

CHAPTER 11
ENERGY DISSIPATORS

Chapter Table of Contents

11.1	Introduction.....	11-3
11.2	Policy and Practice.....	11-3
11.3	Sources of Information	11-4
11.4	Results of Dissipator Design Studies	11-5
11.5	Types and Selection of Energy Dissipators.....	11-5
11.6	Dissipator Design and Check Discharges	11-14
11.7	Hydraulics of Conduit and Channel Outlets.....	11-15
11.7.1	Example: Hydraulic Characteristics of Flow at the Brink of a Box	11-22
11.7.2	Example: Hydraulic Characteristics of Flow at the Brink of an Arch.....	11-24
11.8	Estimating the Size of an Unlined Scour Hole	11-26
11.8.1	Example: Estimating the Size of an Unlined Scour Hole.....	11-29
11.9	Design of Unlined Pools.....	11-31
11.10	Design of Internal Energy Dissipators	11-33
11.11	Design of Riprap Pads	11-40
11.11.1	Example: Designing a Riprap Pad	11-47
11.12	Design of Riprap Lined Basins	11-48
11.12.1	Scour Protection Downstream From Basin Outlet.....	11-56
11.12.2	Example: Designing a Riprap Lined Basin.....	11-57

--Figures--

Figure 11-1 Unlined Pool.....	11-7
Figure 11-2 Internal Energy Dissipators.....	11-9
Figure 11-3 Riprap Pad.....	11-10
Figure 11-4 Riprap Lined Basin	11-11
Figure 11-5 Drop Structures in a Storm Sewer.....	11-12
Figure 11-6 Complex Dissipators	11-13
Figure 11-7 Flow in a Steep Channel Entering a Pool.....	11-18
Figure 11-8 Brink Depth Rating Curves for Rectangular Conduits on Mild Slopes	11-20
Figure 11-9 Brink Depth Rating Curves for Circular Conduits on Mild Slopes	11-21
Figure 11-10 Scour Hole Geometry	11-28
Figure 11-11 Outlet Into Unlined Pool	11-35
Figure 11-12 Upper Ends of Tee Dissipators	11-36
Figure 11-13 Slope Pipe, Cleanout Gate, and Slip Joint Details for Tee Dissipators.....	11-38
Figure 11-14 Lower End of Tee Dissipator	11-39
Figure 11-15 Riprap Around Outlet of Tee Dissipator.....	11-41
Figure 11-16 Dimensions of Riprap Pads.....	11-42
Figure 11-17 ODOT Class 50 Riprap Energy Dissipator Selection Chart	11-43
Figure 11-18 ODOT Class 100 Riprap Energy Dissipator Selection Chart	11-44
Figure 11-19 ODOT Class 200 Riprap Energy Dissipator Selection Chart	11-45
Figure 11-20 ODOT Class 700 Riprap Energy Dissipator Selection Chart	11-46
Figure 11-21 Elevation View of Riprap Lined Basins - Fish Passage Not Required.....	11-49
Figure 11-22 Elevation View of Riprap Lined Basin - Fish Passage Required.....	11-50
Figure 11-23 Cross-Sections of Riprap Lined Basin	11-51
Figure 11-24 Plan View of Riprap Lined Basin	11-52
Figure 11-25 Weir Coefficients and Submergence Factors for Unsubmerged and Partially Submerged Riprap Lined Basin Outlets.....	11-55

--Tables--

Table 11-1 Coefficients for Unlined Scour Holes in Cohesionless Materials	11-30
---	-------

11.1 Introduction

This chapter provides information for the planning and design of energy dissipators at the outlets of open-channels or closed conduits such as culverts or storm sewers. The methodology is intended for those with an understanding of basic hydrologic and hydraulic methods and some experience in the design of hydraulic structures. Most of this chapter is based on information and methodology in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular Number 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (HEC-14).

Flowing water, due to its movement, possesses kinetic energy. This energy is often expressed in hydraulic engineering as "velocity head" and it is related to the velocity of the fluid as follows:

$$H_{\text{velocity}} = \frac{V^2}{2g} \quad \text{(Equation 11-1)}$$

Where:

H_{velocity} = Velocity head in feet

V = flow velocity in feet per second

g = the acceleration of gravity (32.2 feet per second squared, or ft/s²)

As shown by the preceding equation, the kinetic energy of water is highly dependent on its velocity, and the faster the water travels the greater its velocity head and kinetic energy.

The kinetic energy of a flow is directly related to its ability to move particles, and the faster the flow, the larger the particles that it can move. This ability to move material gives water the means to erode and scour channels and conduits. As a result, a durable hydraulic conveyance system should have flow velocities that do not exceed the maximum velocities the conduit and channel lining materials can withstand.

The ability of a lining material to withstand scour and erosion damage varies considerably. The hard materials used to line conduits and artificial channels such as metal, concrete, steel, plastic, wood and rock can withstand fairly high velocities, as discussed in **Chapter 5**. Conversely, the softer materials that line typical natural channels like soil and vegetation can withstand much slower velocities, as discussed in **Chapter 8**.

Energy dissipators are used to reduce the velocity, and consequently, the erosion potential of flowing water. Their most common use is to reduce the outlet flow velocities from conduits that discharge onto embankments, into natural or unlined channels, or into drainage swales. Occasionally they are also used to slow the flow out of lined open channels such as spillways, ditches, and other hydraulic structures.

11.2 Policy and Practice

General policies of the Federal government and ODOT pertaining to hydraulic design are discussed in **Chapter 3** of this manual. General Geo-Environmental Section practice specific to energy dissipator selection and design include:

- coordination with other Federal, state, and local agencies concerned with water resources planning will have a high priority,
- safety of the general public and protection of property is an important consideration,
- the passage of ice or debris, if present, shall be considered,
- the frequency and type of maintenance and allowances for the access of maintenance equipment and personnel shall be considered,
- sediment deposition and scour shall be considered and minimized when possible,
- environmental impacts such as fish passage and disturbance of fish habitat, wetlands, and riparian areas shall be considered,
- the hydraulic performance of the dissipator shall consider the class of the roadway, consequences of scour and erosion, economics, and local site conditions.
- The dissipator should reduce the velocity to or below that which exists in the natural channel

11.3 Sources of Information

The type, source, and amount of data needed for a dissipator design will vary depending on the size and complexity of the dissipator. Large or complex dissipators typically require data similar to a large culvert location survey and the needed information is usually supplied with the culvert location data. If the dissipator is to be used on a storm sewer outlet or other application, a special survey is often needed to provide the needed information. Large or complex dissipator designs typically require:

- a comprehensive contour map or terrain model of the site providing the centerlines of the upstream and downstream conduits and/or channels, the roadway, the locations of utilities and developed property such as buildings, the edges of agency right-of-way and drainage easements,
- cross-sections and profiles of the conduits and/or channels and the roadway,

- description of the streambed or other items to be protected, including soil and geological information,
- photographs of the site,
- navigation, environmental, and fish passage requirements, and
- known history of the site, including recollections of past floods and damages, damage complaints, and names and phone numbers of involved agencies and neighboring landowners.

Simple or small dissipators can often be designed using the roadway location data. The dissipator design requires calculation of the site hydrology/hydraulics and additional information is often needed, such as the watershed area and the type of land cover (forest, suburban lots, etc). An inspection of the site and interviews with maintenance personnel can often be a valuable supplement to the location survey.

11.4 Results of Dissipator Design Studies

A dissipator at the end of a conduit is usually designed at the same time as the facility that provides the incoming flow. In these cases the information required from the dissipator design is listed in the guidelines in **Chapter 4** of this manual for culverts or storm sewers. Occasionally a dissipator is designed separately from the upstream channel or conduit or the dissipator is designed to handle flow from a source other than a culvert or a storm sewer. In this case the following information is needed:

- location of the dissipator including highway, milepoint, etc,
- design flow and recurrence interval,
- dimensions of the dissipator, including a sketch,
- recommended materials,
- details of channel changes, if needed,
- descriptions of special design considerations including drawings as needed,
- fish passage and other environmental data as needed, and
- descriptions of maintenance needs if they will be atypical or unusually frequent.

11.5 Types and Selection of Energy Dissipators

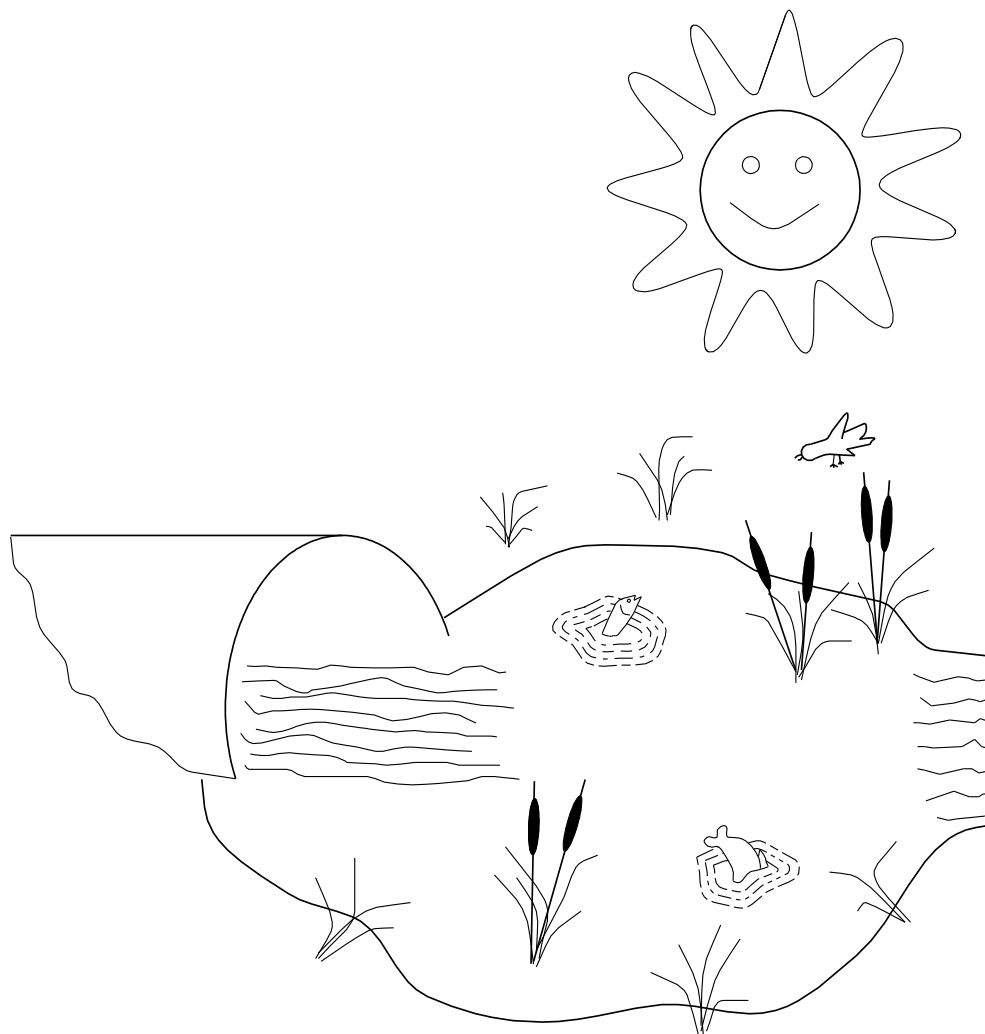
There are many kinds of energy dissipators, and most are either internal or external. Internal dissipators are located within the upstream conduit and they slow the flow before it leaves the conduit. External dissipators are located at the outlet end of conduits or channels. Some of the more common types are listed in this section with guidance on their selection.

Unlined Pools - These external dissipators consist of water filled pools, as shown in Figure 11-1. These pools can be located at the outlets of conduits or channels. The sides and bottom of the pool are not completely covered by revetment, and hence the name "unlined pool." Energy is dissipated as turbulence when the higher velocity flow from the upstream conduit or channel collides with the water that is ponded in the pool.

The pool can be formed by the scouring forces from the higher velocity flow that exits the upstream conduit or channel. This type of pool is commonly called a "natural scour hole," and its size can be predicted by methods in Section 11.8 of this chapter and its design is discussed in Section 11.9. Considerable sediment can be washed downstream when this pool forms, and the effects of this material on the downstream channel should be considered. The banks of these pools are often composed of the soils and rocks that underlie the site with a covering of the vegetation that naturally establishes itself over time.

The natural scour hole can be excavated by construction equipment and personnel instead of natural forces. This excavation reduces the amount of sediment that is carried downstream. At a minimum, these pools usually have the shape and size of a naturally occurring scour hole. The banks can be planted with vegetation and sometimes revetment is used on sections of the bank to protect critical areas from erosion.

These basins can be designed to provide fish habitat and passage. Typically an excavated scour hole is used in this application because there is less chance that a mound around the exterior of the pool may form that can obstruct fish passage.



Note: The designer must make provisions to insure that the stream and the pool do not degrade to a point where fish passage is blocked. This may be accomplished by providing a stable liner for the outlet channel or the pool

Figure 11-1 Unlined Pool

Note: An unlined pool in many circumstances can be considered to be a wetland by resource management agencies and regulations may prevent or limit future modifications or maintenance. These factors should be considered before this type of energy dissipator is selected.

Unlined pools are often used where:

- fish passage and/or wetland habitat are desired,
- the site has enough room to accommodate the pool and it is located within agency right-of-way or drainage easements,
- the outlet of the upstream channel or conduit, the highway facilities, and neighboring property can be protected from undermining or damage due to scour or erosion, and
- the pool is not a nuisance or safety hazard to the highway users or to the general public.

Internal Dissipators - This dissipator is used inside a closed conduit to slow the flow to an acceptable velocity before it enters the downstream channel. One type, roughness rings, uses concentric rings inside the conduit to slow the flow, and energy losses are created by the turbulence at the interface where the water contacts the rings. An example is shown in Figure 11-2a. Another type of internal dissipator is the "pipe tee" shown in Figure 11-2b. This device dissipates hydraulic energy by splitting the flow and abruptly changing its direction of travel. Regardless of the type of internal dissipator, riprap pads are usually placed at the outlets to further spread the flow and reduce the velocity to prevent scour of the channel or embankment. The design of these dissipators is discussed in Section 11.10.

Internal dissipators are often used where:

- their cost is justified by the ability to use a smaller, less costly, or more environmentally friendly type of external dissipator,
- ice or debris passage is not a problem,
- moderate velocity reduction is needed, and
- fish passage is not required.
- right-of-way is limited

Riprap Pad - This is the most common type of external energy dissipator for smaller conduits or channels. When this dissipator is used the flow crosses over the riprap before it enters the downstream channel. The roughness of the riprap creates turbulence in the flow and considerable hydraulic energy is dissipated. An example of a pad downstream from a conduit is

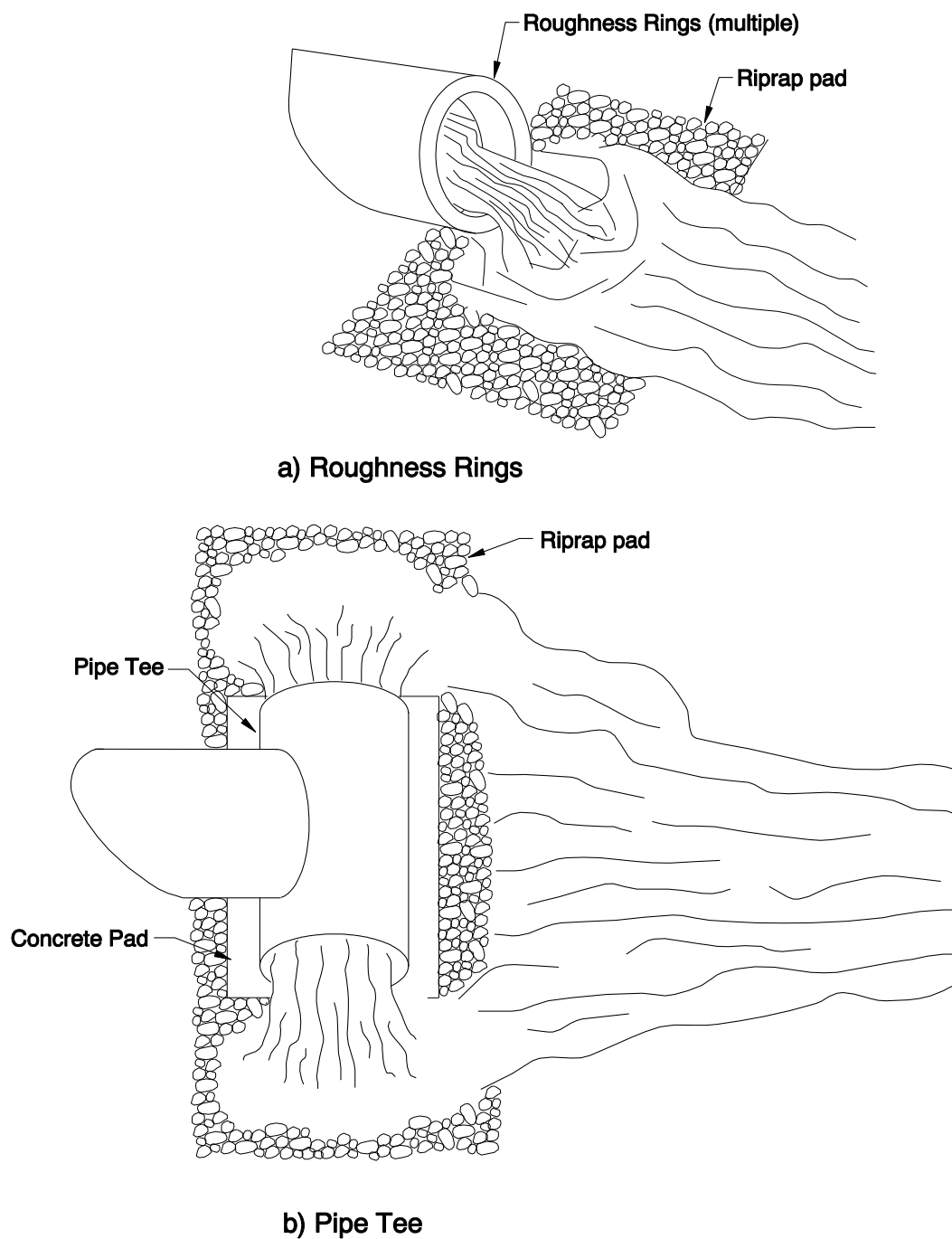


Figure 11-2 Internal Energy Dissipators

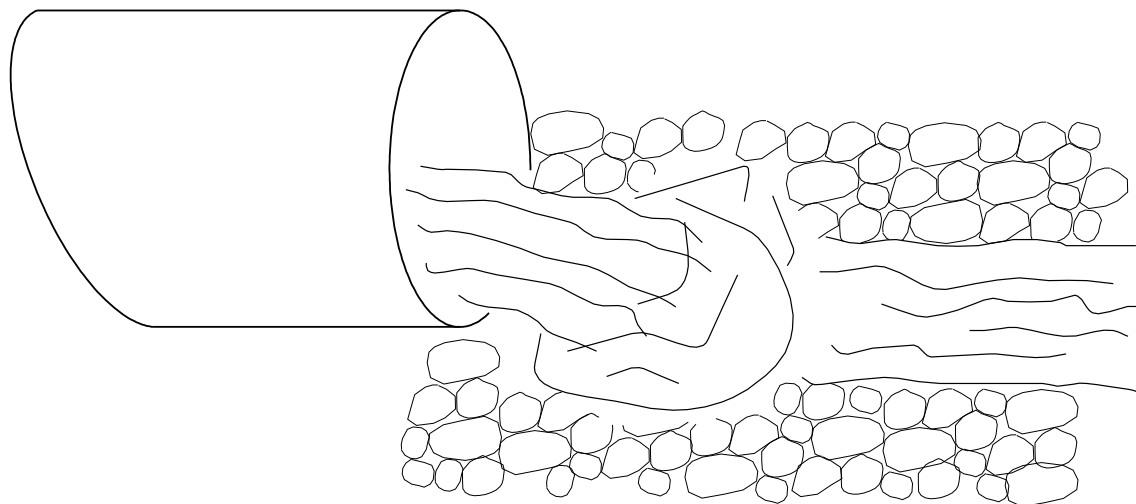


Figure 11-3 Riprap Pad

shown in Figure 11-3. These pads are also used in conjunction with many other types of dissipators and they are located immediately downstream from the dissipator outlet. The design of these dissipators is discussed in Section 11.11.

These pads are best suited for conduits with outlet flows having specific characteristics. The outlet should be free-flowing, in other words, the outlet should not be submerged by tailwater. The effectiveness of the riprap in creating turbulence is greatest at shallow flow depths and it lessens as the flow deepens. In addition, the outlet velocities should not be excessively high. Fast flow can displace the riprap and wash it downstream.

Riprap pads are often used where:

- a low-cost and easily constructed dissipator is needed,
- flow from the outlet of the conduit has moderate to low velocity and depth, and
- fish passage is not required.

Riprap Lined Basins - These external dissipators are most often used at the outlets of larger conduits or channels. They consist of a pool known as a preformed scour hole that is lined with riprap, as shown in Figure 11-4. Like an unlined pool, the velocity flow entering these dissipators collides with the relatively tranquil water in a pool and considerable hydraulic energy is dissipated. The riprap lining

protects the surrounding soil from the scour and erosion caused by the turbulent flow in the basin. The design of these dissipators is presented in Section 11.12.

Note: The depth and dimensions of the preformed scour hole depend on the size and quality of the rock riprap available.

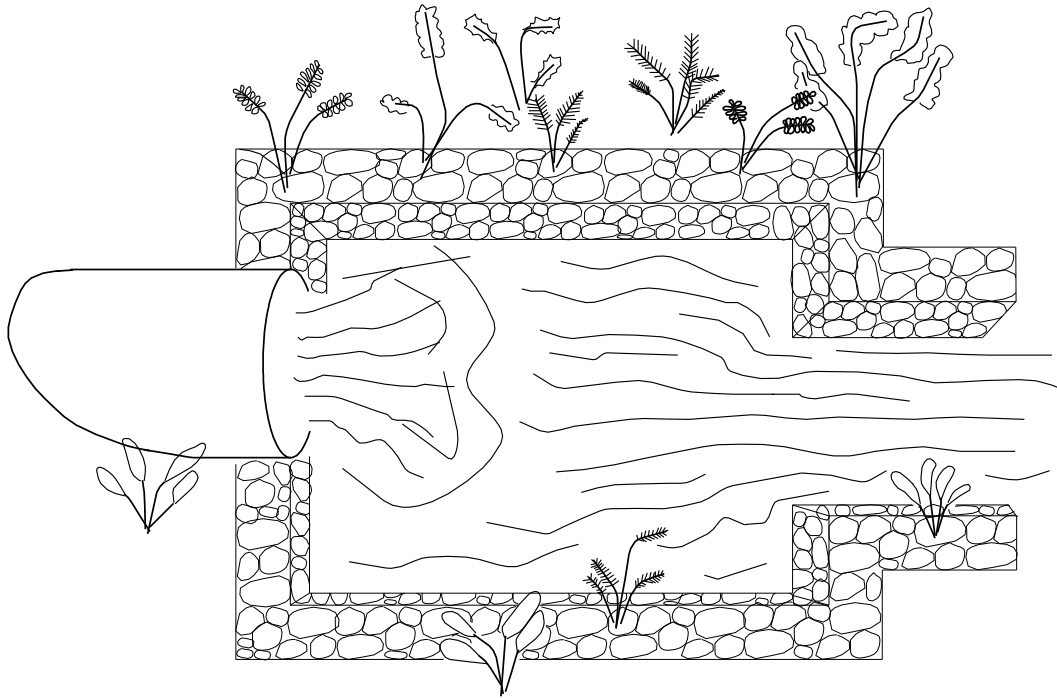


Figure 11-4 Riprap Lined Basin

These basins can be designed to pass fish and riparian plantings can be installed in the riprap at the basin edges in some locations. As a result, with the exception of unlined basins, these basins are some of the most environmentally friendly dissipators.

Riprap lined basins are often used where:

- fish passage and/or wetland habitat are desired and the site cannot accommodate an unlined pool,
- flow depths and velocities are too high for a riprap pad, and
- the pool is not a nuisance or safety hazard to the highway users or to the general public.

Drop Structures - In general, the flow velocity in a conduit is proportional to its slope, and the steeper the conduit, the higher the velocity. Drop structures are often used to flatten the slopes of the conduits in a drainage system, and consequently, they can reduce the flow velocities in the system and at the outlet. An example is shown in Figure 11-5.

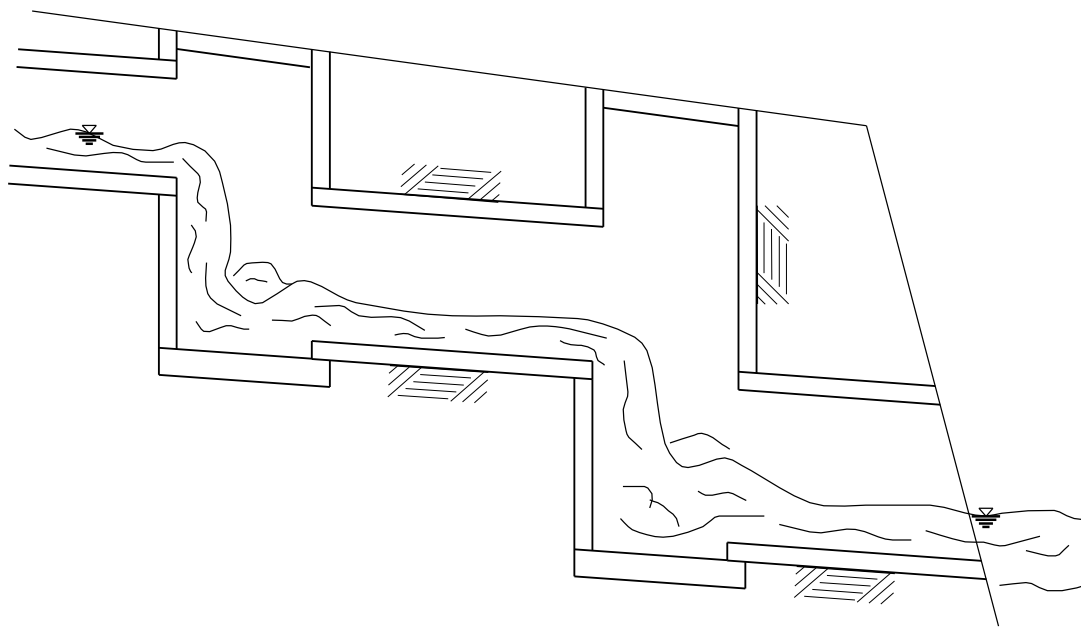
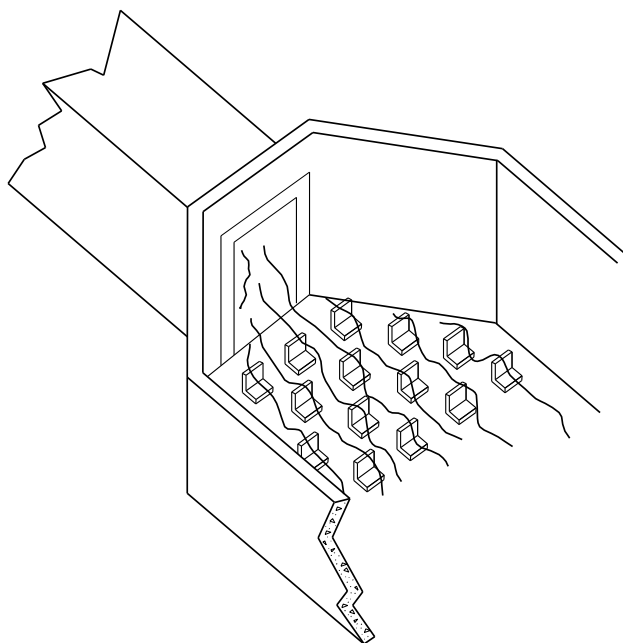


Figure 11-5 Drop Structures in a Storm Sewer

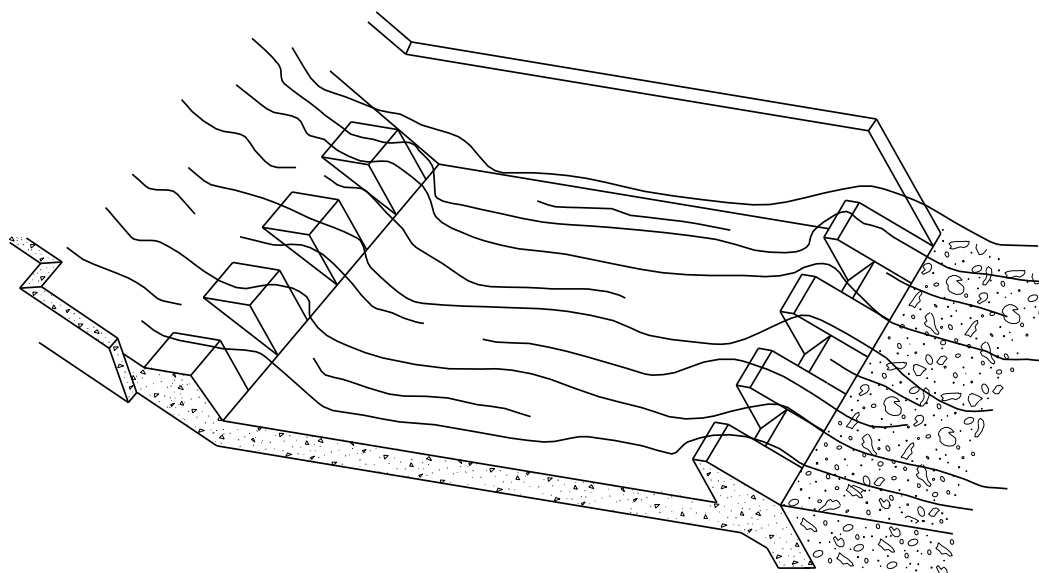
Drop structures are often used where:

- it is desired to reduce the velocities in conduits leading to the outlet as well as the velocity at the outlet,
- their cost is justified by the ability to use a smaller, less costly, or more environmentally friendly type of external dissipator,
- ice or debris passage is not a problem, and
- fish passage is not required.

Complex Dissipators - These external dissipators are used at the outlets of both conduits and channels. Most types use blocks, sills, walls, or other obstructions to slow the flow, they are made of cast-in-place reinforced concrete, and they are specifically designed for each site. They are called complex dissipators because they require considerable effort to design and build.



Colorado State University Rigid Boundary Basin



United States Bureau of Reclamation Type II Basin

Figure 11-6 Complex Dissipators

Examples of two of the many types of complex dissipators are shown in Figure 11-6. The design of these dissipators is not presented in this manual. The topic is presented in detail in the Federal Highway

Administration Hydraulic Engineering Circular No. 14: "Hydraulic Design of Energy Dissipators for Culverts and Channels" and the American Association of State Highway and Transportation Officials "Model Drainage Manual."

Complex dissipators are often used where:

- the outlet velocities or discharge volumes are too great for other types of dissipators,
- the areas occupied by other types of dissipators would be too large for the site, and
- normally, fish passage is not required, but there are circumstances where complex dissipators could be designed to incorporate and even assist fish passage.

11.6 Dissipator Design and Check Discharges

An essential step in the dissipator design is to determine the design and check discharges. Design and check flows for various types of drainage facilities are listed in **Chapter 3** and the hydrologic methods of calculating these discharges are presented in **Chapter 7**.

The dissipator should adequately dissipate the energy from the design flow and it should not be damaged by this flow. In most cases, the design discharges for the dissipator and the facility that discharges into the dissipator are the same. As an example, a culvert under a main highway has a 50-year design discharge. The dissipator would also have a 50-year design flow.

Dissipator design discharges that are larger than the upstream facility design flows are allowed in certain circumstances, as follows.

1. The failure of the dissipator or the inability of the dissipator to reduce flow energy could result in a catastrophic failure of a roadway or adjacent facility that would threaten human life or cause extensive property damage. As an example, the abutment of a railroad bridge needs to be protected from the discharge out of an ODOT culvert and a 25-year design flow was used to size the culvert. In this case a larger design flow may be justified to assure the bridge will not be damaged, such as the 50-year, 100-year, 500-year, or overtopping flow (see Note at end of this section).
2. Local ordinances, requirements of regulatory agencies, or agreements with neighboring property owners require a larger design flow than ODOT standards.

Check dissipators for a range of discharges as notes in table 3.1, **Chapter 3**. The dissipator should protect structures from undermining or other scour related damage during the check discharge. Structures include, but are not limited to: the outlet of the conduit if the diameter or span is 72-inch or larger, the dissipator itself if it is a complex dissipator, or a nearby structure. Typically culvert outlets,

complex dissipators, and similar structures are designed to provide a 75-year life without structural failure. Designing the dissipator to prevent damage during the 100-year flood helps to assure the structure will last as long as a 75 year design life.

Note: In some instances a lesser and more frequent flow than the design or check discharge can cause greater hydraulic stresses on the dissipator. This happens most often when road overtopping occurs more frequently than the design or check flow. If a more frequent flood causes greater hydraulic stresses than the design or check flood, the dissipator should be designed to dissipate the energy from, and withstand damage from, the more frequent flood.

11.7 Hydraulics of Conduit and Channel Outlets

The hydraulic characteristics of the flow at the outlet of a conduit or channel are the major factors that determine the type and size of the energy dissipator. The outlet of the conduit or channel is often called the "brink" and flow characteristics are calculated at this location. Several terms are used in energy dissipator design to describe this flow, as follows.

Brink Depth - There are two types of brink depths, actual and equivalent. The actual brink depth, y , is the actual depth of the flow at the brink.

y = Brink depth in feet

The equivalent brink depth, y_e , is a hypothetical depth of flow at the brink. It is used in certain hydraulic calculations to represent the brink depth in non-rectangular conduits or channels. It is based on the following equation:

$$y_e = \left(\frac{a}{2} \right)^{0.5} \quad \text{(Equation 11-2)}$$

Where:

y_e = Equivalent brink depth in feet

a = Cross-sectional area of flow in square feet

Froude Number - The Froude number is a dimensionless expression of the ratio of the inertial force of moving water to the force of gravity. Two forms of this expression are commonly used in dissipator design. One expression, Fr , is used to describe the flow out of rectangular shaped conduits or channels, as follows:

$$Fr = \frac{v}{(32.2y)^{0.5}} \quad \text{(Equation 11-3)}$$

Where:

Fr = Froude number of the flow at brink of rectangular conduit or channel, flowing full or partially full

V = Average flow velocity in feet per second

y = Brink depth in feet

The Froude number calculated by the preceding equation is applicable to rectangular conduits or channels flowing full or partially full. In most instances it does not provide realistic Froude numbers for non-rectangular sections. In some applications where non-rectangular conduits or channels flow full or partially full, an equivalent Froude number is used in dissipator design, and it is calculated as follows:

$$Fr_e = \frac{v}{(32.2y_e)^{0.5}} \quad \text{(Equation 11-4)}$$

Where:

Fr_e = Equivalent Froude number of the flow at the brink of a non-rectangular conduit or channel, flowing full or partially full

v = Average flow velocity in feet per second

y_e = Equivalent brink depth in feet

The flow characteristics at the brink of a conduit or channel usually depend on the magnitude of the flow, the slope of the conduit or channel, and the tailwater depth. Descriptions of several commonly encountered outlet conditions follow. In these descriptions there are many references to the terms and concepts of conduit and open-channel hydraulics. Concepts such as determining the properties of normal flow and critical depth in conduits are discussed in detail in **Chapter 9**. Useful information such as critical depth charts and the hydraulic properties of common conduits are presented in Appendix B, of **Chapter 8**. Concepts related to open-channel flow are discussed in **Chapter 8**, and Appendix A of **Chapter 8** lists the roughness values of many common conduits and channels.

Case 1: Partially Full Conduit or Channel on a Steep Slope with a Free Flowing Outlet - This flow type occurs when a conduit or channel is on a steep slope and the outlet is free flowing. In other words, there is supercritical flow in the conduit or channel and the tailwater depth does not influence the depth of the flow at the outlet. An example of a culvert with this profile is shown in Figure 9-11a. The most common occurrences of this profile are in culverts under inlet control with low tailwater or storm sewers and channels on steep slopes.

In Case 1 flow:

- the brink depth is the depth of normal flow,
- the equivalent brink depth is based on the area of normal flow, and
- the brink velocity is the velocity of normal flow.

Case 2: Partially Full Conduit on a Steep Slope with a Submerged or Partially Submerged Outlet

- This type of flow occurs at the outlets of conduits when there is supercritical flow at normal depth upstream from the outlet and the outlet is submerged or partially submerged. A culvert with this type of flow is shown in Figure 9-11b. The most common occurrence of this flow profile is in culverts under inlet control and storm sewers on steep slopes.

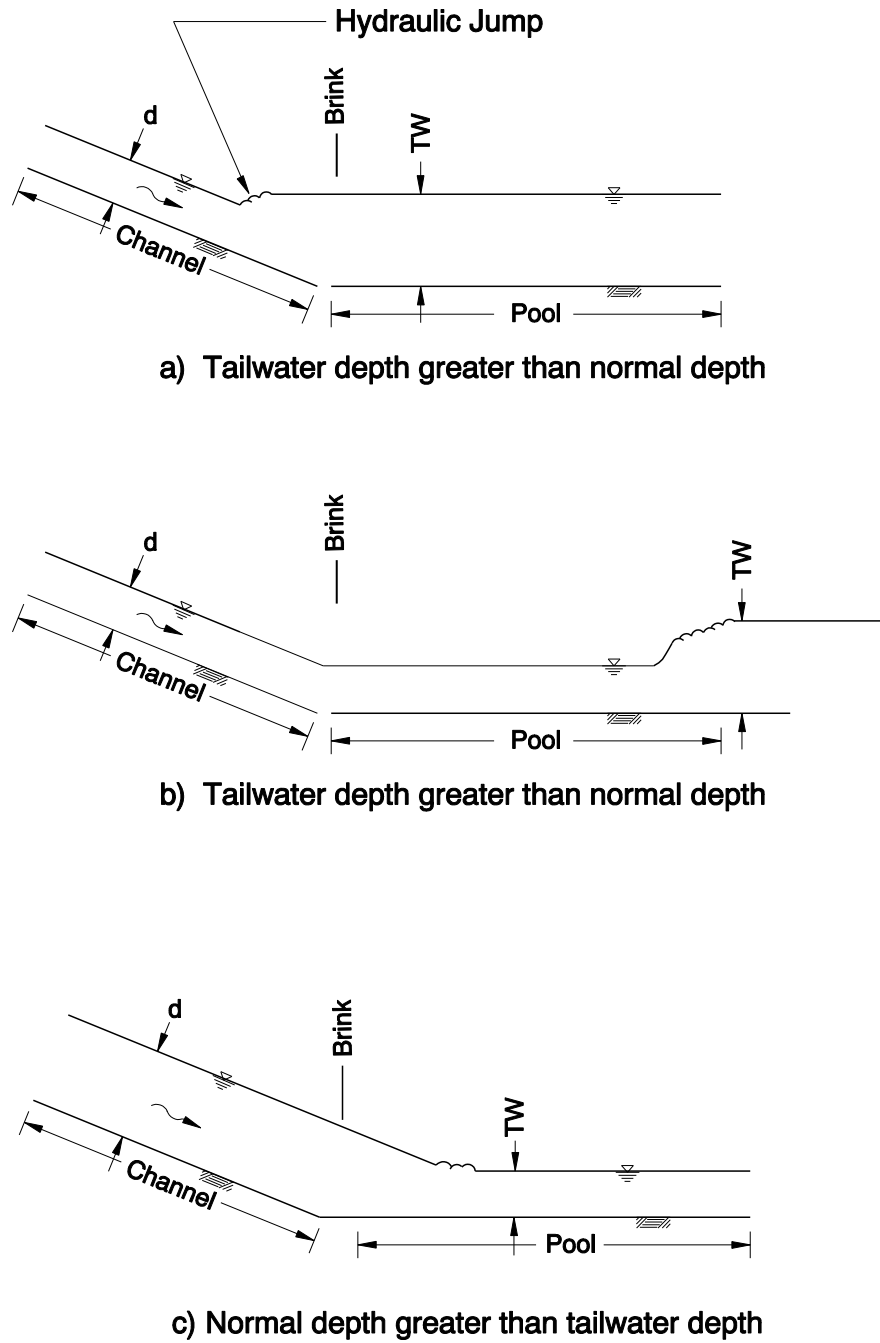
In Case 2 flow:

- the brink depth is the diameter or rise of the conduit if the outlet is fully submerged,
- the brink depth is the depth of the tailwater above the outlet invert, if this depth is greater than the depth of normal flow in the conduit,
- the brink depth is the depth of normal flow in the conduit, if this distance is greater than the depth of the tailwater above the outlet invert,
- the equivalent brink depths are based on the flow areas corresponding to the brink depths previously listed, and
- the brink velocities are based on the flow areas corresponding to the brink depths previously listed.

Case 3: Channel on a Steep Slope with Pooled Water at the Outlet - This flow type occurs in channels when there is supercritical flow at normal depth upstream from the outlet and there is slower moving subcritical flow or ponded water at the outlet. This profile often occurs at the outlets of lined ditches or channels that empty into ponds, canals, or other relatively slow moving or stagnant water bodies.

In Case 3 flow:

- the brink depth is the depth of the tailwater above the bottom of the upstream channel at the outlet, if this depth is greater than the depth of normal flow in the upstream channel, as shown in Figure 11-7a, unless the tailwater depth is greater than the resultant depth for a hydraulic jump, the brink depth will be normal depth because the jump will not occur here as shown in figure 11-7b.



d = Depth of normal flow in upstream channel
 TW = Depth of tailwater

Figure 11-7 Flow in a Steep Channel Entering a Pool

- the brink depth is the depth of normal flow in the upstream channel, if this depth is greater than the depth of the tailwater above the bottom of the upstream channel at the outlet, as shown in Figure 11-7c,
- the equivalent brink depths are based on the flow areas corresponding to the brink depths previously listed, and
- the brink velocities are based on the flow areas corresponding to the brink depths previously listed.

Case 4: Full Conduit on a Mild Slope with a Submerged Outlet - This type of flow occurs at the outlet of a conduit when there is full flow in the conduit and the outlet is submerged. This type of flow in a culvert is shown in Figure 9-14a. The most common occurrence of this flow profile is in culverts or storm sewers on mild slopes that discharge into other waterways.

In Case 4 flow:

- the brink depth is the diameter or rise of the conduit,
- the equivalent brink depths are based on the full flow area of the conduit, and
- the brink velocities are based on the flow areas corresponding to the brink depths previously listed.

Case 5: Full Conduit Under Surge with a Submerged, Partially Submerged, or Free-Flowing Outlet - This type of flow occurs when the conduit is full throughout its entire length and the outlet is partially submerged or free-flowing, as shown in Figure 9-14b. In this case the critical depth of the flow in the conduit is at least as large as the conduit diameter or rise. This type flow commonly occurs in culverts and storm sewers operating under surcharge.

In Case 5 flow:

- the brink depth is the diameter or rise of the conduit,
- the equivalent brink depth is based on the cross-sectional area of the full conduit, and
- the brink velocities are based on the flow areas of the brink depths previously listed.

Note: Outlet velocities in Case 5 flow can be extremely high. Complex dissipators are often needed to handle this type of flow.

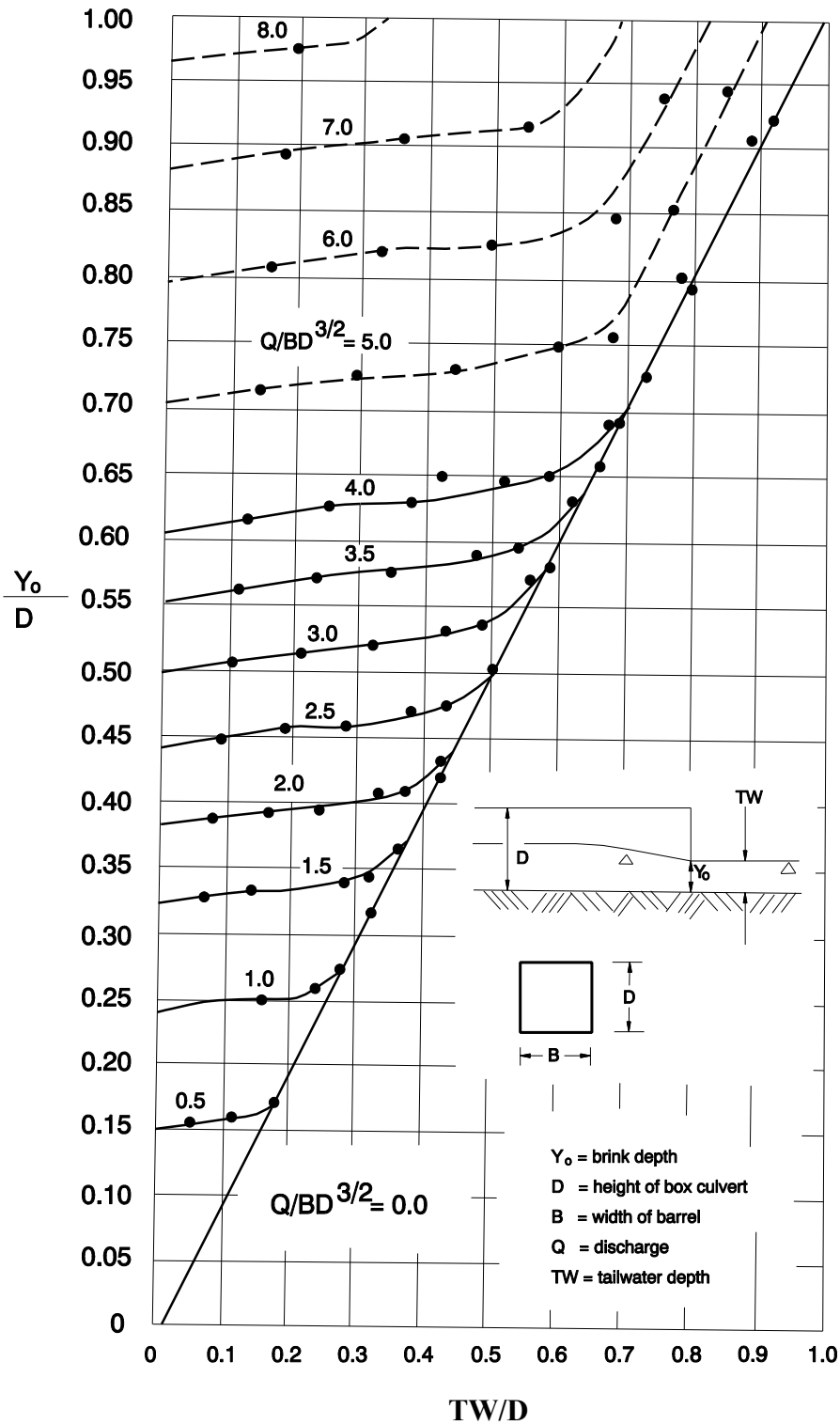


Figure 11-8 Brink Depth Rating Curves for Rectangular Conduits on Mild Slopes

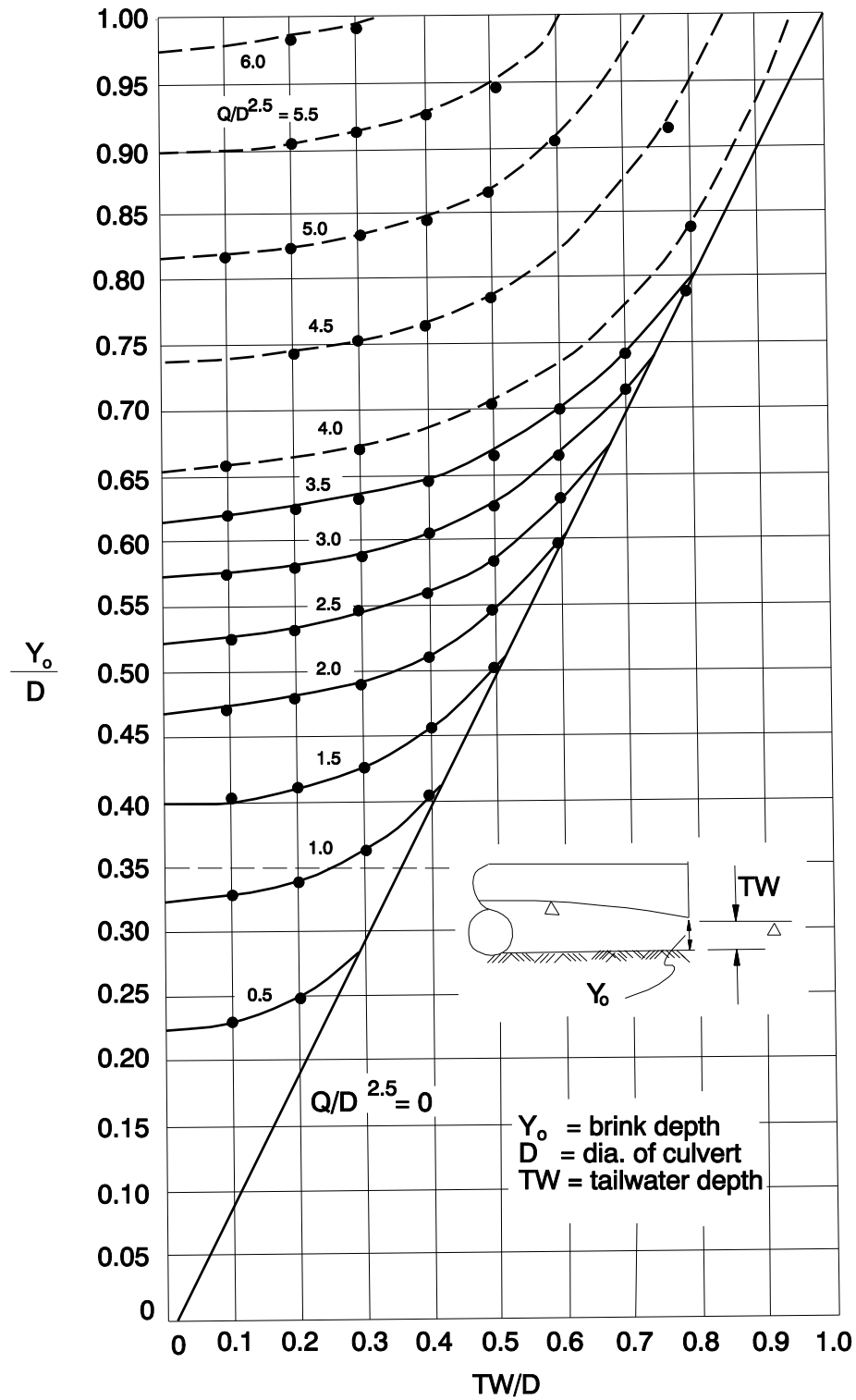


Figure 11-9 Brink Depth Rating Curves for Circular Conduits on Mild Slopes

Case 6: Full or Partially Full Conduit or Channel on a Mild Slope with a Partially Submerged or Free Flowing Outlet

This is the most common type of flow at the outlet of both conduits and channels, and it occurs when there is subcritical flow just upstream from the outlet and the outlet is not submerged. Culverts with this flow type are shown in Figures 9-14c and 9-14d, and these profiles can occur in other closed conduits such as storm sewers. Open-channels with this type of flow have profiles similar to Figure 9-14d.

During Case 6 flow the depth of flow at the brink is influenced by the amount of the discharge, the conduit or channel size and shape, and the tailwater elevation. The brink depths for rectangular and circular conduits can be determined from the rating curves in Figures 11-8 and 11-9, respectively. The brink depths for conduits and channels with other shapes can be approximated as follows:

- if the depth of the tailwater above the outlet invert is greater than critical depth in the upstream conduit or channel, assume the brink depth is the depth of the tailwater above the invert, or
- if critical depth in the upstream conduit or channel is greater than the depth of the tailwater above the outlet invert, assume the brink depth is the critical depth in the upstream conduit or channel.

11.7.1 Example: Hydraulic Characteristics of Flow at the Brink of a Box

The brink depth, brink velocity, and Froude number are needed for an old box culvert. The 50-year design flow is 459 cubic feet per second, the box has a 7 foot span and a 6 foot rise, and the culvert has a 0.01 foot per foot slope. Tailwater depths are assumed to be negligible.

Step 1: The first step is to determine the type of flow at the outlet. Since there is negligible tailwater, the outlet is free-flowing, and Case 1, Case 5, or Case 6 flow types can occur. In order to determine whether the conduit is flowing full with surcharge or partially full, the discharge that occurs when the conduit is full but not surcharged is calculated by Manning's equation for discharge (**Chapter 8**). The Manning's roughness coefficient, n , is assumed to be 0.015 because the box is made from worn concrete (Table 8-A-3 of Appendix A, **Chapter 8**). The cross-sectional area of the full conduit multiplied by the hydraulic radius to the $2/3$ power, $AR^{2/3}$, is 57.8. Using Manning's equation:

$$Q_{\text{FULL}} = \left(\frac{1.49}{0.015} \right) (57.8) (0.01^{0.5}) = 574 \text{ cubic feet per second}$$

$Q = 459$ cubic feet per second and Q is less than or equal to Q_{FULL} . Consequently, the conduit is partially full and Case 5 flow cannot exist.

A hydraulics elements chart for a 7 foot span by 6 foot rise box (Chart 11 of Appendix B, **Chapter 8**) is used to get the depth of normal flow, as follows:

$$\frac{Q}{Q_{\text{FULL}}} = \frac{q}{Q_{\text{FULL}}} = \frac{459}{574} = 0.80 \quad \text{From the hydraulic elements chart, } \frac{d}{D_{\text{FULL}}} = 0.70$$

$$D = 6 \text{ feet (the rise of the box). } d = (0.70)(6.0) = 4.2 \text{ feet}$$

In order to determine if the conduit is on a mild or steep slope the critical depth, D_c , is calculated. If the normal depth, d , is greater than D_c , there is subcritical flow in the barrel and the conduit is on a mild slope. Conversely, if D_c is greater than d , there is supercritical flow in the barrel and the conduit is on a steep slope. The critical depth chart for rectangular sections (Chart 16 of Appendix B, **Chapter 8**) is used to find D_c , as follows:

$Q = 459$ cubic feet per second and $B = 7.0$ feet (the span of the box).

$$\frac{Q}{B} = \frac{459}{7} = 65.6 \text{ cubic feet per second per foot}$$

From the critical depth chart, $D_c = 5$ feet. Since the d less than or equal to D_c , there is supercritical flow in the conduit. Now the flow type at the outlet can be determined. Case 1 flow occurs because the conduit is partially full, it is on a steep slope, and the outlet is free flowing.

Step 2: In this step the brink depth and brink velocity are calculated. If the conduit were a shape other than rectangular, an equivalent brink depth would be calculated. The brink depth, y , is assumed to be the depth of normal flow, d , calculated in the previous step.

At the brink, $y = d = 4.2$ feet.

The brink velocity is calculated using the hydraulic elements chart as follows:

$A_{\text{FULL}} = 42$ square feet (Table 9-B-2). Using the continuity equation (**Chapter 8**):

$$V_{\text{FULL}} = \frac{Q_{\text{FULL}}}{A_{\text{FULL}}} = \frac{574}{42} = 13.7 \text{ feet per second}$$

$$\frac{q}{Q_{\text{FULL}}} = 0.80 \quad \text{From the hydraulic elements chart (Chart 11, Appendix B, **Chapter 8**),}$$

$$\frac{v}{V_{\text{FULL}}} = 1.11, \text{ and } v = (1.11)(13.7) = 15.07 \text{ feet per second.}$$

At the brink, $v = 15.07$ feet per second.

Step 3: The last step is to calculate the Froude number at the brink using Equation 11-3. The brink depth and velocity were calculated in the previous step, and the Froude number is:

$$Fr = \frac{15.07}{[(32.2)(4.2)]^{0.5}} = 1.3$$

At the brink: $Fr = 1.3$

11.7.2 Example: Hydraulic Characteristics of Flow at the Brink of an Arch

The equivalent brink depth, brink velocity, and equivalent Froude number are needed for an arch culvert that discharges directly into a pool. The 50-year design flow is 70 cubic feet per second, the structural plate arch has a 10 foot span and a 5.25 foot rise and an earth bottom, and the slope of the arch is 0.005 foot per foot. The tailwater depth is 1.6 feet during the design flood.

Step 1: Since there is tailwater and the outlet is partially submerged, Case 2, Case 5, or Case 6 flows can occur. In order to determine if the culvert is flowing full, Manning's Equation is used. The Manning's roughness coefficients, n , for arches with various bottoms and cross-sectional areas are listed on the outlet control nomographs (Appendix A, **Chapter 9**). The cross-sectional area of the conduit flowing full (Table 4, Appendix B, **Chapter 8**) is: $A = 41$ square feet. The rise/span ratio is: $5.25 / 10 = 0.52$. From Chart 23 for an arch culvert with an earth bottom, a rise/span ratio ≥ 0.5 , and a cross-sectional area from 4.1 to 150 square feet: $n = 0.029$. From Table 4: Appendix B, **Chapter 8** $(41)(1.60)^{2/3} = 56.1$. Using Manning's equation for discharge (Equation 8-15):

$$Q_{\text{FULL}} = \left(\frac{1.49}{0.029} \right) (56.1) (0.005^{0.5}) = 204 \text{ cubic feet a second}$$

$Q = 70$ cubic feet per second and Q is less than or equal to Q_{FULL} . The conduit is partially full and Case 5 flow cannot exist.

The hydraulics element chart for a semicircular arch (Chart 3 of Appendix B, **Chapter 8**) is used to get d , as follows:

$$\frac{Q}{Q_{\text{FULL}}} = \frac{q}{Q_{\text{FULL}}} = \frac{70}{204} = 0.34$$

$= 0.32$

From the hydraulic $\frac{d}{D_{\text{FULL}}}$ elements chart,

$$D = 5.25 \text{ feet (the rise of the arch). } d = (0.32) (5.25) = 1.68 \text{ ft}$$

D_c is calculated by the iterative method using Equation 8-19 in **Chapter 8**, and $D_c = 0.03$ feet. Appendix B, **Chapter 8**, the critical depth chart for corrugated metal arch culverts, can also be used to find D_c . Since the d is more than or equal to D_c , the barrel is on a mild slope. There is

Case 6 flow at the outlet because the barrel is partially full, it is on a mild slope, and the outlet is submerged or free-flowing.

Step 2: In this step the equivalent brink depth and brink velocity are calculated. The flow area at the brink is assumed to be the cross-sectional area of the arch below the tailwater elevation because the 1.6 foot tailwater depth is greater than the 0.03 foot critical depth. This flow area is calculated using the hydraulic elements chart (Chart 3, Appendix B, **Chapter 8**) as follows:

$$d = y = TW = 1.6 \text{ feet} \quad D = 5.25 \text{ feet} \quad A_{\text{FULL}} = 41 \text{ ft}^2 \quad \frac{d}{D} = \frac{1.6}{5.25} = 0.31$$

From the hydraulic elements $\frac{a}{A_{\text{FULL}}} = 0.42$ and $a = (0.42)(41) = 17.2$ square feet chart,

The equivalent brink depth is calculated using Equation 11-2 as follows:

$$y_e = \left(\frac{17.2}{2} \right)^{0.5} = 2.9 \text{ feet}$$

The brink velocity is calculated using the continuity equation (**Chapter 8**) and the flow area below the tailwater elevation, as follows:

$$v = \frac{Q}{a} = \frac{70}{17.2} = 4.1 \text{ feet per second}$$

At the brink: $y_e = 2.9$ feet and $v = 4.1$ feet per second

Step 3: The equivalent Froude number at the brink is calculated using Equation 11-4. The equivalent brink depth and brink velocity were calculated in the previous step, and the equivalent Froude number is:

$$Fr_e = \frac{4.1}{[(32.2)(2.9)]^{0.5}} = 0.42$$

At the brink: $Fr_e = 0.42$

11.8 Estimating the Size of an Unlined Scour Hole

In some cases it is necessary to estimate the size and shape of the scour hole that would occur if there were no scour or erosion protection beyond the conduit or channel outlet. Methods have been developed for predicting the size of scour holes and they are discussed in detail in the FHWA Hydraulic Engineering Circular No. 14 "Hydraulic Design of Energy Dissipators for Culverts and Channels" September 1983 with March 1996 Amendment (HEC-14). A computer program that can calculate scour hole dimensions is included in the HY-8 module of the FHWA's HYDRAIN software package.

A simplified version of the method of predicting scour holes in non-cohesive soils is presented in this manual. This method is suitable for the most applications because the majority of streambeds are composed of cohesionless materials. Based on the agency's experience this method tends to be fairly conservative. In other words, the scour holes that actually occur are rarely larger than the holes predicted by the equations.

A method of predicting scour hole dimensions in cohesive soils is not given in this manual, but it is included in HEC-14 and HY-8. It is applicable for sandy clays with a Plasticity Index based on Atterburg Limits (PI) not lower than 5 or higher than 16. Prior to using this design method, laboratory tests are needed on representative samples of the soils at the location of the scour hole, such as the PI (AASHTO T 90-96) and the unconfined compressive strength (AASHTO T 208-96). The Geo-Environmental Section can provide details about the needed tests.

Predicting the exact dimensions of a scour hole is difficult because of the many factors involved, some of which are: discharge, conduit shape, soil type, flow duration, conduit gradient, height of the drop from the conduit outlet to the channel bed, and the tailwater depth. In addition, the dimensions of the scour hole can change over time. If the stream transports sediment the hole may be quite large during the time of peak flow and it may fill with sediment during periods of lower flows. This occurs more often with culverts and very rarely with storm sewers. A typical scour hole is shown in Figure 11-10a, and in general, research has shown:

- the scour hole geometry varies with tailwater conditions,
- the largest scour holes occur when the tailwater depths are less than half of the conduit rise, and
- the maximum scour depth occurs at a distance of $(0.4)(L_s)$ downstream from the conduit outlet, where L_s is the length of the scour hole.

The following equations are used to calculate the dimensions of scour holes in non-cohesive soils such as silts, sands, or gravels. They are intended to be used along with the maintenance history and the results of a site reconnaissance to estimate the scour potential of the site. The maximum dimensions of the scour hole can be estimated as follows:

$$d_s = C_g C_h R \left(\frac{2.27}{\sigma^{0.33}} \right) \left(\frac{0.176 Q}{R^{2.5}} \right)^{0.39} \left(\frac{t}{316} \right)^{0.06} \quad (\text{Equation 11-5})$$

$$w_s = C_g C_h R \left(\frac{6.94}{\sigma^{0.33}} \right) \left(\frac{0.176 Q}{R^{2.5}} \right)^{0.53} \left(\frac{t}{316} \right)^{0.08} \quad (\text{Equation 11-6})$$

$$l_s = C_g C_h R \left(\frac{17.10}{\sigma^{0.33}} \right) \left(\frac{0.176 Q}{R^{2.5}} \right)^{0.47} \left(\frac{t}{316} \right)^{0.10} \quad (\text{Equation 11-7})$$

$$v_s = C_g C_h \left(R^3 \right) \left(\frac{127.08}{\sigma^{0.33}} \right) \left(\frac{0.176 Q}{R^{2.5}} \right)^{1.24} \left(\frac{t}{316} \right)^{0.18} \quad (\text{Equation 11-8})$$

Where:

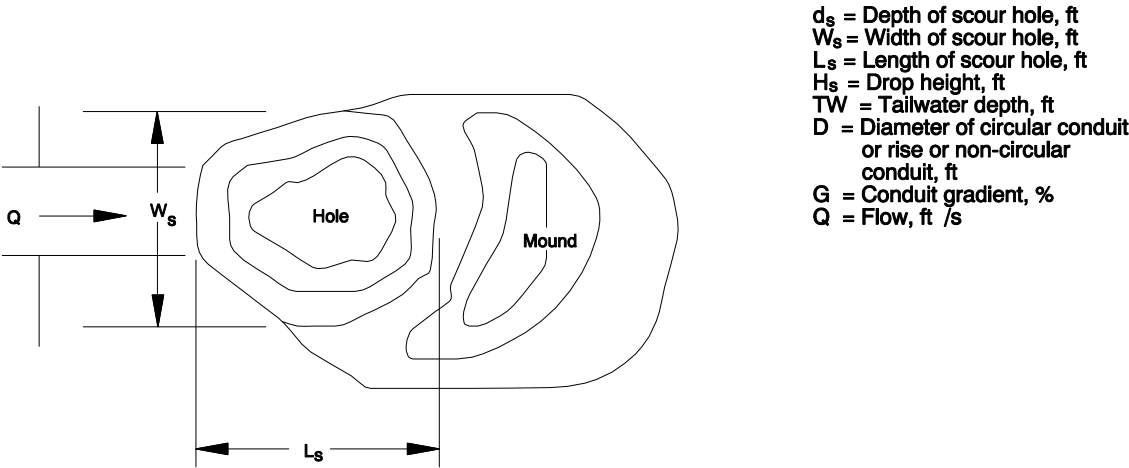
- d_s = Maximum depth of scour hole in cohesionless soil in feet
- w_s = Maximum width of scour hole in cohesionless soil in feet
- l_s = Maximum length of scour hole in cohesionless soil in feet
- v_s = Maximum volume of scour hole in cohesionless soil in cubic feet
- C_g = Dimensionless coefficient to account for the gradient of the conduit. See Table 11-1a and Figure 11-10b.
- C_h = Dimensionless coefficient to account for the ratio of the drop height to the conduit diameter, H_s / D . See Table 11-1b and Figure 11-10c.

Where:

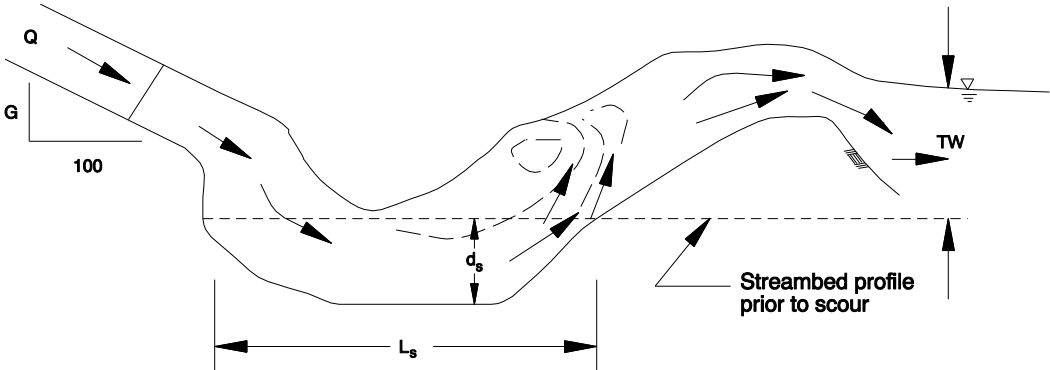
- H_s = Vertical distance between the conduit invert at the outlet and the streambed prior to scour, in feet.
- D = Diameter of conduit in feet.
- R = Hydraulic radius of the flow at the brink in feet. This is the flow area of the brink in square feet divided by the wetted perimeter of the brink in feet. See Appendix B of **Chapter 8** for tables listing hydraulic radii of common conduits flowing full. If the conduit is flowing partially full, the hydraulic radius can be determined using the hydraulic element charts in Appendix B of **Chapter 8**.
- σ = Standard deviation of the bed-material grain size distribution using the following formula:

$$\sigma = \left(\frac{d_{84}}{d_{16}} \right)^{0.5} \quad (\text{Equation 11-9})$$

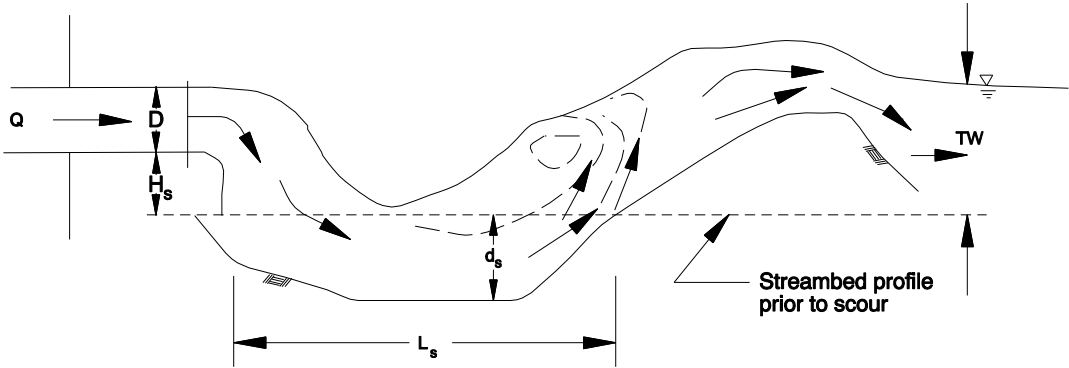
The values d_{84} and d_{16} are based on a distribution curve determined from a particle size analysis of the soil at the location of the scour hole (AASHTO T 88-97). A σ value



a) Typical Scour Hole



b) Scour Hole Profile Showing Conduit Gradient



b) Scour Hole Profile Showing Drop Height

Figure 11-10 Scour Hole Geometry

based on test results is preferred. If these results are not available, it can be assumed that σ is 2.10 for gravel and 1.87 for sand.

Q = Discharge in cubic feet per second

t = 30 minutes or the time of concentration, if longer. If t is not known a value of 30 minutes should be used.

11.8.1 Example: Estimating the Size of an Unlined Scour Hole

The Wirfsville storm sewer outfall is to be relocated. The 36-inch diameter corrugated metal pipe ($n = 0.021$) is at a 0.20 percent grade, the design discharge is 15 cubic feet per second, and the outfall will be onto fairly uniformly graded beach sand with a σ value of 1.2. There will be a 1.6 foot drop between the outlet invert and the channel bed. Peak flow is estimated to occur when the time of concentration is 120 minutes. The size and shape of the scour hole is needed.

The C_g coefficients for a 0.20 percent gradient were interpolated for depth, width, length, and volume using the values in Table 11-1a, as follows: C_g (Depth) = 1.00, C_g (Width) = 1.03, C_g (Length) = 1.02, and C_g (Volume) = 1.03

The drop height to conduit diameter ratio is: 1.6 feet drop / 3 feet diameter = 0.53. The C_h coefficients for a ratio of 0.53 were interpolated for depth, width, length, and volume using the values in Table 11-1b, as follows: C_h (Depth) = 1.12, C_h (Width) = 1.28, C_h (Length) = 0.85, and C_h (Volume) = 1.15

The hydraulic radius at the brink is needed. To calculate this radius, it is necessary to determine the characteristics of the flow at the outlet. This procedure is presented in detail in Section 7 and it is summarized in this example.

Full flow without surcharge using Manning's equation for discharge and an $AR^{2/3}$ value of 5.8:

Table 11-1 Coefficients for Unlined Scour Holes in Cohesionless Materials**a) Culvert Gradient Coefficient, C_g**

Conduit Gradient, G, in Percent	C_g for Depth Equation	<u>Coefficient C_g for Conduit Gradient</u>			C_g for Volume Equation
		C_g for Width Equation	C_g for Length Equation		
0	1.00	1.00	1.00		1.00
2	1.03	1.28	1.17		1.30
5	1.08	1.28	1.17		1.30
> 7	1.12	1.28	1.17		1.30

b) Drop Height to Diameter Ratio Coefficient C_h

Drop Height to Diameter Ratio, H_s / D	C_h for Depth Equation	<u>Coefficient C_h for Outlets Above the Streambed</u>			C_h for Volume Equation
		C_h for Width Equation	C_h for Length Equation		
0	1.00	1.00	1.00		1.00
1	1.22	1.51	0.73		1.28
2	1.26	1.54	0.73		1.47
4	1.34	1.66	0.73		1.55

$$Q_{\text{FULL}} = \left(\frac{1.49}{0.021} \right) (5.8) (0.002^{0.5}) = 18.4 \text{ cubic feet per second}$$

Depth of normal flow in the conduit using the hydraulic elements chart for circular pipe:

$$\frac{q}{Q_{\text{FULL}}} = \frac{15}{18.4} = 0.82 \quad \text{From Chart 1, Appendix B, Chapter 8,} \quad \frac{d}{D_{\text{FULL}}} = 0.69$$

$$d = (0.69) (3) = 2.07 \text{ ft}$$

$Q = 15$ cubic feet per second, depth of critical flow using the critical depth chart for circular pipe:

$$D_c = 1.2 \text{ feet}$$

Since q less than Q_{FULL} , d more than D_c , and TW less than D there is Case 6 flow at the outlet and the brink depth can be determined using Figure 11-9. Using the figure, $y / D = 0.32$ and $y = (0.32)(3) = .96$ feet.

The ratio of the brink depth to the diameter of the conduit is used with the hydraulic elements chart (Chart 1 of Appendix B of **Chapter 8**), and the hydraulic radius of the conduit at full flow (Table 1 of Appendix B of **Chapter 8**), to determine the hydraulic radius at the brink, as follows:

$$\frac{y}{D} = 0.32 \quad \text{From Chart 8: } \frac{r}{R_{FULL}} = 0.74 \quad \text{From Table 8-B-1: } R_{FULL} = 0.75 \text{ feet}$$

$$r = R = (0.74)(.75) = .56 \text{ ft}$$

$$\sigma = 1.2 \text{ (see given data)}$$

$$Q = 15 \text{ cubic feet per second (see given data)}$$

$t = 120$ minutes (see given data). If t was less than 30 minutes, or if t was not known, a value of 30 minutes would be used.

The maximum dimensions of the scour hole are estimated using Equations 11-5 through 11-8 to be:

$$\begin{aligned} d_s &= (1.00)(1.12)(.56) \left(\frac{2.27}{1.2^{0.33}} \right) \left[\frac{(0.176)(15)}{.56^{2.5}} \right]^{0.39} \left(\frac{120}{316} \right)^{0.06} = 3.25 \text{ feet} \\ w_s &= (1.03)(1.28)(.56) \left(\frac{6.94}{1.2^{0.33}} \right) \left[\frac{(0.176)(15)}{.56^{2.5}} \right]^{0.53} \left(\frac{120}{316} \right)^{0.08} = 16.1 \text{ feet} \\ l_s &= (1.02)(0.85)(.56) \left(\frac{17.10}{1.2^{0.33}} \right) \left[\frac{(0.176)(15)}{.56^{2.5}} \right]^{0.47} \left(\frac{120}{316} \right)^{0.10} = 22.1 \text{ feet} \\ v_s &= (1.03)(1.15)(.56^3) \left(\frac{127.08}{1.2^{0.33}} \right) \left[\frac{(0.176)(15)}{.56^{2.5}} \right]^{1.24} \left(\frac{120}{316} \right)^{0.18} = 421 \text{ cubic feet} \end{aligned}$$

The maximum depth is estimated to occur at a distance of 8.9 feet downstream from the end of the conduit, based on the following: $0.4 \times L_s = 0.4 \times 22.3 = 8.9$ feet

11.9 Design of Unlined Pools

The unlined pool is the simplest form of energy dissipator. The main design tasks are to assure that the highway embankment, culvert outlet, and other facilities will not be damaged by scour or erosion, and that the pool will be within ODOT right of way or drainage easements. Further discussion about this dissipator and guidance on its selection is in Section 11.5 and a pool is shown in Figure 11-1.

The outlet should have a cutoff wall to prevent scour damage to the conduit or the lining of the upstream channel. The cutoff wall should be designed to resist undermining and this is typically done using one of the following three methods.

1. The cutoff wall is keyed into non-erodible material as shown in Figure 11-11a. As additional and optional scour protection, the excavation that is made on the stream side of the wall, in order to place the wall, can be backfilled with loose riprap. ODOT Class 50 or Class 100 loose riprap is recommended.
2. The cutoff wall extends down into erodible material; and scour depth is not calculated, or the scour depth is calculated and the wall does not extend below the predicted scour elevation. In this case riprap should be placed around the stream side of the wall, as shown in Figure 11-11b. The riprap size should be determined by the methods used to size the rock in riprap pads, as discussed in Section 11.11.
3. The cutoff wall extends down into erodible material, the scour depth has been calculated, and the cutoff wall extends to an elevation below the predicted scour depth, as shown in Figure 11-11c. As additional and optional scour protection, the excavation that was made on the stream side of the wall, in order to place the wall, can be backfilled with loose riprap. ODOT Class 50 or Class 100 loose riprap is recommended.

The unlined pool can be excavated during construction or it can form due to scour from floods that occur subsequent to construction. Regardless of the method used, if the streambed or other material at the outlet is non-cohesive, the unlined pool may eventually resemble the scour hole shown in Figure 11-10 and its dimensions can be predicted by the equations in Section 11.8.

The right-of way or drainage easements should be of adequate size to contain the pool and the mound downstream from the pool. This mound is most likely to occur when scour from floods is used to create the pool, or when the pool is downstream from a conduit or channel that carries sediment or other bed load during peak flows. The mound is less likely to occur if the pool is excavated during construction, or if the flow carries little sediment, such as the discharge from a storm sewer. Critical structures and other features that may be undermined or damaged should be protected. Methods of bank protection are discussed in **Chapter 15** of this manual.

Vegetation can be planted in and around the embankment protection, as discussed in **Chapter 9**. Vegetation can also be planted in and around the outlet of the conduit or channel and along the edges of the pool. The vegetation should not obstruct the discharge from the outlet unless the effects of this obstruction are considered in the hydraulic design. Large vegetation should not be planted too close to any structure, such as a conduit, channel lining, or end treatment. The plants could grow large enough to damage the structure. As a general rule, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet from the structure, and large bushes such as sitka willows,

pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the structure. A structural designer should be consulted if more detailed guidance is needed.

11.10 Design of Internal Energy Dissipators

The internal dissipator is contained within the upstream conduit. Two of the more common types of internal dissipators are roughness rings and tees. Guidance on the selection of these dissipators is provided in Section 11.5 and they are shown in Figure 11-2. The design of roughness rings is not presented in this manual. It is included in the FHWA's Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels."

The design of tee dissipators is addressed in this section. The most common use of this dissipator is at the outlet of a culvert, storm sewer, inlet box, gutter, or other water collection facility where flow must be conveyed down a steep embankment slope. The dissipators perform the dual functions of conveying the flow down the slope without causing embankment erosion and they dissipate much of the flow energy at the outlet.

A standard design is presented in this section for tee dissipators that convey design flows up to and including 16 cubic feet per second. If a dissipator is needed for greater flows or for situations where the standard design cannot be used, the Geo-Environmental Unit should be contacted for assistance. The components of the dissipator are as follows.

Upstream Drainage System - The tee dissipator is located at the outfall of an upstream drainage system. This outfall could be a culvert, a storm sewer, an inlet box, or a flared or sloped inlet located behind a gap in a drainage curb. Examples of these outfalls are shown in Figure 11-12, and detailed guidance on the design of inlet boxes, gutters, and other types of roadway drainage is provided in **Chapter 13**.

The maximum allowable design discharges from the outfall are listed in the table in Figure 11-12. The discharge should not contain large debris or large amounts of sediment because these dissipators are susceptible to clogging. Trash racks or grates should be installed at all inlets where water entering the upstream drainage system carries debris. The openings in the rack or grate should be small enough to retain any debris that is large enough to clog the smallest pipe in the downstream system. A settling basin is recommended upstream from an inlet if the water entering the inlet carries a large amount of sediment.

Lateral Connection - The upstream drainage system attaches to the pipe going down the slope and to the stub to the cleanout gate using a lateral connection, as shown in Figure 11-12. The connection to the cleanout stub should have the same diameter as the outlet of the upstream drainage system. The connection to the pipe going down the slope should have the same diameter as the outlet of the upstream drainage system if debris can enter the system. If debris cannot get into the system, the pipe going down the slope can be reduced one size in comparison to the upstream pipes. The reduction in pipe size should be at the lateral connection.

Cleanout Gate - A stub with a cleanout hatch or gate is attached to the lateral connection to provide access for debris removal, as shown in Figures 11-12 and 11-13. This access should be provided whether or not the inlets to the system are covered by grates or trash racks. Typically the access hatch is latched or locked shut when not in use. This prevents water pressure from opening the hatch.

Slope Pipe - The pipe down the slope is usually buried within the embankment as shown in Figure 11-12, 11-13, and 11-14. The maximum difference in elevation between the inlet and outlet of the slope pipe depends on the slope pipe diameter. These maximum differences are listed in the table in Figure 11-13. Typically the slope pipe as well as the other components of

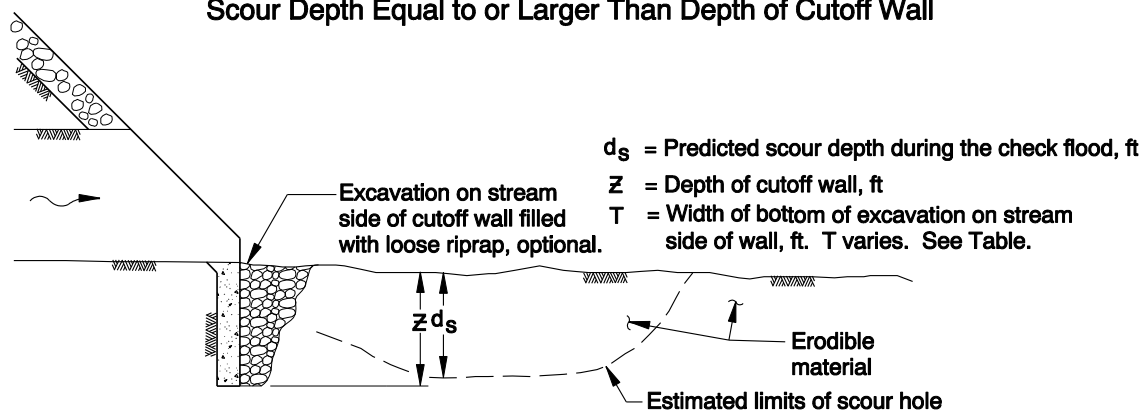
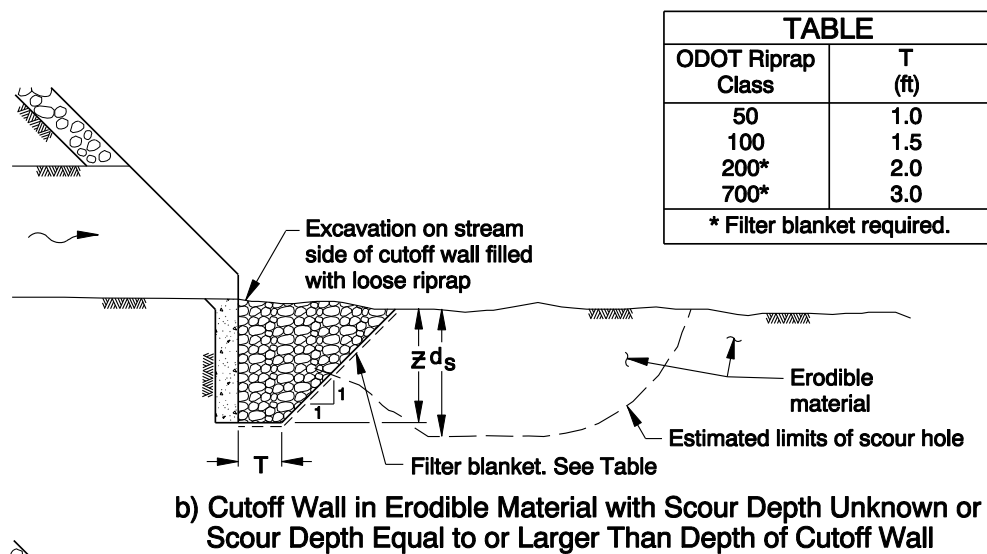
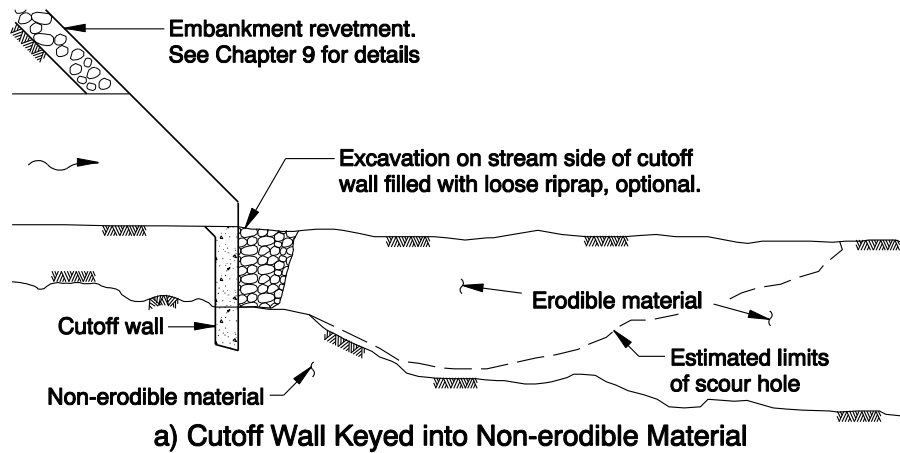


Figure 11-11 Outlet Into Unlined Pool

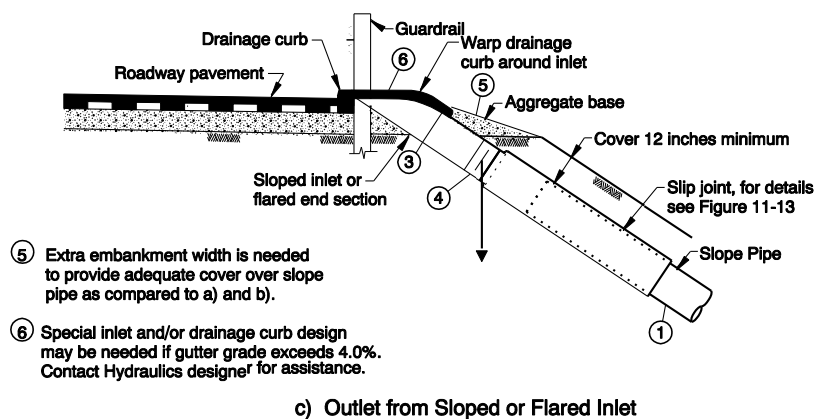
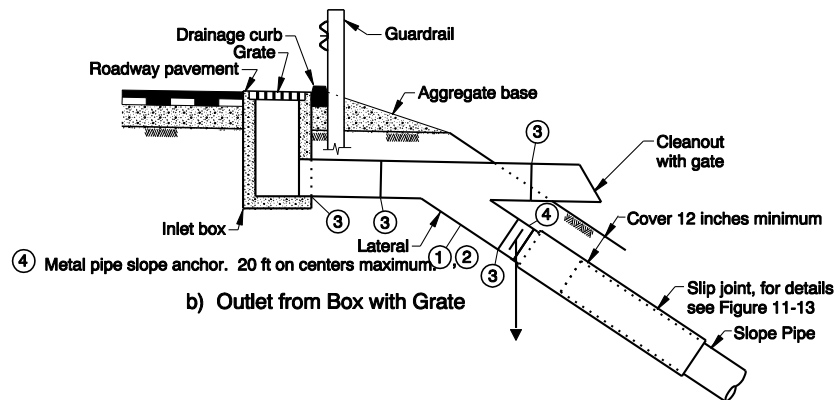
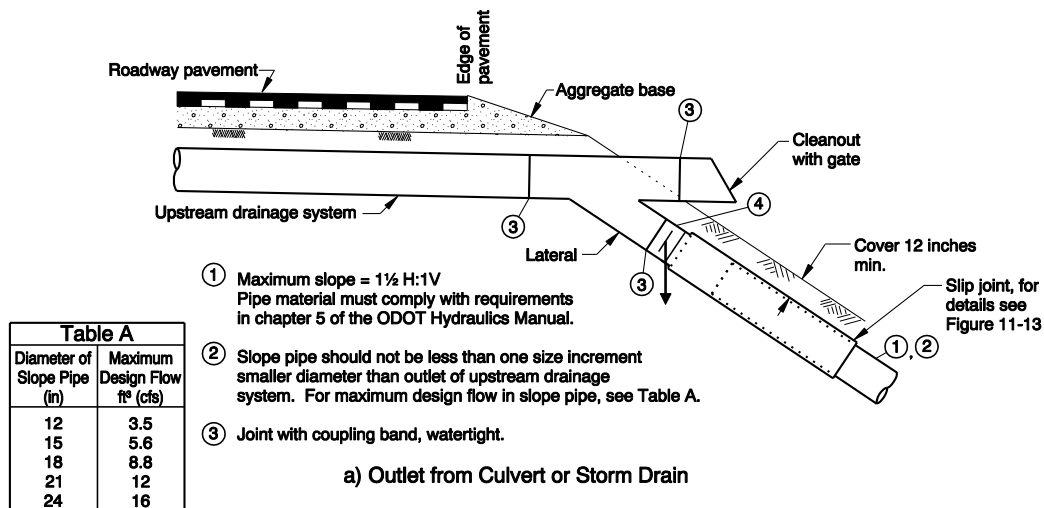


Figure 11-12 Upper Ends of Tee Dissipators

this system are made from corrugated metal. Segmented concrete pipe and other smooth walled conduits are not recommended. The slope pipe is one of the components of this system that is most susceptible to invert erosion. Care should be taken to select materials that will provide an adequate design life, and pipe material selection is discussed in **Chapter 5**. Pipe joints on the slope pipe, elbow, tee, and all other components of the dissipator should be watertight.

Slip Joint - A slip joint is provided in some installations to prevent damage to the slope pipe and the dissipator if the embankment fill settles or shifts, as shown in Figure 11-12 and 11-13. The Geo-Environmental Unit or Region Geology staff can be contacted to provide guidance on whether or not a slip joint is needed. The pipe size should not be changed at the slip joint. Any reduction in pipe size should be at the lateral connection.

Elbow and Tee - The elbow and the tee perform the functions of splitting the flow and dissipating much of the flow energy as turbulence. The elbow and the stem of the tee should be the same diameter as the slope pipe, and the diameter of the end of the tee should be 6-inches larger than the stem, as shown in Figure 11-14. The minimum length of the stem of the tee is 10 feet. If there is sufficient room beyond the toe of the fill, it is recommended that a longer stem is used, and the stem should project at least 10 feet beyond the toe.

Thrust Block - The thrust block supports the tee and its dimensions are shown on Figure 11-14. The block should be placed on soil or rock with an ultimate bearing capacity of at least 2089 pounds per square foot. If a larger or longer tee than the one shown in the drawings is used, the size of the thrust block should be increased so that the tee is fully supported.

Riprap - The thrust block is surrounded on three sides by riprap, as shown in Figure 11-15. The riprap provides additional energy dissipation and it protects the thrust block from undermining. ODOT Class 50 loose riprap is adequate for the design flows listed in Figure 11-12.

Vegetation can be planted on the highway embankment near the dissipator. The vegetation planted over and near the slope pipe should be sufficiently small so that the roots do not displace the buried conduit and the plants do not obstruct access to the cleanout gate. Generally small bushes and grasses are used and large bushes and trees are avoided. Vegetation planted on the embankment near the roadway should not grow up and obstruct the motorist's ability to see ahead. ODOT district maintenance personnel can provide information about these visibility requirements.

Vegetation can be planted around the tee dissipator. The plants should not obstruct the flow from the dissipator onto the riprap pad and they should not grow large enough to displace or damage the dissipator. In general, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet from the dissipator, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the dissipator. A structural designer should be consulted if more detailed guidance is needed.

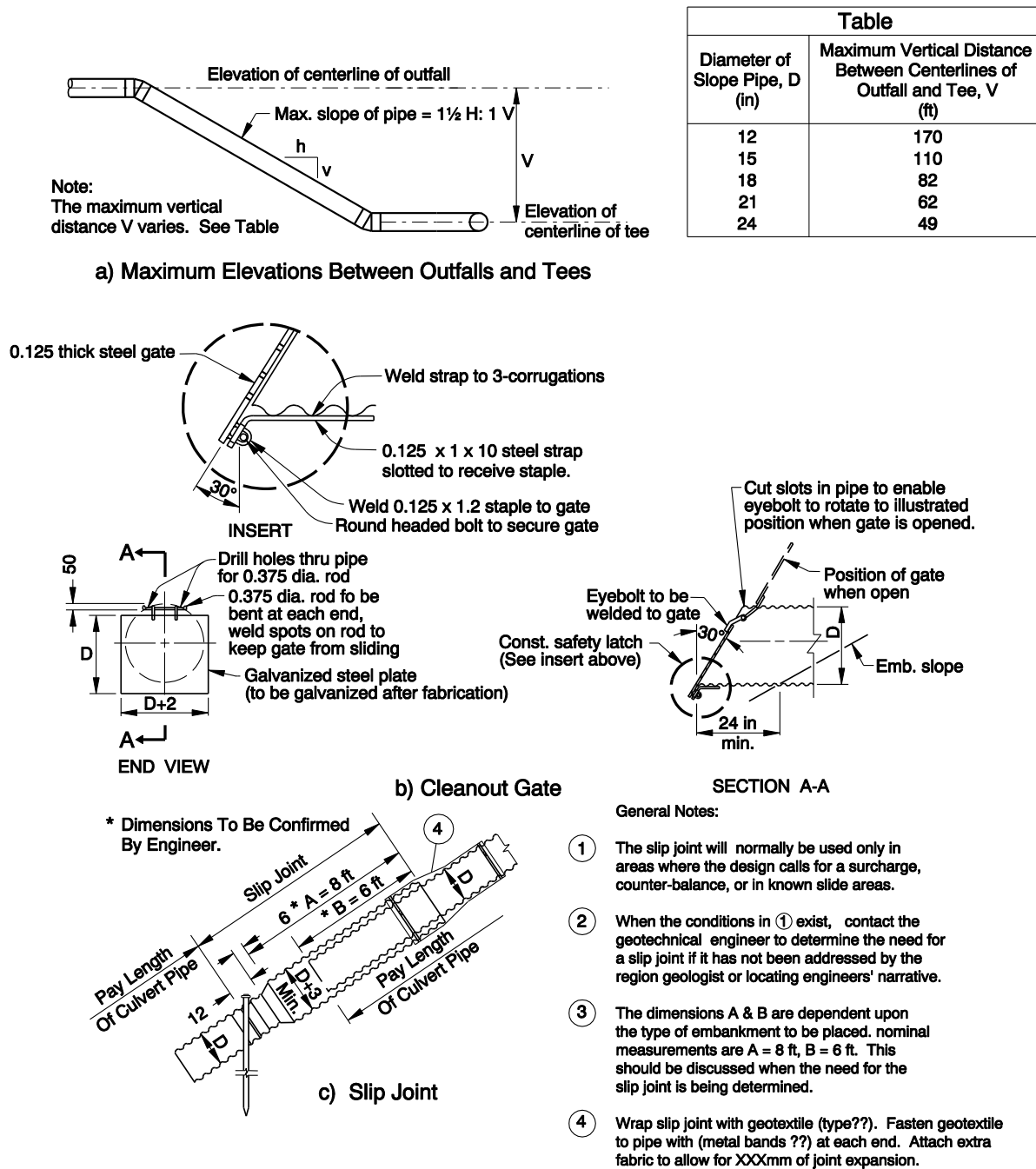
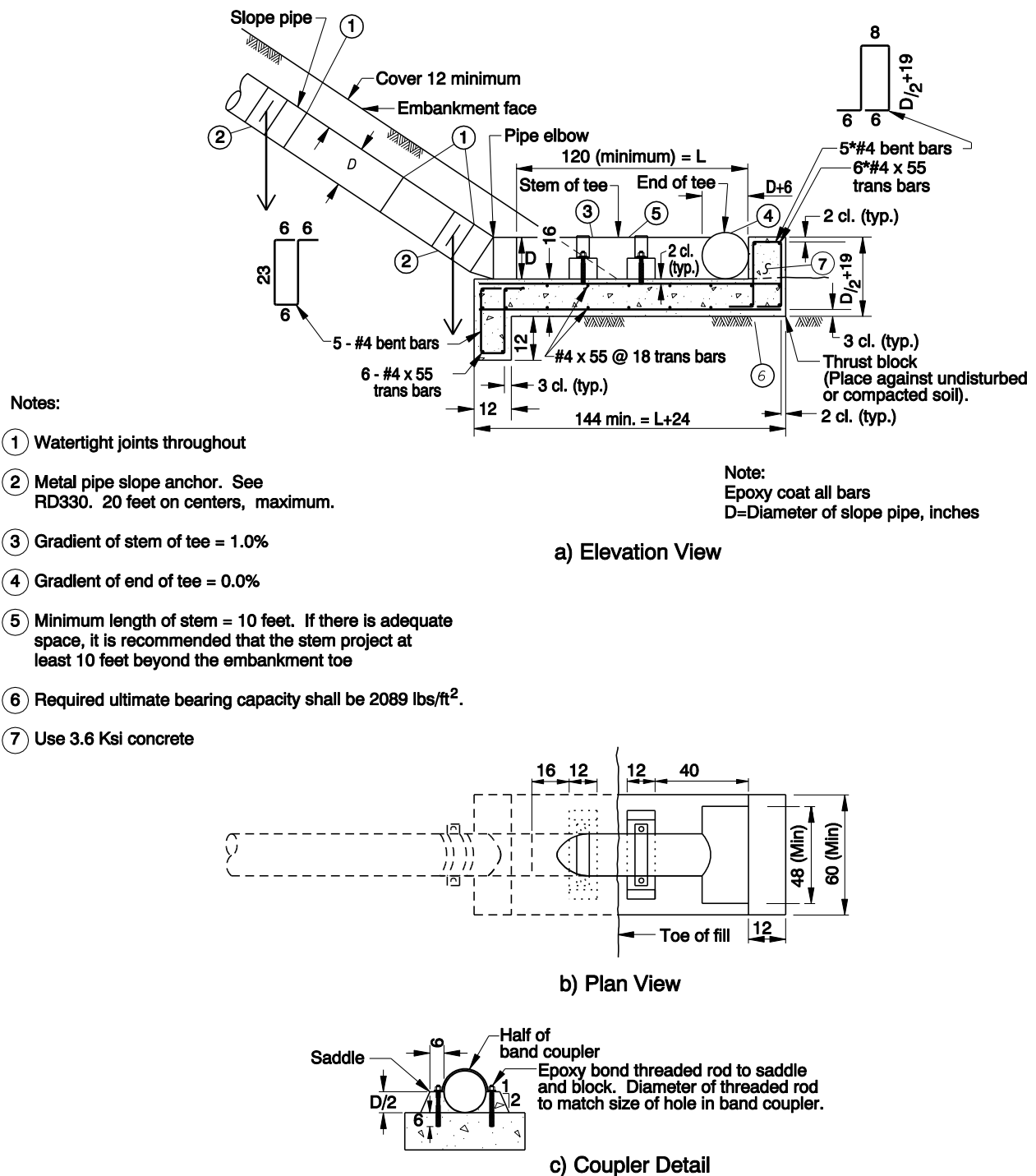


Figure 11-13 Slope Pipe, Cleanout Gate, and Slip Joint Details for Tee Dissipators



All dimensions in inches unless noted otherwise

Figure 11-14 Lower End of Tee Dissipator

Vegetation can be planted around the edges of the riprap. The vegetation should not obstruct the flow off of the riprap unless the effects of this obstruction are considered in the hydraulic design.

11.11 Design of Riprap Pads

The design of riprap pads is presented in this section. Guidance on the selection of these dissipators is provided in Section 11.5 and pads are shown in Figures 11-3 and 11-16. These dissipators protect the outlet of the conduit or channel from undermining and they also dissipate considerable flow energy. As a result, considerably more riprap is used in these pads than in the outlets into unlined pools that were discussed in the previous section.

The first step in designing a riprap pad is to determine the flow characteristics at the outlet of the upstream conduit or channel, such as the brink depth and the Froude number. The methods used to determine these characteristics are in Section 11.7. Actual brink depths and Froude numbers should be used for rectangular conduits and channels, and equivalent brink depths and equivalent Froude numbers should be used for non-rectangular conduits and channels.

The second step is to determine if a riprap pad, riprap lined basin, or a complex reinforced concrete dissipator is needed using the selection charts in Figures 11-17 through 11-20.

The third step is to determine the dimensions of the riprap pad using Figure 11-16.

Vegetation can be planted on the highway embankment as discussed in **Chapter 9**. Vegetation can also be planted around the outlet of the conduit or channel or along the edges of the pad. This vegetation should not obstruct the flow unless the effects of the obstruction are considered in the hydraulic study. Large vegetation should not be planted too close to any structure such as conduits, lined channels, or conduit and channel end treatments. The plants could grow large enough to damage the structure. As a general rule, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet of the structure, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the structure. A structural designer should be consulted if more detailed guidance is needed.

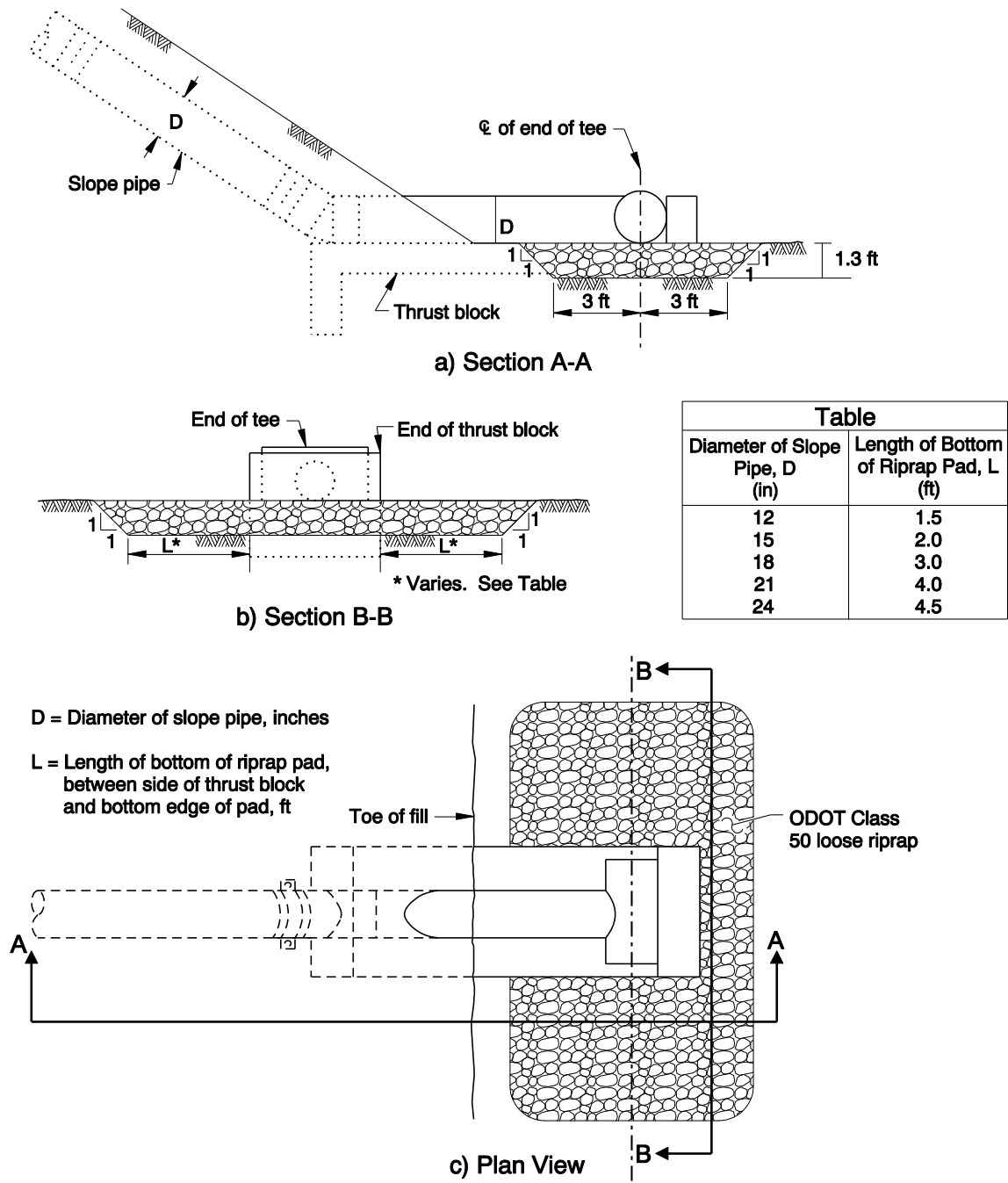


Figure 11-15 Riprap Around Outlet of Tee Dissipator

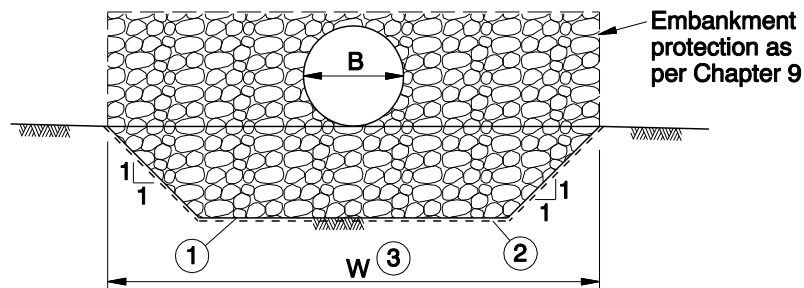
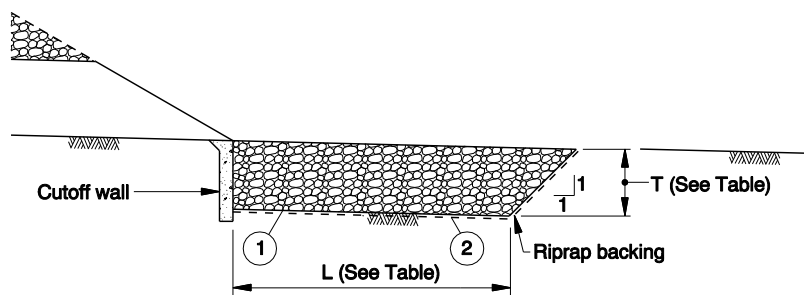
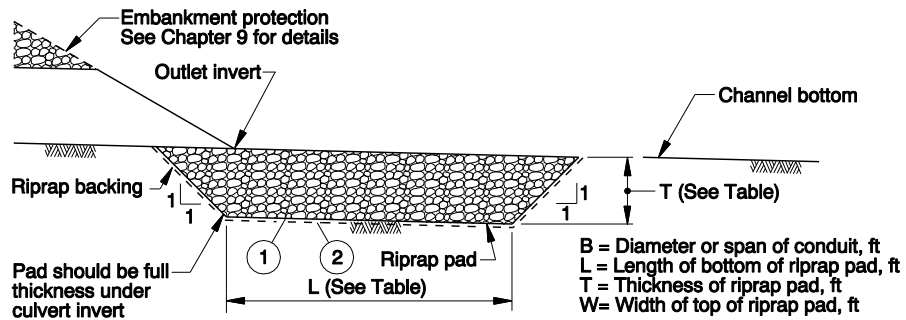


TABLE		
Riprap Class	L* (ft)	T (ft)
50	4B or 1.3	2.3
100	4B or 1.6	3.3
200	4B or 2.0	4.3
700	4B or 3.3	5.6

* L is the greater of 4B or the listed dimension.

Notes:

- ① Do not excavate non-erodible rock in order to place riprap.
- ② Use riprap backing under Class 200 and Class 700 loose riprap.
- ③ Top width of the riprap pad is the larger of 5B or the width of the embankment protection (see Chapter 9).

Figure 11-16 Dimensions of Riprap Pads

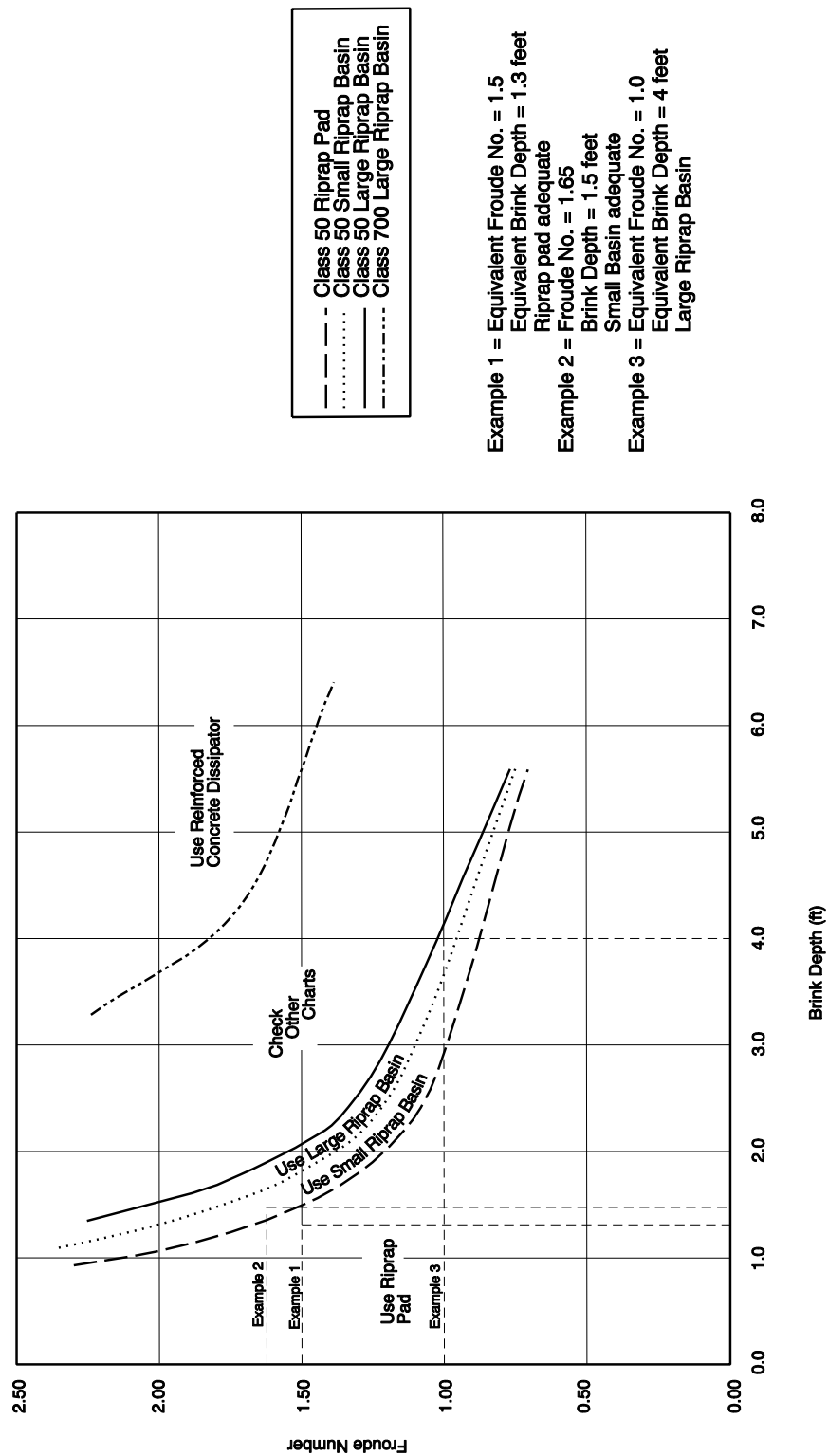


Figure 11-17 ODOT Class 50 Riprap Energy Dissipator Selection Chart

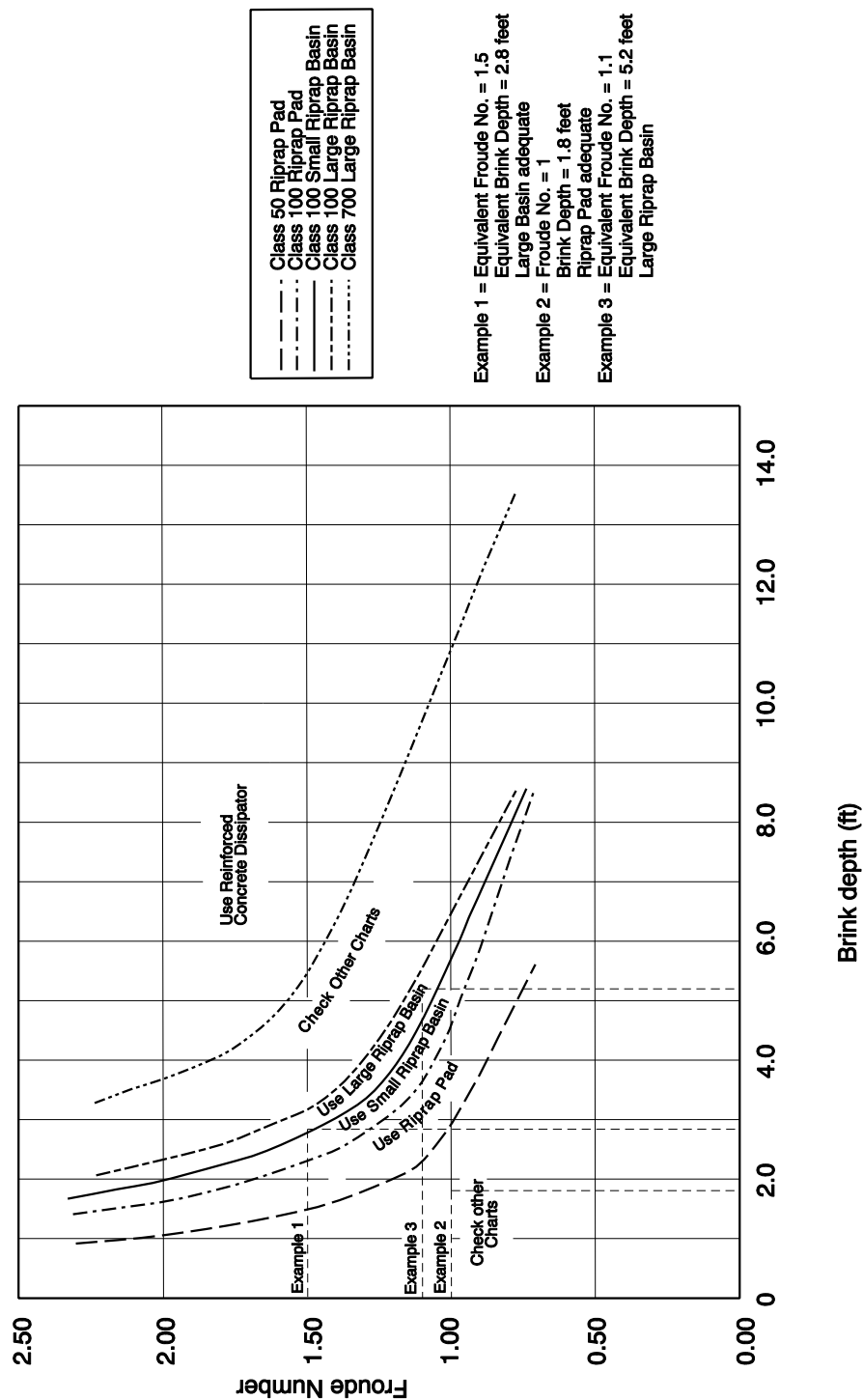


Figure 11-18 ODOT Class 100 Riprap Energy Dissipator Selection Chart

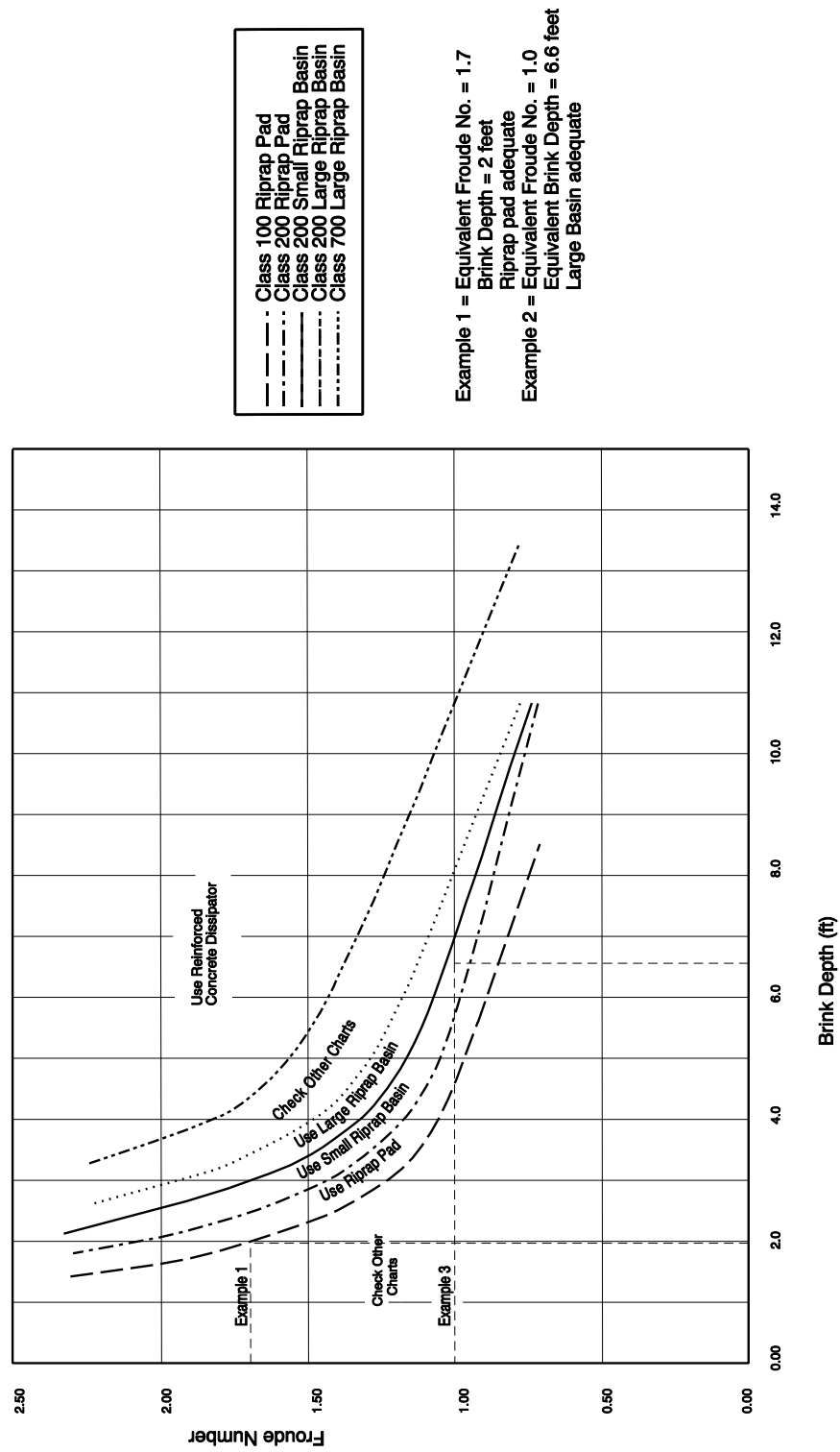


Figure 11-19 ODOT Class 200 Riprap Energy Dissipator Selection Chart

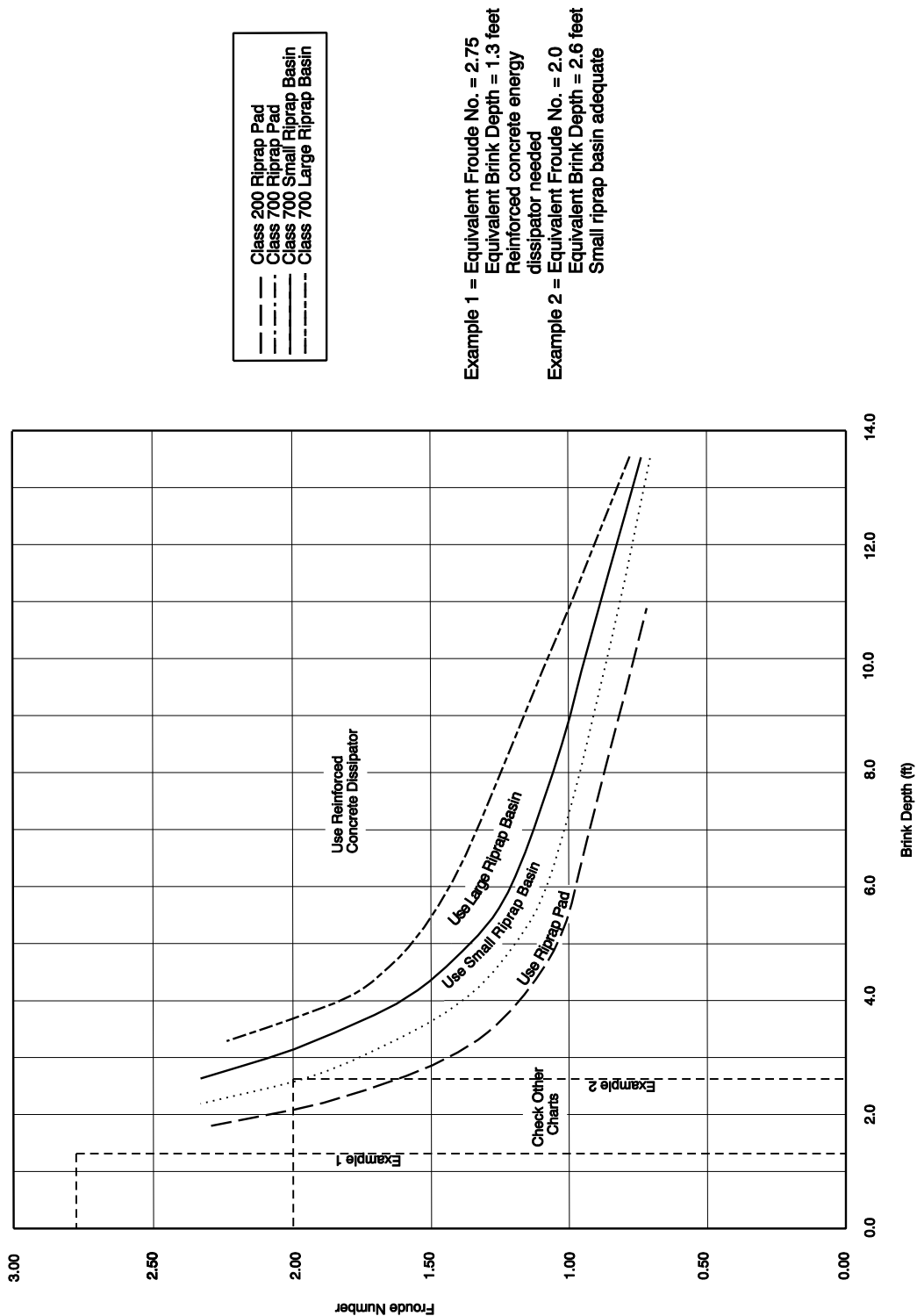


Figure 11-20 ODOT Class 700 Riprap Energy Dissipator Selection Chart

11.11.1 Example: Designing a Riprap Pad

The storm sewer outfall from the Bat Hills Subdivision has a design flow of 39 cubic feet per second out of a 36-inch diameter circular corrugated metal pipe on a 0.05 foot per foot slope. It is assumed the tailwater depth is negligible. A riprap pad needs to be designed to dissipate the flow energy and prevent a scour hole from developing.

Note: The procedure in this example does not address erosion that could occur downstream from the pad. This topic is discussed in Subsection 11.12.1.

Step 1: The equivalent brink depth and equivalent Froude number are needed. The calculation procedure is presented in detail in Section 11.7 and it is summarized for this example as follows:

Full flow without surcharge using Manning's equation for discharge, a roughness coefficient of 0.021, and an $AR^{2/3}$ value of 5.8:

$$Q_{\text{FULL}} = \left(\frac{1.49}{0.021} \right) (5.8) (0.05^{0.5}) = 92 \text{ cubic feet per second}$$

Depth of normal flow in the conduit using the hydraulic elements chart for circular pipe:

$$\frac{q}{Q_{\text{FULL}}} = \frac{39}{92} = 0.42 \quad \text{Using the chart, } \frac{d}{D_{\text{FULL}}} = 0.46 \quad d = (0.46)(3) = 1.38 \text{ feet}$$

Depth of critical flow using the critical depth chart for circular pipe:

$$D_c = 2 \text{ feet}$$

Since d is less than D_c and there is negligible tailwater, there is Case 1 flow at the brink and the flow area is assumed to be the area of normal flow. Using the hydraulic elements chart for circular pipe:

$$A_{\text{FULL}} = 7.07 \text{ square feet} \quad \frac{q}{Q_{\text{FULL}}} = \frac{39}{92} = 0.42 \quad \text{Using the chart, } \frac{a}{A_{\text{FULL}}} = 0.45$$

$$a = (0.45)(7.07) = 3.2 \text{ square feet}$$

$$\text{The equivalent brink depth : } y_e = \left(\frac{3.2}{2} \right)^{0.5} = 1.3 \text{ feet}$$

$$v = \frac{Q}{a} = \frac{39}{3.2} = 12.2 \text{ feet per second}$$

$$\text{The equivalent Froude number : } Fr_e = \frac{12.2}{[(32.2)(1.3)]^{0.5}} = 2$$

Step 2: In this step the equivalent brink depth and equivalent Froude number are used with the charts in Figures 11-17 through 11-20 to determine if a riprap pad can be used and the class of the riprap in the pad. Based on the charts, a riprap basin will be needed if Class 50 riprap is used and riprap pads are adequate if Class 100, 200, and 700 riprap are used. A riprap pad using ODOT Class 100 loose riprap is selected for the design.

Step 3: The riprap pad dimensions are determined in this step using Figure 11-16. The length of the bottom of the pad, L, will be four times the diameter of the culvert:

$$L = 4B = (4)(3) = 12 \text{ feet}$$

The thickness, T, of the pad is 3.3 feet (See table in figure 11-16).

The width of the embankment revetment is five times the diameter of the culvert, as shown in Figure 9-21 of **Chapter 9**. The total width of the riprap pad, W, will also be this width.

$$W = 5B = (5)(3) = 15 \text{ feet}$$

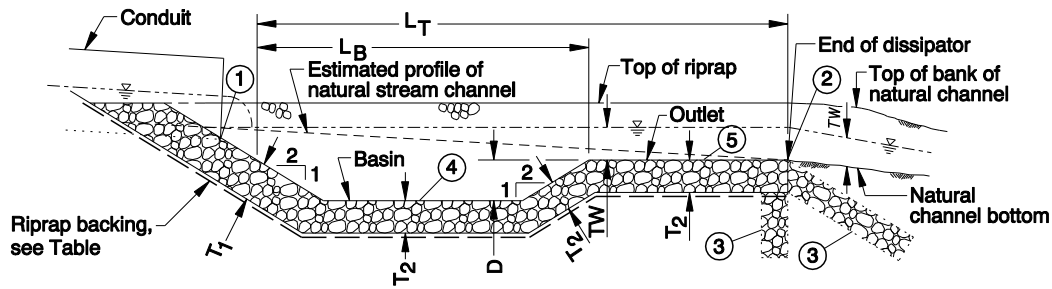
11.12 Design of Riprap Lined Basins

The design of a riprap lined basin is presented in this section. The selection of this type of dissipator is discussed in Section 11.5 and basins are shown in Figure 11-4 and Figures 11-21 through 24. These dissipators protect the outlet of the conduit or channel from undermining and they dissipate considerable flow energy.

The first step in riprap lined basin design is to determine the flow characteristics at the outlet of the upstream conduit or channel, such as the brink depth and the Froude number. The methods used to determine these flow characteristics are in Section 11.7. Actual brink depths and Froude numbers should be used for rectangular conduits and channels, and equivalent brink depths and equivalent Froude numbers should be used for non-rectangular conduits and channels.

The second step is to determine if a riprap pad, riprap lined basin, or a complex reinforced concrete dissipator is needed using the selection charts in Figures 11-17 through 11-20.

The third step is to determine the dimensions of the riprap lined basin using Figures 11-21 through 11-24. The dissipator outlet should be wide enough to match the existing channel. If



For dimensions, see Table

B = Diameter or span of conduit outlet, ft See Figure 11-23

D = Depth of basin below elevation of end of dissipator, ft

L_T = Total length of dissipator, ft

L_B = Length of basin, at elevation of end of dissipator, ft

T_1 = Thickness of revetment blanket on inlet side of basin, ft

T_2 = Thickness of blanket on remainder of basin, ft

TW = Tailwater depth during design flow, ft

Notes:

- ① Outlet invert is at elevation of natural stream bottom
- ② End of dissipator is at same elevation as bottom of natural channel
- ③ Thickened or sloping toe optional - construct if degradation is expected in downstream channel. Extend toe downward to an elevation lower than expected degradation.
- ④ Basin bottom is flat and at a depth "D" below the elevation of the end of the dissipator
- ⑤ Outlet bottom is flat and at same elevation as the end of the dissipator

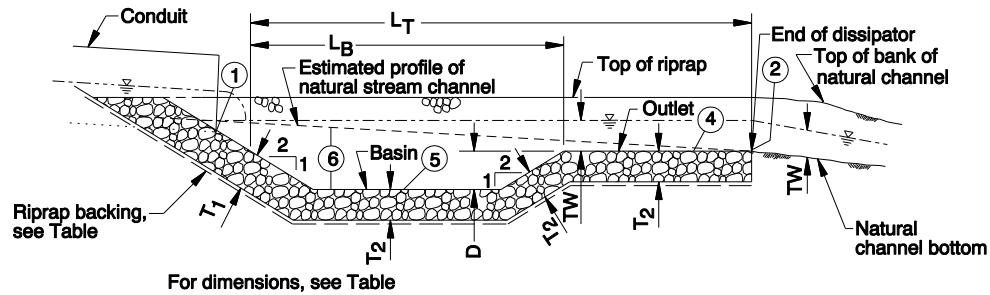
TABLE								
ODOT Riprap Class	D in feet		L_T ** in feet		L_B * in feet		T_1 in feet	T_2 in feet
	Small Basin	Large Basin	Small Basin	Large Basin	Small Basin	Large Basin	Large and Small Basin	Large and Small Basin
50	1.6	2.3	26 or 4B	33 or 4B	16 or 3B	23 or 3B	1.3	1.6
100	2.6	3.3	39 or 4B	49 or 4B	26 or 3B	33 or 3B	1.6	2.6
200 ***	3.3	4.3	49 or 4B	62 or 4B	62 or 3B	43 or 3B	2.3	3.3
700 ***	3.9	5.2	59 or 4B	79 or 4B	39 or 3B	52 or 3B	2.6	3.9

* L_T is the greater of:
4B or the listed dimension

** L_B is the greater of:
3B or the listed dimension

*** Riprap backing required

Figure 11-21 Elevation View of Riprap Lined Basins -
Fish Passage Not Required



B = Diameter or span of conduit outlet, ft See Figure 11-23
 D = Depth of basin below elevation of end of dissipator, ft
 L_T = Total length of dissipator, ft
 L_B = Length of basin, at elevation of end of dissipator, ft
 T_1 = Thickness of revetment blanket on inlet side of basin, ft
 T_2 = Thickness of blanket on remainder of basin, ft
 TW = Tailwater depth during design flow, ft

Notes:

- 1 Outlet invert is set at elevation that produces jump pool with desired depth. See Appendix D, Chapter 9.
- 2 End of dissipator is at same elevation as bottom of natural stream channel if no degradation is expected in downstream channel. End of dissipator is at elevation of degraded channel if degradation is expected.
- 3 L_B must not be less than the jump pool length needed for fish passage. See Appendix D, Chapter 9.
- 4 Outlet bottom is flat and at same elevation as end of dissipator.
- 5 Basin bottom is flat and at a depth "D" below the lower of: the outlet invert of the conduit, or the end of the dissipator.
- 6 The total jump pool depth should be checked at the fish passage design flow. The total jump pool depth must not be less than the minimum required for fish passage. See Appendix D of Chapter 9.

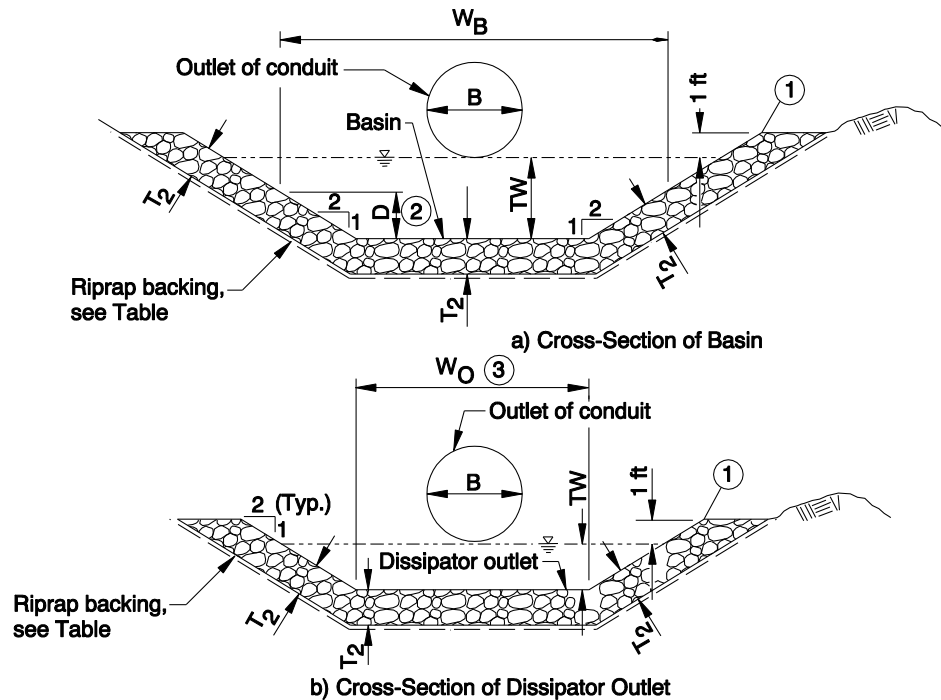
TABLE								
ODOT Riprap Class	D in feet		L_T * in feet		L_B ** in feet		T_1 in feet	T_2 in feet
	Small Basin	Large Basin	Small Basin	Large Basin	Small Basin	Large Basin	Large and Small Basin	Large and Small Basin
50	1.6	2.3	26 or 4B	33 or 4B	16 or 3B	23 or 3B	1.3	1.6
100	2.6	3.3	39 or 4B	49 or 4B	26 or 3B	33 or 3B	1.6	2.6
200***	3.3	4.3	49 or 4B	62 or 4B	33 or 3B	43 or 3B	2.3	3.3
700***	3.9	5.2	59 or 4B	79 or 4B	39 or 3B	52 or 3B	2.6	3.9

* L_T is the greater of:
4B or the listed dimension

** L_B is the greater of:
3B or the listed dimension

*** Riprap backing required

Figure 11-22 Elevation View of Riprap Lined Basin - Fish Passage Required



- B = Diameter or span of conduit outlet, ft
 D = Depth of basin, ft See Figure 11-21 if fish passage not required. See Figure 11-22 if fish passage required.
 T_2 = Thickness of revetment blanket, ft See Figure 11-21 or 11-22
 TW = Tailwater depth during design flow, ft
 W_B = Basin width, at elevation of dissipator outlet, ft See Table
 W_O = Outlet width, at bottom of outlet, ft See Note ③

Notes:

- 1 Extend riprap up side of basin to an elevation 1 foot higher than elevation of design tailwater
- 2 See Table, Figures 11-21 or 11-22
- 3 The width of the dissipator outlet:
 - is the width of the downstream channel bottom if a channel exists, or
 - is wide enough to create outlet flow velocities that do not erode the materials downstream from the outlet, or
 - if fish passage is needed, the outlet is wide enough to create the desired jump pool depth

TABLE		
ODOT Riprap Class	W_B^* in feet	
	Small Basin	Large Basin
50	17 or 3B	23 or 3B
100	26 or 3B	33 or 3B
200**	33 or 3B	43 or 3B
700**	40 or 3B	53 or 3B

*W_B is the greater of:
3B or the listed dimension

** Riprap backing required

Figure 11-23 Cross-Sections of Riprap Lined Basin

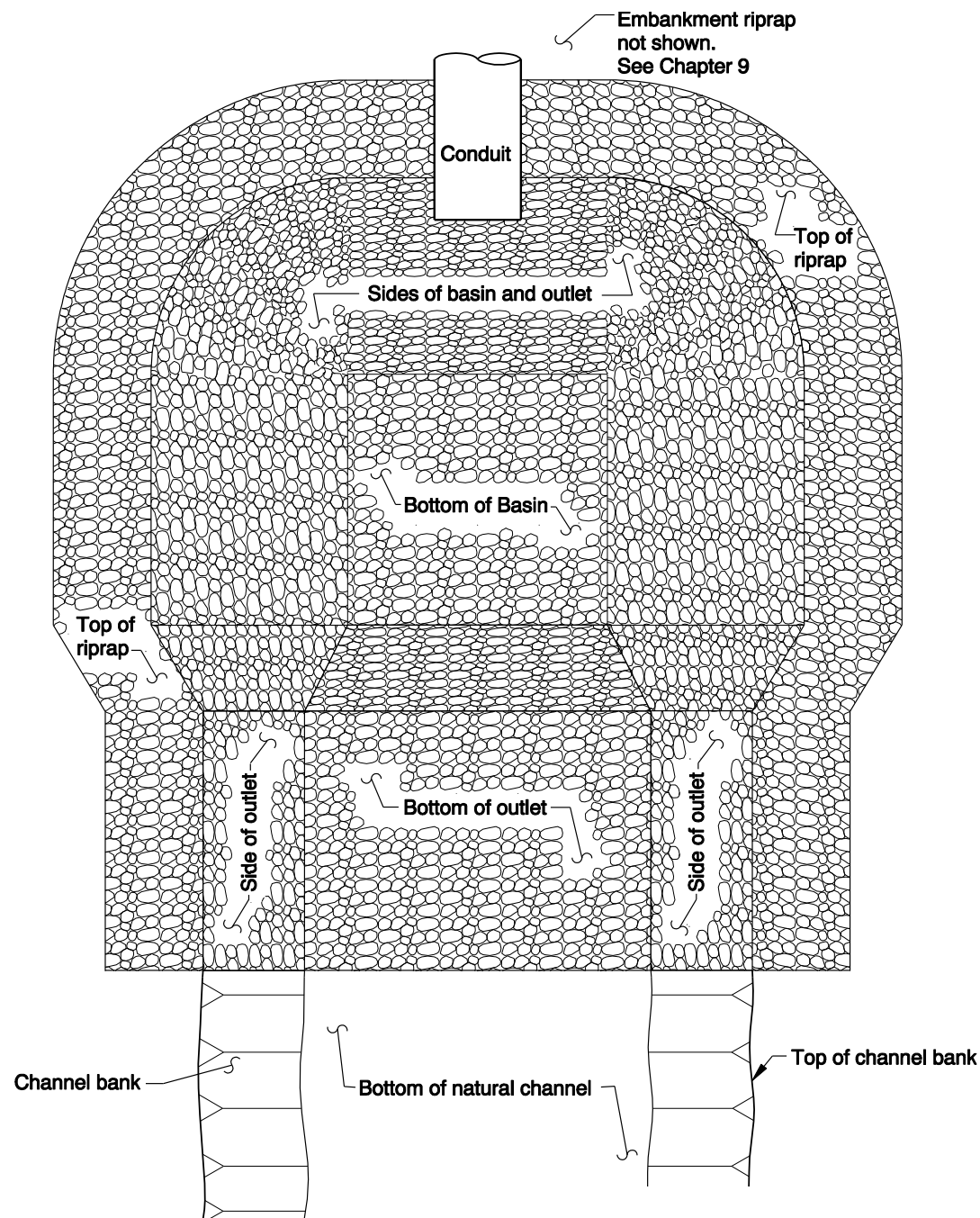


Figure 11-24 Plan View of Riprap Lined Basin

there is no existing channel, the need for a new outlet channel should be evaluated. The outlet from the dissipator should be sufficiently wide so there are non-erosive velocities at the upstream end of the channel, or the surface the water flows over if there is no channel. A method of determining this width is described in the following subsection.

The fourth step is to determine the depth of the water in the basin, and this depth typically depends on these factors: the discharge, the width of the outlet, and the depth of the tailwater above the bottom of the basin outlet. Three conditions usually occur:

1. The outlet is unsubmerged. This is the most common flow condition. It occurs when the dissipator outlet is less than 76 percent submerged by the tailwater. During this condition it is assumed the basin outlet is an unsubmerged broad-crested trapezoidal weir.
2. The outlet is partially submerged. This condition occurs when the dissipator outlet is not over 100 percent submerged or less than 76 percent submerged. During this condition it is assumed the outlet is a partially submerged broad-crested trapezoidal weir.
3. The outlet is fully submerged. This condition occurs when the outlet is over 100 percent submerged. During this condition it is assumed the basin outlet does not function as a weir. It is assumed the elevation of water in the dissipator basin is the same as the elevation of the tailwater.

The percent submergence of the dissipator outlet is calculated by the following equation:

$$\text{Percent Submergence} = 100 \left(\frac{\text{TW}}{\text{H}} \right) \quad (\text{Equation 11-11}) \quad \text{Where:}$$

TW = Depth of the tailwater above the bottom of the dissipator outlet, in feet

H = Difference in elevation between the surface of the water in the dissipator and the bottom of the outlet, in feet.

Unsubmerged or Partially Submerged Outlet - If it is assumed the dissipator outlet is unsubmerged or partially submerged, the elevation of the water surface in the dissipator is:

$$\text{EL}_{\text{Water Surface}} = \text{EL}_{\text{Dissipator Outlet}} + \text{H} \quad (\text{Equation 11-12})$$

Where:

EL_{Water Surface} = The elevation of the water surface in the dissipator in feet

EL_{Dissipator Outlet} = The elevation of the bottom of the dissipator outlet in feet

H = Difference in elevation between the water surface in the basin and the bottom of the dissipator outlet in feet. See Equation 11-13.

The distance H is calculated as follows:

$$H = \left[\frac{Q}{(C_s C_f W_o)} \right]^{2/3} \quad (\text{Equation 11-13})$$

Where:

H = Difference in elevation between water surface in the basin and the bottom of the dissipator outlet in feet

Q = The dissipator design discharge, in cubic feet per second

C_f = Weir coefficient. See Figure 11-25a.

C_s = Submergence factor.

If percent submergence is less than 76, C_s = 1.0

If 76 is less than or equal to percent submergence is less than or equal to 100, C_s varies.

See Figure 11-25b.

If percent submergence is more than 100, assume the outlet is fully submerged.

W_o = Width of the bottom of the dissipator outlet, in feet

The preceding equations are used in an iterative manner to determine the elevation of the water in the basin, as follows: C_f and C_s are assumed, H is calculated, and C_f and C_s are checked to see if the initial assumptions are correct. If the initial assumptions are not correct, C_f and C_s are reestimated and H is recalculated. The process is repeated until H, C_f, and C_s are in agreement.

Submerged Outlet - If the dissipator outlet is submerged, the water surface elevation in the dissipator is:

$$EL_{\text{Water Surface}} = \text{Tailwater Elevation}$$

Where:

EL_{Water Surface} = The elevation of the water surface in the dissipator in feet

Once the water surface elevation in the dissipator has been established, the minimum elevation of the top of the revetment on the sides of the dissipator can be determined. The revetment should extend upward along the sides of the dissipator to an elevation at least 1 foot higher than the water surface elevation, as shown in Figure 11-23.

These basins can also be used to provide a stable jump pool for fish passage at the downstream end of a culvert.

Vegetation can be planted on the highway embankment as discussed in **Chapter 9**. Vegetation can also be planted at the edges of the basin and around the outlet of the conduit or channel. The vegetation around the outlet should not obstruct the flow unless these effects have been

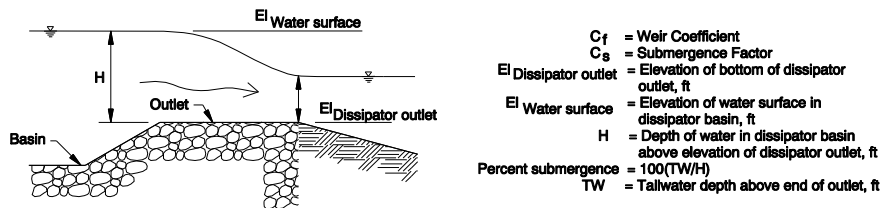
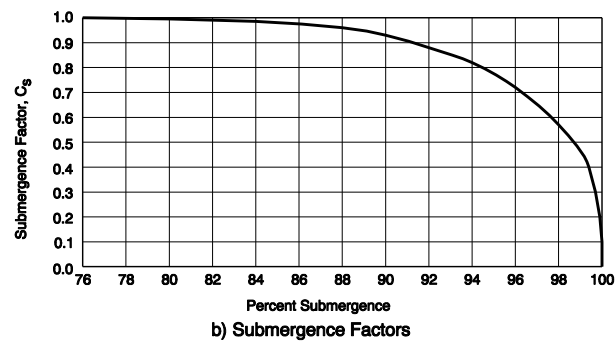
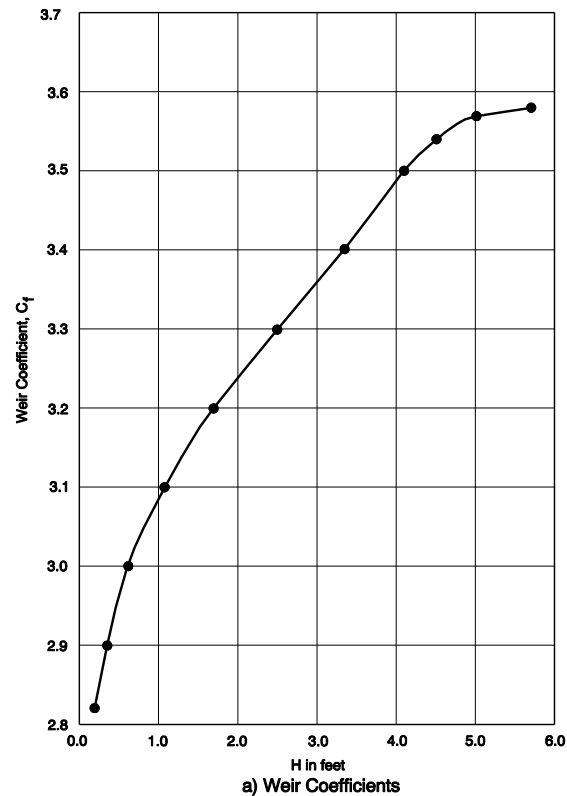


Figure 11-25 Weir Coefficients and Submergence Factors for Unsubmerged and Partially Submerged Riprap Lined Basin Outlets

considered in the hydraulic study. Large vegetation should not be planted too close to structures such as conduits, lined channels, or conduit and channel end treatments. The plants could grow large enough to damage the structure. As a general rule, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet of the structure, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the structure. A structural designer should be consulted if more detailed guidance is needed.

11.12.1 Scour Protection Downstream From Basin Outlet

In most cases the dissipator discharges into a channel and the width of the bottom of the dissipator outlet is the same as the width of the bottom of the channel. In some instances, however, the dissipator discharges into an open area where there is no channel and the flow is not constrained laterally. This open area could be a field, an embankment, a grassy swale, etc. If this occurs, the dissipator outlet should be sufficiently wide so it does not concentrate the flow and cause erosion of the downstream open area.

The basic concept of preventing scour downstream from the dissipator outlet is to assure that the shear stress exerted by the water on the underlying material is less than the shear stress the material can withstand. Although this concept appears to be simple, a precise mathematical analysis is quite complex. This manual presents a simplified analysis method that is sufficiently accurate for most applications.

The designer should be aware that if preventing erosion downstream from the dissipator is especially critical, a more detailed analysis may be warranted. This analysis may require a more precise determination of the erosion or scour at the end of the dissipator as well as an analysis of the scour or erosion caused by the flow through the entire area of concern.

The simplified method is as follows:

Step 1: Determine the maximum permissible shear stress that the material downstream from the dissipator outlet can withstand, τ_p , in pounds per square foot. The figures and tables in **Chapter 8** can be used for most materials. Shear stresses for temporary erosion control mats, grasses, and riprap are listed in **Chapter 8**. Shear stresses for non-cohesive and cohesive soils are shown in **Chapter 8**. These charts are applicable for flows up to 50 cubic feet per second. Allowable shear stresses for materials subject to higher flows are listed in most texts about river mechanics and sediment transport.

Step 2: Determine the maximum depth of flow that the underlying material downstream from the outlet can tolerate, as follows:

$$y = \frac{\tau_p}{\gamma(S)} = \frac{\tau_p}{(62.4)(S)}$$

(See Chapter 8) Where:

- y = Maximum depth of flow in feet
 τ_p = Maximum permissible shear stress on underlying material in pounds per square foot
 S = Slope of energy grade line in foot per foot. In most cases this slope can be assumed

Step 3: Determine the bottom width of a dissipator outlet that provides a flow depth equal to or less than the maximum depth determined in the previous step. It can be assumed the outlet has a trapezoidal shape and the flow is at normal depth. The depth can be determined by a trial-and-error solution using Manning's equation as presented in **Chapter 8**, or computer analysis. Regardless of the method used, the input variables are defined as follows:

- y = Maximum depth of flow in feet
 B = W_o = Outlet bottom width at end of the dissipator, in feet
 Q = Design discharge in cubic feet per second
 N = Manning's roughness coefficient for the surface downstream from the dissipator outlet that the water flows over
 S = Slope of energy grade line in foot per foot. This slope can often be assumed to be the slope of the surface downstream from the dissipator outlet that the water flows over.

The dissipator outlet can be quite wide if there is substantial discharge from the dissipator and the downstream surface is highly erodible. If this wide outlet is impractical, a more erosion resistant surface or channel may be needed downstream from the dissipator. This situation is discussed in the following example.

11.12.2 Example: Designing a Riprap Lined Basin

A riprap lined basin will be designed for the storm sewer outfall from the Bat Hills Subdivision. ODOT Class 50 riprap will be used. The design discharge is 39 cubic feet per second out of a 36-inch diameter circular corrugated metal pipe on a 0.05 foot per foot slope. The tailwater depth is assumed to be negligible. The discharge will leave the dissipator outlet and flow down the side of a grass lined swale. The swale is covered with a fair stand of mowed grass and it has a slope of 6 foot horizontal distance to 1 foot vertical distance. The grass is allowed to grow to a height of 1 foot before it is mowed.

Basin Design Step 1: The first step is to determine the flow characteristics at the brink of the storm sewer outlet. A riprap pad was designed for this outfall in the design example in Subsection 11.11.1. The equivalent flow depth, y_e , and equivalent Froude number, Fr_e , were calculated in the prior example, and they are:

$$y_e = 1.3 \text{ feet, and } Fr_e = 2.0$$

Basin Design Step 2: In this step, Figure 11-17 is used to determine if a small or large basin is needed. Based on the Figure and the y_e and Fr_e values from Step 1, a large basin will be adequate.

Basin Design Step 3: In this step, Figures 11-21, 11-23, and 11-24 are used to determine the dimensions of the large basin. If a small basin were to be designed, its dimensions would be determined in a similar manner.

Based on Figure 11-21, the basin depth = $D = 2.3$ feet.

The total length of dissipator, L_T , is the greater of 33 feet or $4B$. $4B = (4)(3 \text{ ft}) = 12$ feet
 $L_T = 33$ feet

The length of basin, L_B , is the greater of 23 feet or $3B$. $3B = (3)(3) = 9$ feet $L_B = 23$ feet

The thickness of the riprap blanket on the upstream end of the dissipator = $T_1 = 1.3$ feet

The thickness of the riprap blanket on the remainder of the dissipator = $T_2 = 1.6$ feet

Based on Figure 11-23, the basin width, W_B , is the greater of 23 feet or $3B$.
 $3B = (3)(3) = 9$ feet $W_B = 23$ feet

The outlet width, W_O , will be wide enough to have non-erosive velocities at the outlet to the channel. This width will be calculated using the method in Subsection 11.12.1 as follows:

Outlet Design Step 1: In this step the maximum shear stress the surface of the swale can withstand is calculated. The grass on the side of the swale has a retardance of C , based on Retardance Classes of Grass Channel Linings Table of **Chapter 8**. The maximum shear this covering can withstand is 1.0 pounds per square foot, based on Permissible Shear Stresses for Linings Table of **Chapter 8**.

Outlet Design Step 2: In this step the maximum depth of flow down the side of the swale is calculated. The slope of the energy grade line is assumed to be the same as the 1V: 6H slope of the swale face. $1 / 6 = 0.17$ $S = 0.17$ foot per foot Using Equation 11-10, the maximum allowable flow depth is:

$$y = \frac{1}{(62.4)(0.17)} = 0.09 \text{ feet}$$

Outlet Design Step 3: In this step the minimum width of the bottom of the dissipator outlet, W_O , is determined. It is assumed the deepest flow, and consequently, highest shear stress, occurs immediately beyond the end of the dissipator outlet. As a result, the flow depth at this location should not be deeper than the maximum depth determined in the previous step. The flow out of the end of the dissipator will be modeled by Manning's equation assuming normal flow occurs. The input into Manning's equation follows.

The maximum flow depth is 0.09 feet.

The slope of the swale face is the slope of the energy grade line, 0.17 foot per foot.

The shape of the outlet is a trapezoid with 1V: 2H side slopes.

The design discharge is 39 cubic feet per second.

A Manning's roughness coefficient of 0.15 is selected, based on the values in **Chapter 8**. This is the roughness value for sheet flow over a meadow or pasture. It is selected because the 1 foot grass height is considerably larger than the 0.09 foot flow depth, and the flow down the swale face will resemble overland sheet flow rather than channel flow.

Using Manning's equation, the minimum width of the bottom of the outlet is 492 feet. It is impractical to build a dissipator outlet this wide.

A channel with a lining that can withstand a shear larger than 1 pound per square foot will have a smaller bottom width than 492 feet. A shallow grass lined trapezoidal channel with a permanent flexible liner will be designed using the methods in **Chapter 8** to convey the flow down the slope to the bottom of the swale. Liners are available that can withstand a shear stress of 7.3 pounds per square foot, and a liner that can withstand this stress will be specified for this channel. The design is as follows:

Outlet Redesign Step 1: The maximum allowable shear stress on the liner is 7.3 pounds per square foot.

Outlet Redesign Step 2: The maximum allowable flow depth is:

$$y = \frac{7}{(62.4)(0.17)} = 0.69 \text{ feet}$$

Outlet Redesign Step 3: The .69 foot flow depth in the channel is too deep to be overland sheet flow. Instead, it is channel flow. The Manning's roughness coefficients for grass lined highway channels are listed in Appendix A, **Chapter 8**. A fair stand of grass with a height of 24-inches is assumed because the vegetation in the channel may be trimmed with a brush cutter rather than mowed. Listed roughness coefficients are 0.09 for a flow velocity of 6 feet per second, and 0.17 for a velocity of 2 feet per second.

Outlet bottom widths are calculated for a 0.69 foot flow depth using roughness values of 0.09 and 0.17, as listed in the following table. Channel velocities are also calculated, and Manning's equation is used as described earlier in this example. Using a roughness value of 0.09, the calculated flow velocity is 4.85 feet per second. The applicable velocity for this roughness is 6 feet per second based on Appendix A, **Chapter 8**. Likewise, at a roughness value of 0.17, the calculated flow velocity is different than the applicable flow velocity. The velocities are

also calculated for roughness values of 0.14, 0.15, and 0.16. These velocities are compared to interpolated velocities based on the values listed in Appendix A, **Chapter 8**. At a roughness value of 0.15 the calculated and interpolated flow velocities are similar. The outlet width at this roughness is 17.4 feet. This will be the width of the dissipator outlet and the channel bottom.

Manning's Roughness Coefficient	Calculated Velocity (feet/sec)	Interpolated Velocity (feet/sec)	Difference Between Velocities	Outlet Width (feet)
0.09*	4.85	5.91*	- 1.06	10.3
0.17*	2.67	1.97*	0.70	
0.14	3.21	3.45	- 0.24	
0.16	2.83	2.46	0.37	
0.15	3.01	2.95	0.06	17.4

*Values listed in Appendix A, **Chapter 8**

In summary: the dissipator outlet width, W_O , is 17.4 feet. The minimum bottom width of the channel is also 17.4 feet. The channel must be deep enough to accommodate a flow depth of 0.69 feet. The maximum slope of the channel sides is 1V: 2H. Flatter side slopes can be used if desired. The channel liner must withstand shear forces up to 7.3 pounds per square foot. If grass is planted in the channel, it must not get taller than 24-inches.

Basin Design Step 4 - In this step, the elevation of the water in the basin is calculated and the minimum elevation of the revetment on the sides of the basin is determined. The 17.4 feet bottom width of the outlet, W_O , is the length of the weir. Earlier in this example it was assumed the tailwater depth is negligible because the water would spill out of the dissipator outlet onto the side of the swale. This is no longer a valid assumption because the discharge from the basin will be confined in a channel. Consequently, the tailwater depth, TW , is the 0.69 feet depth of flow in the downstream channel.

An iterative method is used to calculate H , as follows: H is estimated. The percent submergence is calculated using Equation 11-11. C_s is read from Figure 11-25b. C_f is read from Figure 11-25a. H is calculated using Equation 11-13. Assumed and calculated H values are compared. The process is repeated until the assumed and calculated H values are similar. This process is illustrated in the following table.

Assumed H (feet)	Percent Submergence	C_s for Assumed H	C_f for Assumed H	Calculated H (feet)	Difference Between Assumed and Calculated H (feet)
0.72	95	0.77	3.03	0.97	-0.25
0.92	75	1.00	3.06	0.81	0.11
0.82	84	0.98	3.04	0.82	0.00

Based on the preceding table and Equation 11-12, the water surface elevation in the dissipator basin is 0.82 feet higher than the elevation of the bottom of the dissipator outlet. The minimum elevation of the top of the revetment on the sides of the basin is 1 foot higher than this, or 1.82 feet above the elevation of the dissipator outlet.
