ft, d = (10) (0.098) = 0.98 ft, b = 8 ft, d = (8) (0.14) = 1.12 ft

   Select:

   b = 10 ft, such that R is approximately 0.80 ft

   z = 3

   d = 1 ft

   v = 3.9 ft/s (equation 4.4.1)

   Fr = 0.76 (equation 4.4.8)

   Flow is subcritical

   Design capacity calculations for this channel are presented in Example 3 below.

Example 3 -- Grassed Channel Design Capacity

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.

Assume a depth of 1.0 ft and calculate the following (see Figure 4.4-5):

A = (b + zd) d = [10 + (3) (1)] (1) = 13.0 square ft

R = $[(b + zd)\ d]/\{b + [2d(1 + z^2)^{0.5}]\} = \{[10+(3)(1)]1\}/\{10+[(2)(1)(1+3^2)^{0.5}]\}$

R = 0.796 ft

Find the velocity: v = Q/A = 50/13.0 = 3.85 ft/s

Find the value of vR: vR = (3.85) (0.796) = 3.06

Using the vR product from Step 3, find Manning's n from Figure 4.4-1 for retardance Class C (n = 0.047)

Use Figure 4.4-2 or equation 4.4.1 to find the velocity for S = 0.015, R = 0.796, and n = 0.047: v = 3.34 ft/s

Since 3.34 ft/s is less than 3.85 ft/s, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 4.4-1)	Velocity (Eq. 4.4.1)
1.0	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.047	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 ft. Design data for the trapezoidal channel are summarized as follows:

   Vegetation lining = grass mixture, v_m = 4 ft/s

   Q = 50 cfs

   b = 10 ft, d = 1.3 ft, z = 3, S = 0.015
   Top width = (10) + (2) (3) (1.3) = 17.8 ft

   n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (equation 4.4.8)
   n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (equation 4.4.8)

Example 4 -- Riprap Design

A natural channel has an average bankfull channel velocity of 8 ft per second with a top width of 20 ft and a bend radius of 50 ft. The depth over the toe of the outer bank is 5 ft. Available stone weight is 170 lbs/ft^3. Stone placement is on a side slope of 2:1 (horizontal:vertical). Determine riprap size at the outside of the bend.

1. Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.
2. Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 4.4-7, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.
3. Determine the correction coefficient for the specific weight of 170 lbs from Figure 4.4-8 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.
4. Determine minimum d_{30} from Figure 4.4-9 or equation 4.4.13 as about 10 inches.
5. Use a gradation with a minimum d_{30} size of 10 inches.
6. *(Optional)* Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 4.4-10, this gradation would be acceptable if the blanket thickness was increased from the original d_{100} (diameter of the largest stone) thickness by 35% (a ratio of 1.35 on the horizontal axis).
7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 4.4-6.

4.4.11 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-RAS, developed by the U.S. Army Corps of Engineers and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step method (Chow, 1959, TxDOT, 2002). In the Direct Step method an increment of water depth is chosen, and the distance over which the depth change occurs is computed. This method is often used in association with culvert hydraulics. It is most accurate when the slope and depth increments are small. It is appropriate for prismatic channel sections which occur in most conduits, and can be useful when estimating both supercritical and subcritical profiles. For supercritical flow, the water surface profile is computed downstream. For subcritical flow, the water surface profile is computed upstream.

For an irregular nonuniform channel, the Standard Step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for Standard Step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity the designed waterway. Channel cross sections will be required at each location along the waterway where there are changes in channel shape or dimension, changes in the flowline slope, and changes in vegetation or channel lining. These sections are in addition to any sections necessary to define obstructions such as culverts, bridges, damns, energy dissipation features, or aerial crossings (pipelines). Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

4.4.12 Rectangular, Triangular and Trapezoidal Open Channel Design

4.4.12.1 Introduction

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial and error solution of Manning's Equation must be used. However, it is anticipated that available software programs will be the first choice for solving these design computations.

4.4.12.2 Description of Figures

Figures given in FHWA, HDS No. 3, 1973 and Atlanta Regional Commission, 2001 are for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft). A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but also between the inclined lines representing depth and slope.

The chart for a triangular cross section (see Figure 3.2-1) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V-shaped sections by following the instructions on the chart.

4.4.12.3 Instructions for Rectangular and Trapezoidal Figures

Figures such as Figure 4.4-24 provide a solution of the Manning equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the figure for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the figures can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity V from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

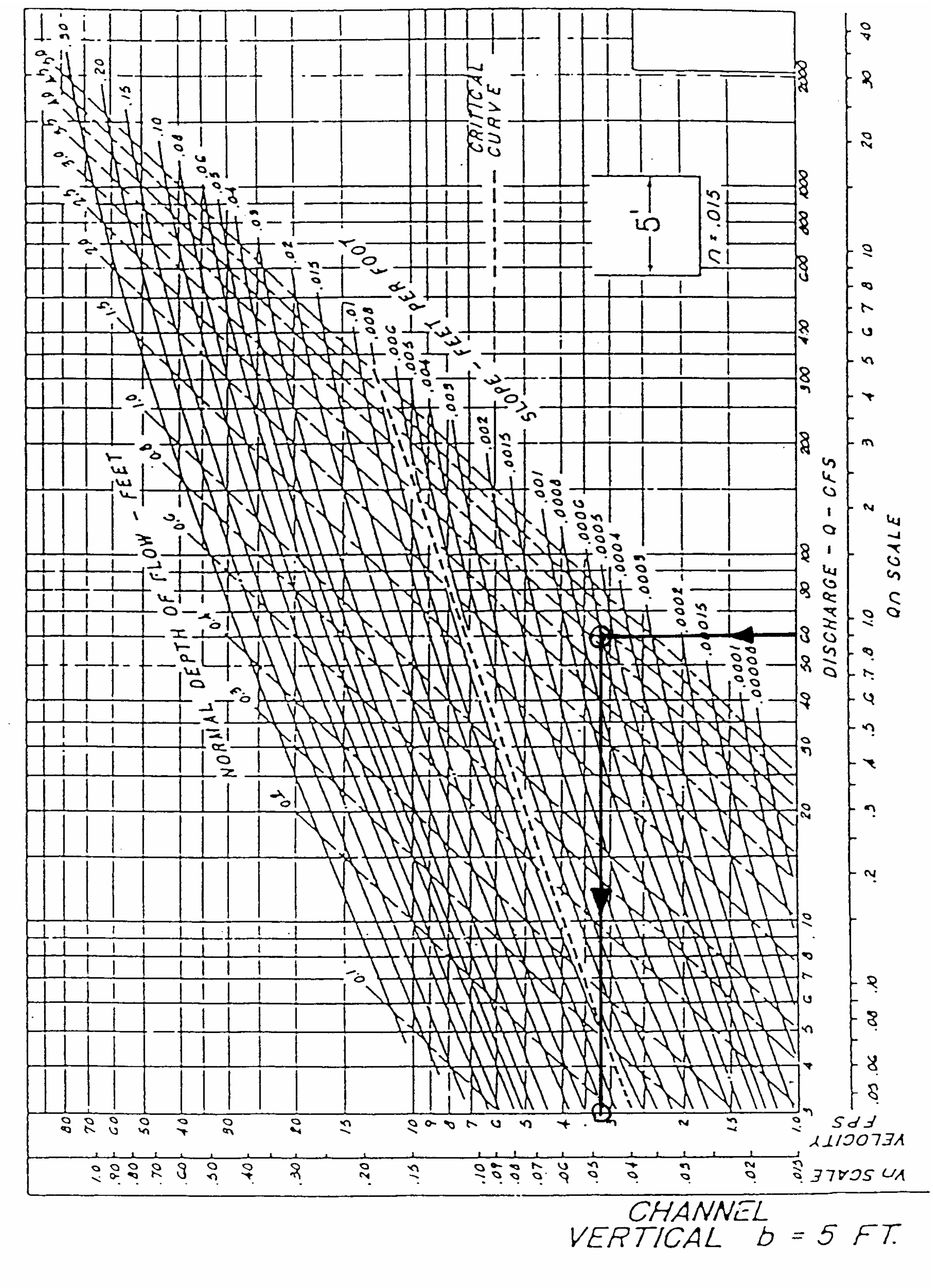
Example Design Problem 1

Given: A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure:

1. From subsection 4.4.12, select the rectangular figure for a 5-ft width (Figure 4.4-24).
2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read d_n = 3.7 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.
4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.



Source: Federal Highway Administration

Figure 4.4-24 Example Nomograph #1

Example Design Problem 2

Given: A trapezoidal channel with 2:1 side slopes and a 4 ft bottom width, with n = 0.030, 0.2% slope (S = 0.002), discharging 50 cfs.

Find: Depth, velocity, type flow.

Procedure:

1. Select the trapezoidal figure for b = 4 ft (see Figure 4.4-25).
2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d_n = 2.2 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/s, on the ordinate scale. The intersection lies below the critical curve, and the flow is therefore subcritical.

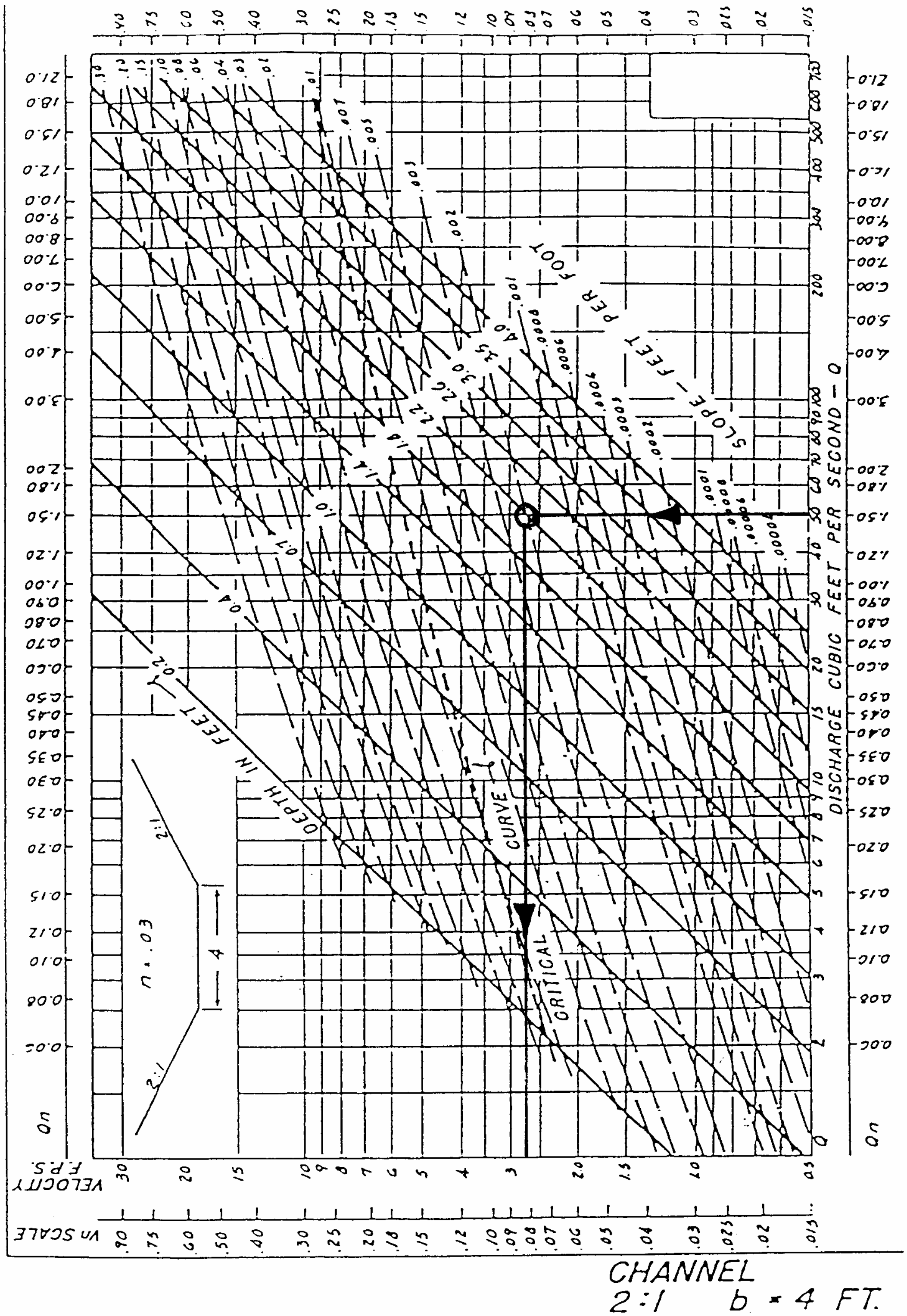
Example Design Problem 3

Given: A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, 0.5% slope (S = 0.005), discharging 80 cfs.

Find: Depth velocity and type of flow

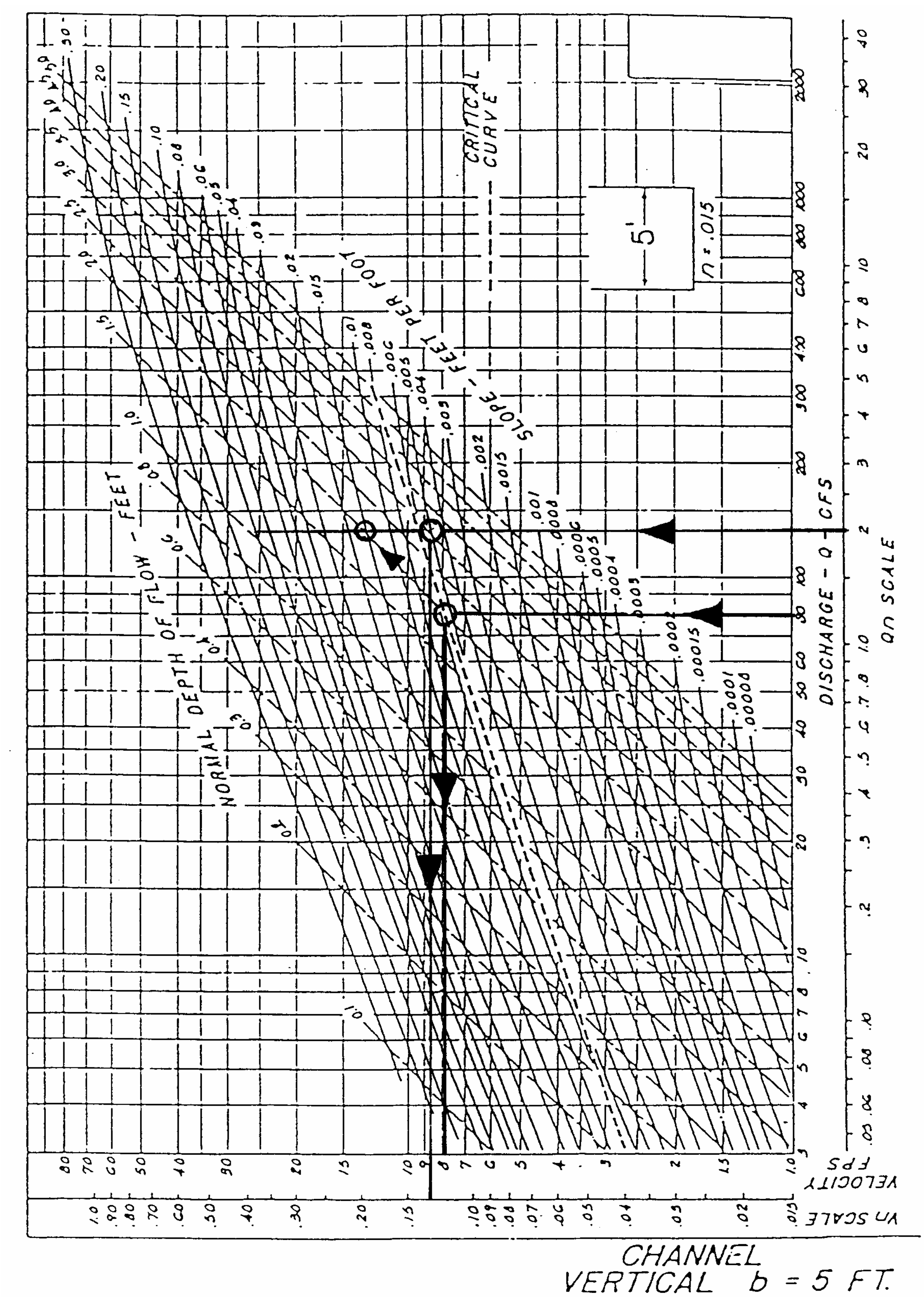
Procedure:

1. Select the rectangular figure for a 5 ft width (Figure 4.4-26).
2. Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.
3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read d_n = 3.1 ft.
4. Move horizontally from the intersection and read Vn = .13, then Vn/n = 0.13/0.025 = 5.2 ft/s.
5. Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 4.4-13, d_c = 2.0 ft and V_c = 7.9 ft/s. The normal velocity, 5.2 ft/s (from step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.
6. To determine the critical slope for Q = 80 cfs and n = 0.025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding d_c (2.0 ft) at this point. Follow along this d_c line to its intersection with a vertical line through Qn = 2.0 (step 2), at this intersection read the slope value S_c = 0.015.



Source: Federal Highway Administration

Figure 4.4-25 Example Nomograph #2



Source: Federal Highway Administration

Figure 4.4-26 Example Nomograph #3

4.4.12.4 Grassed Channel Figures

The Manning equation can be used to determine the capacity of a grass-lined channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass-lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass-lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

To aid in the design of grassed channels, the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in subsection 4.4.14. Following is a brief description of general design criteria, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial-and-error solution of the Manning equation must be used.

4.4.12.5 Description of Figures

A set of figures in FHWA, NDS No. 3, 1973 and Atlanta Regional Commission, 2001 are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (n) changes as higher velocities and/or greater depths change the condition of the grass. The effect of velocity and depth of flow on n is evaluated by the product of velocity and hydraulic radius V times R. The variation of Manning's n with the retardance (Table 4.4-6) and the product V times R is shown in Figure 4.4-1. As indicated in Table 4.4-6, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. Table 4.4-6 is used to determine retardance classification.

The grassed channel figures each have two graphs, the upper graph for retardance Class D and the lower graph for retardance Class C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic.

Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

4.4.12.6 Instructions for Grassed Channel Figures

The grassed channel figures like those in Figure 4.4-11 provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have length sufficient to establish uniform flow. The figures are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations: (1) selecting a section that has the capacity to carry the design discharge on the available slope and (2) checking the velocity in the channel to ensure that the grass lining will not be eroded. Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance Class C and the velocity using retardance Class D. The calculated velocity should then be checked against the permissible velocities listed in Tables 4.4-2 and 4.4-3. The use of the figures is explained in the following steps:

Step 1 Select the channel cross section to be used and find the appropriate figure.

Step 2 Enter the lower graph (for retardance Class C) on the figure with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is subcritical; if it is above, the flow is supercritical.

Step 3 To check the velocity developed against the permissible velocities (Tables 4.4-2 and 4.4-3), enter the upper graph on the same figure and repeat Step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

Example Design Problem 1

Given: A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4:1 side slopes, and a 4 ft bottom width on slope of 0.02 ft per foot (S=.02), discharging 20 cfs.

Find: Depth, velocity, type of flow, and adequacy of grass to prevent erosion

Procedure:

1. From subsection 4.4.13 select figure for 4:1 side slopes (see Figure 4.4-27).
2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S=0.02. At this intersection read d_n = 1.0 ft, and normal velocity V_n 2.6 ft/s.
3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance Class D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/s. This is about three-quarters of that listed as permissible, 4.0 ft/s in Table 4.4-3.

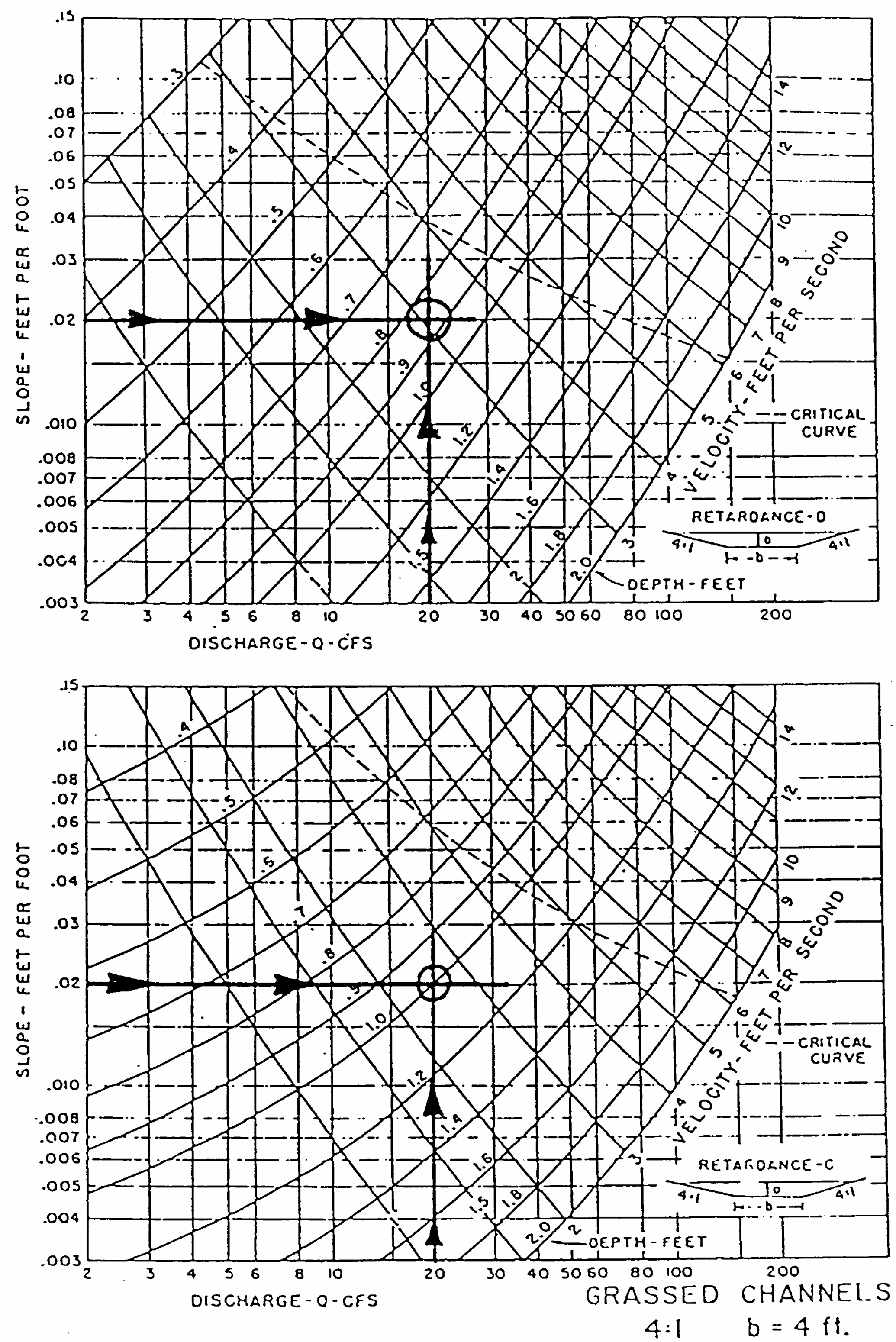
Example Design Problem 2

Given: The channel and discharge of Example 1.

Find: The maximum grade on which the 20 cfs could safely be carried

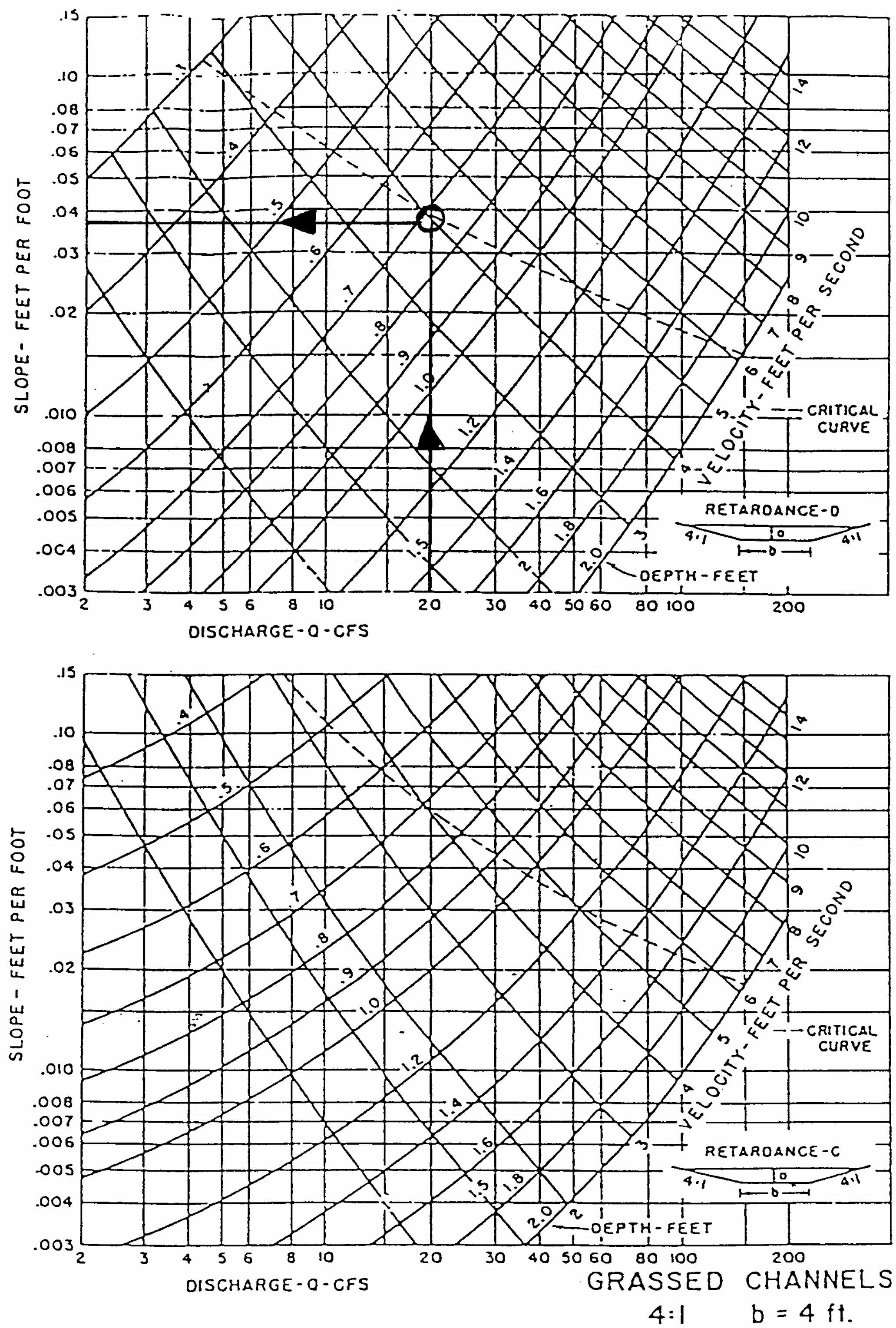
Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 4.4-3). On the upper graph of Figure 4.4-28 for short grass, the intersection of the 20 cfs line and the 4 ft/s line indicates a slope of 3.7% and a depth of 0.73 ft.



Source: Federal Highway Administration

Figure 4.4-27 Example Nomograph #4



Source: Federal Highway Administration

Figure 4.4-28 Example Nomograph #5

References

Atlanta Regional Commission, 2001, Georgia Stormwater Management Manual, Volume 2, Technical Handbook, First Edition, pages 4.4-40 through 4.4-73 (http://www.georgiastormwater.com/vol2/4-4.pdf).

Chow, V. T., 1959. Open Channel Hydraulics. McGraw-Hill Book Co. New York, New York.

French, R. H., 1985. Open Channel Hydraulics. McGraw-Hill Book Co. New York, New York.

Federal Highway Administration, 1989. Bridge Waterways Analysis Model (WSPRO), Users Manual, FHWA IP-89-027.

Gregory, G.T., June 1987. Stone Riprap Design Guidelines. Unpublished paper, Fort Worth, Texas.

Harza Engineering Company, 1972. Storm Drainage Design Manual. Prepared for the Erie and Niagara Counties Regional Planning Bd. Harza Engineering Company, Grand Island, NY.

Maynord, S. T., 1987. Stable Riprap Size for Open Channel Flows. Ph.D. Dissertation. Colorado State University, Fort Collins, CO.

Mays, L. W., ed, 1999. Hydraulic Design Handbook. McGraw-Hill, New York, New York.

Morris, J. R., 1984. A Method of Estimating Floodway Setback Limits in Areas of Approximate Study. In Proceedings of 1984 International Symposium on Urban Hydrology, Hydraulics and Sediment Control. Lexington, Kentucky: University of Kentucky.

Peterska, A. J., 1978. Hydraulic Design of Stilling Basins and Energy Dissipators. Engineering Monograph No. 25. U. S. Department of Interior, Bureau of Reclamation. Washington, DC.

Reese, A. J., 1984. Riprap Sizing, Four Methods. In Proceedings of ASCE Conference on Water for Resource Development, Hydraulics Division, ASCE. David L. Schreiber, ed.

Reese, A. J., 1988. Nomographic Riprap Design. Miscellaneous Paper HL 88-2. Vicksburg, Mississippi: U. S. Army Engineers, Waterways Experiment Station.

Texas Department of Transportation, 2002, Hydraulic Design Manual, Revision 2002-2, Austin, Texas

U. S. Army Corps of Engineers, 1991/1994, Hydraulic Design of Flood Control Channels, Engineer Manual, EM 1110-2-1601 (htt;://www.usace.army.mil/usace-docs/).

U. S. Army Corps of Engineers Research and Development Center, 2000. Gabions for Streambank Erosion Control, ERDC TN-EMRRP-SR-22, Vicksburg, Mississippi.

U. S. Department of Transportation, Federal Highway Administration, 1961. Design Charts For Open Channel Flow. Hydraulic Design Series No. 3. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1984. Guide for Selecting Manning's Roughness Coefficients For Natural Channels and Flood Plains. FHWA-TS-84-204. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1988. Design of Stable Channels with Flexible Linings. Hydraulic Engineering Circular No. 15. Washington, DC.

Wright-McLaughlin Engineers, 1969. Urban Storm Drainage Criteria Manual, Vol. 2. Prepared for the Denver Regional Council of Governments. Wright-McLaughlin Engineers, Denver, CO

Section 4.5
Storage Design

4.5.1 General Storage Concepts

4.5.1.1 Introduction

This section provides general guidance on storm water runoff storage for meeting storm water management control objectives (i.e., water quality protection, downstream streambank protection, and flood control).

Storage of storm water runoff within a storm water management system is essential to providing the extended detention of flows for water quality protection and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural storm water controls and/or nonstructural features and landscaped areas. Figure 4.5-1 illustrates various storage facilities that can be considered for a development site.

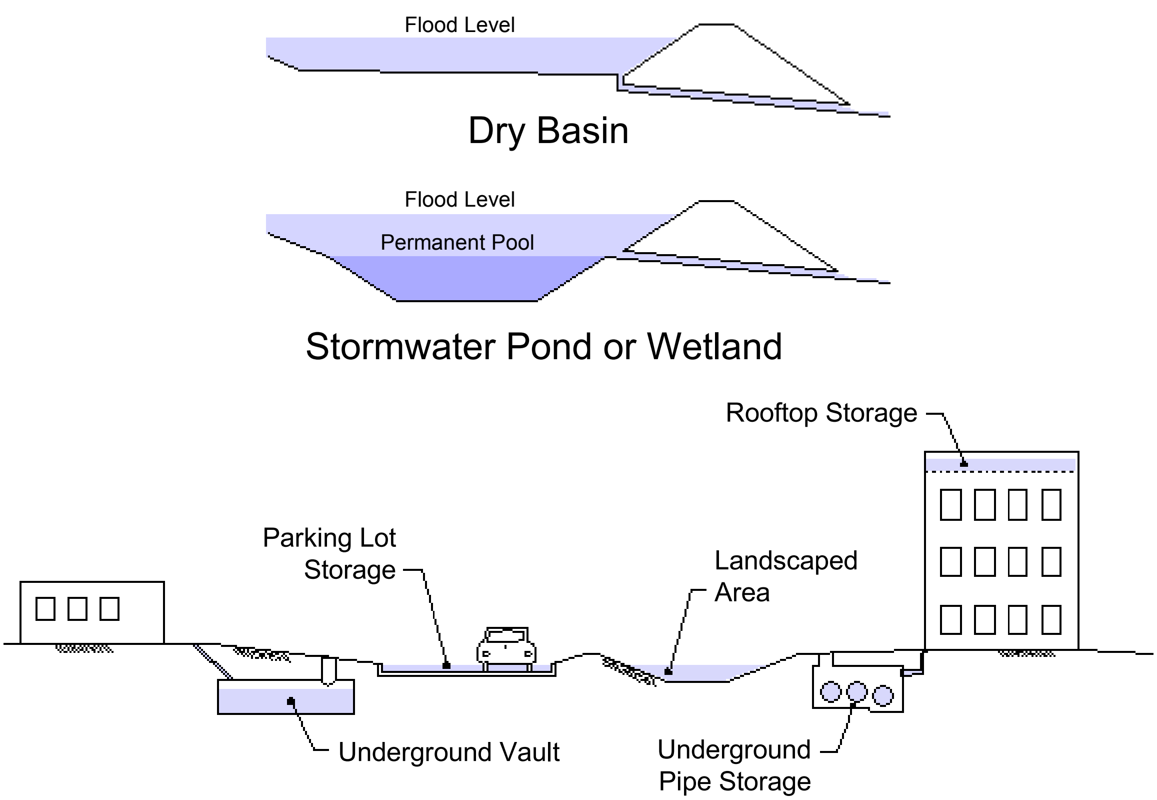


Figure 4.5-1 Examples of Typical Storm Water Storage Facilities

4.5.1.2 Storage Classification

Storm water storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

Storm water *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet streambank protection criteria, and flood criteria where required.

Extended detention (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet streambank protection criteria. Some structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) also include extended detention storage of a portion of the water quality protection volume.

Retention facilities are designed to contain a permanent pool of water, such as storm water ponds and wetlands, which is used for water quality protection.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage storm water runoff from multiple projects and/or properties. A discussion of regional storm water controls is found in Section 5.1.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 4.5-2 illustrates on-line versus off-line storage.

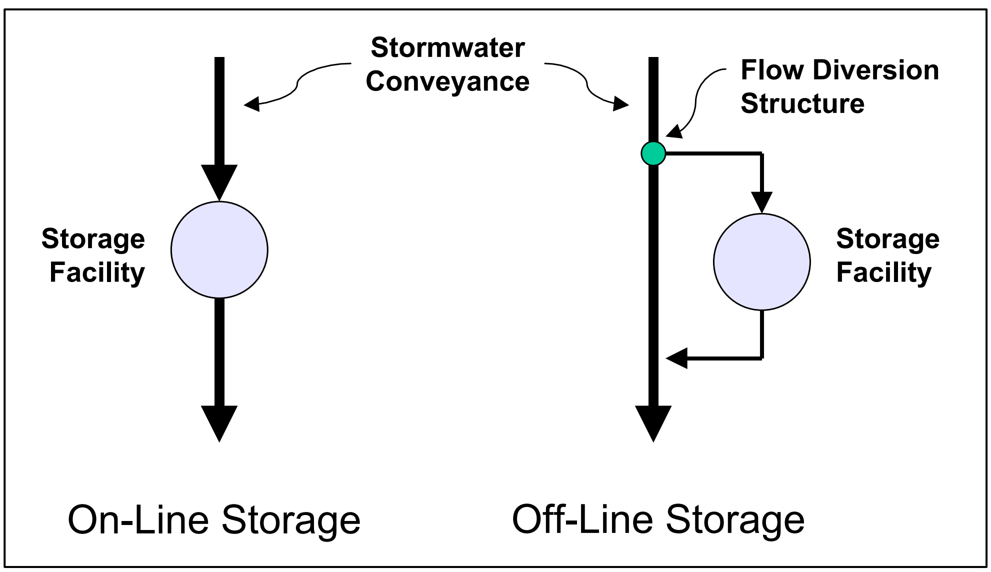


Figure 4.5-2 On-Line versus Off-Line Storage

4.5.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 4.5-3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

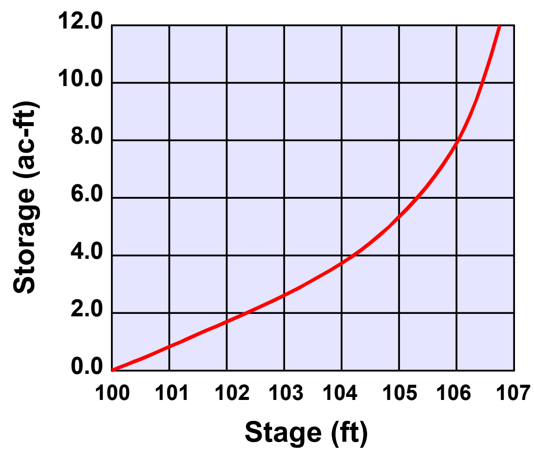


Figure 4.5-3 Stage-Storage Curve

The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see Figure 4.5-4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (4.5.1)$$

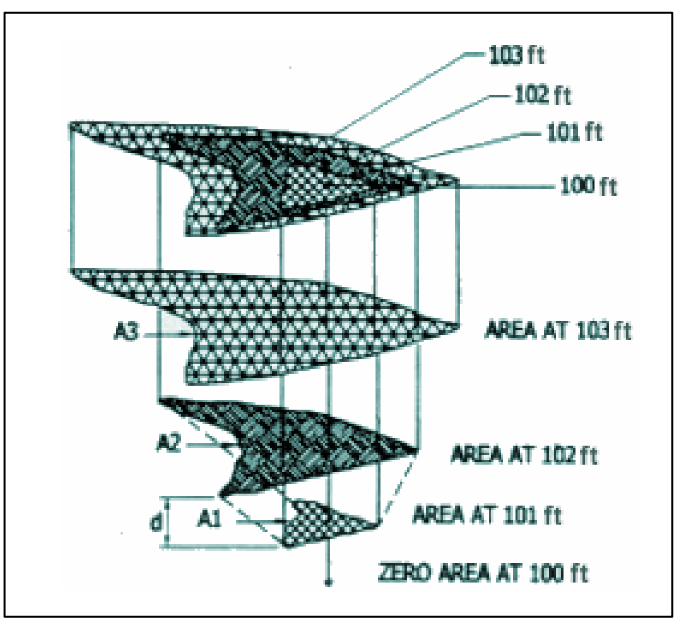


Figure 4.5-4 Double-End Area Method

where:

$V_{1,2}$ = storage volume (ft^3) between elevations 1 and 2

A_1 = surface area at elevation 1 (ft^2)

A_2 = surface area at elevation 2 (ft^2)

d = change in elevation between points 1 and 2 (ft)

The frustum of a pyramid formula is expressed as:

$$\mathbf{V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3} \qquad \mathbf{(4.5.2)}$$

where:

V = volume of frustum of a pyramid (ft^3)

d = change in elevation between points 1 and 2 (ft)

A_1 = surface area at elevation 1 (ft^2)

A_2 = surface area at elevation 2 (ft^2)

The prismoidal formula for trapezoidal basins is expressed as:

$$\mathbf{V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3} \qquad \mathbf{(4.5.3)}$$

where:

V = volume of trapezoidal basin (ft^3)

L = length of basin at base (ft)

W = width of basin at base (ft)

D = depth of basin (ft)

Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$\mathbf{V = 1.047\ D\ (R_1^{\ 2} + R_2^{\ 2} + R_1R_2)} \qquad \mathbf{(4.5.4)}$$

$$\mathbf{V = 1.047\ D\ (3\ R_1^{\ 2} + 3ZDR_1 + Z_2D^2)} \qquad \mathbf{(4.5.5)}$$

where:

R_1, R_2 = bottom and surface radii of the conic section (ft)

D = depth of basin (ft)

Z = side slope factor, ratio of horizontal to vertical

4.5.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 4.5-5). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 4.6, *Outlet Structures*.

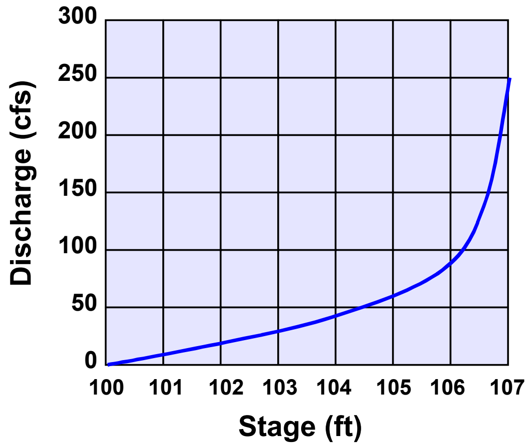


Figure 4.5-5 Stage-Discharge Curve

4.5.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.5-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.5-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	sec
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K	Coefficient	-
I	Inflow rate	cfs
L	Length	ft
Q, q	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
S, V_S	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_I	Duration of basin inflow	hrs
t_P	Time to peak	hrs
V_S, S	Storage volume	ft^3, in, acre-ft
V_r	Volume of runoff	ft^3, in, acre-ft
W	Width of basin	ft
Z	Side slope factor	-

4.5.3 General Storage Design Procedures

4.5.3.1 Introduction

This section discusses the general design procedures for designing storage to provide standard detention of storm water runoff for flood control (Q_f).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural storm water controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see subsection 2.1.9).

In multi-purpose multi-stage facilities such as storm water ponds, the design of storage must be integrated with the overall design for water quality protection objectives. See Chapter 5 for further guidance and criteria for the design of structural storm water controls.

4.5.3.2 Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

4.5.3.3 Design Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

Step 1 Compute inflow hydrograph for runoff from the "Conveyance" (e.g., Q_{p25}) and 100-year (Q_{p100}) design storms using the hydrologic methods outlined in Section 2.1. Both existing- and post-development hydrographs are required for both the "Conveyance" and 100-year design storms.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see subsection 4.5.4).

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

Step 4 Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the "Conveyance" design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.

Step 6 Perform routing calculations using the 100-year hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflows for the “Conveyance” and 100-year storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system to the location where the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system (see subsection 2.1.9).

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including North Central Texas.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

4.5.4 Preliminary Detention Calculations

4.5.4.1 Introduction

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

4.5.4.2 Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 4.5-6.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$\mathbf{V_S = 0.5T_i\,(Q_i - Q_O)} \qquad \textbf{(4.5.6)}$$

where:

V_S = storage volume estimate (ft^3)

Q_i = peak inflow rate (cfs)

Q_O = peak outflow rate (cfs)

T_i = duration of basin inflow (s)

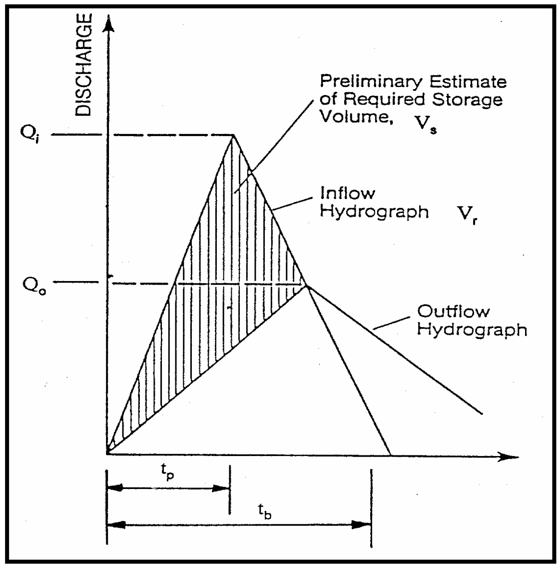


Figure 4.5-6 Triangular-Shaped Hydrographs (For Preliminary Estimate of Required Storage Volume)

4.5.4.3 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

Determine input data, including the allowable peak outflow rate, Q_O, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_p}{t_b}\right)^{0.753}} \qquad \textbf{(4.5.7)}$$

where:

V_S = volume of storage (in)
V_r = volume of runoff (in)
Q_O = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
t_p = time to peak of the inflow hydrograph (hr)

Multiply the volume of runoff, V_r, times the ratio V_S/V_r, calculated in Step 2 to obtain the estimated storage volume V_S.

4.5.4.4 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_i, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph, t_p, and storage volume V_S.

Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff and Singh, 1976):

$$\mathbf{Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546}} \qquad \textbf{(4.5.8)}$$

where:

Q_O = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
V_S = volume of storage (in)
V_r = volume of runoff (in)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
t_p = time to peak of the inflow hydrograph (hr)

Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_O, for the selected storage volume.

References

Metropolitan Government of Nashville and Davidson County, 1988. Storm Water Management Manual - Volume 2 Procedures. Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill.

Wycoff, R. L. and U. P. Singh, 1976. Preliminary Hydrologic Design of Small Flood Detention Reservoirs. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49.

CHAPTER

4

Section 4.6 Outlet Structures

4.6.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.6-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.6-1 Symbols and Definitions

Symbol	Definition	Units
A, a	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
B	Breadth of weir	ft
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K, k	Coefficient	-
L	Length	ft
n	Manning's n	-
Q, q	Peak inflow or outflow rate	cfs, in
V_u	Approach velocity	ft/s
WQ_v	Water quality protection volume	ac ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_g	Angle of the grate with respect to the horizontal	degrees

4.6.2 Primary Outlets

4.6.2.1 Introduction

Primary outlets provide the critical function of the regulation of flow for structural storm water controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the storm water facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for storm water storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

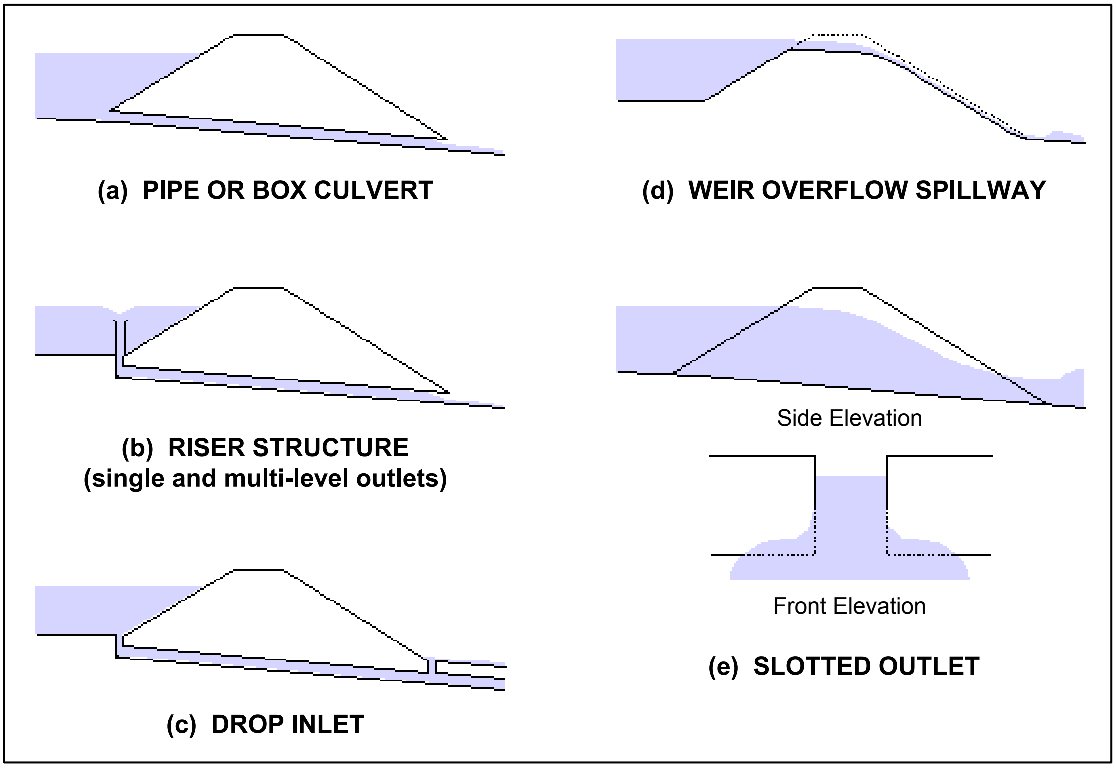


Figure 4.6-1 Typical Primary Outlets

4.6.2.2 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in storm water facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

The design professional must pay attention to material types and construction details when designing an outlet structure or device. Non-corrosive material and mounting hardware are key to device longevity, ease of operation, and low cost maintenance. Special attention must also be paid to not placing dissimilar metal materials together where a cathodic reaction will cause deterioration and destruction of metal parts.

Protective coatings, paints, and sealants must also be chosen carefully to prevent contamination of the storm water flowing through the structure/device. This is not only important while they are being applied, but also as these coating deteriorate and age over the functional life of the facility.

Each of these outlet types has a different design purpose and application:

- Water quality and streambank protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.

- Larger flows, such as flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

4.6.2.3 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 4.6-2(a), the orifice discharge can be determined using the standard orifice equation below.

$$\mathbf{Q = CA\,(2gH)^{0.5}} \qquad \textbf{(4.6.1)}$$

where:

Q = the orifice flow discharge (cfs)
C = discharge coefficient
A = cross-sectional area of orifice or pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
D = diameter of orifice or pipe (ft)
H = effective head on the orifice, from the center of orifice to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 4.6-2(b).

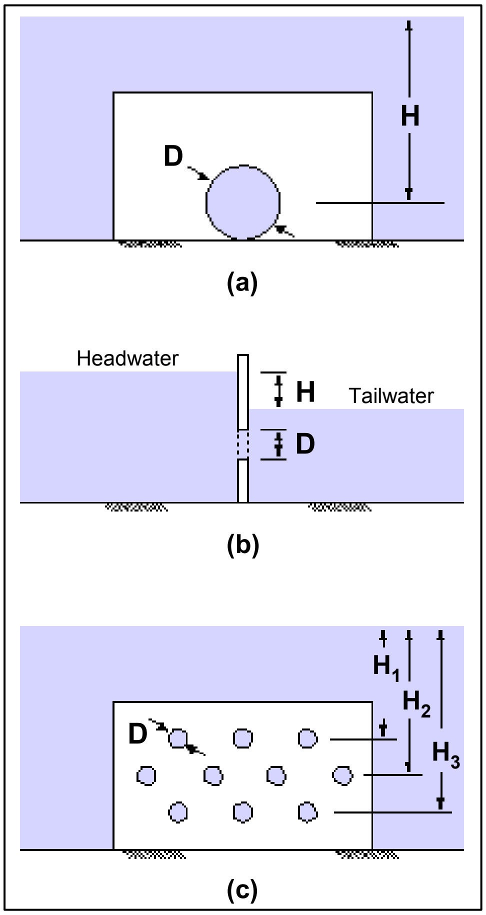


Figure 4.6-2 Orifice Definitions

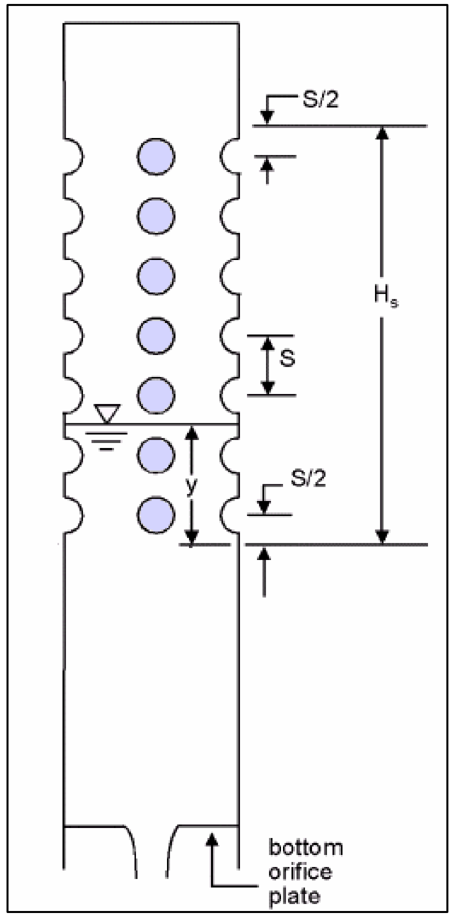


Figure 4.6-3 Perforated Riser

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$Q = 0.6A\ (2gH)^{0.5} = 3.78D^2H^{0.5} \qquad (4.6.2)$$

where:

D = diameter of orifice or pipe (ft)

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 4.6-2(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 4.6-2 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 4.6-2 Circular Perforation Sizing

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in²)		
		1 column	2 columns	3 columns
1/4	1	0.05	0.1	0.15
5/16	2	0.08	0.15	0.23
3/8	2	0.11	0.22	0.33
7/16	2	0.15	0.3	0.45
1/2	2	0.2	0.4	0.6
9/16	3	0.25	0.5	0.75
5/8	3	0.31	0.62	0.93
11/16	3	0.37	0.74	1.11
3/4	3	0.44	0.88	1.32
13/16	3	0.52	1.04	1.56
7/8	3	0.6	1.2	1.8
15/16	3	0.69	1.38	2.07
1	4	0.79	1.58	2.37
1 1/16	4	0.89	1.78	2.67
1 1/8	4	0.99	1.98	2.97
1 3/16	4	1.11	2.22	3.33
1 1/4	4	1.23	2.46	3.69
1 5/16	4	1.35	2.7	4.05
1 3/8	4	1.48	2.96	4.44
1 7/16	4	1.62	3.24	4.86
1 1/2	4	1.77	3.54	5.31
1 9/16	4	1.92	3.84	5.76
1 5/8	4	2.07	4.14	6.21
1 11/16	4	2.24	4.48	6.72
1 3/4	4	2.41	4.82	7.23
1 13/16	4	2.58	5.16	7.74
1 7/8	4	2.76	5.52	8.28
1 15/16	4	2.95	5.9	8.85
2	4	3.14	6.28	9.42
Number of columns refers to parallel columns of holes				
Minimum plate thickness		1/4”	5/16”	3/8”

Source: Urban Drainage and Flood Control District, Denver, CO

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 4.6-4 provides a schematic of an orifice plate outlet structure for a wet extended detention pond showing the design pool elevations and the flow control mechanisms.

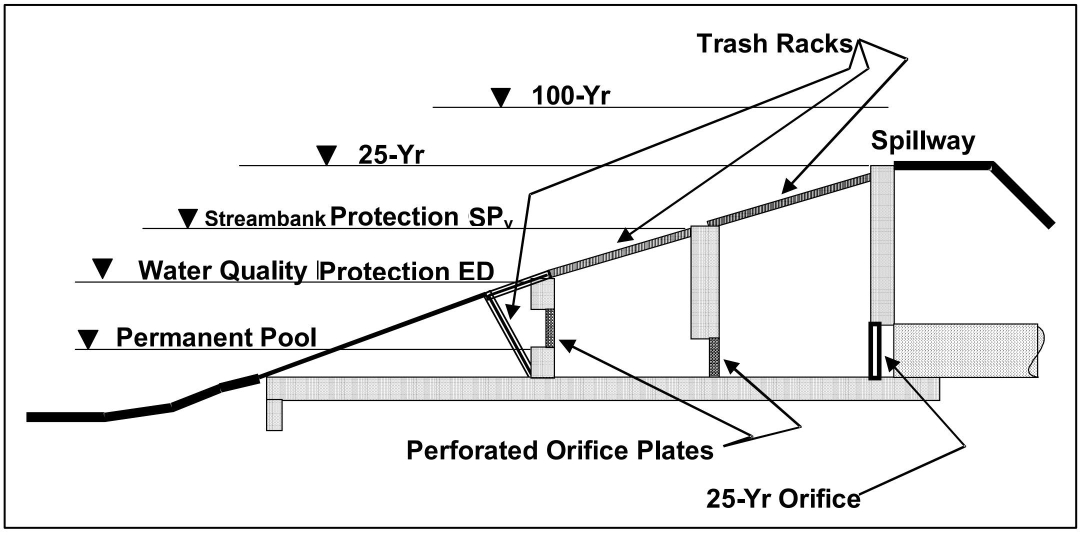


Figure 4.6-4 Schematic of Orifice Plate Outlet Structure

4.6.2.4 Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 4.6-3. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 4.6-3, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2} \qquad \textbf{(4.6.3)}$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft^2)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

4.6.2.5 Pipes and Culverts

Discharge pipes are often used as outlet structures for storm water control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or streambank protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls (see subsection 4.6.2.6). As the stage increases the flow will transition to orifice flow.

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 4.3, *Culvert Design*, or by using equation 4.6.4 (NRCS, 1984).

The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH) / (1 + k_m + k_pL)]0.5 \qquad (4.6.4)$$

where:

- Q = discharge (cfs)
- a = pipe cross sectional area (ft^2)
- g = acceleration of gravity (ft/s^2)
- H = elevation head differential (ft)
- k_m = coefficient of minor losses (use 1.0)
- k_p = pipe friction coefficient = $5087n^2/D^{4/3}$
- L =pipe length (ft)

4.6.2.6 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested* weir. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with compensation for end contractions is illustrated in Figure 4.6-5(a). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_C)]\ LH^{1.5} \qquad (4.6.5)$$

were:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

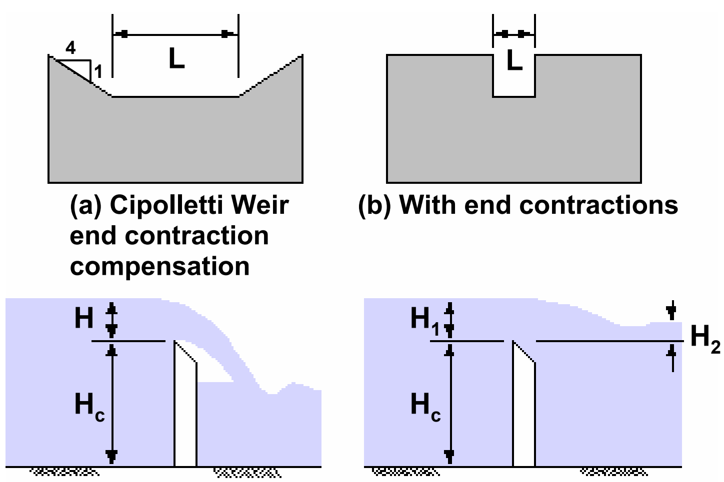


Figure 4.6-5 Sharp-Crested Weir

The discharge equation for the Cipolletti Weir is $Q = 3.367\ LH^{1/2}$

A sharp-crested weir with two end contractions is illustrated in Figure 4.6-5(b). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_C)] (L - 0.2H) H^{1.5} \quad (4.6.6)$$

where:

Q = discharge (cfs)

H = head above weir crest excluding velocity head (ft)

H_C = height of weir crest above channel bottom (ft)

L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_S = Q_f (1 - (H_2/H_1)^{1.5})^{0.385} \quad (4.6.7)$$

where:

Q_S = submergence flow (cfs)

Q_f = free flow (cfs)

H_1 = upstream head above crest (ft)

H_2 = downstream head above crest (ft)

4.6.2.7 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \quad (4.6.8)$$

where:

Q = discharge (cfs)

C = broad-crested weir coefficient

L = broad-crested weir length perpendicular to flow (ft)

H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 4.6-3.

Figure 4.6-6
Broad-Crested Weir

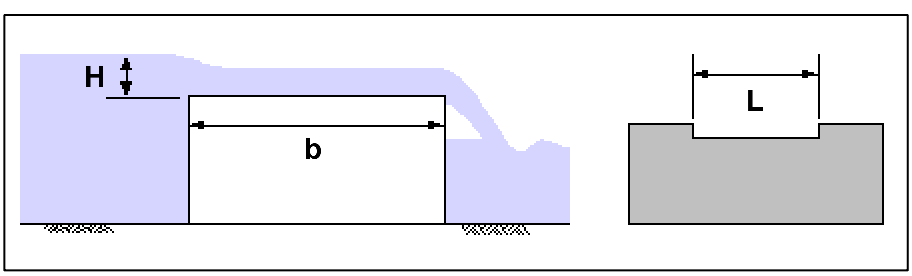


Table 4.6-3 Broad-Crested Weir Coefficient (C) Values

Measured Head (H)*	Weir Crest Breadth (b) in feet										
In feet	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

* Measured at least 2.5H upstream of the weir.
Source: Brater and King (1976)

4.6.2.8 V-Notch Weirs

The discharge through a V-notch weir (Figure 4.6-7) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)\, H^{2.5} \qquad (4.6.9)$$

where:

Q = discharge (cfs)
θ = angle of V-notch (degrees)
H = head on apex of notch (ft)

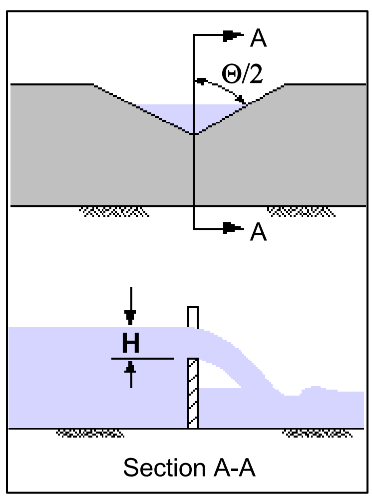


Figure 4.6-7 V-Notch Weir

4.6.2.9 Proportional Weirs

Although it may be more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 4.6-8. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\, a^{0.5}\, b\, (H - a/3) \qquad (4.6.10)$$

$$x/b = 1 - (1/3.17)\, (\arctan\, (y/a)^{0.5}) \qquad (4.6.11)$$

where:

Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 4.6-8

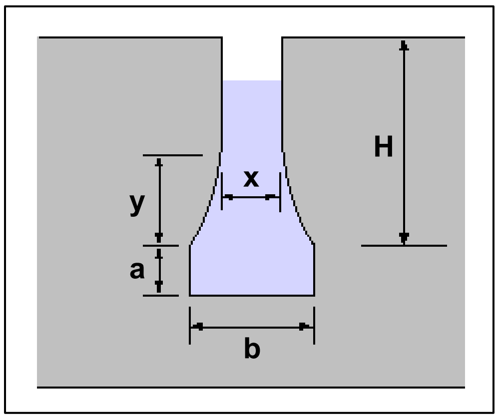


Figure 4.6-8 Proportional Weir Dimensions

4.6.2.10 Combination Outlets

Combinations of orifices, weirs, and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality protection volume, streambank protection volume, and flood control volume).

They are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 4.6-9 shows an example of a riser designed for a wet extended detention pond. The orifice plate outlet structure in Figure 4.6-4 is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 4.6-10) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

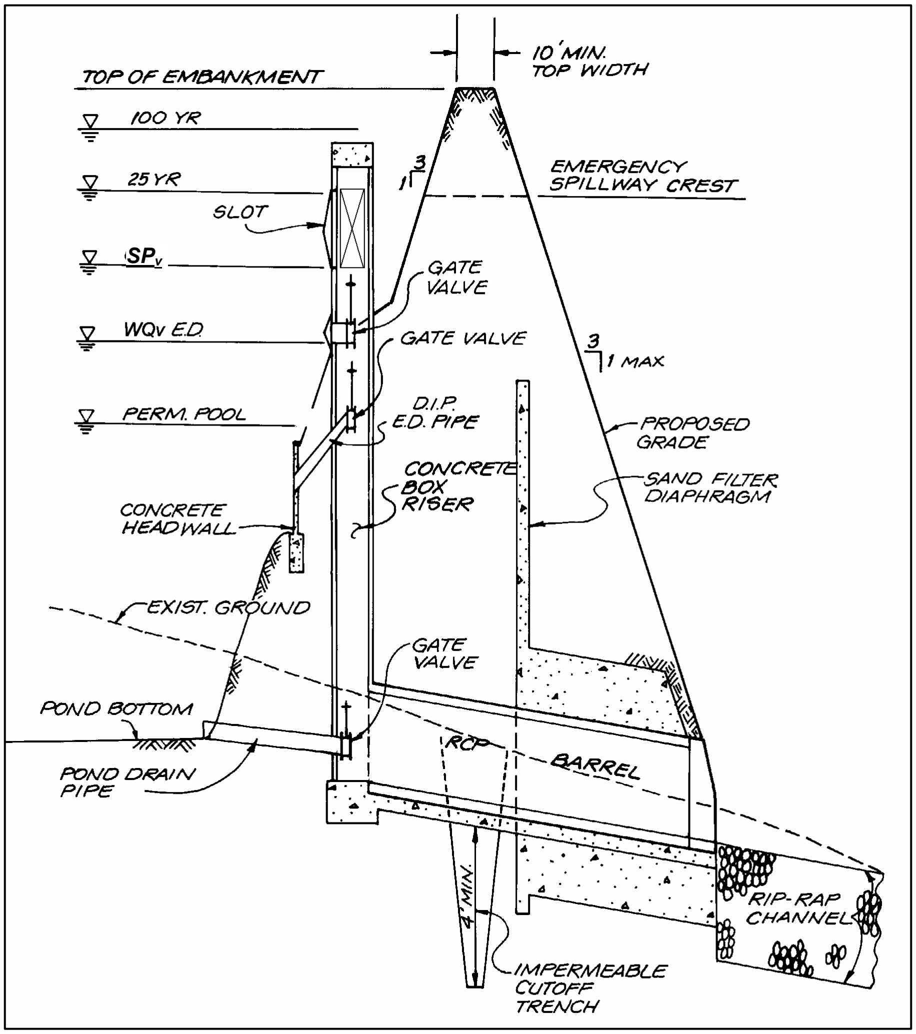


Figure 4.6-9 Schematic of Combination Outlet Structure

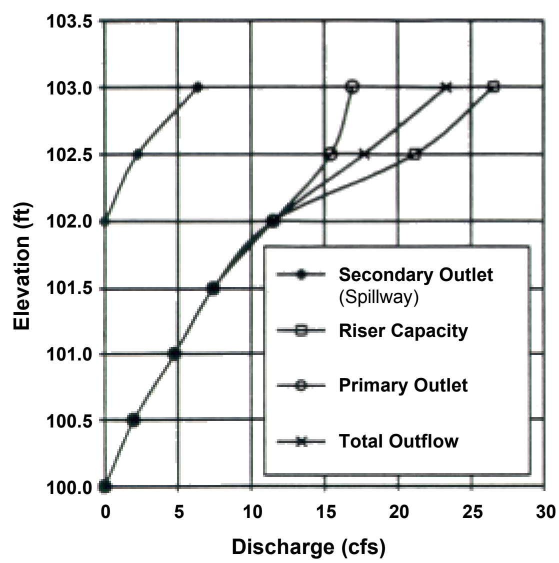


Figure 4.6-10 Composite Stage-Discharge Curve

4.6.3 Extended Detention (Water Quality and Streambank Protection) Outlet Design

4.6.3.1 Introduction

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for downstream streambank protection or the ED portion of the water quality protection volume. The release rate for both the WQ_v and SP_v should be one that discharges the ED volume in a period of 24 hours or longer. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQ_v extended detention and SP_v control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the streambank protection drawdown.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

The outlet hydraulics for peak control design (flood control) is usually straightforward in that an outlet is selected to limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality protection or downstream streambank protection, however, the storage volume is detained and released for each over a specified amount of time (e.g., 24-hours). The release period is a "brim" drawdown time, beginning at the time of peak storage of the WQ_v or SP_v until the entire calculated volume drains out of the basin. This assumes the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- Using the maximum hydraulic head associated with the brim storage volume and maximum discharge, calculate the orifice size needed to achieve the required drawdown time. Route the volume through the basin to verify the actual storage volume used and the drawdown time.
- Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or streambank protection.

4.6.3.2 Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality protection volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality protection design.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1 Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Protection Volume (or Streambank Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs
Q_{max} = 2 * Q_{avg} = 2 * 0.38 = 0.76 cfs

Step 2 Determine the required orifice diameter by using the orifice equation (4.6.8) and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$
$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

Determine pipe diameter from $A = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$
$D = [4(0.071)/3.14]^{0.5} = 0.30$ ft = 3.61 in

Use a 3.6-inch diameter water quality protection orifice.

Routing the water quality protection volume of 0.76 ac ft through the 3.6-inch water quality protection orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours.

4.6.3.3 Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example (4.6.3.2) use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Average Hydraulic Head (h_{avg}) = 2.5 ft (from stage vs storage data)

Step 1 Determine the average release rate to release the water quality protection volume over a 24-hour time period.

$Q = 33{,}106\ ft^3 / (24\ hr)(3{,}600\ s/hr) = 0.38$ cfs

Step 2 Determine the required orifice diameter by using the orifice equation (4.6.8) and the average head on the orifice:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$
$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05\ ft^3$

Determine pipe diameter from $A = 3.14r^2 = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$
$D = [4(0.05)/3.14]^{0.5} = 0.252$ ft = 3.03 in

Use a 3-inch diameter water quality protection orifice.

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

4.6.4 Multi-Stage Outlet Design

4.6.4.1 Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality protection, streambank protection, and flood control) for storm water ponds, storm water wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figures 4.6-4 and 4.6-9 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 4.6-10)

4.6.4.2 Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQ_v, SP_v, and Q_f), then that step in the procedure is skipped.

Step 1 Determine Storm Water Control Volumes. Using the procedures from Sections 2.1 and 2.2, estimate the required storage volumes for water quality protection (WQ_v), streambank protection (SP_v), and flood control (Q_f).

Step 2 Develop Stage-Storage Curve. Using the structure geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

Step 3 Design Water Quality Protection Outlet. Design the water quality protection extended detention (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in subsection 4.6.3. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality protection will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see subsection 4.6.5).

Step 4 Design Streambank Protection Outlet. Design the streambank protection extended detention outlet (SP_v-ED) using either method from subsection 4.6.3. For this design, the storage needed for streambank protection will be greater than the water quality protection volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality protection orifice and the outlet used for streambank protection. The outlet should be protected in a manner similar to that for the water quality protection orifice.

Step 5 Design Flood Control Outlet. The storage needed for flood control will be greater than the water quality protection and streambank protection storage. Establish the Q_f maximum water surface elevation using the stage-storage curve and subtract the SP_v elevation to find the maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment peak discharge rate. Develop a stage-discharge curve for the combined set of outlets (WQ_v, SP_v and Q_f).

Step 6 Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the outlet structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 4.6-11, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this transition will occur. Also note in Figure 4.6 -11 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 4.6-12 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

Step 7 Size the Emergency Spillway. It is recommended that all storm water impoundment structures have a vegetated emergency spillway (see subsection 4.6.7). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed. Also check the dam safety requirements to be sure of an adequate design.

Step 8 Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See Subsection 4.7, *Energy Dissipation Design*, for more information.

Step 9 Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.

Step 10 Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

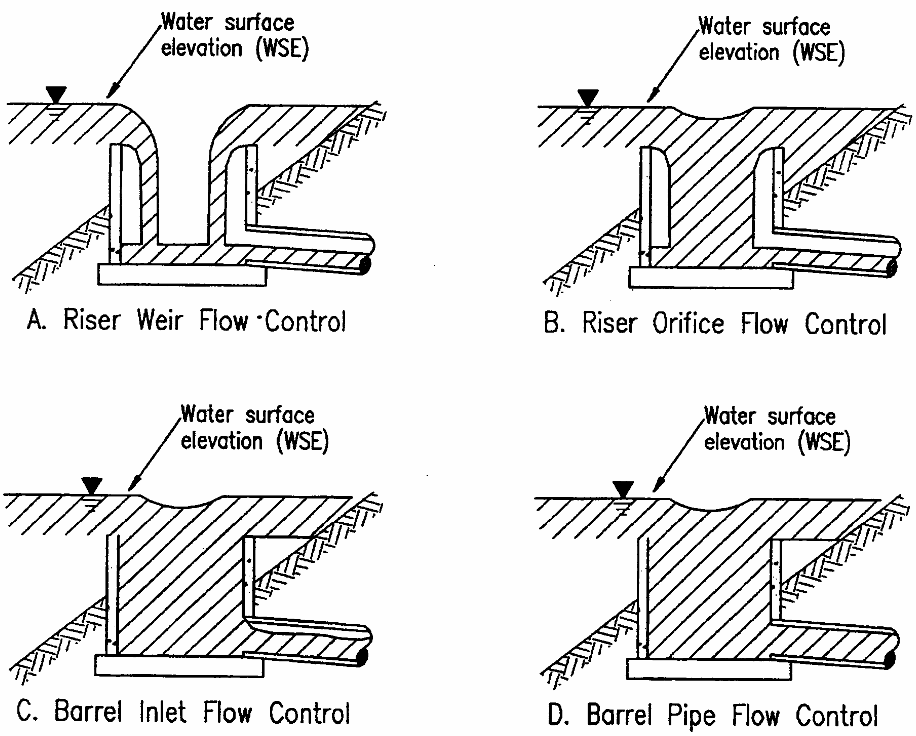


Figure 4.6-11 Riser Flow Diagrams
(Source: VDCR, 1999)

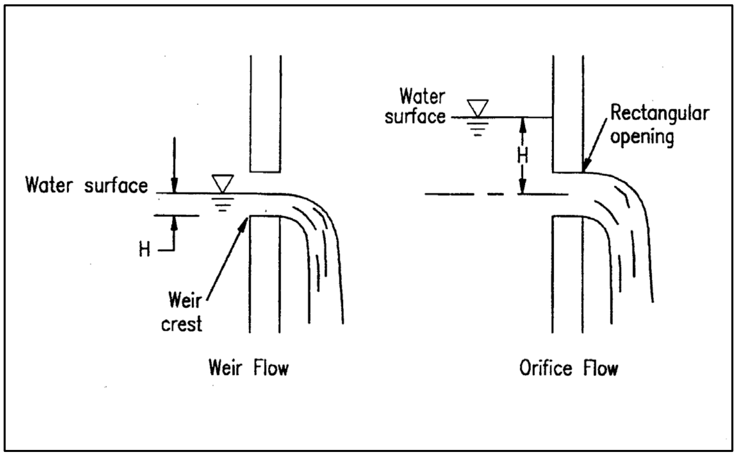


Figure 4.6-12 Weir and Orifice Flow
(Source: VDCR, 1999)

4.6.5 Extended Detention Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

The use of a reverse slope pipe attached to a riser for a storm water pond or wetland with a permanent pool (see Figure 4.6-13). The inlet is submerged a minimum of 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.

The use of a hooded outlet for a storm water pond or wetland with a permanent pool (see Figures 4.6-14 and 4.6-15).

Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 4.6-16).

Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

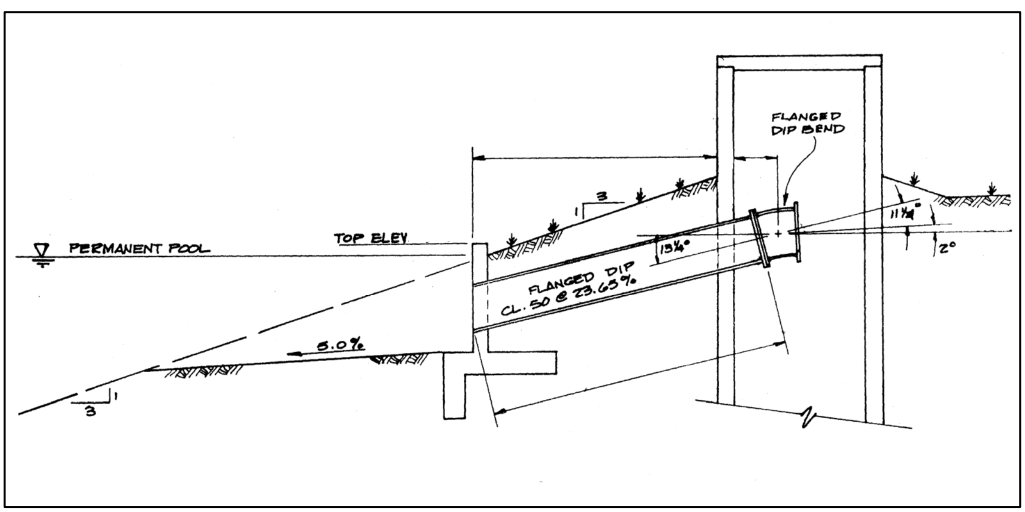


Figure 4.6-13 Reverse Slope Pipe Outlet

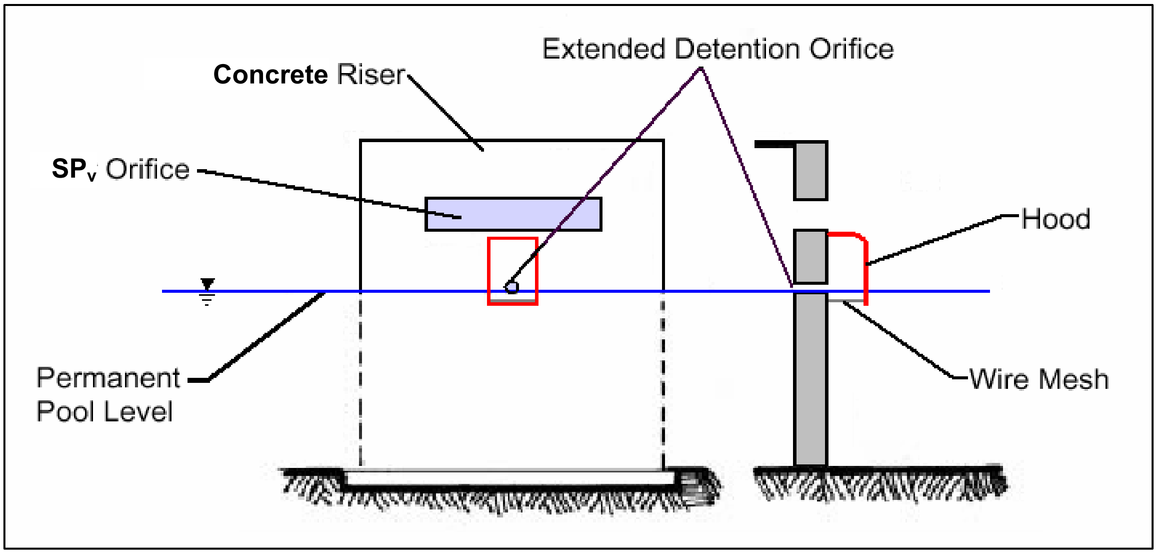


Figure 4.6-14 Hooded Outlet

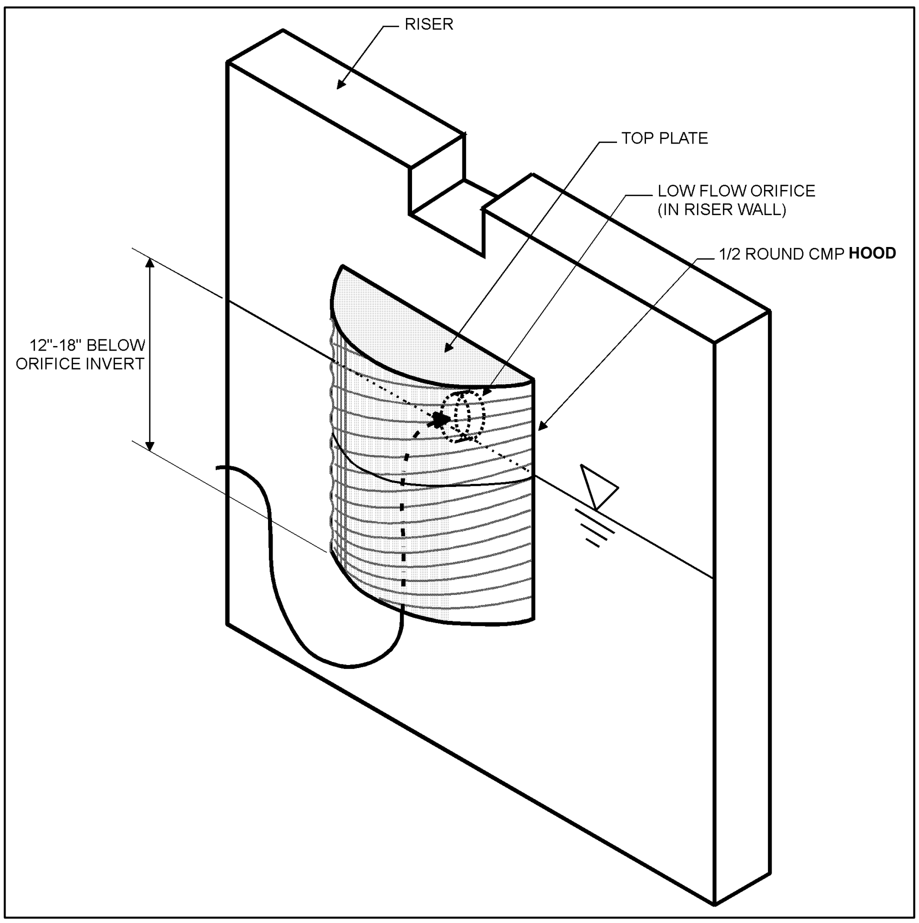


Figure 4.6-15 Half-Round CMP Orifice Hood

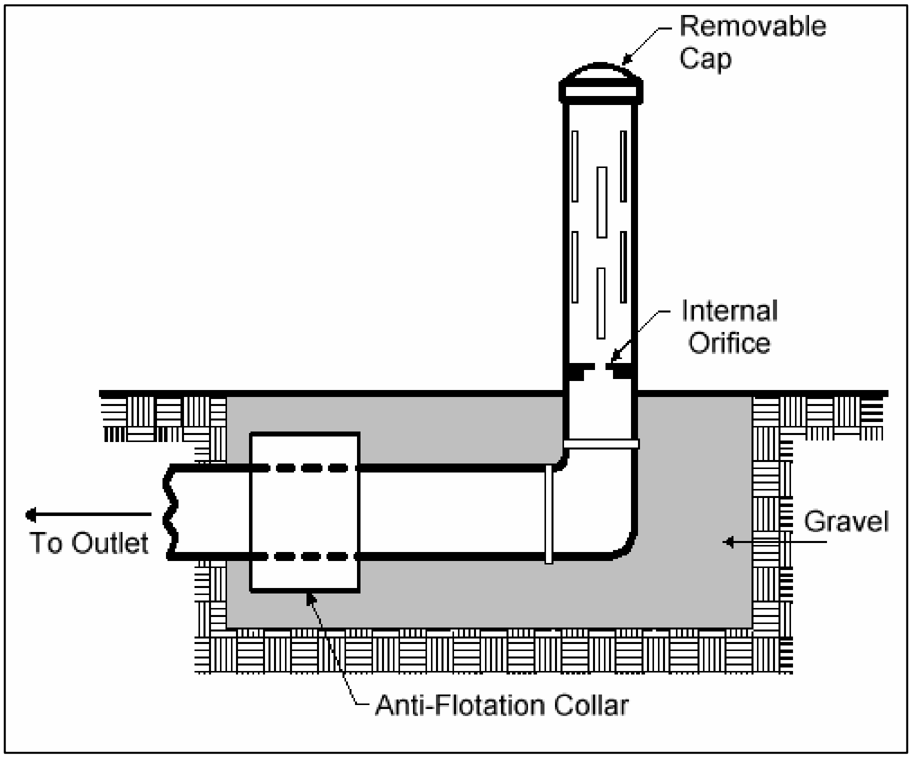


Figure 4.6-16 Internal Control for Orifice Protection

4.6.6 Trash Racks and Safety Grates

4.6.6.1 Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure 4.6-17. Additional track rack design can be found in Appendix C. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

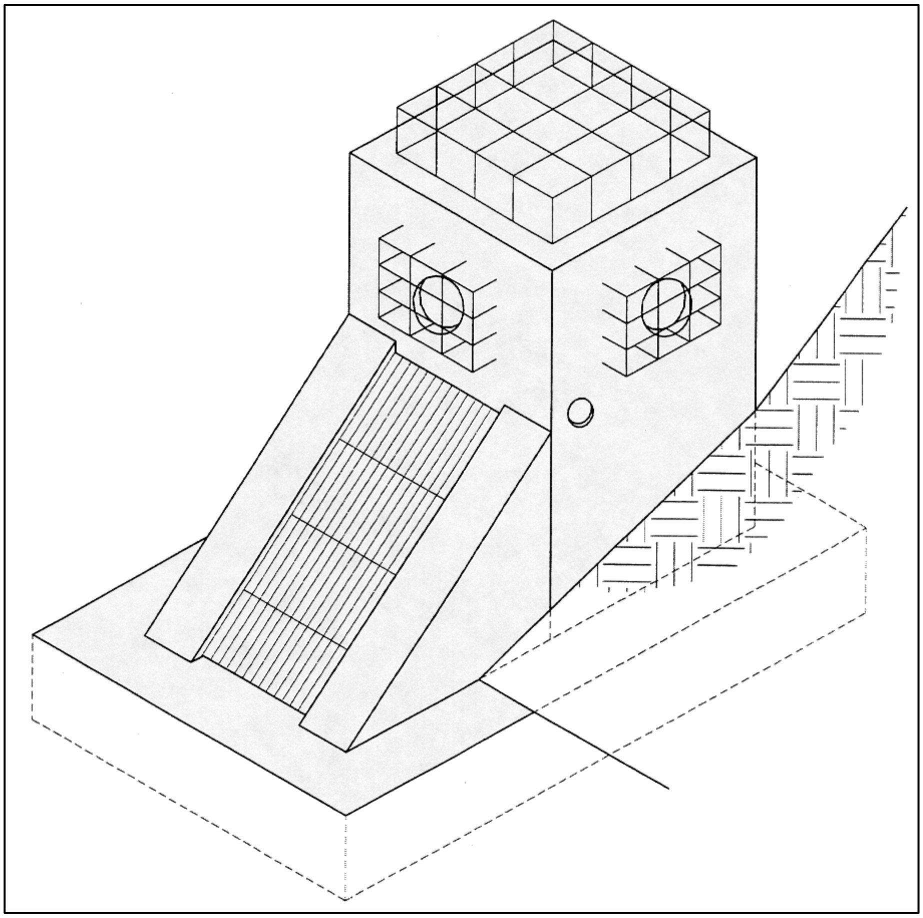


Figure 4.6-17 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

4.6.6.2 Trash Rack Design

Trash racks must be large enough so that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level — the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 4.6-18 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$\mathbf{H_g = K_{g1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g} \qquad \textbf{(4.6.12)}$$

Where:

H_g = head loss through grate (ft)

K_{g1} = bar shape factor:
- 2.42 - sharp edged rectangular
- 1.83 - rectangular bars with semicircular upstream faces
- 1.79 - circular bars
- 1.67 - rectangular bars with semicircular up- and downstream faces

w = maximum cross-sectional bar width facing the flow (in)

x = minimum clear spacing between bars (in)

V_u = approach velocity (ft/s)

g = acceleration due to gravity (32.2 ft/s^2)

θ_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$\mathbf{H_g = \frac{K_{g2} V_u^2}{2g}} \qquad \textbf{(4.6.13)}$$

Where:

K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)

$K_{g2} = 0.00158 - 0.03217\ A_r + 7.1786\ A_r^2$

- sharp edged rectangular (length/thickness = 5)
  $K_{g2} = -0.00731 + 0.69453\ A_r + 7.0856\ A_r^2$
- round edged rectangular (length/thickness = 10.9)
  $K_{g2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- circular cross section
  $K_{g2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

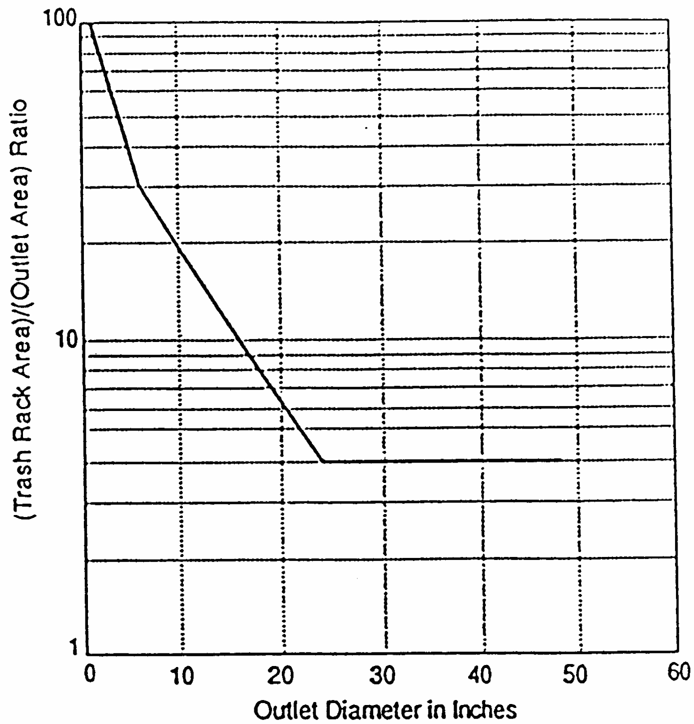


Figure 4.6-18 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

4.6.7 Secondary Outlets

4.6.7.1 Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 4.6-19 shows an example of an emergency spillway.

In many cases, on-site storm water storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

4.6.7.2 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 4.6-19). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Any dam, six feet or higher, must meet appropriate state and federal design standards, especially those regarding spillway design requirements related to passage of the probable maximum flood. In any case, the 100-year flood discharge, assuming blockage of outlet works, must be conveyed with some freeboard as specified by local criteria. Flow in the emergency spillway is open channel flow (see Section 4.4, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

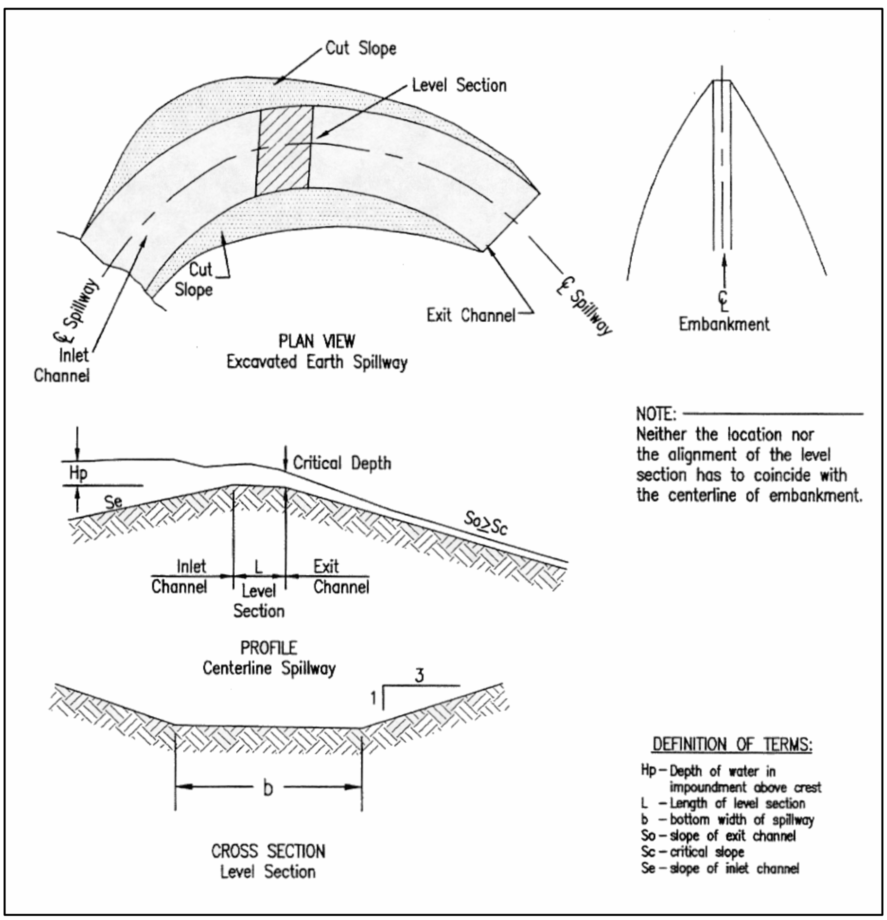


Figure 4.6-19 Emergency Spillway
(Source: VDCR, 1999)

References

Allred-Coonrod, J. E., 1991, Safety Grates in Open Channels, MS Prof. Paper, University of New Mexico, Albuquerque.

Brater, E. F. and H. W. King, 1976. Handbook of Hydraulics. 6th ed. New York: McGraw Hill Book Company.

Bureau of Reclamation. Flood Hydrology Manual, A Water Resources Technical Publication.

Chow, C. N., 1959. Open Channel Hydraulics. New York: McGraw Hill Book Company.

Debo, Thomas N and Andrew J. Reese, 2003. Municipal Storm Water Management. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

McEnroe, B.M., J.M. Steichen and R. M. Schweiger, 1988. Hydraulics of Perforated Riser Inlets for Underground Outlet Terraces, Trans ASAE, Vol. 31, No. 4, 1988.

Metcalf & Eddy, Inc., 1972, Wastewater Engineering, McGraw-Hill, New York.

NRCS, 1984. Engineering Field Manual for Conservation Practices, Soil Conservation Service, Engineering Division, Washington, D.C.

Virginia Department of Conservation and Recreation, 1999. Virginia Stormwater Management Handbook.

Sandvik, A., 1985. Proportional Weirs for Stormwater Pond Outlets. Civil Engineering, March 1985, ASCE pp. 54-56.

Metropolitan Government of Nashville and Davidson County, 1988. Stormwater Management Manual - Volume 2 Procedures. Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill.

United States Bureau of Reclamation, Water Measurement Manual, http://www.usbr.gov/wrrl/fmt/wmm/

U.S. Army Corps of Engineers Engineers Manual (EM 1110-2-1405). Flood –Hydrograph Analysis and Computations.

U.S. Army Corps of Engineers, 1988, Hydraulic Design Criteria (HDC), USAE Waterways Experiment Station, Vicksburg, MS.

Urban Drainage and Flood Control District, 1999. Criteria Manual, Denver, CO.

Wycuff, R. L. and U. P. Singh, 1976. Preliminary Hydrologic Design of Small Flood Detention Reservoirs. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49.

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Section 4.7
Energy Dissipation

4.7.1 Overview

4.7.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Storm water transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at storm water outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

Energy dissipators are engineered devices such as rip-rap aprons or concrete baffles placed at the outlet of storm water conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

4.7.1.2 General Criteria

Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosion velocity of the downstream area channel system.

Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

4.7.1.3 Recommended Energy Dissipators

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, for the design procedures of other energy dissipators.

4.7.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.7-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.7-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
D	Height of box culvert	ft
d_{50}	Size of riprap	ft
d_w	Culvert width	ft
Fr	Froude Number	-
g	Acceleration of gravity	ft/s^2
h_s	Depth of dissipator pool	ft
L	Length	ft
L_a	Riprap apron length	ft
L_B	Overall length of basin	ft
L_s	Length of dissipator pool	ft
PI	Plasticity index	-
Q	Rate of discharge	cfs
S_v	Saturated shear strength	lbs/in^2
t	Time of scour	min.
t_c	Critical tractive shear stress	lbs/in^2
TW	Tailwater depth	ft
V_L	Velocity L feet from brink	ft/s
V_o	Normal velocity at brink	ft/s
V_o	Outlet mean velocity	ft/s
V_s	Volume of dissipator pool	ft^2
W_o	Diameter or width of culvert	ft
W_s	Width of dissipator pool	ft
y_e	Hydraulic depth at brink	ft
y_o	Normal flow depth at brink	ft

4.7.3 Design Guidelines

If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

If outlet protection is not provided, energy dissipation will occur through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, h_s. The scourhole should then be stabilized. If the scourhole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 4.7-1 provides the riprap size recommended for use downstream of energy dissipators.

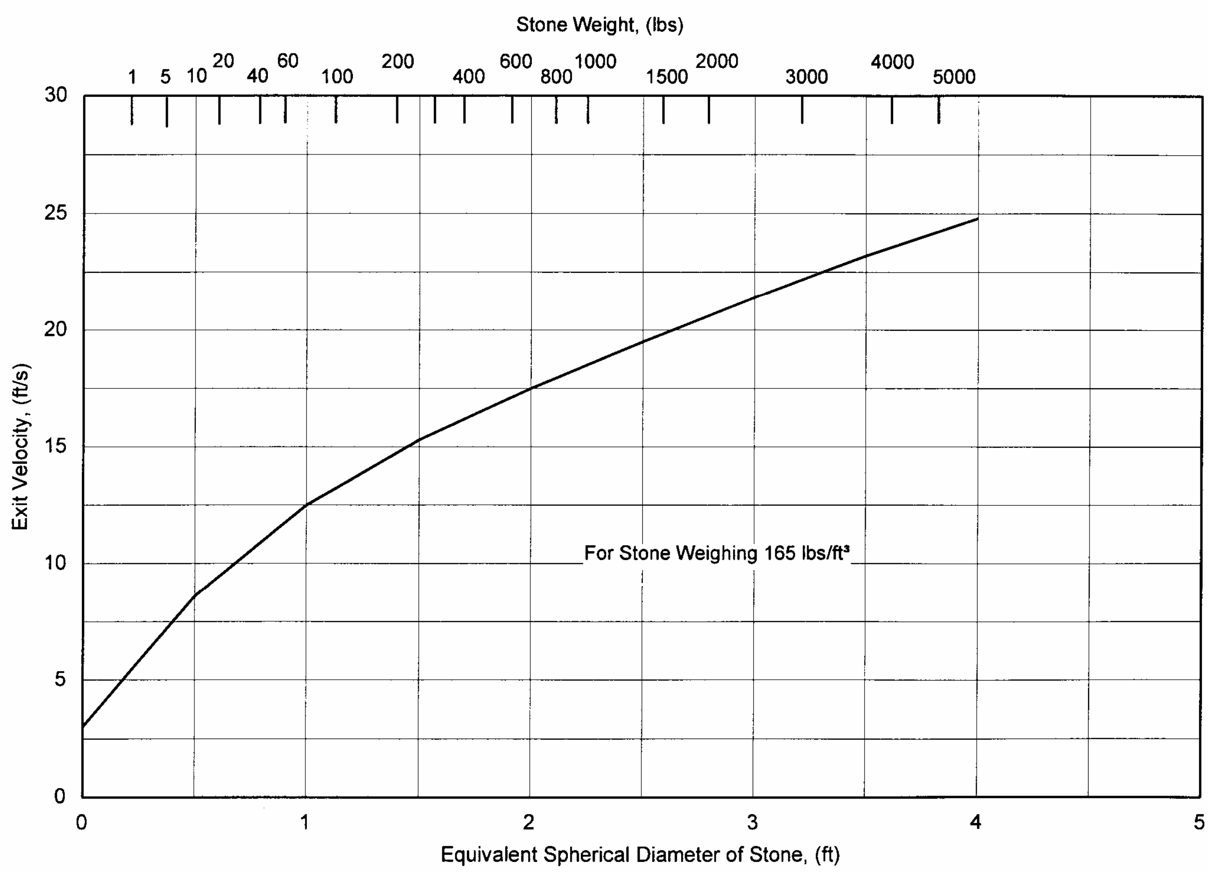


Figure 4.7-1 Riprap Size for Use Downstream of Energy Dissipator
(Source: Searcy, 1967)

4.7.4 Riprap Aprons

4.7.4.1 Description

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.

4.7.4.2 Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 4.7-2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 4.7-3 should be used.

Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 4.7-2 and 4.7-3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 4.7-4.

a. For pipes flowing full:

   Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.

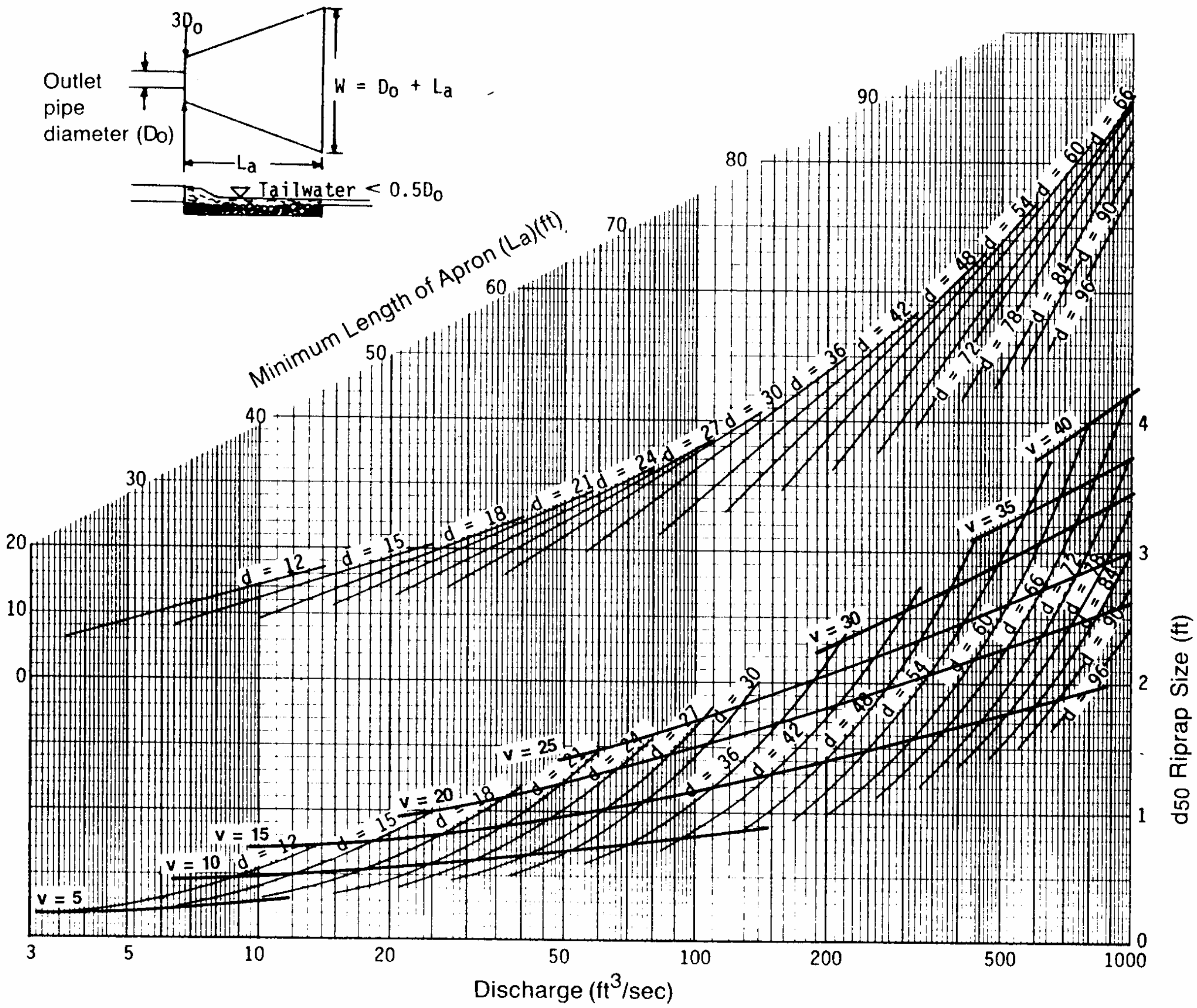
b. For pipes flowing partially full:

   Use the depth of flow, d, in feet, and velocity, v, in ft/s. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, $L_{a,}$ from the scale on the left.

c. For box culverts:

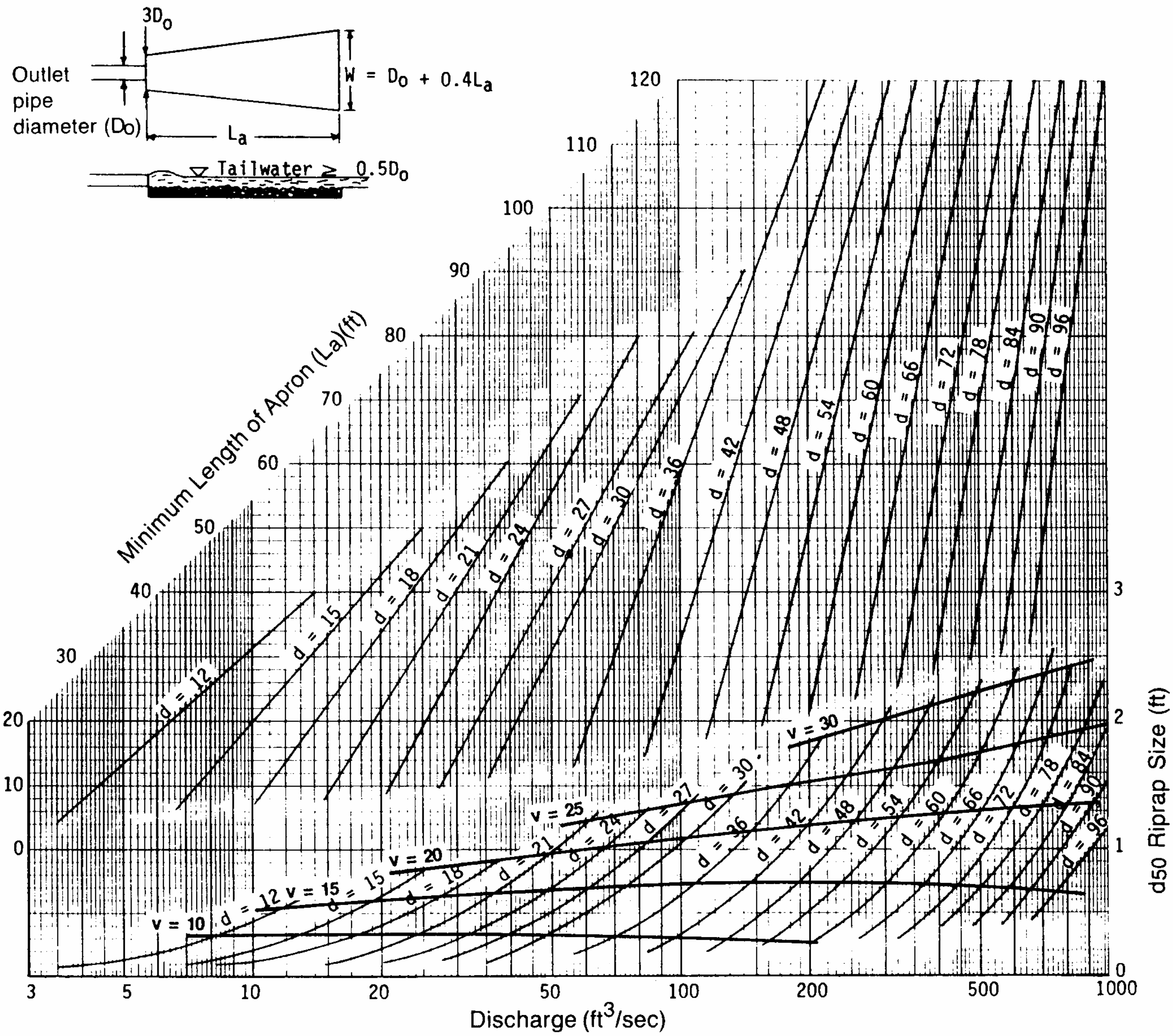
   Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 4.7-4. This will provide protection under either of the tailwater conditions.



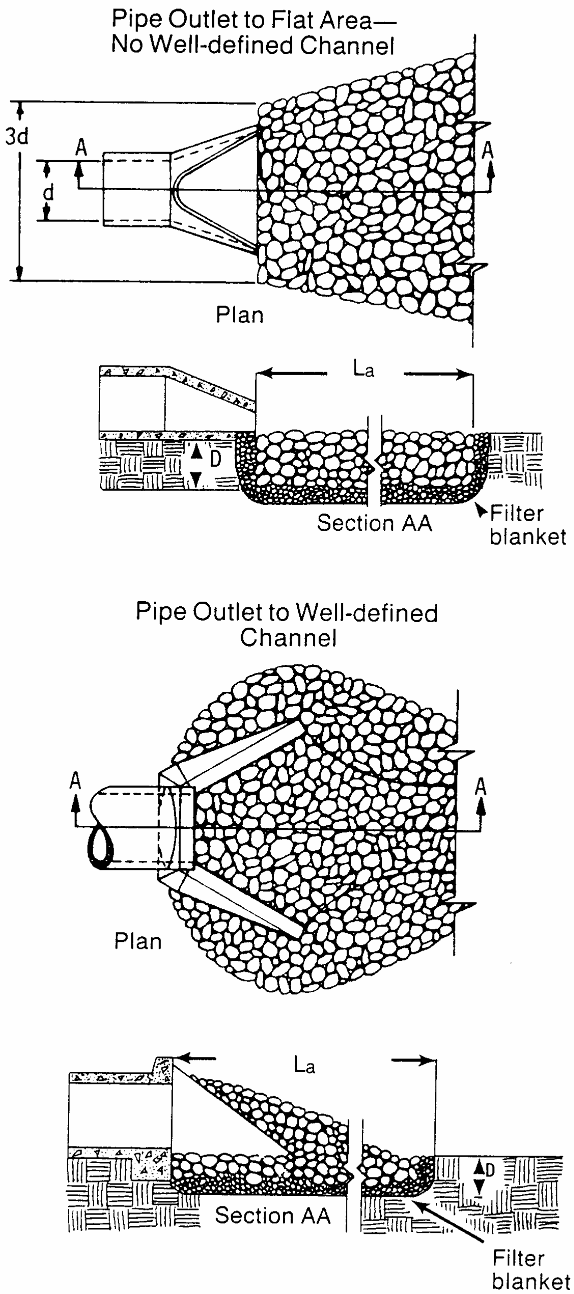
Curves may not be extrapolated.

Figure 4.7-2 Design of Riprap Apron under Minimum Tailwater Conditions
(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.7-3 Design of Riprap Apron under Maximum Tailwater Conditions
(Source: USDA, SCS, 1975)



Notes

1. L_a is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter but not less than 6".
3. In a well-defined channel extend the apron up the channel banks to an elevation of 6" above the maximum tailwater depth or to the top of the bank, whichever is less.
4. A filter blanket or filter fabric should be installed between the riprap and soil foundation.

Figure 4.7-4 Riprap Apron
(Source: Manual for Erosion and Sediment Control in Georgia, 1996)

4.7.4.3 Design Considerations

The following items should be considered during riprap apron design:

The maximum stone diameter should be 1.5 times the median riprap diameter.
d_{max} = 1.5 x d_{50} , d_{50} = the median stone size in a well-graded riprap apron.

The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater.
Apron thickness = 1.5 x d_{max}
(Apron thickness may be reduced to 1.5 x d_{50} when an appropriate filter fabric is used under the apron.)

The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.

If there is a well-defined channel, the apron length should be extended as necessary so the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.

If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.

The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

4.7.4.4 Example Designs

Example 1 Riprap Apron Design for Minimum Tailwater Conditions

A flow of 280 cfs discharges from a 66-in pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

Minimum tailwater conditions = 0.5 d_o, d_o = 66 in = 5.5 ft; therefore, 0.5 d_o = 2.75 ft.

Since TW = 2 ft is less than 2.75 ft, use Figure 4.7-2 for minimum tailwater conditions.

By Figure 4.7-2, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

The downstream apron width equals the apron length plus the pipe diameter:

$W = d + L_a = 5.5 + 38 = 43.5$ ft

Maximum riprap diameter is 1.5 times the median stone size:

$1.5 (d_{50}) = 1.5 (1.2) = 1.8$ ft

Riprap depth = $1.5 (d_{max}) = 1.5 (1.8) = 2.7$ ft.

Example 2 Riprap Apron Design for Maximum Tailwater Conditions

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

Minimum tailwater conditions = 0.5 d_o, d_o = 0.5 (5.0) = 2.5 ft.

Since TW = 5.0 ft is greater than 2.5 ft, use Figure 4.7-3 for maximum tailwater conditions.

v = Q/A = 600/[(5)(10)] = 12 ft/s

On Figure 4.7-3, at the intersection of the curve, d_o = 60 in and v = 12 ft/s, d_{50} = 0.4 ft. Reading up to the intersection with d = 60 in, find L_a = 40 ft.

Apron width downstream = d_w + 0.4 L_a = 10 + 0.4 (40) = 26 ft.

Maximum stone diameter = 1.5 d_{50} = 1.5 (0.4) = 0.6 ft.

Riprap depth = 1.5 d_{max} = 1.5 (0.6) = 0.9 ft.

4.7.5 Riprap Basins

4.7.5.1 Description

Another method to reduce the exit velocities from storm water outlets is through the use of a riprap basin. A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

4.7.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.7-5. Principal features of the basin are:

The basin is preshaped and lined with riprap of median size (d_{50}).

The floor of the riprap basin is constructed at an elevation of h_s below the culvert invert. The dimension h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge. The ratio of h_s to d_{50} of the material should be between 2 and 4.

The length of the energy dissipating pool is 10 x h_s or 3 x W_o, whichever is larger. The overall length of the basin is 15 x h_s or 4 x W_o, whichever is larger.

4.7.5.3 Design Procedure

The following procedure should be used for the design of riprap basins.

Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that TW/y_o $\leq$ 0.75 for the design discharge.

For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 4.7-6 or Figure 4.7-7 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

For streambank protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 4.7-8, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

Size basin as shown in Figure 4.7-5.

Where allowable dissipator exit velocity is specified:

a. Determine the average normal flow depth in the natural channel for the design discharge.

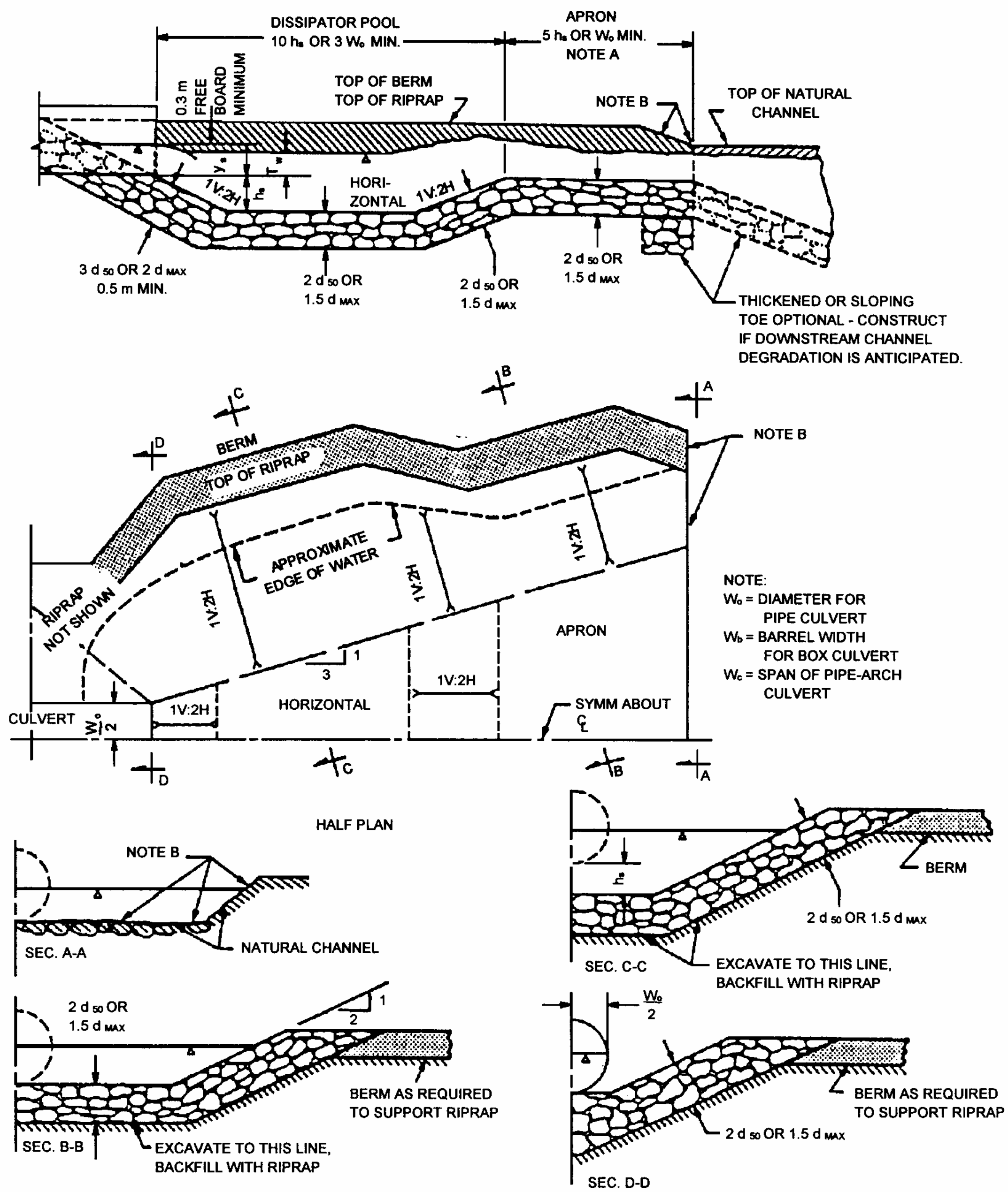
b. Extend the length of the energy basin (if necessary) so the width of the energy basin at section A-A, Figure 4.7-5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

- Design a conventional basin for low tailwater conditions in accordance with the instructions above.
- Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 4.7-9.
- Shape downstream channel and size riprap using Figure 4.7-1 and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled Use of Riprap For Bank Protection.



NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

Figure 4.7-5 Details of Riprap Outlet Basin
(Source: HEC-14, 1983)

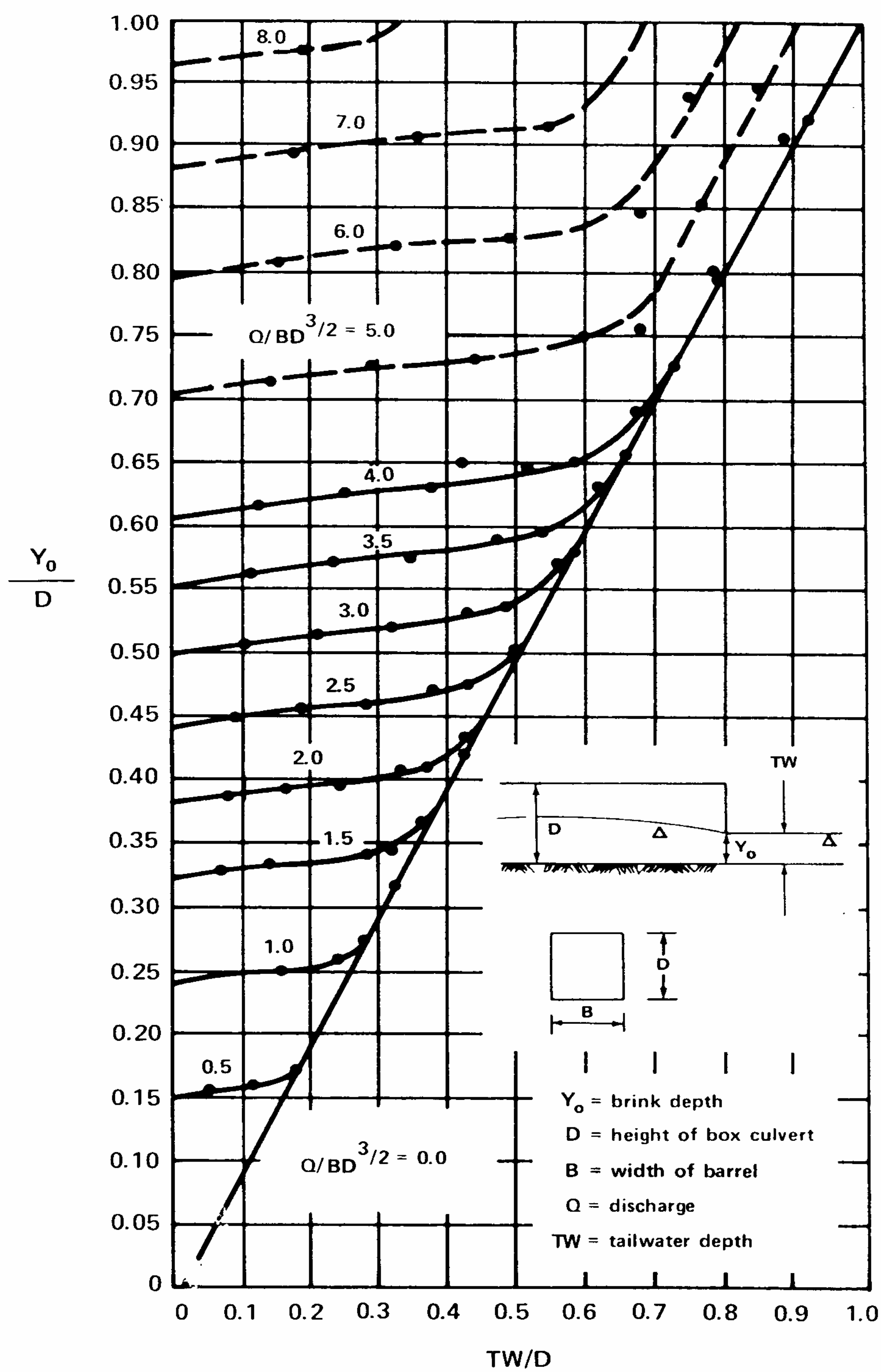


Figure 4.7-6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes
(Source: USDOT, FHWA, HEC-14, 1983)

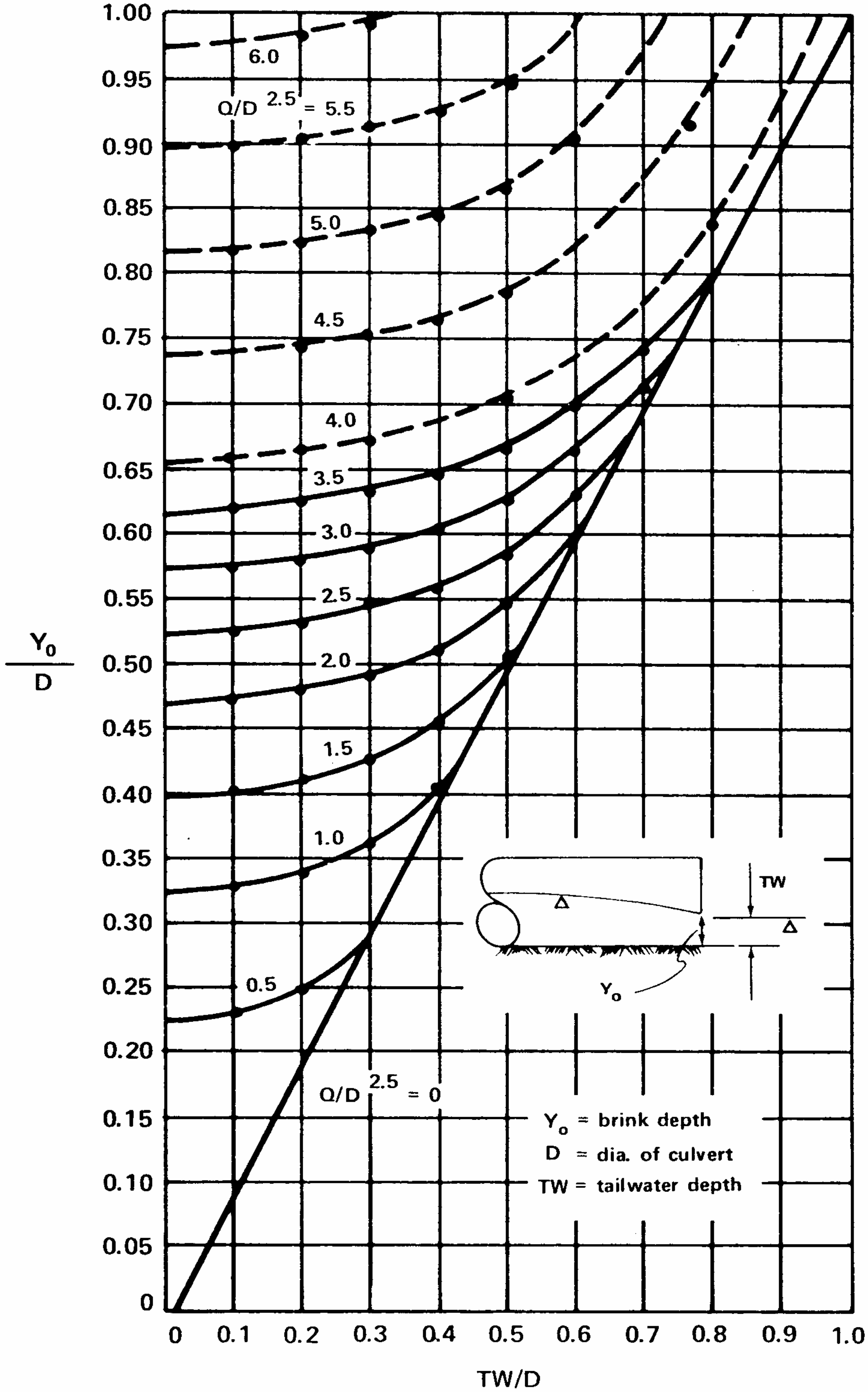


Figure 4.7-7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes
(Source: USDOT, FHWA, HEC-14, 1983)

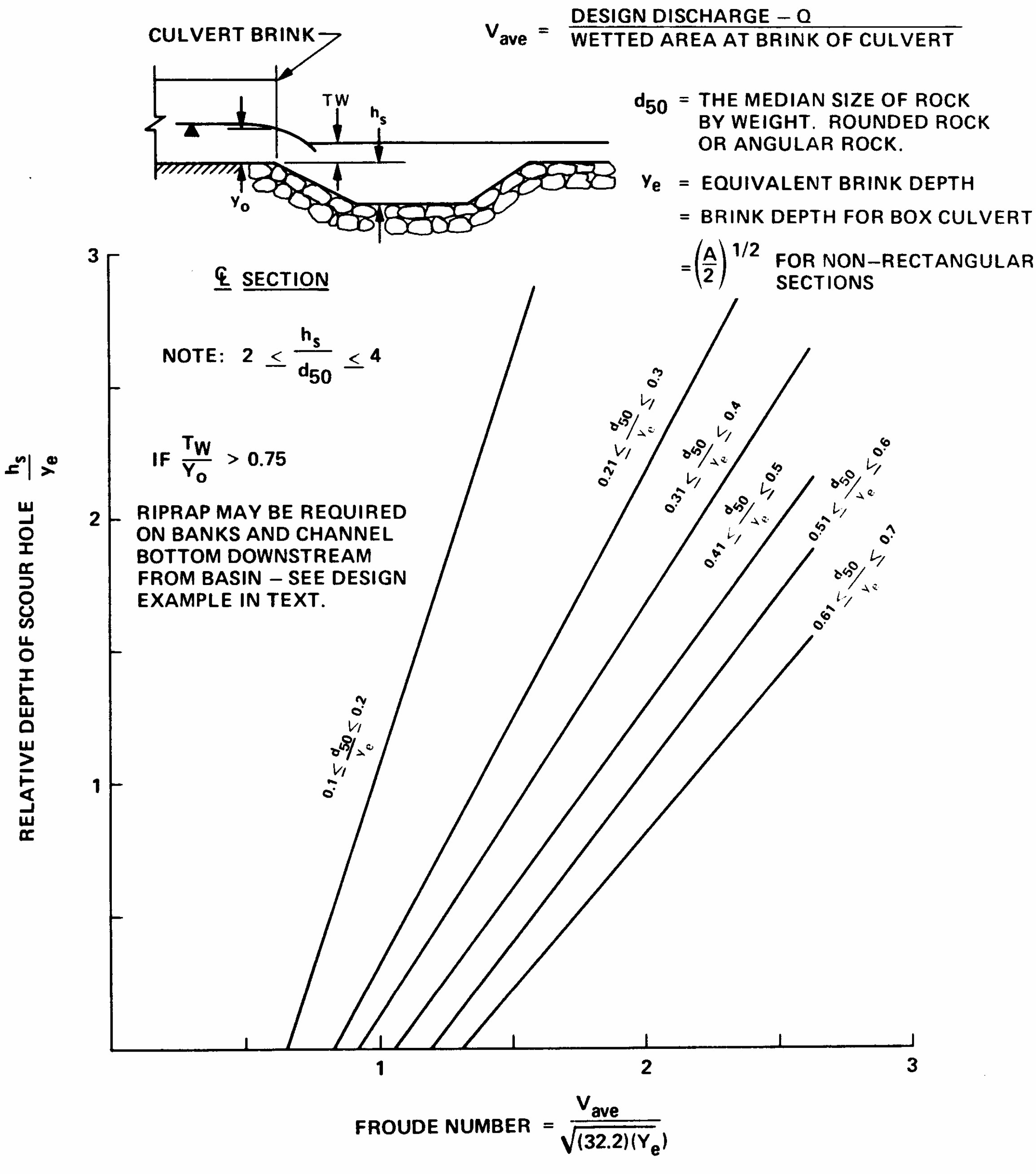


Figure 4.7-8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable
(Source: USDOT, FHWA, HEC-14, 1983)

4.7.5.4 Design Considerations

Riprap basin design should include consideration of the following:

The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.

When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.

The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.

For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

See Standards in the in FHWA HEC No. 11 for details on riprap materials and use of filter fabric.

Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in Section 4.4, *Open Channel Design.*

4.7.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.

Example 1

Given: Box culvert - 8 ft by 6 ft; Design Discharge Q = 800 cfs
Supercritical flow in culvert; Normal flow depth = brink depth
Y_o = 4 ft; Tailwater depth TW = 2.8 ft

Find: Riprap basin dimensions for these conditions

Solution: Definition of terms in Steps 1 through 5 can be found in Figures 4.7-5 and 4.7-8.

$y_o = y_e$ for rectangular section; therefore, with y_o given as 4 ft, y_e = 4 ft.

$V_o = Q/A = 800/(4 \times 8) = 25$ ft/s

Froude Number = $Fr = V/(g \times y_e)^{0.5}$ ($g = 32.2$ ft/s^2)

$Fr = 25/(32.2 \times 4)^{0.5} = 2.20 < 2.5$ O.K.

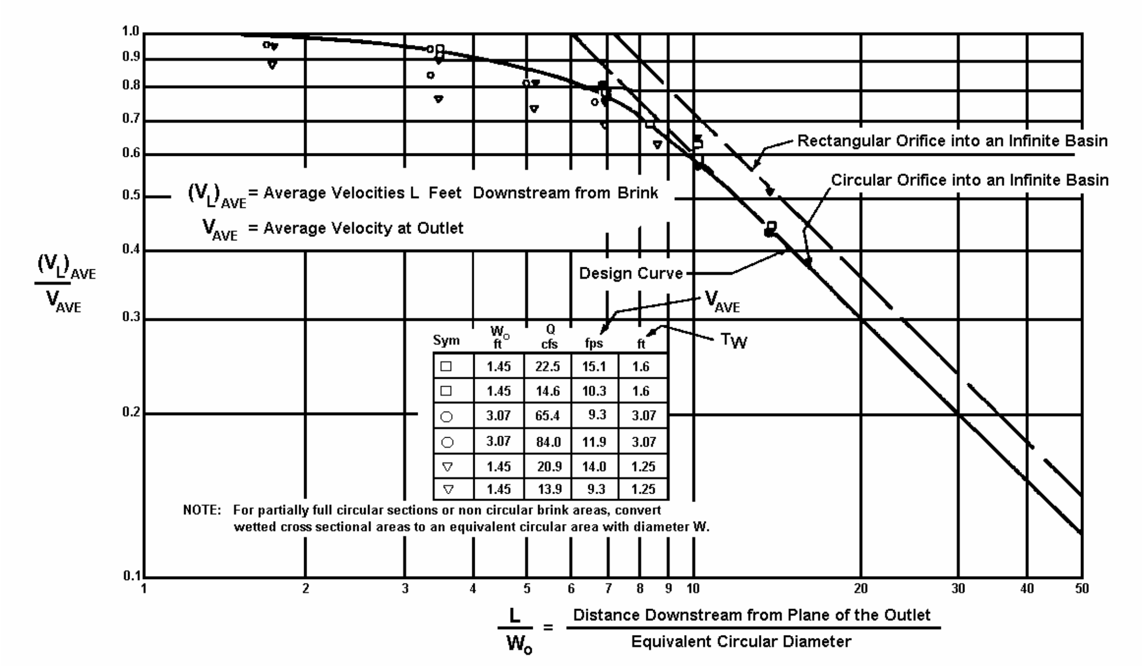


Figure 4.7-9 Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails
(Source: USDOT, FHWA, HEC-14, 1983)

$TW/y_e = 2.8/4.0 = 0.7$ Therefore, $TW/y_e < 0.75$ OK

Try $d_{50}/y_e = 0.45$, $d_{50} = 0.45 \times 4 = 1.80$ ft
From Figure 4.7-8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ ft
$h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 6.4 = 64$ ft (L_s = length of energy dissipator pool)
L_s min $= 3 \times W_o = 3 \times 8 = 24$ ft; therefore, use $L_s = 64$ ft

$L_B = 15 \times h_s = 15 \times 6.4 = 96$ ft (L_B = overall length of riprap basin)
L_B min $= 4 \times W_o = 4 \times 8 = 32$ ft; therefore, use $L_B = 96$ ft

Thickness of riprap: On the approach $= 3 \times d_{50} = 3 \times 1.8 = 5.4$ ft

Remainder $= 2 \times d_{50} = 2 \times 1.8 = 3.6$ ft
Other basin dimensions designed according to details shown in Figure 4.7-5.

Example 2

Given: Same design data as Example 1 except:

Tailwater depth TW = 4.2 ft
Downstream channel can tolerate only 7 ft/s discharge

Find: Riprap basin dimensions for these conditions

Solutions: Note -- High tailwater depth, $TW/y_o = 4.2/4 = 1.05 > 0.75$

From Example 1: $d_{50} = 1.8$ ft, $h_s = 6.4$ ft, $L_s = 64$ ft, $L_B = 96$ ft.

Design riprap for downstream channel. Use Figure 4.7-9 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32$ ft^2

$D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft

$V_o = 25$ ft/s (From Example 1)

Set up the following table:

L/D_e	L (ft)	V_L/V_o	v_1 (ft/s)	Rock Size d_{50} (ft)
(Assume)	(Compute)	(Fig. 4.7-9)	(Fig. 4.7-1)	$D_e = W_o$
10	64	0.59	14.7	1.4
15*	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

*L/W_o is on a logarithmic scale so interpolations must be done logarithmically.

Riprap should be at least the size shown but can be larger. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication.

Example 3

Given: 6-ft diameter CMC
Design discharge Q = 135 cfs
Slope channel $S_o = 0.004$
Manning's n = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 ft
Normal velocity is 5.9 ft/s
Flow is subcritical
Tailwater depth TW = 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

Determine y_o and V_o

$Q/D^{2.5} = 135/6^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

From Figure 4.7-7, $y_o/D = 0.45$

$y_o = .45 \times 6 = 2.7$ ft

$TW/y_o = 2.0/2.7 = 0.74$ $TW/y_o < 0.75$ O.K.

Determine Brink Area (A) for $y_o/D = 0.45$

From Uniform Flow in Circular Sections Table (from Section 4.3)

For $y_o/D = d/D = 0.45$

$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3$ ft^2

$V_o = Q/A = 135/12.3 = 11.0$ ft/s

For Froude number calculations at brink conditions,
$y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

Froude number = Fr = $V_o/(32.2 \times y_e)^{1/2} = 11/(32.2 \times 2.48)^{1/2} = 1.23 < 2.5$ OK

For most satisfactory results, $0.25 < d_{50}/y_e < 0.45$
Try $d_{50}/y_e = 0.25$
$d_{50} = 0.25 \times 2.48 = 0.62$ ft
From Figure 4.7-8, $h_s/y_e = 0.7$; therefore, $h_s = 0.7 \times 2.48 = 1.74$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From Section 4.3)
Check: $h_s/d_{50} = 1.74/0.62 = 2.8$, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 1.74 = 17.4$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft;
therefore, use $L_s = 17.4$ ft

$L_B = 15 \times h_s = 15 \times 1.74 = 26.1$ ft or $L_B = 4 \times W_o = 4 \times 6 = 24$ ft;
therefore, use $L_B = 26.1$ ft

$d_{50} = 0.62$ ft or use $d_{50} = 8$ in

Other basin dimensions should be designed in accordance with details shown on Figure 4.7-5. Figure 4.7-10 is provided as a convenient form to organize and present the results of riprap basin designs.

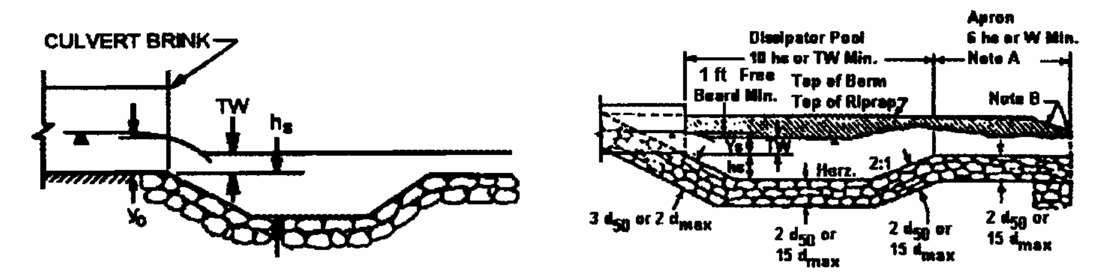
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the $3 \times d_{50}$ thickness of riprap on the approach and the $2 \times d_{50}$ thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

RIPRAP BASIN

Project No. ____________________

Designer ____________________ Date ________

Reviewer ____________________ Date ________



DESIGN VALUES	TRIAL 1	FINAL TRIAL
Equi. Depth, d_E		
D_{50}/d_E		
D_{50}		
Froude No., Fr		
h_s/d_E		
h_s		
h_s/D_{50}		
$2 < h_s/D_{50} < 4$		

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_s$		
	$3W_o$		
Basin length is the larger of:	$15h_s$		
	$4W_o$		
Approach Thickness	$3D_{50}$		
Basin Thickness	$2D_{50}$		

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_E	
TW/d_E	
IF $TW/d_E > 0.75$, calculate riprap downstream	
$D_E = (4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP				
L/D_E	L	V_L/V_o	V_L	D_{50}

Figure 4.7-10 Riprap Basin Design Form
(Source: USDOT, FHWA, HEC-14, 1983)

4.7.6 Baffled Outlets

4.7.6.1 Description

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 4.7-11. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

4.7.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 4.7-11 should be calculated as follows:

Determine input parameters, including:

h = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)

Q = Design discharge (cfs)

v = Theoretical velocity (ft/s = 2gh)

A = Q/v = Flow area (ft^2)

d = $A^{0.5}$= Representative flow depth entering the basin (ft) *assumes square jet*

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

g = Acceleration of gravity (32.2 ft/s)

Calculate the minimum basin width, W, in ft, using the following equation.

$$\mathbf{W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566}} \qquad \mathbf{(4.7.2)}$$

Where:

W = minimum basin width (ft)

d = depth of incoming flow (ft)

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

Calculate the other basin dimensions as shown in Figure 4.7-11, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

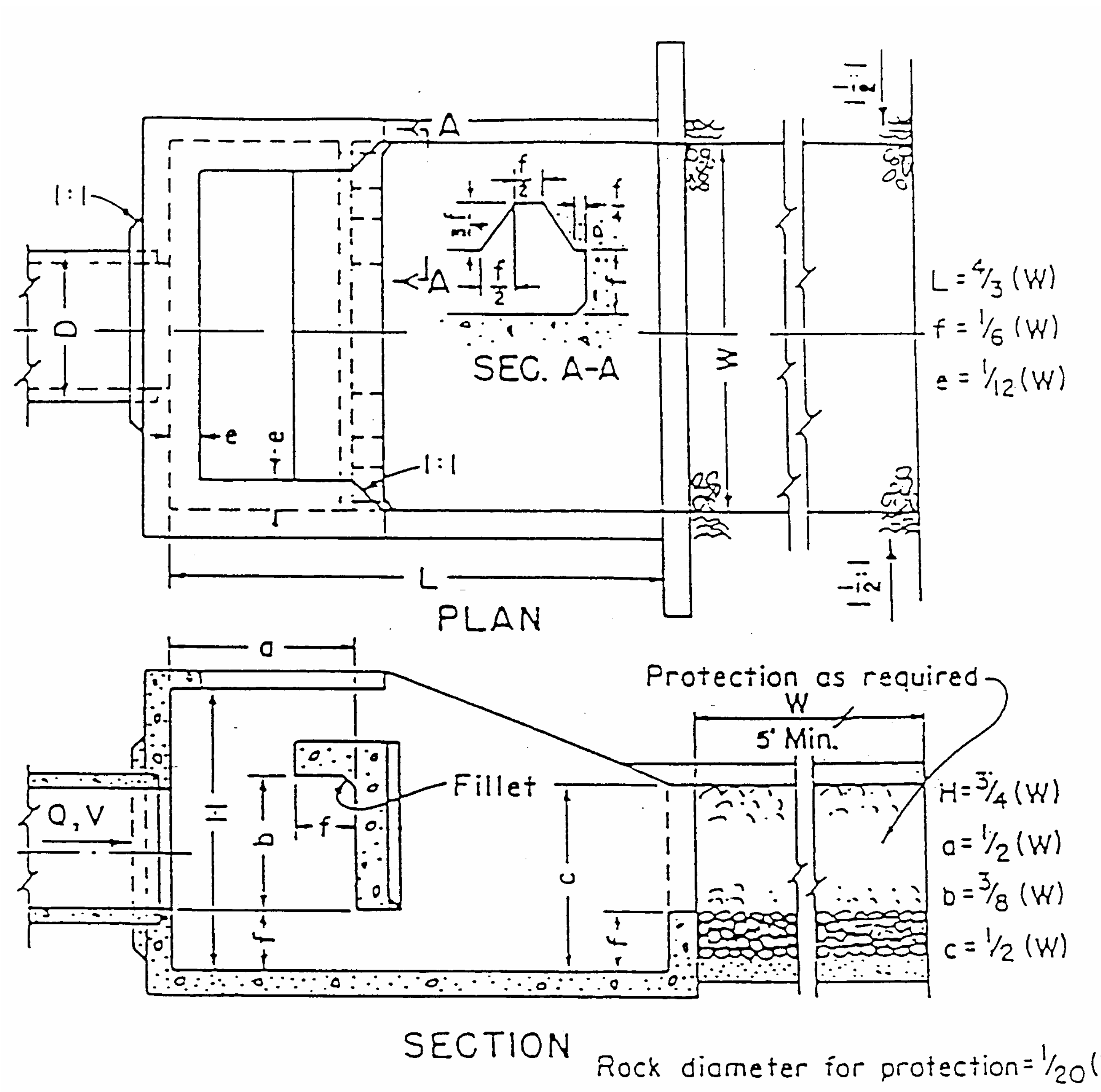


Figure 4.7-11 Schematic of Baffled Outlet
(Source: U.S. Dept. of the Interior, 1978)

If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

4.7.6.3 Example Design

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from
   $v = (2gh)^{0.5} = [2(32.2 \text{ ft/sec}^2)(15 \text{ ft})]^{0.5} = 31.1$ ft/s
   This is less than 50 ft/s, so a baffled outlet is suitable.

2. Determine the flow area using the theoretical velocity as follows:
   $A = Q/v = 150 \text{ cfs}/31.1 \text{ ft/sec} = 4.8 \text{ ft}^2$

3. Compute the flow depth using the area from Step 2.
   $d = (A)^{0.5} = (4.8 \text{ ft}^2)^{0.5} = 2.12$ ft

4. Compute the Froude number using the results from Steps 1 and 3.
   $Fr = v/(gd)^{0.5} = 31.1 \text{ ft/sec}/[(32.2 \text{ ft/sec}^2)(2.12 \text{ ft})]^{0.5} = 3.8$

5. Determine the basin width using equation 4.7.2 with the Froude number from Step 4.
   $W = 2.88\ dFr^{0.566} = 2.88\ (2.12)\ (3.8)^{0.566} = 13.0$ ft (minimum)
   Use 13 ft as the design width.

6. Compute the remaining basin dimensions (as shown in Figure 4.7-11):
   L = 4/3 (W) = 17.3 ft, use L = 17 ft, 4 in
   f = 1/6 (W) = 2.17 ft, use f = 2 ft, 2 in
   e = 1/12 (W) = 1.08 ft, use e = 1 ft, 1 in
   H = 3/4 (W) = 9.75 ft, use H = 9 ft, 9 in
   a = 1/2 (W) = 6.5 ft, use a = 6 ft, 6 in
   b = 3/8 (W) = 4.88 ft, use b = 4 ft, 11 in
   c = 1/2 (W) = 6.5 ft, use c = 6 ft, 6 in

   Baffle opening dimensions would be calculated as shown in Figure 4.7-11.

7. Basin invert should be at b/2 + f below tailwater, or
   (4 ft, 11 in)/2 + 2 ft, 2 in = 4.73 ft
   Use 4 ft 8 in; therefore, invert should be 2 ft, 8 in below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20, or about 8 in.

9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/s.
   $(3.14d)^2 /4 = Q/v$; $d = [(4Q)/3.14v)]^{0.5} = [(4(150 \text{ cfs})/3.14(12 \text{ ft/sec})]^{0.5} = 3.99$ ft
   Use 48-in pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 in.

4.7.7 Grade Control Structures

When channels are relocated through non-stable soils and stream gradients are increased, the stream bottom may degrade or dig itself deeper. This can cause bank instability, increased upstream scouring, and sloughing of natural slopes. The U.S. Soil Conservation Services (SCS) requires that streambed stability be maintained in any of its stream projects. This can be accomplished by grade stabilization structures; in essence a series of low-head weirs.

If designed and constructed with ecological values in mind, these structures can double as habitat enhancement devices. If improperly planned however, they can atually degrade habitat values. The most productive method of installing these structures is to use low weirs that pool water just a short distance (approximately 100 feet) upstream. A plunge pool will form just below the structures, and a riffle area should develop below this pool. The next structure should be located downstream a sufficient distance to avoid impounding the riffle area below the pool at the base of the upstream weir.

Specific construction requirements and techniques can be obtained from the SCS or other agencies upon request. The intent of this general discussion of grade stabilization structures is to promote consideration of such measures early in the planning process.

Source: US Army Corp of Engineers, Nashville District, *"Mitigating the Impacts of Stream Alterations"*, unkn.

References

Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14.

Federal Highway Administration, 1967. Use of Riprap for Bank Protection. Hydraulic Engineering Circular No. 11.

Searcy, James K., 1967. Use of Riprap for Bank Protection. Federal Highway Administration.

USDA, SCS, 1975. Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas, College Park, Maryland.

U.S. Department of Interior, Bureau of Reclamation, 1978. Design of Small Canal Structures.