Chapter 9
Hydraulic Structures

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Appendix A

1.0 Structures in Streams

Photograph 9-1. This grouted boulder drop structure exemplifies the opportunity available for creating an attractive urban hydraulic setting for a riparian corridor.

Hydraulic structures are used to guide and control water flow in streams. Structures described in this chapter consist of grade control structures and outfall structures for various applications and conditions.

The discussion of grade control structures in this chapter addresses the hydraulic design and grouted boulder, sculpted concrete, and vertical drop structures, whereas the *Open Channels* chapter discusses the placement of grade control structures in the stream and the *Stream Access and Recreational Channels* chapter covers safety considerations relevant to all urban streams and specialized design of boatable hydraulic structures.

The outfalls section provides design guidance for various types of pipe end treatment and rock protection to dissipate hydraulic energy at outfalls of storm drains and culverts. Related design information is covered in the *Streets, Inlets, and Storm Drains* and *Culverts and Bridges* Chapters.

Considered environmental, ecological, and public safety objectives in the design of each structure. The proper application of hydraulic structures can reduce initial and future maintenance costs by managing the character of the flow to best meet all project needs.

The shape, size, and features of hydraulic structures vary widely for different projects, depending upon the design discharge and functional needs of the structure. Hydraulic design procedures discussed herein govern design of all structures. For the design of unique structures that may not fit the guidance provided, hydraulic physical modeling or computational fluid dynamics (CFD) modeling may be beneficial.

Guidance for Using this Chapter

- Determine if the project can be designed using the simplified method (Section 2.2) or if a detailed design is required (Section 2.3).
- Perform soils and seepage analyses as necessary for the design of the foundation and seepage control system (Section 2.4). Additional analysis of forces acting on a structure may be necessary and should be evaluated on a case-by-case basis (Section 2.5).
- Use criteria specific to the type of drop structure to determine the final flow characteristics, dimensions, material requirements, and construction methods. Refer to Section 2.6 for Grouted Stepped Boulder (GSB) drop structures or to Section 2.7 for Sculpted Concrete (SC) drops.
- Refer to the *Trails and Recreations Channels* chapter for design of boatable structures and other criteria required for public safety.

2.0 Grade Control Structures

2.1 Overview

Photograph 9-2. Grouted stepped boulder drop structures such as this one in Denver's Bible Park can be safe, aesthetically pleasing, and provide improved aquatic habitat besides performing their primary hydraulic function of energy dissipation.

As discussed in the *Open Channels* Chapter, urbanization increases the rate, frequency and volume of runoff in natural streams and, over time, this change in hydrology may cause streambed degradation, otherwise known as down cutting or head cutting. Stabilization improvements to the stream are necessary prior to or concurrent with development in the watershed. Stream stabilization is the third step of the *Four Step Process to Stormwater Management* (see Chapter 1 of Volume 3 of this manual).

"Drop structures" are broadly defined. Drop structures provide protection for high velocity hydraulic conditions that allow a drop in channel grade over a relatively short distance. They provide controlled and stable locations for a hydraulic jump to occur, allowing for a more stable channel downstream where flow returns to subcritical. This chapter provided specific design guidance for the following basic categories of drop structures:

- Grouted stepped boulder (GSB) drop structures
- Sculpted concrete (SC drop structures
- Vertical drop structures

The design of the drop structure crest and the provision for the low flow channel directly affect the ultimate configuration of the upstream reach. A higher unit flow will pass through the low flow area than will pass through other portions of the stream cross section. Consider the situation in design to avoid destabilization of the drop structure and the stream. It is also important to consider the major flood, the path of which frequently extends around structure abutments.

Design grade control structures for fully developed future basin conditions, in accordance with zoning maps, master plans, and other relevant documents. The effects of future hydrology and potential down cutting will negatively impact the channel.

There are two fundamental systems of a drop structure that require design consideration: the hydraulic surface-drop system and the foundation and seepage control system. The surface drop system is based on project objectives, stream stability, approach hydraulics, downstream tailwater conditions, height of the drop, public safety, aesthetics, and maintenance considerations. The material components for the foundation and seepage control system are a function of soil and groundwater conditions. One factor that influences both systems is the potential extent of future downstream channel degradation. Such degradation could cause the drop structure to fail.

See the *Stream Access and Recreational Channels* chapter for special design issues associated with drop structures in boatable channels.

Drops in series require full energy dissipation and return to normal depth between structures or require specialized design beyond the scope of this manual.

Evaluate drop structures during and after construction. Secondary erosion tendencies will necessitate additional bank and bottom protection. It is advisable to establish construction contracts and budgets with this in mind.

The sections that follow provide guidance on drop structure design using either a simplified design method or a more detailed hydraulic design method. The designer must evaluate each method and determine which is appropriate for the specific project.

Key Considerations during Planning and Early Design of a Drop Structure

- Identify the appropriate range of drop height based on the stable channel slope (as provided in the master plan or based on guidance provided in the *Open Channels* chapter). Limit the net drop height to five feet or less to avoid excessive kinetic energy and avoid the appearance of a massive structure. Vertical drops should not exceed 3 feet at any location to minimize the risk of injury from falling. With a 12-inch stilling basin, this limits the net drop height to two feet.

- Design with public safety in mind. Structures located in streams where boating, including tubing, is anticipated require additional considerations. See the *Stream Access and Recreational Channels* chapter.

- Begin the process of obtaining necessary environmental permits, such as a Section 404 permit, early in the project.

- Evaluate fish passage requirements when applicable. This may also be a requirement of environmental permits.

2.2 Simplified Design Procedures for Drop Structures

2.2.1 Introduction

The simplified design procedure can be used for grade control structures meeting design criteria provided in Table 9-1 and where all of the following criteria are met:

- Maximum unit discharge for the design event (typically the 100-year) over any portion of the drop structure is 35 cfs/ft or less,
- Net drop height (upstream channel invert less downstream channel invert exclusive of stilling basin depth) is 5 feet or less,
- Drop structure is constructed of GSB or SC,
- Drop structure is located within a tangent section and at least twice the distance of the width of the drop at the crest both upstream and downstream from a point of curvature,
- Drop structure is located in a reach that has been evaluated per the design requirements of the *Open Channel* chapter.

The simplified design procedures provided herein do not consider channel curvature, effects of other hydraulic structures, or unstable beds. If any of these conditions exist or the criteria above are not met, a detailed analysis is required per Section 2.3. Even if the criteria are met and the simplified design procedures are applied, checking the actual hydraulics of the structure using the detailed comprehensive hydraulic analysis may yield useful design insight.

There is a basic arrangement of upstream channel geometry, crest shape, basin length, and downstream channel configuration that will result in optimal energy dissipation. The following sections present simplified relationships that provide basic configuration and drop sizing parameters that may be used when the above criteria are met.

2.2.2 Geometry

Table 9-1 below summarizes the specific design and geometric parameters applicable to drop structures designed using the simplified design procedures. Additional discussion is provided in the sections following for some of the specific parameters summarized in the table. Graphical depiction of the geometric parameters listed in Table 9-1 can be found in Figure 9-11 through 9-14 for GSB drop structures and Figures 9-16 through 9-21 for SC drop structures.

Table 9-1. Design criteria for drop structures using simplified design procedures

Design Parameter	Requirement to Use Simplified Design Procedures	
	GSB Drop Structure	**SC Drop Structure**
Maximum Net Drop Height (H_d)	5 feet[1]	
Maximum Unit Discharge over any Portion of Drop Width	35 cfs per foot of drop width (see Section 2.2.3)	
Maximum Longitudinal Slope (Steepest Face Slope)	4(H):1(V) (see Section 2.2.4 for additional discussion)	
Minimum Stilling Basin Depression (D_b)	1 foot (see Section 2.2.6 for additional discussion and requirements for non-cohesive soils)	2 feet (see Section 2.2.6 for additional discussion and requirements for non-cohesive soils)
Minimum Length of Approach Riprap (L_a):	8 feet	
Minimum Stilling Basin Length (L_b):	Determine using Figure 9-1 (see Section 2.2.4)	
Minimum Stilling Basin Width (B)	same as crest width	
Minimum Cutoff Wall Depth	6 feet (for cohesive soils only, see Section 2.2.6 for additional discussion)	
Minimum Length of Riprap Downstream of Stilling Basin	10 feet	
Minimum D_{50} for Approach and Downstream Riprap	12 inches	
Minimum Boulder Size for Drop Structure	Per Figure 9-1	N/A

[1]This is considered a large drop structure and is only appropriate where site specifics do not accommodate installation of smaller drop structures. Urban Drainage and Flood Control District (UDFCD) recommends the height of the drop structure not exceed 3 feet.

2.2.3 Unit Discharge

The unit discharge is an important design parameter for evaluating the hydraulic performance of a drop structure. In order to use the simplified design procedures, the design event maximum unit discharge over any portion of the drop structure width is 35 cfs/ft. This value is derived from recommended values for velocity and depth listed in the *Open Channels* chapter. Typically, this maximum unit discharge will occur in the low-flow channel, but in rare circumstances may be in the overbanks. Determine the design unit discharge at the crest of the drop structure and at a channel cross section 20 to 50 feet upstream of the crest. Depending on the depth of the low-flow channel at these two locations, the unit discharge could differ at the sections. Normally, the maximum unit discharge of the cross sections and exercise judgement regarding the appropriate unit discharge used for the drop structure design. Further discussion on the hydraulic evaluation of a channel cross section is in Section 2.3.6.

2.2.4 Longitudinal Slope of the Drop Structure Face

The longitudinal slope of the structure face should be no steeper than 4(H):1(V), while even flatter slopes will improve safety. Flatter longitudinal face slopes (i.e., flatter than 8(H):1(V), help to mitigate overly retentive hydraulics at higher tailwater depths that can cause submerged hydraulic jump formation and create reverse rollers with “keeper” waves which are a frequent cause of drowning deaths in rivers. Where possible roughen the face of the drop by developing a series of slopes rather than a smooth surface. Individual steps and differences in vertical elevation should be no greater than 3 feet in any location to limit consequence associated with slip and fall during dry conditions. The *Stream Access and Recreational Channels* chapter provides additional longitudinal slope considerations for water-based recreation and in-channel safety as well as other avoidance techniques for overly-retentive drop structures.

Overly Retentive Hydraulics

Drop faces should have a longitudinal slope no steeper than 4(H):1(V). The formation of overly retentive hydraulics is a major drowning safety concern when constructing drop structures. Longitudinal slope, roughness and drop structure shape all impact the potential for dangerous conditions. See the *Stream Access and Recreational Channels* chapter for additional criteria.

2.2.5 Stilling Basin

Typically, drop structures include a hydraulic jump dissipater basin. The stilling basin should be depressed in order to start the jump near the toe of the drop face, per the requirements in Table 9-1. A sill should be located at the basin end to create a transition to the downstream invert elevation. The profiles for GSB (Figure 9-12) and SC (Figure 9-17) drop structures include options for both non-draining and draining stilling basins. Where it is undesirable to have standing water, provide an opening in the end sill.

When using the simplified design, the length of the stilling basin (L_b) can be determined using Figure 9-1. Figure 9-1 provides the required stilling basin length for both GSB and SC drop structures up to a unit discharge of 35 cfs/ft. If the proposed drop structure does not fit within the requirements of the simplified design, complete a detailed hydraulic analysis as described in Section 2.3.

In non-cohesive soil channels and channels where future degradation is expected, especially where there is no drop structure immediately downstream, it is generally recommended that the stilling basin be eliminated and the sloping face extended five feet below the downstream future channel invert elevation (after accounting for future streambed degradation). A scour hole will form naturally downstream of a structure in non-cohesive soils and construction of a hard basin is an unnecessary cost. Additionally, a hard basin would be at risk for undermining. See Figure 9-12 for the profile of the GSB and Figure 9-17 for that of an SC in this configuration. In some cases, the structure may have a net drop height of zero immediately after construction, but is designed with a long-term net height of 3 to 5 feet to accommodate future lowering of the channel invert.

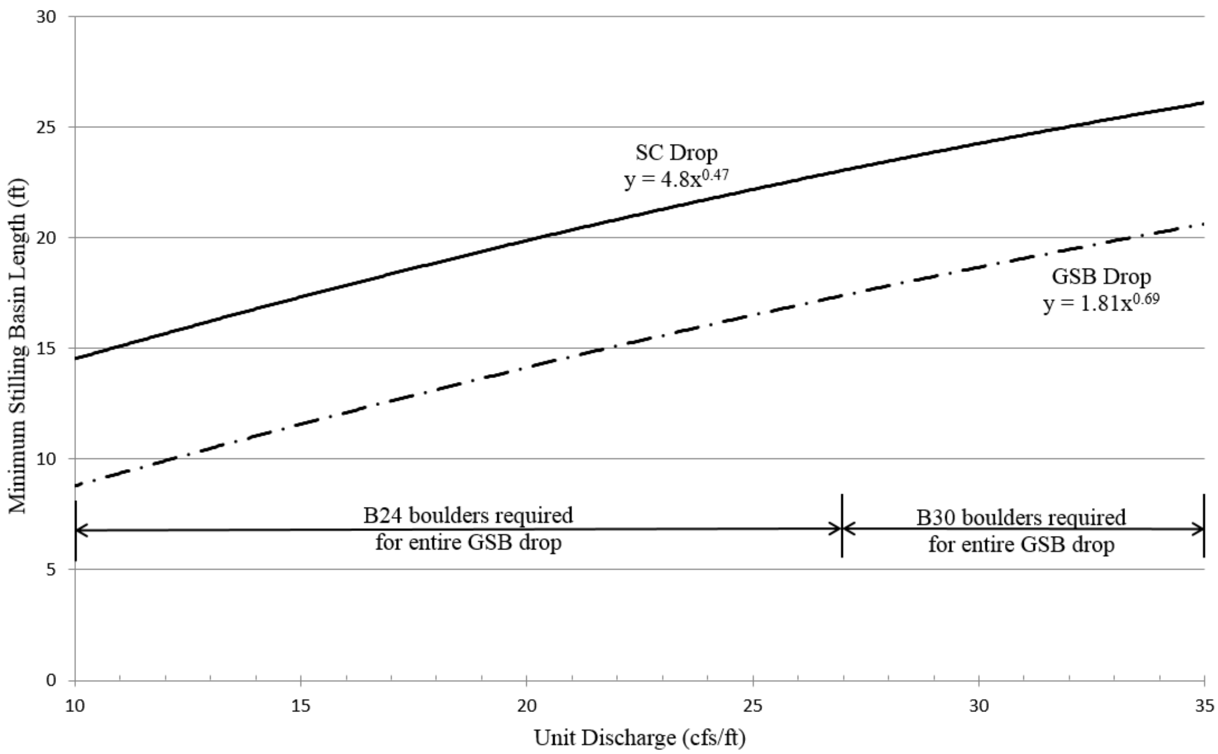


Figure 9-1. Stilling basin length based on unit discharge (for simplified design procedure)

2.2.6 Seepage Analysis and Cutoff Wall Design

The simplified drop structure design only applies to drops with cutoffs located in cohesive soils. Therefore, it is necessary to determine surface and subsurface soil conditions in the vicinity of a proposed drop structure prior to being able to use the simplified approach for cutoff design. For a drop structure constructed in cohesive soils meeting all requirements of a simplified design, the cutoff wall must be a minimum of six feet deep for concrete and ten feet deep for sheet pile.

If a proposed drop structure meets the requirements of the simplified approach, but is located in non-cohesive soils, guidance on determining the required cutoff wall depth is described in Section 2.4.

Photograph 9-4. View of the sheet pile cutoff wall and steel reinforcement for a sculpted concrete drop structure prior to the concrete placement. Note the steel reinforcement has been spot welded to the sheet pile.

The vertical seepage cutoff wall should be located upstream of the crest and can be constructed of either concrete or sheet pile. One of the most important details for grade control structures involves the interface between the seepage cutoff wall and the remainder of the structure. Regardless, of the material used for the cutoff wall, the structure should completely bury the interface between the wall and structure. This eliminates the unattractive view of the cutoff wall within the drop structure and provides a more effective seal at the interface. To ensure a good seal, specify that the contractor must fully clean the surface of the cutoff wall prior to the construction of the interface. Figures 9-7 through 9-9 provide multiple options (for both GSB and SC drop structures) for connecting the verticle cutoff wall to the drop structure. Additionally, the cutoff wall should extend beyond the low-flow channel and five to ten feet into the bank on each side of the structure as shown in Figure 9-27.

Take special care when designing cutoff walls for drops in series. This typically requires a deeper wall or a wall at each crest.

2.2.7 Low-flow Channel

The crest of the drop structure is frequently shaped similarly to, although sometimes slightly shallower than, the upstream low-flow channel. It is also typical that the shape transition along the face of the structure in an effort to disperse the flow and dissipate energy over the width of the drop structure. This geometry is recommended unless the stream is boatable. The low-flow channel can then be re-established beyond the end sill of the drop structure. In some circumstances protection in the low-flow channel may need to extend further downstream than protection in the main channel. This should be evaluated on a case-by-case basis. When the stream is boatable, it is typically preferred that flows remain concentrated through the drop.

2.3 Detailed Drop Structure Hydraulic Analysis

2.3.1 Introduction

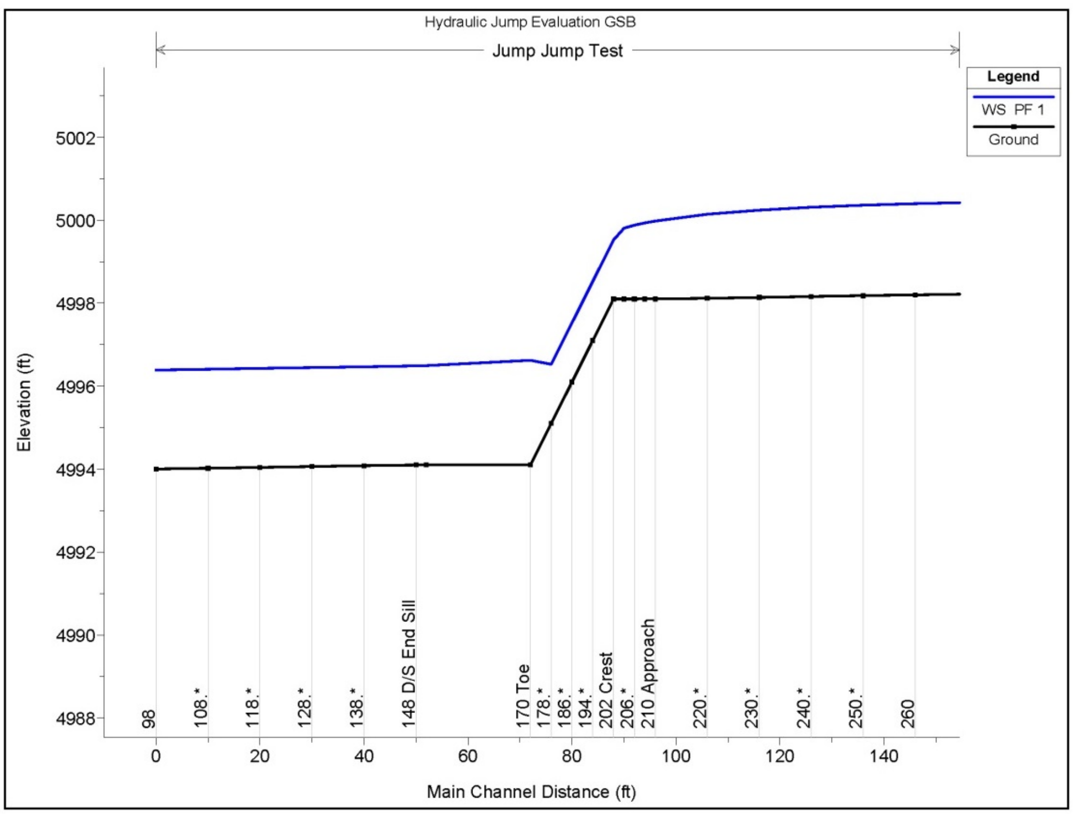
When the parameters of a proposed drop structure do not fit within the criteria of a simplified design (see Section 2.2), or when a designer desires a more thorough analysis of drop structure hydraulics, a detailed hydraulic analysis is conducted. The guidelines presented in this section assume that the designer is using HEC-RAS to assist with the detailed computations necessary for drop structure analysis. It is important to be familiar with the HEC_RAS variables selected for the computations and the effect these variables have on the results of the analysis. The analysis guidelines discussed in this section are intended to assist the engineer in addressing critical hydraulic design factors.

2.3.2 Cross Section Placement

Appropriate placement of cross sections is important when completing a hydraulic analysis of a drop structure using HEC_RAS. Place cross-sections at the following locations:

- Upstream of Drop (50 feet +/-) where channel is at normal depth
- Drop Approach (5 feet +/- upstream of drop crest)
- Drop Crest
- Toe of Drop
- Upstream and at Drop End Sill
- Downstream of Drop (50 feet +/-) where channel has recovered to normal depth

In addition to the locations above, use the “cross section interpolation” option in HEC_RAS. At a minimum, add interpolated cross sections (denoted with * in Figure 9-2) along the drop face. Interpolated cross sections upstream of the drop crest and downstream of the end sill may also be beneficial.
Figure 9-2 provides a sample channel profile from HEC_RAS with cross section locations for reference.



*Denotes Interpolated Cross Section

Figure 9-2. Sample HEC_RAS profile with cross section locations for hydraulic analysis

2.3.3 Mannings's Roughness Coefficient for Drop Structures

Depending on the type of materials and the relative depth, select the appropriate roughness parameters for the HEC-RAS model. Table 9-2 provides roughness parameter recommendations and references for both sculpted concrete and grouted boulder drop structure.

Table 9-2. Approximate Manning's roughness at design discharge for stepped drop structure

Stepped sculpted concrete where step heights equal 25% of drop	0.025[1]
Grouted Boulders	See Figure 9-3

[1] This assumes an approach channel depth of at least 5 feet. Values would be higher at lesser flow depths.

The equations typically used for riprap and provided in the *Open Channels* chapter do not apply to boulders and grouted boulders because of their near uniform size and because the voids may be completely or partially filled with grout. Therefore, the Manning's roughness values for grouted boulders are based on (Chow 1959; Oliver 1967; Anderson et. Al 1973; Henderson 1966; Barnes 1967; Smith and Murray 1975; Stevens et. Al. 1976; Bathurst, Li and Simons 1979; and Stevens 1984). The roughness coefficient varies with the depth of flow relative to the size of the boulders and the depth of grout used to lock them in place.

The following equations may be used to find the recommended Manning's *n* as a function of flow depth over height of the boulders, *y/D*, as represented by the curves in Figure 9-3:

When the upper one-half (plus or minus 1inch) of the rock height is ungrouted, the equation for *n* is:

$$n_{24"-42"(1/2)} = \frac{0.097\left(y/D\right)^{0.16}}{\ln\left(2.55y/D\right)} \qquad \text{Equation 9-1}$$

When the upper one-third (plus or minus 1 inch) of the rock height is ungrouted, the equation for *n* is:

$$n_{24"-42"(2/3)} = \frac{0.086\left(y/D\right)^{0.16}}{\ln\left(2.55y/D\right)} \qquad \text{Equation 9-2}$$

Where:

y = depth of flow above top of rock (feet)

D = diameter of the boulder (feet)

The upper limit for Equation 9-1 is n $\leq$ 0.104 and for Equation 9-2 is *n* $\leq$ 0.092. Determine the value for "y" by reviewing the HEC_RAS cross sections and determining an appropriate representation of the average flow depth over the structure. If the value for *y/D* is < 1, use 1.

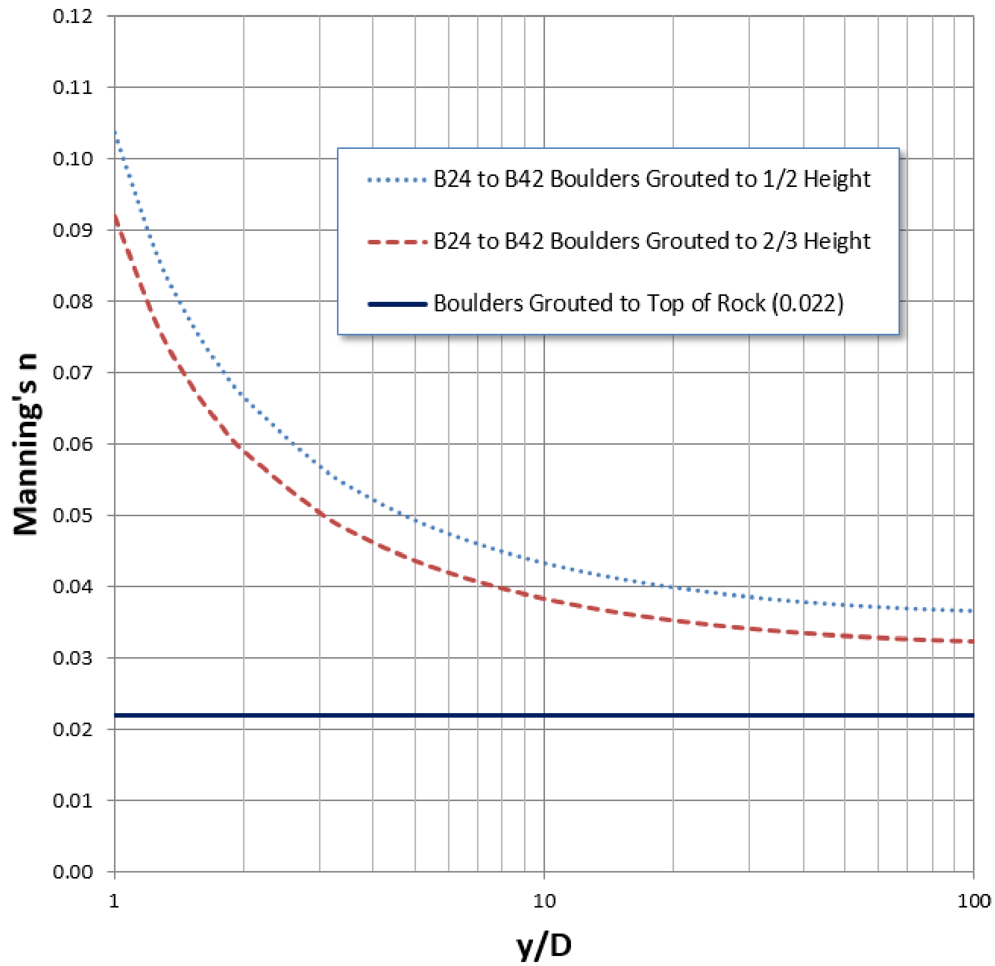


Figure 9-3. Recommended Manning's n for flow over B24 to B42 grouted boulders

Using a stepped grouted rock placement and grouting only the lower ½ of the rock on the drop face creates a significantly higher Manning's n roughness coefficient and, as a result, greater flow depth and lower velocity, reducing the boulder size needed to have a stable structure. Refer to Section 2.6.3 for discussion on boulder sizing for GSB drop structures.

2.3.4 Hydraulic Jump Formation

Once the location and geometry of the drop structure cross sections have been determined, evaluate the HEC-RAS model for the design flow under both subcritical and supercritical flow conditions. To minimize the stilling basin length, use a downstream tailwater depth great enough to force a hydraulic jump to start near the toe of the drop face. This requires that the specific force of the downstream tailwater be greater than the specific force of the supercritical flow at the toe of the drop. The tailwater is modeled by a subcritical water surface (M1 backwater or M2 drawdown curve) profile analysis that starts from a downstream control point and works upstream to the drop structure basin. Model the depth and specific force at the toe of the drop by a supercritical water surface (S2 drawdown curve) profile analysis starting at the crest of the drop and running down the drop face.

Using the output from the subcritical and supercritical HEC-RAS hydraulic models, calculations should be completed to verify that the specific force associated with the downstream tailwater is greater than the specific force of the supercritical flow at the toe of the drop, not only for the design discharge, but for flows corresponding to more frequent events. Specific force can be calculated using equation 9-3 (Chow 1959):

$$F = \frac{Q^2}{gA} + \bar{z}A \qquad \text{Equation 9-3}$$

Where:

F = specific force

Q = flow at cross section

g = acceleration of gravity

$\bar{z}$ = distance from the water surface elevation to the centroid of the flow area (A)

A = area of flow

The required tailwater depth is determined using Equation 9-4 (Chow 1959). This equation applies to rectangular channel sections and should be applied to a rectangular portion of flow within a drop structure. For irregular (non-rectangular) channel shapes, the designer should apply Equation 9-4 using the unit discharge within a rectangular segment of the drop crest. Assuming the low-flow channel is incorporated into the drop crest and this portion of the crest has the largest unit discharge, the rectangular portion would extend over the bottom width of the low-flow channel. See Section 2.3.6 for additional discussion on evaluating the conditions in both the low-flow channel and the overbanks.

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) \qquad \text{Equation 9-4}$$

Where:

y_2 = required depth of tailwater (also called the sequent depth, in feet)

y_1 = depth of water at drop toe, feet (taken from cross section at drop toe, supercritical HEC-RAS model)

F_1 = Froude Number = $V_1/(gy_1)^{1/2}$ (based on depth and velocity at drop toe)

Calculate the required tailwater depth (y_2) using Equation 9-4. Compare the results of this calculation to the modeled tailwater depth determined in the subcritical HEC-RAS model at the upstream side of the end sill (channel depth plus Db). The modeled tailwater depth must be greater than or equal to the calculated required headwater depth for a hydraulic jump to start near the toe of the drop. If the modeled tailwater depth is less than required, the drop structure geometry must be re-evaluated. One option is to increase the depth of the stilling basin, thereby increasing the effective tailwater depth and specific force, and another is to widen the crest of the drop or reduce the depth of the low-flow channel to produce a smaller unit discharge.

2.3.5 Hydraulic Jump Length

After the hydraulic jump has been analyzed using the guidelines provided in Section 2.3.4, the jump length must be calculated. This will aid the designer in determining the appropriate stilling basin length and the need for additional rock lining downstream of the end sill. The following values are required to determine the hydraulic jump length:

y_2 = required depth of tailwater (feet)

F_1 = Froude Number = $V_1/(gy_1)^{1/2}$ (based on depth and velocity at drop toe)

Use the above values to determine the length of the hydraulic jump (L) in Figure 9-4. Note that this figure is for horizontal channels, which is appropriate for most applications in the UDFCD region. Curves for sloping channels (from 5 to 25%) are in Chow, 1959.

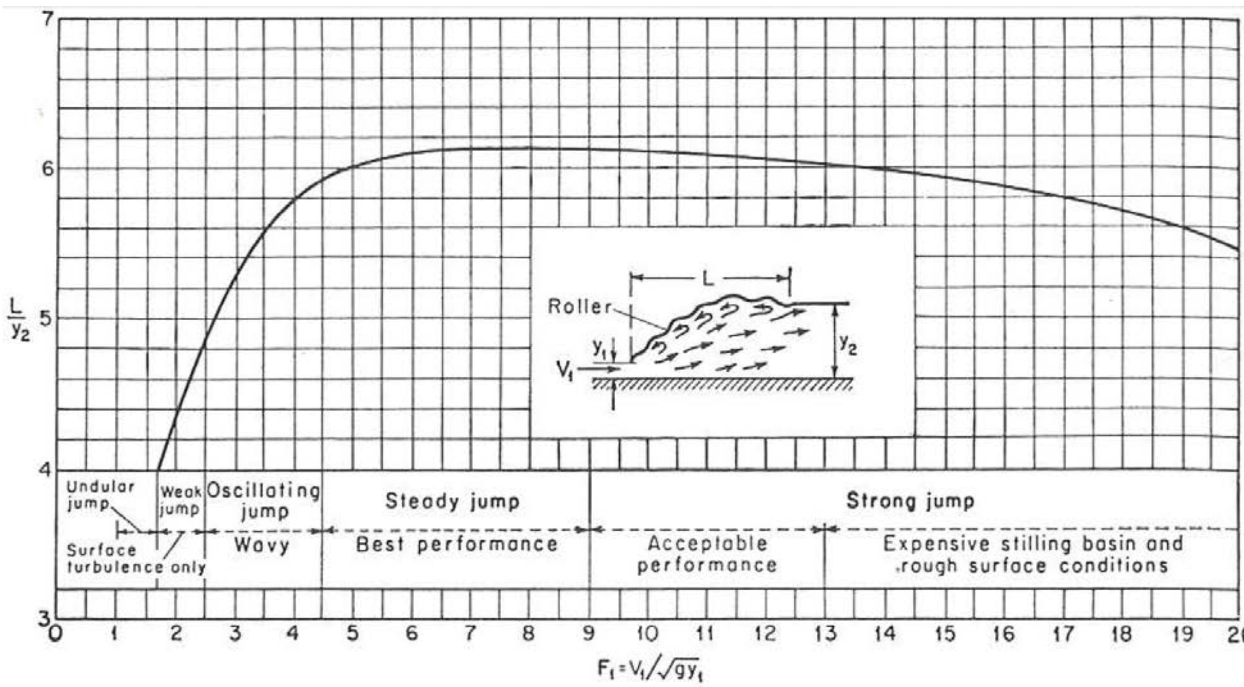


Figure 9-4. Length in terms of sequent depth of jumps in horizontal channels
(Source: US Bureau of Reclamation, 1955)

UDFCD recommends a hard-lined stilling basin (sculpted concrete, grouted boulders, or concrete grout) that is at least 60% of the hydraulic jump length (L). Extend riprap downstream of the sill and provide protection for at least the balance of the full hydraulic jump length (see Figure 9-5). Determine riprap size using the equations provided in the *Open Channels* chapter for channel lining.

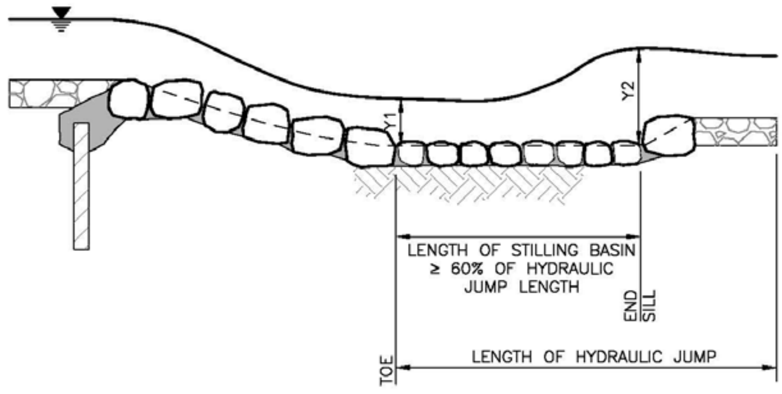


Figure 9-5. Stilling basin profile

2.3.6 Evaluation of Low-flow Channel versus Overbanks

Review the HEC-RAS model to evaluate the hydraulic conditions in both the low-flow channel and the overbanks at the crest and 20 to 50 feet upstream of the crest and determine the maximum representative unit discharge (See Section 2.2.3). Check the shear velocity in the overbanks of low-flow drops to determine if protection in this area is appropriate.

Use the "worst case" hydraulic scenario to design the entire drop structure. In most conditions, the low-flow channel will see the greater unit discharge and velocity and therefore represent the "worst case." HEC-RAS provides output tables to assess the conditions in both the low-flow and overbanks (see Figure 9-6).

Certain site conditions may warrant a separate evaluation for the low-flow channel and overbanks. In some cases, the designer may elect to extend the stilling basin longer in the low-flow channel area than the overbanks; however, in such cases the transition in basin length should be gradual rather than abrupt.

Plan: extrasec Jump Jump Test RS: 202 Profile: PF 1

E.G. Elev (ft)	5001.06	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.65	Wt. n-Val.	0.05	0.05	0.05
W.S. Elev (ft)	5000.41	Reach Len. (ft)	4	4	4
Crit W.S. (ft)	5000.41	Flow Area (sq ft)	69.05	23.09	69.05
E.G. Slope (ft/ft)	0.027778	Area (sq ft)	69.05	23.09	69.05
Q Total (cfs)	1000	Flow (cfs)	400.13	199.74	400.13
Top Width (ft)	118.47	Top Width (ft)	54.23	10	54.23
Vel Total (ft/s)	6.2	Avg. Vel. (ft/s)	5.8	8.65	5.8
Max Chl Dpth (ft)	2.31	Hydr. Depth (ft)	1.27	2.31	1.27
Conv. Total (cfs)	6000	Conv. (cfs)	2400.8	1198.4	2400.8
Length Wtd. (ft)	4	Wetted Per. (ft)	54.56	10	54.56
Min Ch El (ft)	4998.1	Shear (lb/sq ft)	2.19	4	2.19
Alpha	1.09	Stream Power (lb/ft s)	12.72	34.64	12.72
Frctn Loss (ft)	0.1	Cum Volume (acre-ft)	0.03	0.46	0.03
C & E Loss (ft)	0	Cum SA (acres)	0.03	0.2	0.03

Figure 9-6. Sample HEC-RAS output for cross section located at drop crest

2.3.7 Evaluate Additional Return Period Flow Rates

Evaluate the design flow and then assess additional return-period flow rates, as appropriate. For all flows, the actual downstream tailwater should be greater than the tailwater required to force a hydraulic jump to start near the toe of the drop structure face. When this condition is met for a range of events a stilling basin length of 60% of the hydraulic jump length should be adequate.

2.3.8 Rock Sizing for Drop Approach and Downstream of End Sill

Calculate the appropriate rock size for the drop approach and downstream of the end sill. The hydraulic conditions at the approach include the acceleration effects of the upstream drawdown as the water approaches the drop crest. Turbulence generated from the hydraulic jump will impact the area downstream of the end sill. Determine riprap size using the equations provided in the *Open Channels* chapter for channel lining. Because normal depth conditions do not exist upstream and downstream of the drop structure, refer to the HEC-RAS output and use the energy grade line slope (rather than channel slope) to determine the appropriate riprap size.

Riprap at the approach and downstream of the end sill should be a minimum D_{50} of 12-inches, or larger as determined using the channel lining equation in the *Open Channels* chapter. Use either void-filled or soil-filled riprap in these areas.

2.4 Seepage Control

2.4.1 Introduction

Subgrade erosion caused by seepage and structure failures caused by high seepage pressures or inadequate mass are two failure modes of critical concern.

Seepage analyses can range from hand-drawn flow nets to computerized groundwater flow modeling. Use advanced geotechnical field and laboratory testing techniques confirm permeability values where complicated seepage problems are anticipated. Several flow net analysis programs are currently available that are suitable for this purpose. Full description of flow net analysis is beyond the scope of the Urban Storm Drainage Criteria Manual (USDCM). Referred to Cedergren 1967; USBR 1987; and Taylor 1967 for more information and instruction in the use of flow net analysis techniques. See Section 2.4.3 for Lane's Weighted Creep method, a simplified approach.

2.4.2 Weep Drains

Install weep drains in all grade control structures greater than 5 feet in net height or as recommended by the geotechnical engineer. Weep drains assist in reducing the uplift pressure on a structure by providing a location for groundwater to escape safely through a filter. For concept, see Figure 9-10. Weep drains should be placed outside of the low-flow path of the structure and spaced to provide adequate relief of subsurface pressures.

2.4.3 Lane's Weighted Creep Method

As a minimum level of analysis and as a first order of estimation, Lane's Weighted Creep (Lane's) Method can be used to identify probable seepage problems, evaluate the need for control measures, and estimate rough uplift forces. It is not as definitive as the flow net analyses mentioned above. Lane's method was proposed by E.W. Lane in 1935. This method was removed from the 1987 revision of *Design of Small Dams* (USBR 1987), possibly indicating greater use of flow net and computer modeling methods or perhaps for other reasons not documented. Although Lane's method is relatively well founded, it is a guideline, and when marginal conditions or complicated geological conditions exist, use the more sophisticated flow-net analysis.

The essential elements of Lane's method are as follows:

1. The weighted-creep distance through a cross section of a structure is the sum of the vertical creep distances, Lv (along contact surfaces steeper than 45 degrees), plus one-third of the horizontal creep distances, L_H (along contact surfaces less than 45 degrees).

2. The weighted-creep head ratio is defined as:

$$C_W = \frac{\left(\frac{L_H}{3} + L_V\right)}{H_S}$$ Equation 9-5

Where:

C_W = creep ratio

H_S = differential head between analysis points (ft)

3. Reverse filter drains, weep holes, and pipe drains help to reduce seepage problems, and recommended creep head ratios may be reduced as much as 10% if they are used.

4. In the case where two vertical cutoffs are used, then Equation 9-6 should be used along with Equation 9-2 to check the short path between the bottom of the vertical cutoffs.

$$C_{W2} = \frac{\left(L_{V-US} + \quad 2L_{H-C} + L_{V-DS}\right)}{H_S}$$ Equation 9-6

Where:

C_{W2} = creep ratio where two vertical cutoffs are used

$L_{V\text{-}US}$ = vertical distance on the upstream side of the upstream cutoff (ft)

$L_{V\text{-}DS}$ = vertical distance on the downstream side of the downstream cutoff (ft)

$L_{H\text{-}C}$ = horizontal distance between the two vertical cutoffs (ft)

5. If there are seepage lengths upstream or downstream of the cutoffs, they should be treated in the numerator of Equation 9-6 similar to Equation 9-5. Seepage is controlled by increasing the total seepage length such that C_W or C_{W2} is raised to the value listed in Table 9-3. Test soils during design and again during construction.

6. Estimate the upward pressure in design by assuming that the drop in uplift pressure from headwater to tailwater along the contact line of the drop structure is proportional to the weighted-creep distance.

Table 9-3. Lane's weighted creep: Recommended minimum ratios

Material	Ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.0
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	3.0
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

2.4.4 Foundation/Seepage Control Systems

As a general rule, groundwater flow cutoffs should not be installed at the downstream ends of drop structures. They can cause greater hydraulic uplift forces than would exist without a downstream cutoff. The design goal is to relieve the hydrostatic pressures along the structure and not to block the groundwater flow and cause higher pressures to build up.

The hydraulic engineer must calculate hydraulic loadings that can occur for a variety of conditions such as dominant low flows, flood flows, design flows and other critical loading scenarios. A geotechnical engineer should combine this information with the on-site soils information to determine foundation requirements. Both engineers should work with a structural engineer to establish final loading diagrams and to determine and size structural components.

The designer needs to be cognizant of field conditions that may affect construction of a drop structure, including site water control and foundation moisture and compaction. A common problem is destabilization of the foundation soils by rapid local dewatering of fine-grained, erosive soils or soils with limited hydraulic conductivity. Since subsurface water control during construction is so critical to the successful installation of a drop structure, the designer needs to develop ways to ensure that the contractor adequately manages subsurface water conditions.

During construction, check design assumptions in the field including the actual subgrade condition with respect to seepage control assumptions be inspected and field verified. Ideally, the engineer who established the design assumptions and calculated the required cutoffs should inspect the cutoff for each drop structure and adjust the cutoff for the actual conditions encountered. For example, if the inspection of a cutoff trench reveals a sandy substrate rather than clay, the designer may choose to extend the cutoff trench, or specify a different cutoff type. Pre-construction soil testing is an advisable precaution to minimize changes and avoid failures.

Proper dewatering in construction will also improve conditions for construction structures. See Fact Sheet SM-08, Temporary Diversion Methods, located in Volume 3 of this manual.

2.5 Detailed Force Analysis

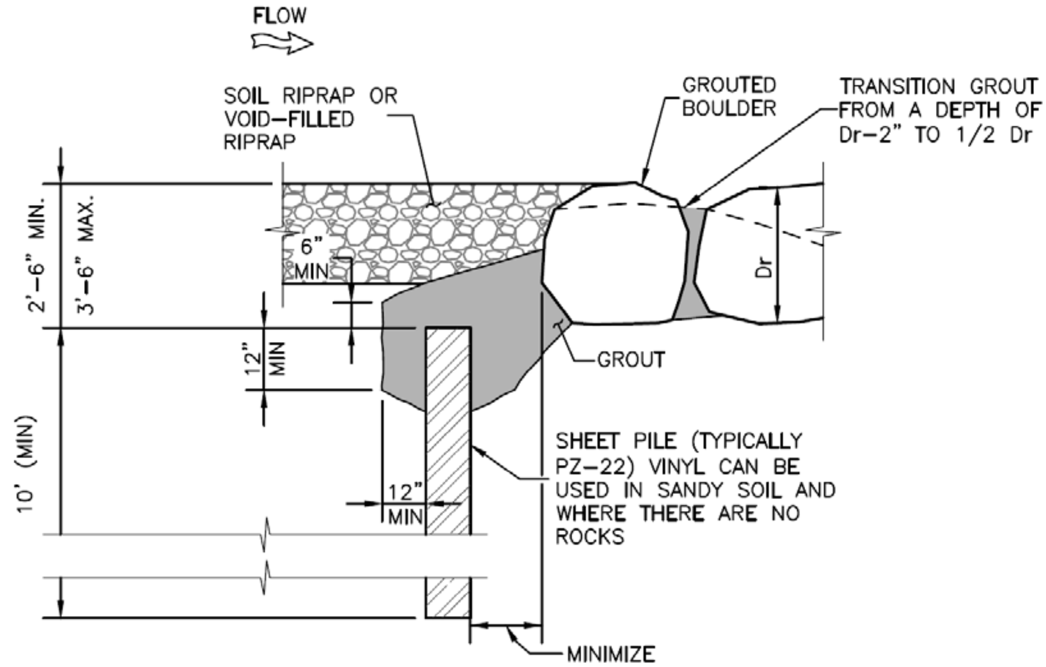
Each component of a drop structure has forces acting upon it that the design engineer should consider. While a brief summary of these forces is provided in this section, it is beyond the scope of this manual to provide detailed guidance on the evaluation of these forces. It is the design engineer's responsibility to properly account for potential forces in the drop structure design.

While a detailed force analysis may not be necessary for drop structures developed using the guidelines presented in the simplified design procedures, the designer may want to check forces acting on a drop structure. The critical design factors are seepage cutoff and relief and pressure fluctuations associated with the hydraulic jump that can create upward forces greater than the weight of water and structure over the point of interest.

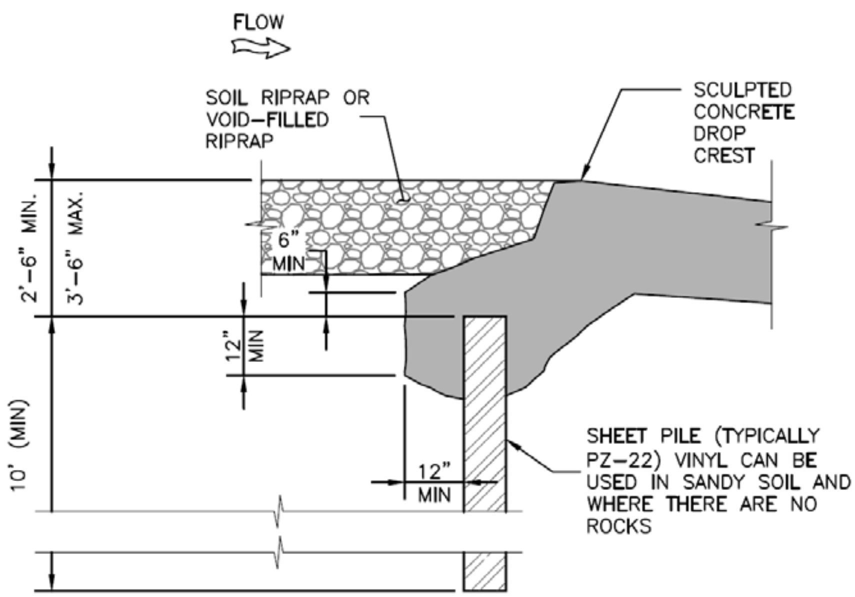
In addition to seepage uplift pressure, the designer should also evaluate the following forces on a drop structure:

- Shear Stress
- Buoyant Weight of Structure
- Impact, Drag and Hydrodynamic Lift Forces
- Turning Force
- Friction
- Frost Heave
- Dynamic Pressure Fluctuations

See Appendix A for additional discussion regarding drop structure force analysis.



SHEET PILE CUTOFF CONNECTION
(GSB)



SHEET PILE CUTOFF CONNECTION
(SC)

Figure 9-7. Sheet pile cutoff wall upstream of drop structure

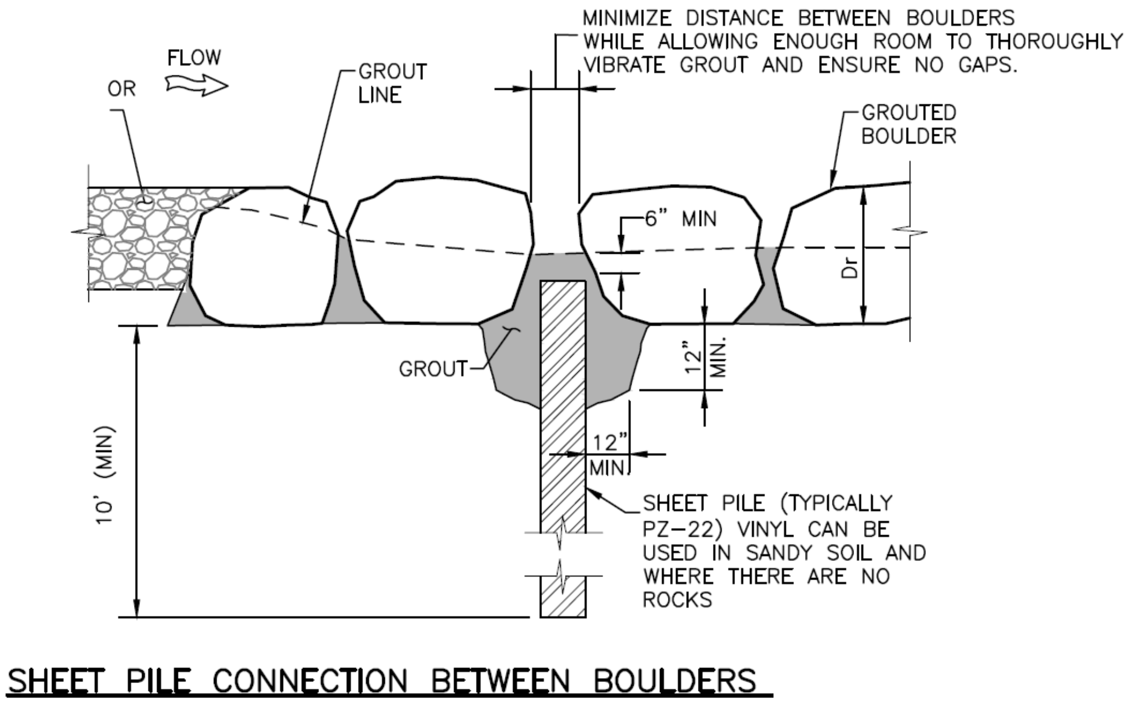


Figure 9-8. Sheet pile cutoff wall connections between boulders

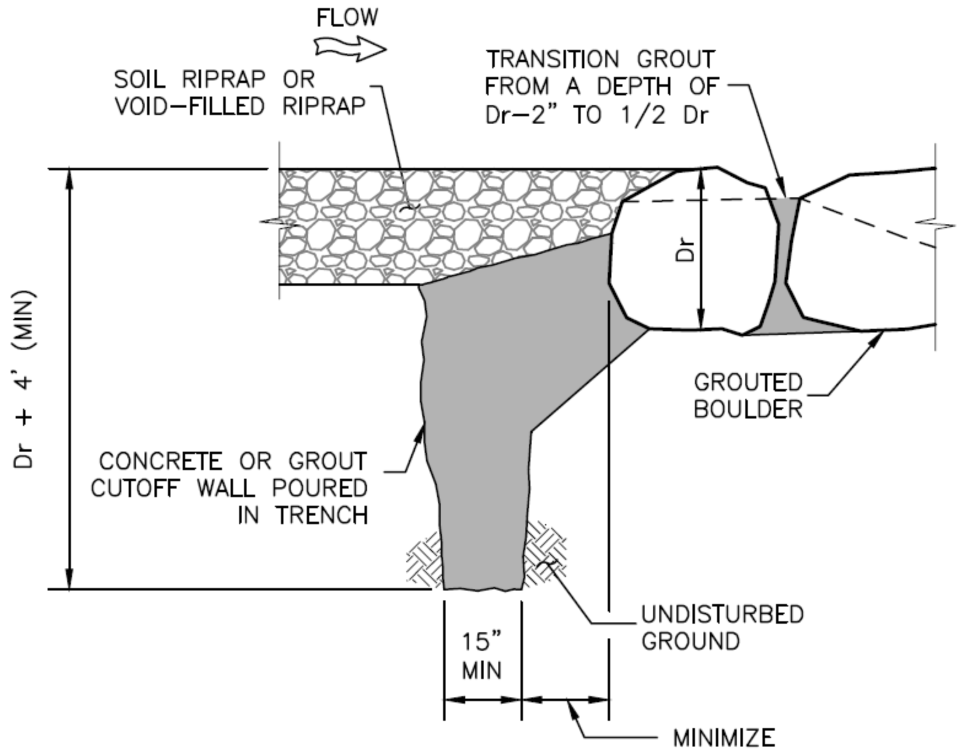


Figure 9-9. Concrete or grout cutoff wall upstream of drop structure

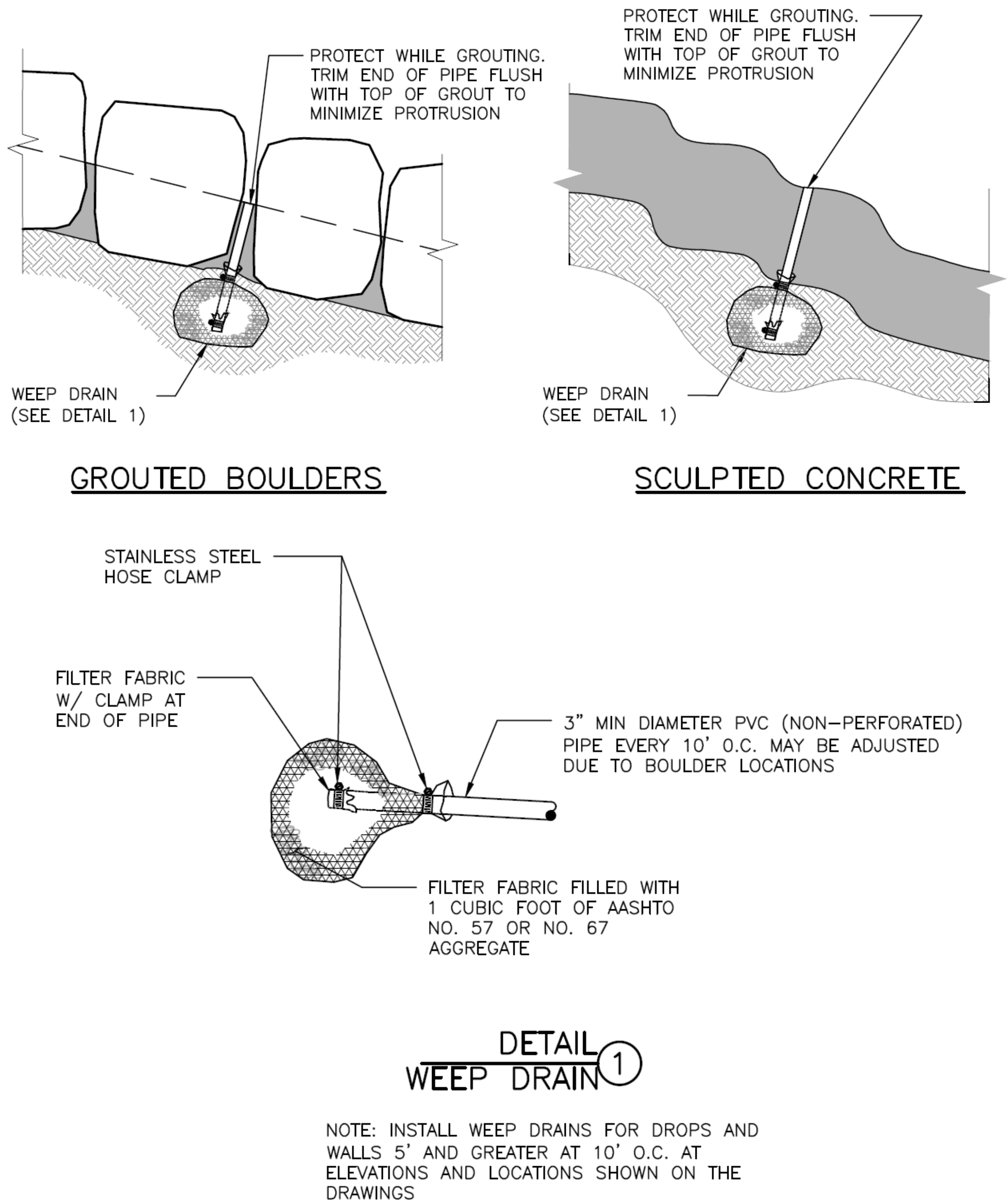


Figure 9-10. Weep drains

2.6 Grouted Stepped Boulder Drop Structures

2.6.1 Description

Grouted stepped boulder (GSB) drop structures have gained popularity in the UDFCD region due to close proximity to high-quality rock sources, design aesthetics, and successful applications. The quality of rock used and proper grouting procedure are very important to the structural integrity.

To improve appearance, cover the grouted boulders above the low-flow section and on the overbanks with local topsoil and revegetated. This material has potential to wash out but when able to become vegetated, has a more attractive and natural appearance.

2.6.2 Structure Complexity

An enlarged plan view of the structure will be necessary for all projects. The amount of detail shown on that plan view will vary depending on the structure complexity, which should be determined early in the design phase.

Photograph 9-5. Example of stepped downstream face for a grouted boulder drop structure. Note dissipation of energy at each step for low flows.

Sample plans for GSB drop structures are provided in this chapter and are referred to as either “basic” or “complex”. A basic structure generally has more of a linear shape with little variation in the step widths and heights. A complex structure will be non-linear with more variation, which may result in a need for more details and cross sections. It is imperative that adequate detail be provided for a complex structure to be constructed as intended.

Figures 9-11 through 9-13 illustrate the general configuration of a GSB drop structure. These figures include plan view, profile, and cross sections at key locations along the drop structure. Figure 9-14 provides an example configuration for a complex GSB drop structure, including a plan view and profile. These figures also serve as an example of the recommended level of detail for construction drawings.

2.6.3 Design Criteria

Hydraulic analysis and design of GSB drop structures should be according to Section 2.2 (simplified design procedures) or Section 2.3 (detailed hydraulic analysis), as appropriate. In addition, the following guidance also applies to structures constructed of grouted boulders.

Boulder Sizing
Boulder sizing for GSB drop structures constructed using the simplified method can be determined using Figure 9-1. For drop structures that do not meet the criteria for the simplified design method, the following procedure should be used to determine boulder size.

1. If the vertical distance from the drop toe to the drop crest is less than or equal to six feet, determine the critical velocity for the design flow in both the low-flow channel and the overbanks. This velocity occurs just upstream of the drop crest. For drop structures up to six feet in height, gradually varied flow acceleration is considered negligible. If the vertical distance from the drop toe to the drop crest is greater than six feet, determine the actual velocity at the drop toe using S2 curve drawdown calculations for the design flow in both the low-flow channel and the overbanks. This can be done using either the standard step or the direct step method. If a detailed hydraulic analysis has been completed using HEC-RAS (see Section 2.3), then the actual velocity is provided in the HEC-RAS output and the critical velocity can be taken from the section just upstream of the drop structure.

2. Calculate rock-sizing parameter, R_p (dimensionless), for both segments of the cross section (overbanks and in the low-flow channel):

$$R_p = \frac{VS^{0.17}}{(S_s - 1)^{0.66}} \qquad \text{Equation 9-7}$$

Where:

V = critical velocity, V_c (for drop structure heights up to six feet) or drawdown velocity at the toe of the drop (for drop height exceeding six feet)

S = slope along the face of the drop (ft/ft)

S_s = specific gravity of the rock (Assume 2.55 unless the quarry certifies a higher value.)

Note that for drop heights exceeding six feet, Equation 9-7 becomes iterative, since Manning's roughness coefficient is a function of the boulder size, from Equation 9-1 or 9-2.

3. Select minimum boulder sizes for the cross-section segments within and outside the low-flow channel cross-section from Table 9-4. If the boulder sizes for the low-flow channel and the overbank segments differ, UDFCD recommends using only the larger sized boulders throughout the entire structure. Mistakes during construction are more common when specifying multiple rock sizes within the same structure.

Table 9-4. Boulder sizes for various rock sizing parameters

Rock Sizing Parameter, R_p	Grouted Boulders [1] Boulder Classification [2]
Less than 5.00	B24
5.00 to 5.59	B24
5.60 to 6.99	B36
7.00 to 8.00	B48

[1] Grouted to no less than $^1/_3$ the height (+1"/- 0"), no more than $^1/_2$ (+0"/- 1") of boulder height.
[2] See *Open Channels* chapter.

Grout
Grout all boulders to a depth of one-half their height through the approach, sloping face, and basin areas. Grout should extend near full depth of the rock at the upstream crest and around the perimeter of the structure where it is adjoining the earth in order to provide stability of the approach channel. See Figure 9-15 for grout placement and material specifications.

Edge Wall
Construct a wall that extends roughly 3 feet below the top surface of the structure around the entire perimeter of the GSB drop structure. See Figure 9-22 for an edge wall detail. An edge wall is especially necessary for structures designed to convey less than the 100-year flow but is also beneficial for structures that do span the 100-year flow. In addition, use buried riprap around the perimeter of the structure when this is the case. The transition between soil and the grouted boulders can become a problem if not properly addressed during design and construction. Ensure compaction around the perimeter of the structure and grade this area higher than the structure to promote sheet flow onto the structure.

Additional Design Guidance
Grouted boulders must cover the crest and cutoff and extend downstream through the stilling basin (when applicable), or through the embedded toe of the drop structure when a stilling basin is not included. Place boulders to create a stepped appearance, which helps to increase roughness. Additional information regarding riprap and boulders is in the *Open Channels* chapter.

2.6.4 Construction Guidance

Grouted boulder drop structures require significant construction oversight. During placement of the rock and construction in general, disturb the subgrade as little as possible to reduce the potential for piping under the structure. Good subgrade preparation, careful rock placement, and removal of loose materials will reduce potential piping. Do not place granular bedding (or subgrade fill using granular materials) between subgrade and the boulders. This can cause piping. Place boulders directly on undisturbed subgrade where possible. Where the design requires over excavation and/or fill or where wet or poor subgrade exists onsite, ensure proper density and compaction. See Division 31 specifications available at www.udfcd.org. When fill is required, it is best to fill and compact to a set elevation (or sloped surface) and then “carve” the surface as necessary to place boulders. See figure 9-15 for a placement detail.

Proper grout placement provides overall mass sufficient to offset uplift and reduces piping under the structure. The greatest risk lies with a “sugar-coated” grout job, where the grout does not penetrate the voids fully between the rock and the subgrade and leaves voids below the grout that act as a direct piping route for water, guaranteeing early failure. Ensure grout thickness set at one-half the boulder height, but no more than two-thirds the boulder height (except at the crest and around the perimeter of the structure where the grout should be near grade). Limiting grout thickness also improves the overall appearance of the grouted boulder structure.

Problems with rock density, durability and hardness are of concern and can vary widely for different locations. Inspect the rock at regular intervals to meet minimum physical dimensions, strengths, durability and weights as defined in the specifications.

As stated earlier, it is important to compact the soil around the perimeter of the structure and leave it slightly higher than the structure to promote sheet flow onto the structure. If the soil settles, surface erosion along the edge of the concrete and ultimately structure piping may occur.

Grout used for GSB drop structures shall receive cold or hot weather protection in accordance with the UDFCD construction specifications (see www.udfcd.org).

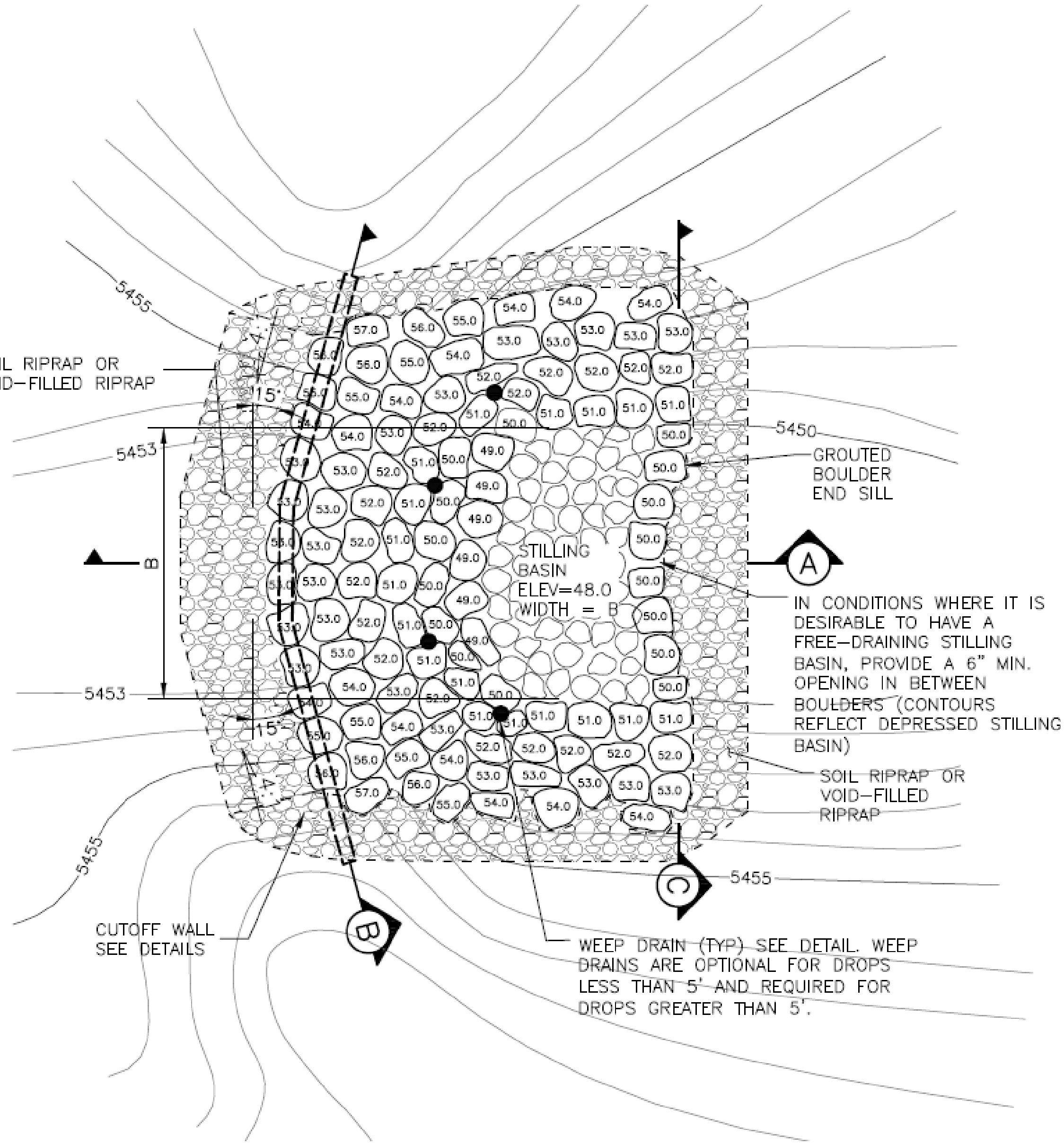


Figure 9-11. Example plan view of basic grouted stepped boulder drop structure

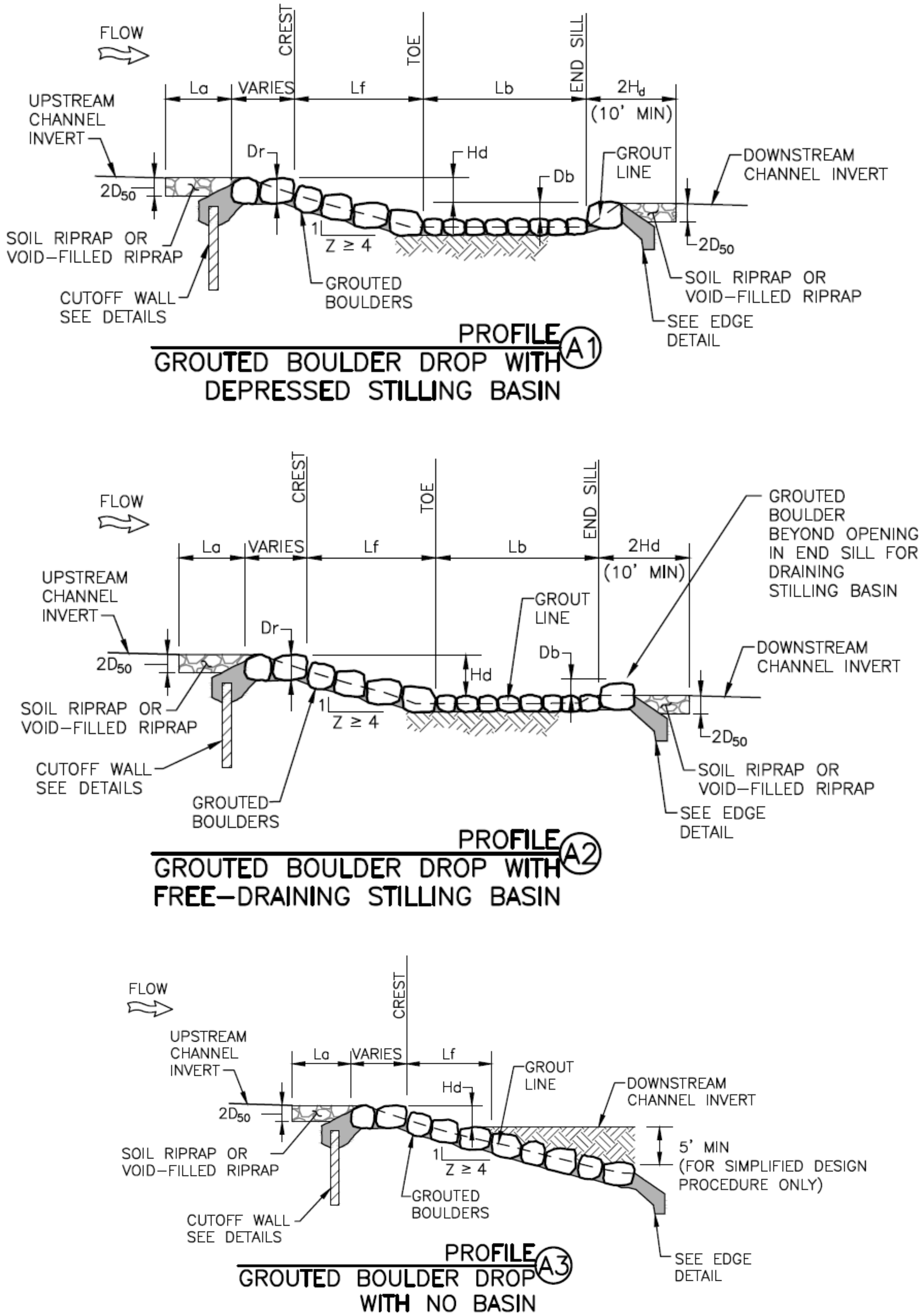
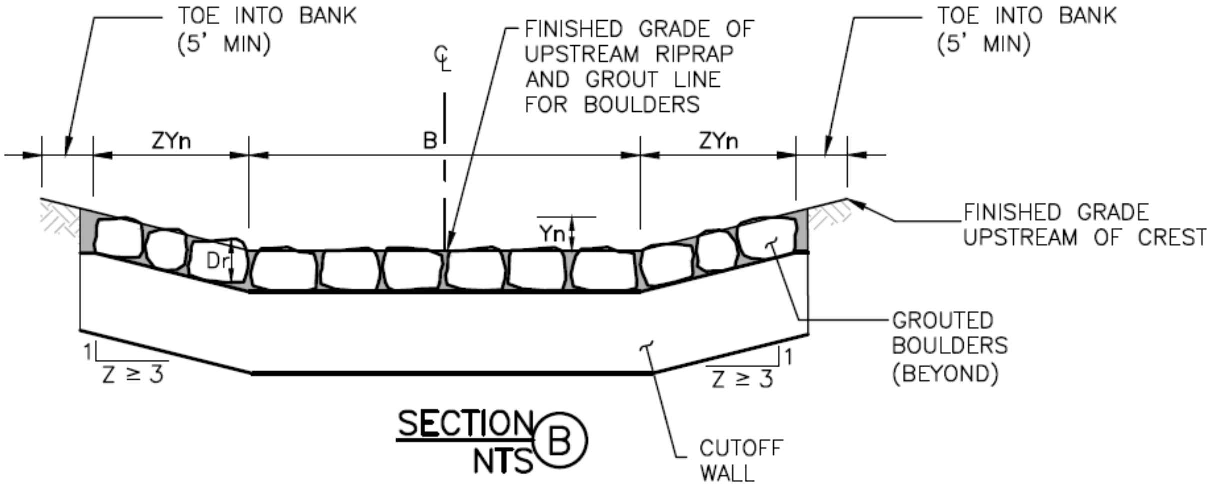


Figure 9-12. Cross sections of basic grouted stepped boulder drop structure



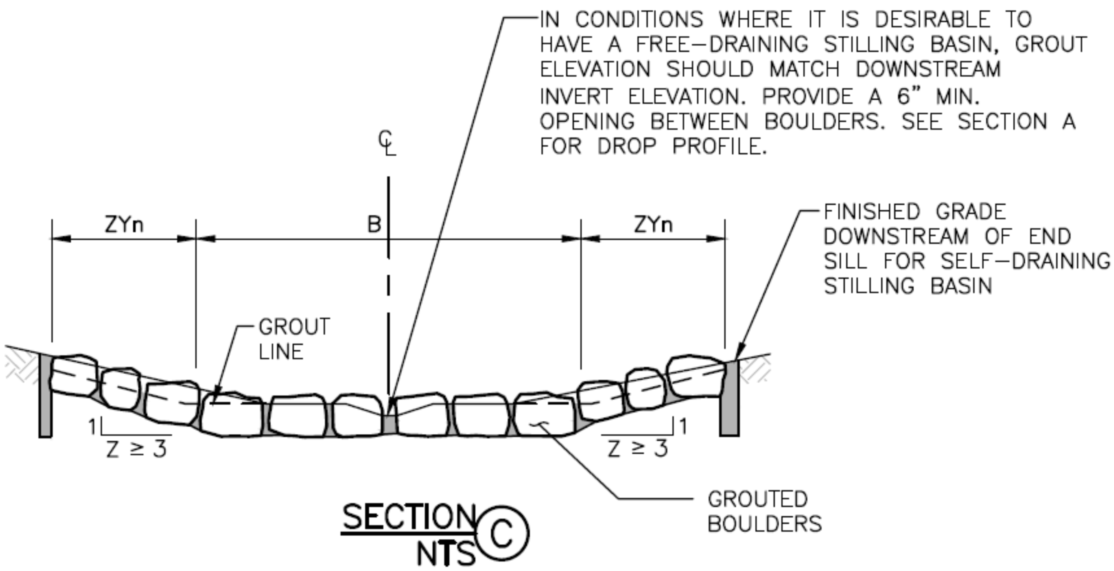


Figure 9-13. Cross sections of basic grouted stepped boulder drop structure

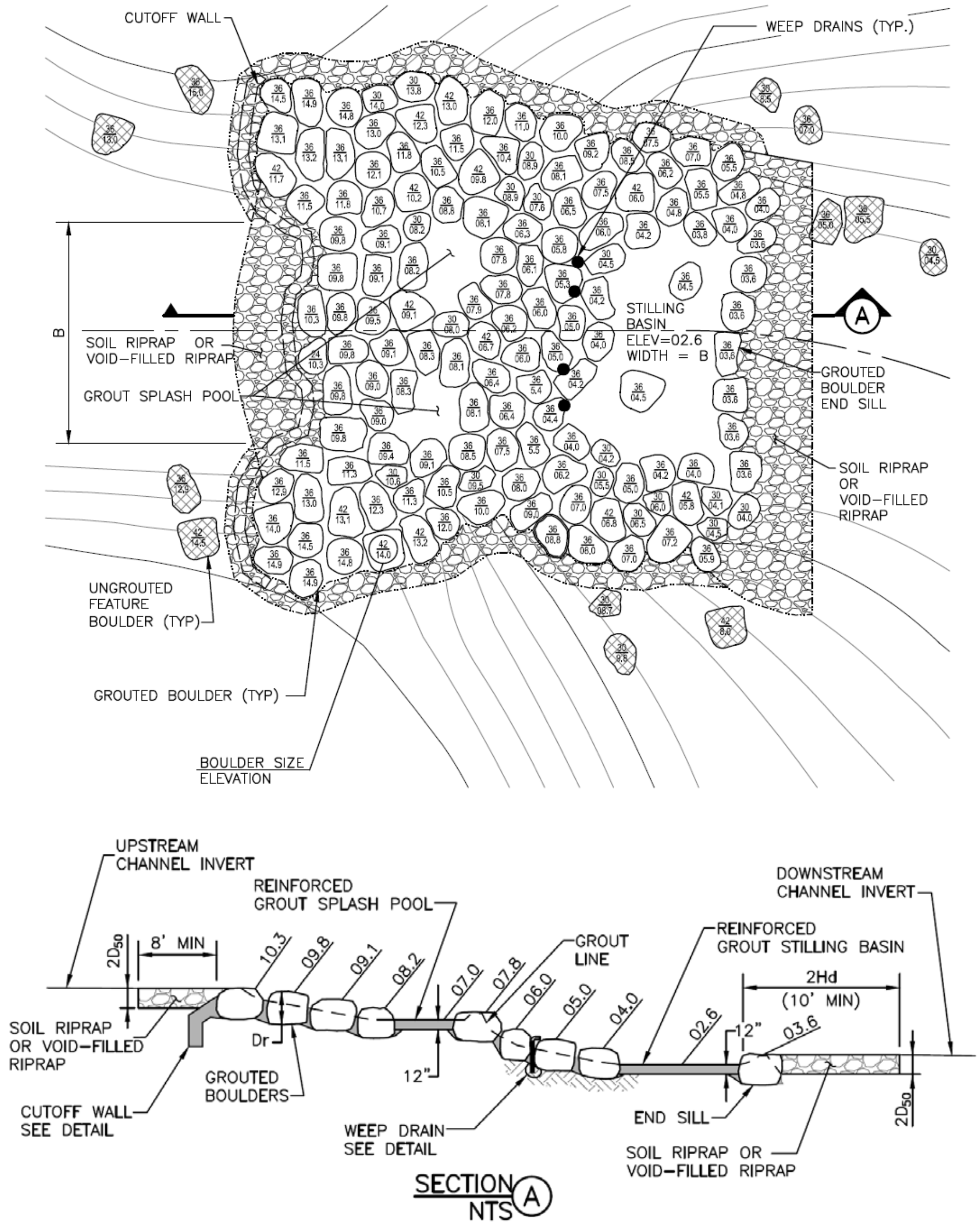
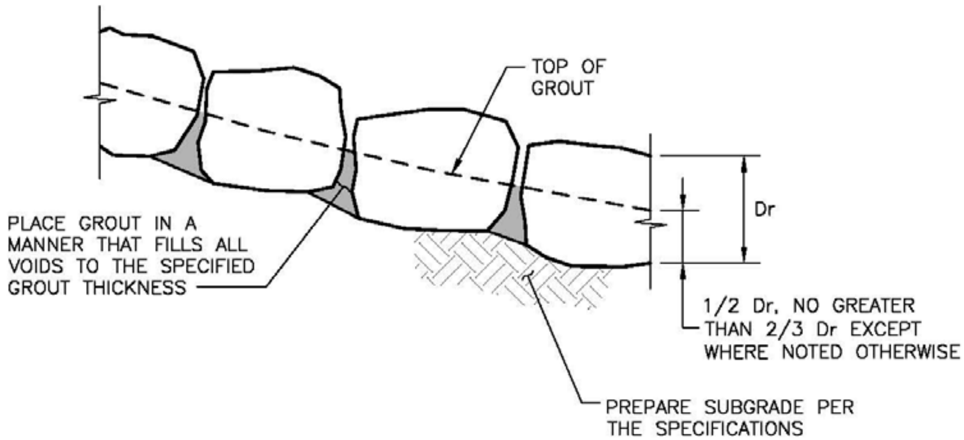


Figure 9-14. Example of complex grouted stepped boulder drop structure



BOULDER PLACEMENT NOTES:

1. PLACE BOULDERS WITH THE REQUIRED BOULDER HEIGHT VERTICAL. PLACE BOULDERS AS TIGHTLY TOGETHER AS POSSIBLE (WITHOUT TOUCHING) WHILE PROVIDING ENOUGH ROOM BETWEEN THEM TO THOROUGHLY VIBRATE THE GROUT AND TO ENSURE NO GAPS IN THE GROUT. THE SMALL DIMENSION OF A 2X4 CAN BE USED AS A GUIDE TO CHECK MINIMUM SPACING.
2. BEFORE GROUTING, CLEAN ALL DIRT AND MATERIAL FROM ROCK THAT COULD PREVENT THE GROUT FROM BINDING TO THE ROCK. KEEP BOULDERS FROM TOUCHING. AVOID SLIDING BOULDERS AGAINST SUBGRADE TO PROPERLY POSITION.

MATERIAL SPECIFICATIONS:

1. ALL GROUT SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH EQUAL TO 3200 PSI.
2. ONE CUBIC YARD OF GROUT SHALL HAVE A MINIMUM OF SIX (6) SACKS OF TYPE II PORTLAND CEMENT.
3. A MAXIMUM OF 25% TYPE F FLY ASH MAY BE SUBSTITUTED FOR THE PORTLAND CEMENT.
4. THE AGGREGATE SHALL BE COMPRISED OF 70% NATURAL SAND (FINES) AND 30% ⅜-INCH ROCK (COARSE).
5. THE GROUT SLUMP SHALL BE BETWEEN 4-INCHES TO 6-INCHES.
6. AIR ENTRAINMENT SHALL BE BETWEEN 5.5% AND 7.5%.
7. TO CONTROL SHRINKAGE AND CRACKING, 1.5 POUNDS OF FIBERMESH, OR EQUIVALENT, SHALL BE USED PER CUBIC YARD OF GROUT.
8. COLOR ADDITIVE IN REQUIRED AMOUNTS SHALL BE USED WHEN SO SPECIFIED BY CONTRACT.

GROUT PLACEMENT SPECIFICATIONS:

1. SPECIAL PROCEDURES SHALL BE REQUIRED FOR GROUT PLACEMENT WHEN THE AIR TEMPERATURES ARE LESS THAN 40°F OR GREATER THAN 90°F. CONTRACTOR SHALL OBTAIN PRIOR APPROVAL FROM THE DESIGN ENGINEER OF THE PROCEDURES TO BE USED FOR PROTECTING THE GROUT.
2. GROUT SHALL BE DELIVERED BY MEANS OF A LOW PRESSURE (LESS THAN 10 PSI) GROUT PUMP USING A 2-INCH DIAMETER (MAXIMUM) NOZZLE.
3. FULL DEPTH PENETRATION OF THE GROUT INTO THE BOULDER VOIDS SHALL BE ACHIEVED BY INJECTING GROUT STARTING WITH THE NOZZLE NEAR THE BOTTOM AND RAISING IT AS THE GROUT FILLS, WHILE VIBRATING GROUT INTO PLACE USING A PENCIL VIBRATOR.
4. ALL GROUT BETWEEN BOULDERS SHALL BE TREATED WITH A BROOM FINISH.
5. AFTER GROUT PLACEMENT, EXPOSED BOULDER FACES SHALL BE CLEANED AND FREE OF GROUT.
6. ALL FINISHED GROUT SURFACES SHALL BE SPRAYED WITH A CLEAR LIQUID MEMBRANE CURING COMPOUND AS SPECIFIED IN ASTM C309.

Figure 9-15. Grouted boulder placement detail

2.7 Sculpted Concrete Drop Structure

Due to increased construction complexity associated with large vertical drops the scope of this section is limited to sculpted concrete drops six feet or less.

Photograph 9-6. Exposed sedimentary rock.

2.7.1 Description

Concrete faux rock is simply concrete that is sculpted, carved, textured, and colored to emulate real rock. In the past, sculpted concrete has been successfully used for retaining wall type structures and stream grade control structures. It can be an aesthetic alternative to grouted boulders in locations where natural sedimentary rock might be expected.

Geology in the UDFCD region east of the foothills primarily consists of sedimentary rock, of which there are five common types including sandstone, shale, conglomerate, limestone, and claystone. Claystone can be found in eroded streams where less dense soils have been washed away. Claystone is similar to sandstone; however, it is composed of finer particles. These layers of sedimentary rock become exposed due to uplift and erosion.

Photograph 9-7. An eroded channel with exposed claystone layers.

When considering the design for a new sculpted concrete structure, existing exposed sedimentary rock in the vicinity of the project should be used for guidance. Section 2.7.4 provides additional guidance for determining the appropriate finish for sculpted concrete.

2.7.2 Structure Complexity

Early in the design, determine what the expectations are regarding the appearance of the structure. An enlarged plan view of the structure will be necessary for all projects. The amount of detail shown on that plan view will vary depending on the complexity of the design.

Note that an overly complex design does not always result in a more aesthetically pleasing structure. Many quality structures have been constructed using very basic design plans and details. Simplifying the design can reduce confusion and misinterpretation during construction, and also matches the skill level of a greater number of potential bidding contractors.

For the purpose of presenting criteria for sculpted concrete drop structures, this manual refers to sculpted concrete structures as either “basic” or “complex”. Structure complexity is generally tied to the following three items.

1. **Overall structure footprint:** Non-linear shaped structures with varied edge delineations can be more attractive but require more detailing.

2. **Structure step widths and height:** Varied step widths and heights can improve the appearance of a structure but also adds construction complexity. Step widths can easily be delineated using boundary lines. Varying step heights requires finish grade point elevations to be added to the plan. The quantity of point elevations largely depends on the amount of desired elevation change.

3. **Sloped steps/flat steps:** Flat steps can be constructed based on a single contour or point elevation. If the surface is sloped, slope arrows and a series of point elevations to identify portions of the sloped top surface are beneficial.

Photograph 9-8. The first sculpted concrete structure in the UDFCD region was along Grange Hall Creek in Northglenn, Colorado. The shape and color was chosen to blend into the existing landscape which consisted of native grasslands.

If the design includes any of the three items discussed above (non-linear shape, varied step widths, or sloped steps), consider the proposed structure to be complex and prepare a more detailed plan view of the structure. Figures 9-19 and 9-20 present an example of such a plan. Note the additional finished grade point elevations and slope arrows compared to Figures 9-16 through 9-18, which provide details for a basic structure. Also included with the complex structure plan is a legend and notes with additional information regarding vegetation beds within the structure and surface treatment. This manual provides further discussion regarding these items later in the chapter, but it is important to note that these elements can also be incorporated into a simple structure without adding complexity. None of the figures in this section are intended as typical details but are provided as an example of the level of detail recommended for this type of design.

Photograph 9-9. A sculpted concrete drop structure along Marcy Gulch in Highlands Ranch, CO represents a basic structure design.

2.7.3 Design Criteria

Hydraulic analysis and design of SC drop structures should be according to Section 2.2 (simplified design guidance) or Section 2.3 (detailed hydraulic analysis), as appropriate. The following also apply to structures constructed of sculpted concrete.

Reinforcing Steel

Steel reinforcement is recommended in order to control temperature and shrinkage cracks. It is the responsibility of the designer to verify all structural components of SC drop structures during the design phase. Figure 9-21 provides guidance for rebar placement for SC structures with a flat subgrade and on an undulated subgrade. Larger walled sections within a given structure may require additional evaluation and design.

Photograph 9-10. This drop structure located along Oak Hills Tributary represents a complex design example.

Edge Wall

Provide an edge wall that extends roughly 3 feet below the top surface of the structure around the entire perimeter of the SC drop structure. See Figure 9-22 for an edge wall detail. An edge wall is especially important for structures designed to convey less than the 100-year flow but is also beneficial for structures that do span the 100-year flow. The transition between soil and the sculpted concrete can become a problem if not properly addressed during design and construction. During construction ensure compaction of the soil around the perimeter of the structure and grade the area to sheet flow onto the structure. If the soil settles, surface erosion along the edge of the concrete and ultimately structure piping may occur. In addition to the edge wall, install buried soil riprap around the perimeter of the structure when the drop structure does not span the 100-year floodplain. This reduces potential erosion.

Concrete Thickness

The concrete should be a minimum of 10 inches thick. As with the steel reinforcement, it is the design engineer's responsibility to complete a structural analysis to determine adequate concrete thickness for structure stability. It is preferred that the subgrade be excavated to closely mirror the finished structure surface, which will allow for the placement of concrete with a consistent thickness. In isolated locations, it may be necessary to thicken the concrete to meet design grades. Ideally, the thickened areas should not exceed 2 feet. Avoid multiple pours of separate layers of concrete over the majority of the structure.

Concrete versus Shotcrete

Either concrete mix or shotcrete mix are suitable for construction of sculpted concrete drop structures, however designers should be aware that there are advantages and disadvantages for each (See Table 9-5).

Table 9-5. Comparison of concrete and shotcrete

	Concrete	**Shotcrete**
	▪ Handling and placement can be performed by a large number of general contractors. ▪ Can be rapidly placed with the use of a concrete pump truck, roughly twice as fast as shotcrete. Construction of very large structures as a single pour in 1 day is possible.	▪ Generally has greater compressive strength and is more impervious than concrete. ▪ Can be placed in a uniform and consistent manner. Vibrating the shotcrete is not required. ▪ Can be placed to create vertical faces and overhangs.
		▪ Shotcrete placement is considered specialty type work and is performed by a limited number of contractors. ▪ Shotcrete placement is slow in comparison to concrete placement. ▪ Shotcrete structures may be more expensive than concrete.

2.7.4 Decorative Elements (Finishing)

Sculpted concrete finishing refers to modifications intended for visual enhancement. The contractor plays an important role in the finishing process and making the structure look attractive. When contractor selection is limited (e.g., the project is open bid), designers must provide adequate finishing guidance and recommendations on the construction plans.

Finishing is an all-encompassing word that can include:

- Troweling, sculpting, and carving
- Stamping
- Top dressing with sand, gravel, cobbles, or other materials
- Vegetation seams, pockets, or beds
- Coloring/Staining

Depending on the design objectives, a project may include a couple or all of these techniques. This section provides an overview and photograph illustrations of the techniques listed above.

Photo 9-11. Rock formation with horizontal weathering and surface erosion. Random pockets of vegetation create significant interest. Overall color is gray-white with lichen and other organic surface growth.

Photo 9-12. Generally horizontal layered rock formation. Surface texture varies with small pockets of vegetation. Overall color is brown with dark staining in the cracks. Close up view reveals significant granular material bedded into the surface.

Examples in Nature

An abundance of natural formations exist throughout the UDFCD region. Rock formations can vary significantly even if separated by only a short distance. Differences in color, surface roughness, bed angles or strata line angles, and vegetation are apparent. A photographic log of different formations can be a valuable resource when designing, constructing, and finishing sculpted concrete structures. Photographs 9-11, 9-12, and 9-13 show three different rock formations found in the UDFCD region.

Troweling, Sculpting, and Carving

Troweling, sculpting, and carving are all terms for the same general action. The contractor typically uses a concrete trowel, float, or other tool to shape the concrete and then carve lines, crevices, or cracks that emulate natural rock features. This requires a contractor with sculpted concrete experience and skill.

Some of the difficulties that may arise during this process include the following:

- Typically, several concrete finishers will work on the same structure producing several different styles of finish treatment within the same structure. Finishers should work together using the same general techniques and producing similar and uniform results.

- Proper sense of scale. Finishers perform the work from an arm's reach. At this close range, the finisher may over-carve the material giving an appearance of a busy and unnatural looking structure when viewed from a distance.

- Style selection. There are many different styles of sculpting and carving. The owner, engineer, and finishers may all have a different vision.

Photographs can be helpful in developing consensus between owner, engineer and contractor. Be as specific as possible with the contractor regarding all of the structure attributes when reviewing photographs. It may be preferred to replicate some characteristics in the photographs and leave out some of the others. Construction of a test panel of sculpted concrete before performing the final structures can also be beneficial. A test panel that is approximately 10 feet by 10 feet is typically adequate in order to practice overall form as well as some of the detailing. If the first panel does not achieve the objectives, construct a second. This is a better alternative to practicing and developing techniques on the structures.

For proper sense of scale, periodically take time to step away from the structure and look at it from a more typical viewing distance. This will allow the finishers to see the structure as a whole.

Photo 9-13. Severely uplifted rock formation with layers standing nearly vertical. Surface texture varies within the layers but is mostly smooth. Vegetation appears to grow out of the seams, not necessarily from pockets. Overall color is a light chalky tan.

Photograph 9-14. Subtle carving and shaping can often produce the desired results. Notice the single horizontal carving that runs through just one of the steps and is extended into the crest of the structure. Horizontal carvings on all of the steps would be excessive and distract from the overall aesthetics.

Stamping

Stamping adds surface texture and requires less skill compared to sculpting and carving. Stamping is most often performed using texture mats or skins, which are rubber molds made from real rock surfaces. When pressing the texture mats into wet concrete, the concrete takes the textured surface of the mat. Texture mats are available with a variety of texture styles and relief. Use a liquid or powder release agent to keep the concrete from sticking to the mats. While texture mats are specifically for texturing concrete, a finisher could use an unlimited amount of other materials to create unique or desired finishes.

Photograph 9-15. Texturing a sculpted concrete drop with a rubber skin.

Top Dressing with Sand, Gravel, Cobbles or Other Materials

Top dressing a structure with sand, gravel, or cobbles adds texture to the surface of sculpted concrete. While some natural rock formations have a very smooth surface finish, many contain grains of sand and pebbles cemented together. This is typical of the sandstone and conglomerate types of sedimentary rock common in the UDFCD region. It is important to press the material into the sculpted concrete shortly after carving and before the concrete sets. Additionally, the material should be washed clean and free of debris to promote bonding to the concrete. The majority of the material will remain in place over time, but with freeze-thaw, some of it will dislodge. Wetting the material immediately before placement can help reduce the percentage of material that dislodges over time.

Photograph 9-16. Completed sculpted concrete drop structure with loose sand, gravel and cobble embedded into the surface.

Photograph 9-17. Small vegetation pockets can be formed using PVC, lumber, or other items. Removed these items shortly before or after the concrete cures. If done after the concrete cures, coat the items with a lubricant to facilitate removal.

Vegetation Seams, Pockets or Beds

A characteristic of natural rock formations is that grasses, shrubs, and trees can be found living in cracks within the rock. Pairing rock and vegetation helps make the structure appear natural. However, it can be difficult to establish vegetation within a concrete structure. Depending on the stream, dryland vegetation beds and seams should stay above certain minimum flood elevations as they won't tolerate frequent flooding. If placed too low and vegetation does not become established, this leaves a vulnerable area in the structure for piping. A slightly thickened edge of sculpted concrete around the seam or bed is typically adequate. In some cases where flow overtopping is a more significant concern, toewalls around the perimeter of the bed or seam may be necessary along with filter material in the bottom of the bed to guard against piping. Filter material should not be installed along the entire structure, but rather at the specific vegetation bed or seam to reduce the likelihood of piping under the structure, especially at the crest. Geotextile can be used for this purpose or a graded filter system could be constructed.

Plants do not necessarily need a large bed to be sustainable. Consult with an ecologist or other qualified specialist for both proper plant selection and bed construction. For example, a plant species that thrives on the north facing side of a sculpted concrete structure may not be able to live on the south facing side of the same structure where sunlight and heat are more intense. Consider the daily amount of sunlight anticipated, reflective and absorptive heat of the sculpted concrete, and water requirements.

The incorporation of wetland vegetation planting pockets should also be considered and have a higher success rate as conditions for wetland vegetation are favorable within a depressed concrete lined portion of the structure as long as the stream base flow is routed through the basin. These planting pockets should be located outside of the primary energy dissipation area of the structure. This will allow the plants to develop a healthy root structure and hold the plants in place during large flows.

Photograph 9-18. The surface of the sculpted concrete can be depressed to capture and direct rainwater to the vegetation.

Photograph 9-19. Grass growth in a vegetation pocket shortly after seeding.

Photograph 9-20. During the structure subgrade preparation, vegetation beds and pockets were delineated and soil was removed. After the sculpted concrete placement was complete, topsoil was placed in the beds and seeding and planting was performed.

Another method to add color to sculpted concrete is to apply a stain to the finished and cured concrete surface. There are several products available specifically made for concrete. Another acceptable method is to add water to exterior acrylic latex paint until it has the consistency of a stain. This allows for a wide range of available colors. Stains are typically applied by hand-held bottle sprayers, mechanical sprayers, and sponges.

Photograph 9-24. Stained concrete in foreground with natural rock in the background.

If a test panel has already been constructed, use this panel to practice to develop the desired color scheme. The process of staining includes layering multiple shades of color. Typically light colored stains are applied first followed by darker accent shades. Applying a light coat of watered down black stain as the final coat will create a weathered or aged surface appearance. As with any finishing technique, the experience level of the finisher plays a key role in the outcome.

Photograph 9-25. An example of a "skim coat" applied to the prepared subgrade.

2.7.5 Construction Guidance

For sculpted concrete drop structures, concrete is often placed in a few hours. It is therefore very important to plan the overall appearance of the structure well in advance of concrete placement. Coordinating details with the contractor should occur during subgrade preparation and tying of steel reinforcement. Construction guidance is provided below.

- **Subgrade Preparation.** The structure subgrade should be adequately dewatered prior to the commencement of excavation or fill. All fill material should be placed on a minimum 12-inch depth of stripped, scarified, moisture conditioned, and compacted subgrade. During excavation, it is recommended that the contractor cover the exposed subgrade with blanket to avoid excessive drying or erosion. If excessive drying does occur, surface wetting of the soil should be performed.

Photograph 9-26. Spray paint is used on the "skim coat" of a sculpted concrete structure prior to concrete placement to identify the locations for fracture lines and other features.

- **Skim Coat.** Keeping steel reinforcement clean can be a challenge. The use of blanketing can help. Another option is to apply a "skim coat" (also sometimes referred to as a "flash coat"). Skim coating consists of placing approximately 1 to 2 inches of shotcrete on the prepared subgrade. This alternative can be used with either concrete or shotcrete structures. Avoid the use of aggregate to stabilize or protect the subgrade. Skim coating will also protect the subgrade from weather and provide a clean and stable surface for placing and tying steel.

- **Concrete Placement.** Concrete placement is a quick process. In order to be properly prepared for a concrete pour, it is important to coordinate the desired finished structure appearance with the contractor. Example photographs of similar sedimentary rock can be used to help communicate the desired finish. A test panel or section is recommended when varied textures and finishing will be incorporated. Another successful approach is to spray paint fracture lines or mark locations on the subgrade where texturing or features are desired. It is imperative that the contractor have an adequate number of workers present to place the concrete, survey design grades, trowel and carve the concrete, as well as perform all other finishing details.

Cautions Associated with Sculpted Concrete Construction

1. Skill and experience on the part of the contractor helps produce an attractive structure.

2. Subgrade excavation/compaction and placement of reinforcing steel must conform to complex, irregular shapes and slopes within design tolerances.

3. Placing, shaping, and carving of concrete/shotcrete must take place within a narrow range of water content and a short window of time. This requires planning, favorable weather conditions, an adequately-sized crew, appropriate pace, and a high degree of organization on the contractor's part.

4. Care needs to be taken to avoid overworking concrete/shotcrete as vertical faces are shaped and trowelled; otherwise, cracking and sloughing can occur.

5. Inspection and adjustment of grades to meet the design intent must take place during placement of concrete/shotcrete.

6. Skill is required to shape, carve and stain the exposed surfaces of the sculpted concrete in an attractive manner that emulates natural rock formations.

7. Consider hot and cold weather conditions to ensure satisfactory finishing and curing of the concrete/shotcrete.

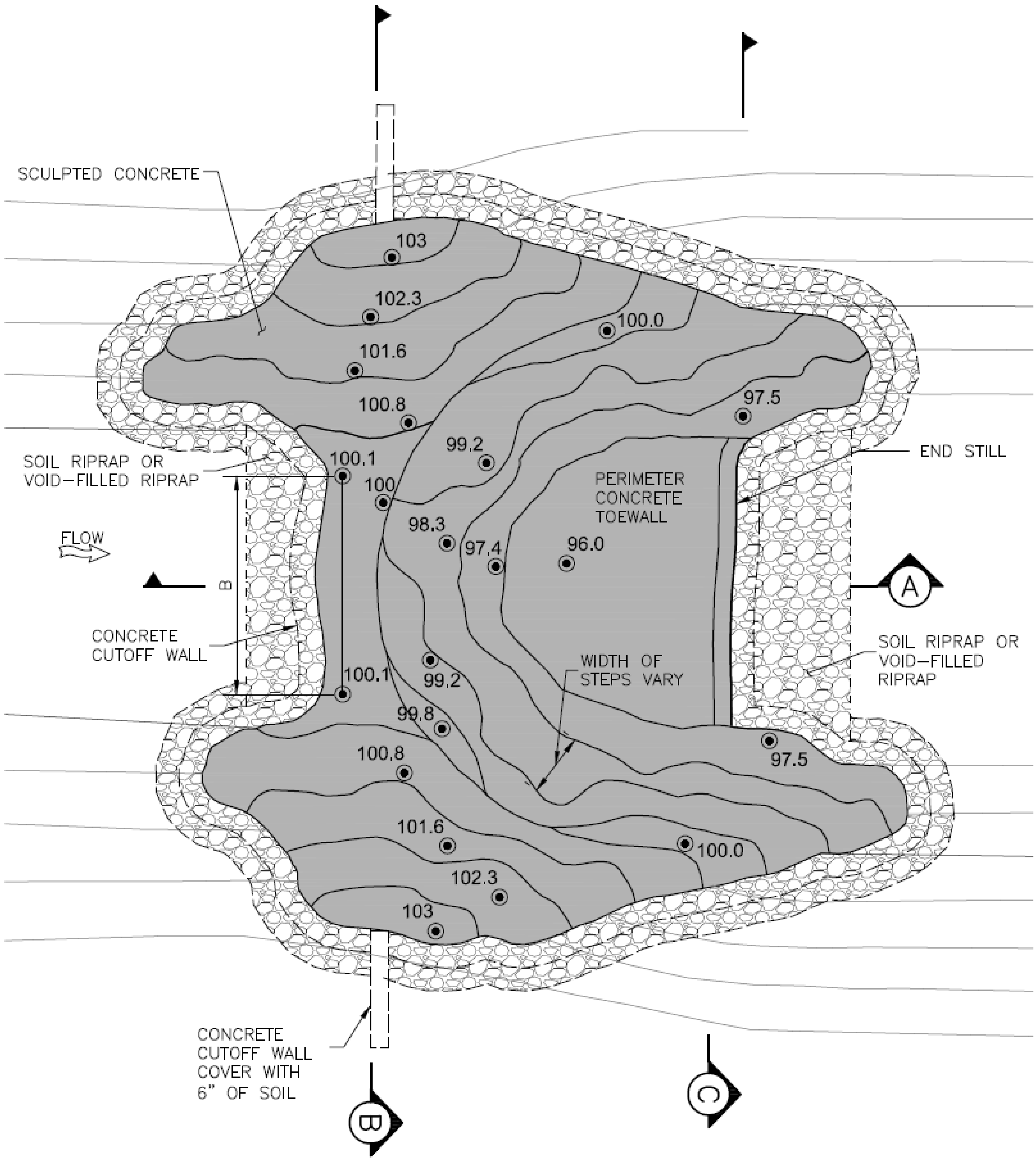


Figure 9-16. Example plan view of basic sculpted concrete drop structure

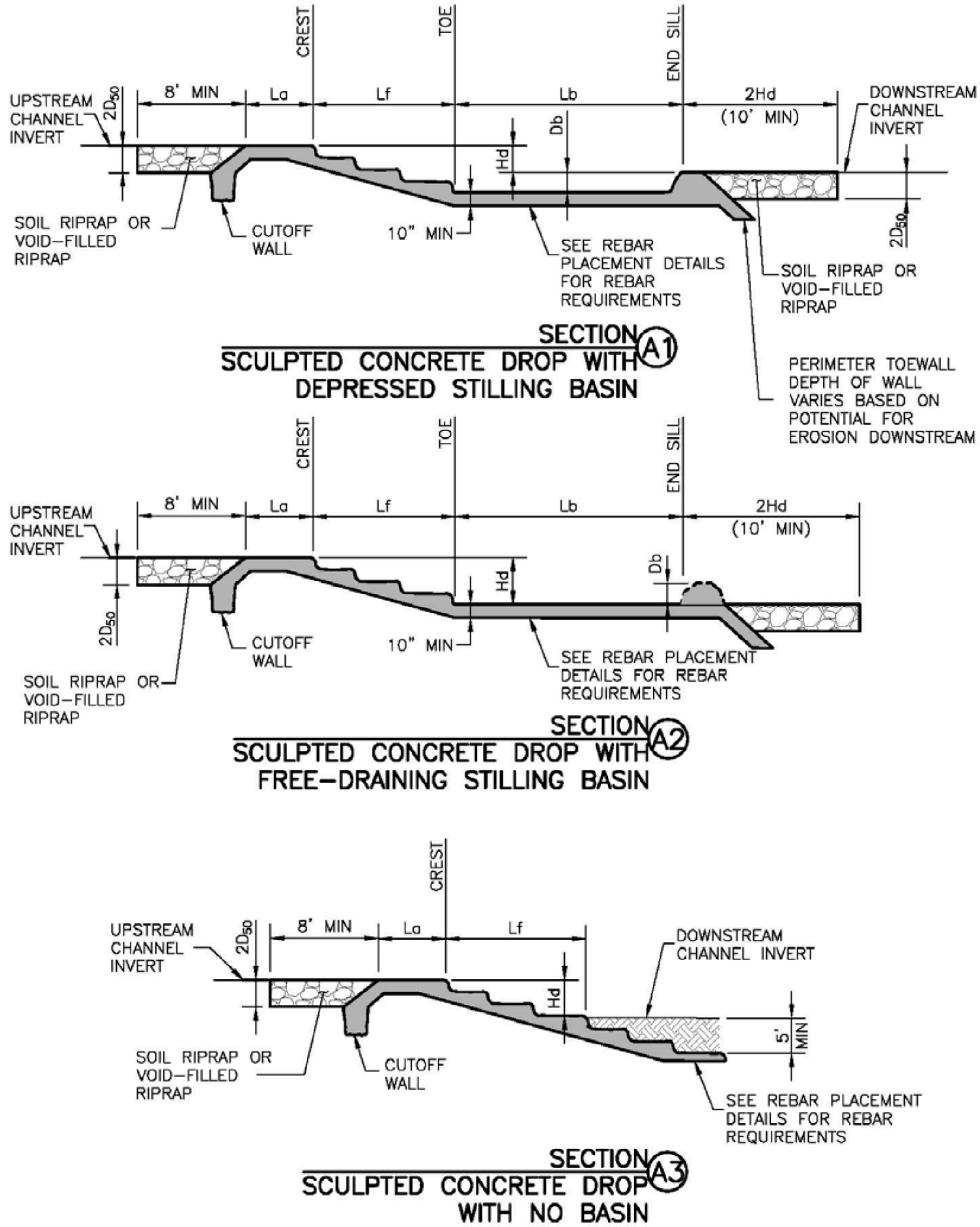


Figure 9-17. Example profiles of basic sculpted concrete drop structure

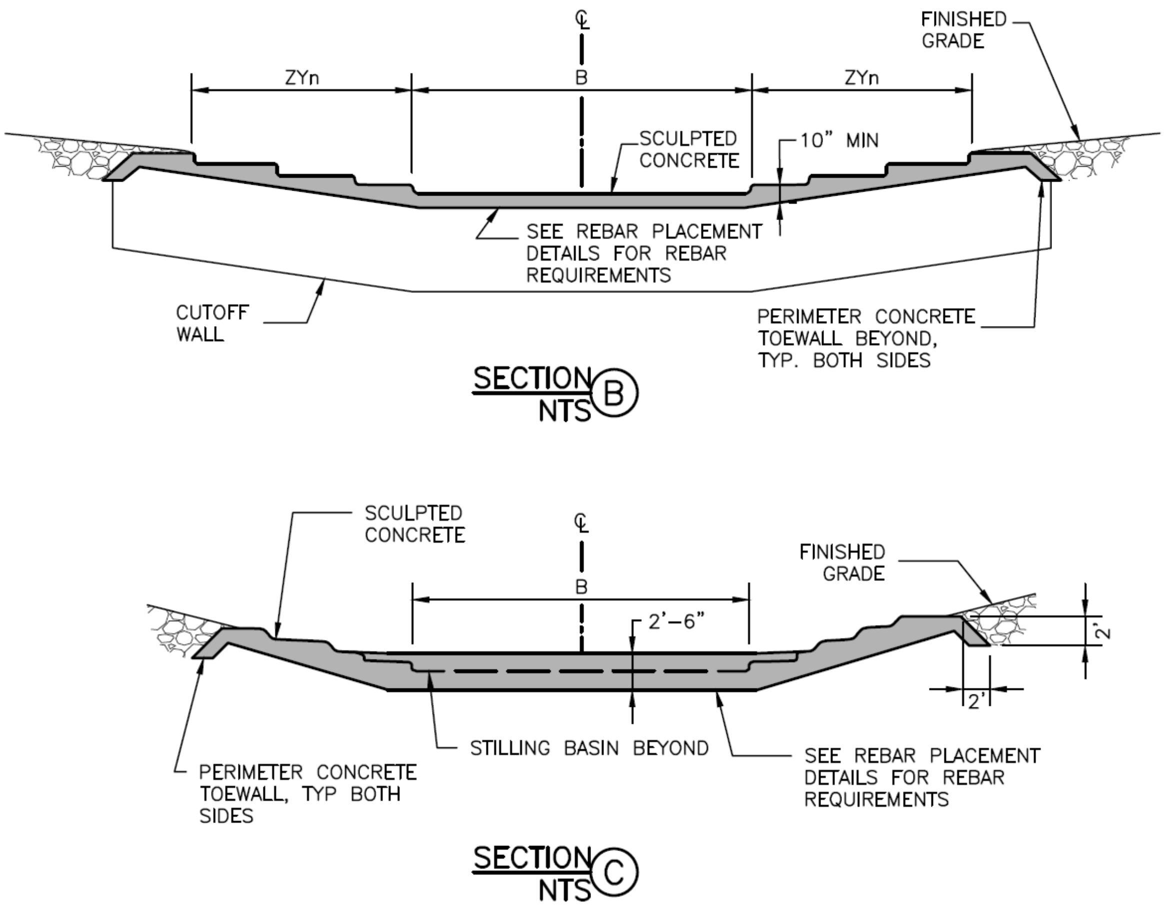


Figure 9-18. Example cross sections of basic sculpted concrete drop structure

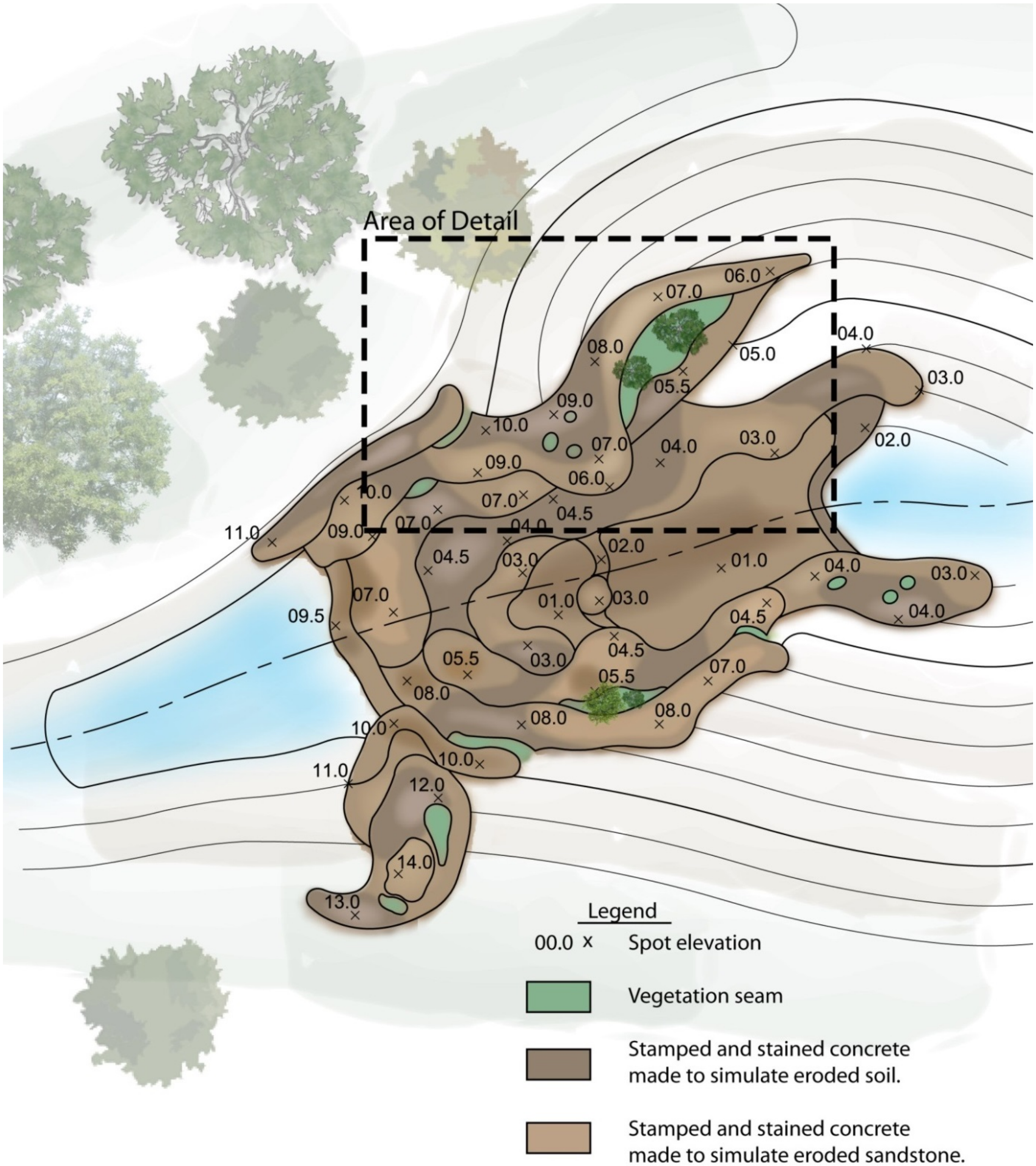


Figure 9-19. Example plan view of complex sculpted concrete drop structure

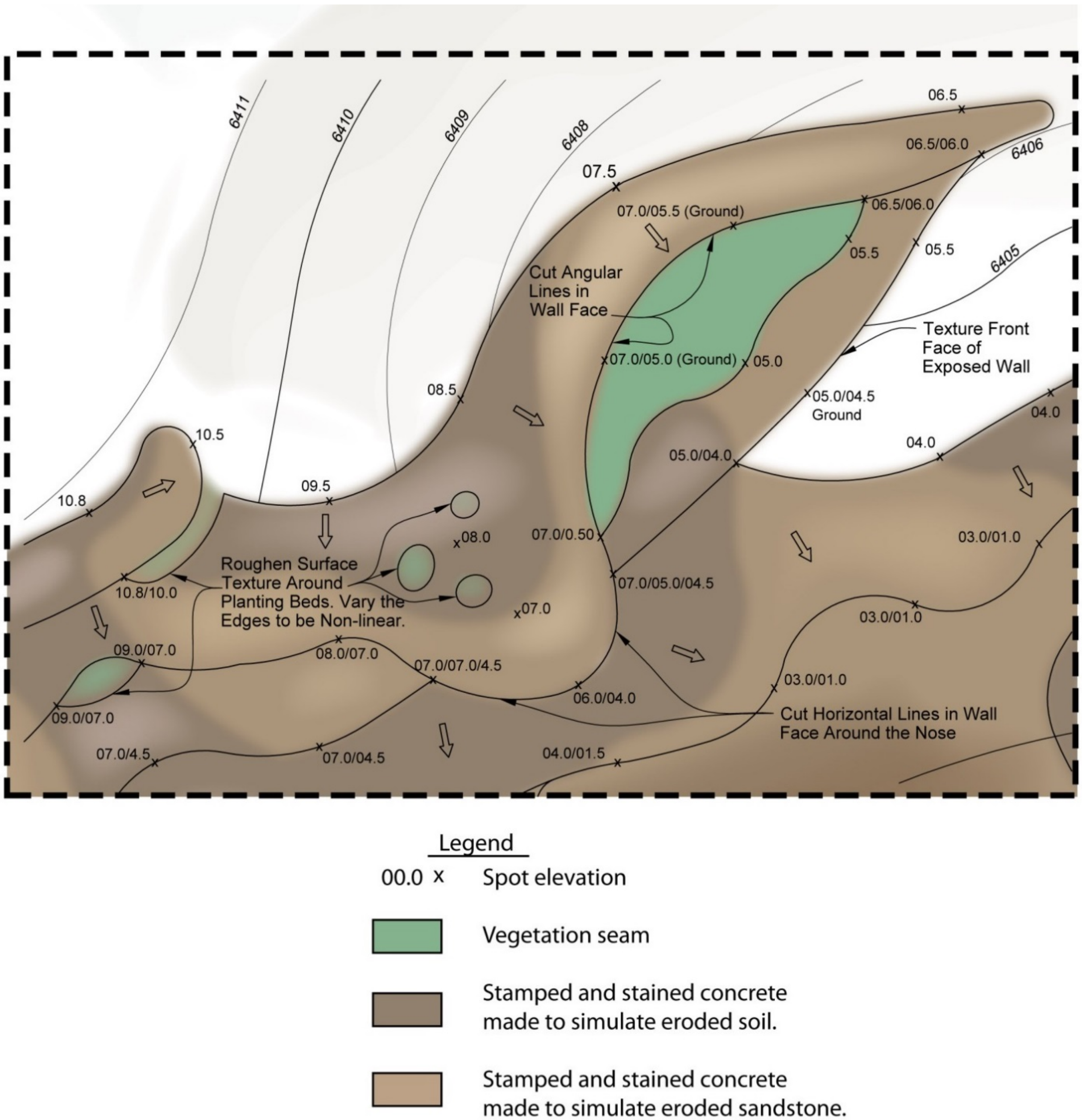


Figure 9-20. Example detailed view of complex sculpted concrete drop structure

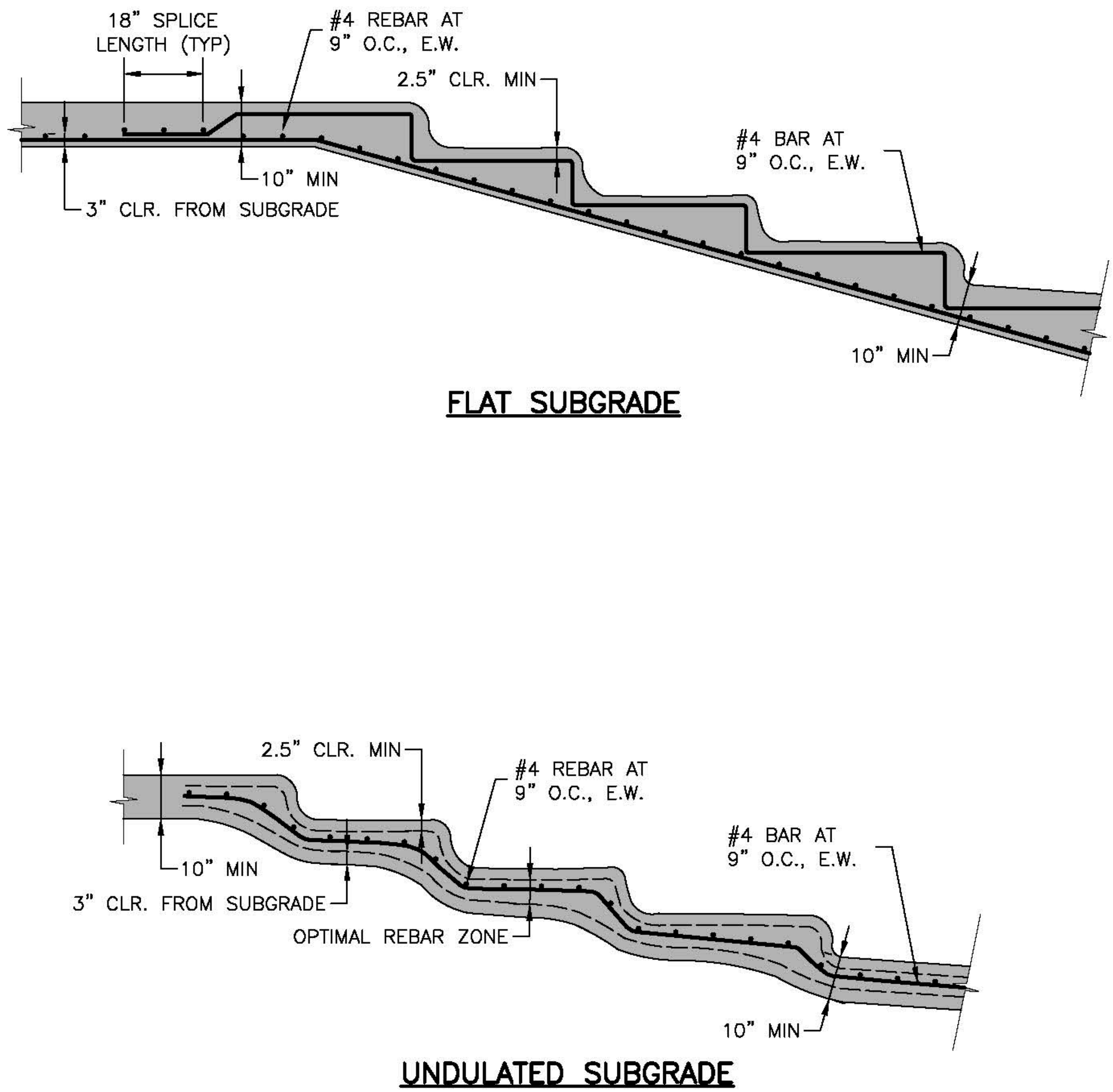
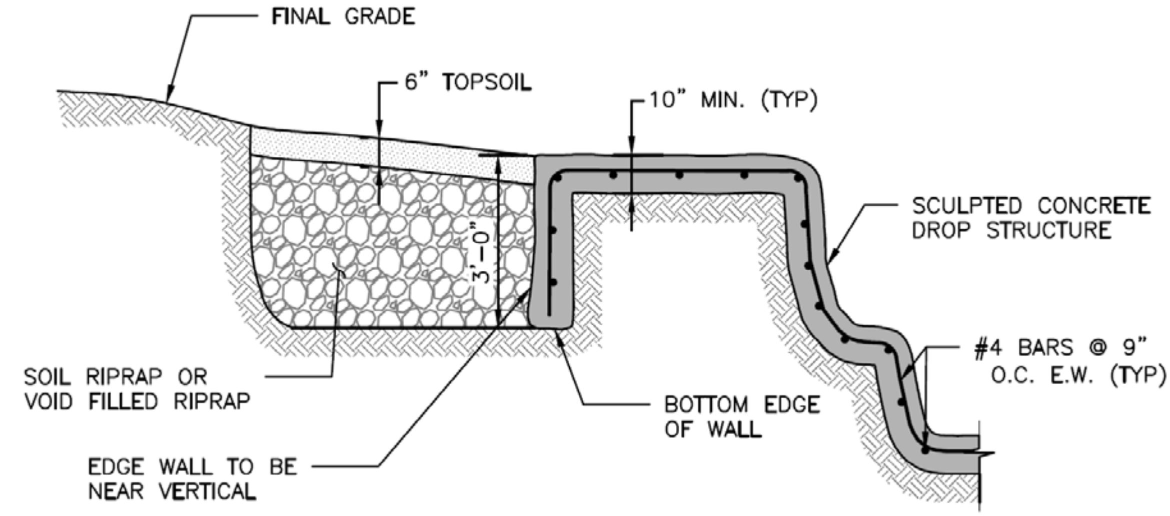
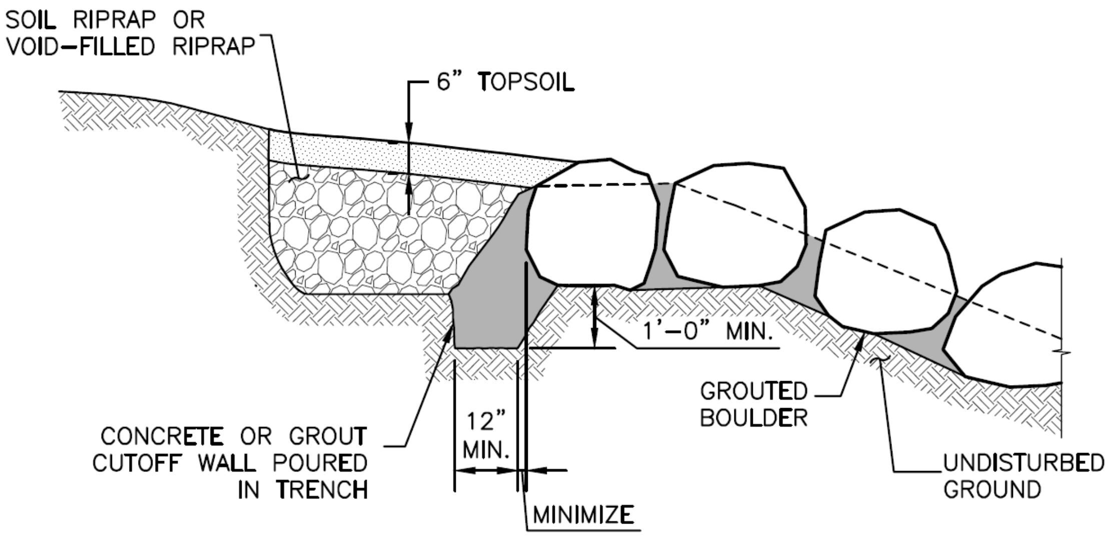


Figure 9-21. Rebar placement for sculpted concrete drop structures



NOTES:
1. DETAIL APPLIES TO THE PERIMETER OF THE ENTIRE STRUCTURE
2. SEE THE REBAR PLACEMENT DETAIL FOR CLEARANCE FROM TOP AND BOTTOM OF THE STRUCTURE

STRUCTURE EDGE WALL DETAIL (SC)



STRUCTURE EDGE WALL DETAIL (GSB)
NTS

Figure 9-22. Structure edge wall details

2.8 Vertical Drop Structure Selection

2.8.1 Description

Photograph 9-27. Keeping vertical drops small improves safety during both wet and dry conditions.

Vertical drop structures are discouraged for a number of safety reasons but can be an effective tool for controlling grade especially in locations where it is important to minimize the footprint of the drop structure and where there is little-to-no chance of recreation or access by minors. It is important to note that vertical structures can cause dangerous hydraulic conditions, including keeper waves, during wet weather and should be used only where appropriate. In addition, vertical drop structures are to be avoided due to impingement energy, related maintenance and turbulent hydraulic potential (ASCE and WEF 1992). Vertical drop structures should not be used on a channel where fish passage is a concern. Whenever used, it is recommended that the net drop structure height (upstream invert to downstream invert) be limited to 2 feet. This will allow for the addition of a 1-foot deep stilling basin immediately downstream of the crest. Drop structures frequently attract children during dry and wet conditions. Heights in excess of 3 feet are a falling hazard. In addition, a vertical drop structure should never be constructed where the design flow exceeds 500 cfs or a unit discharge of 35 cfs/ft.

2.8.2 Design Criteria

The hydraulic phenomenon provided by a vertical drop structure is a jet of water that overflows the crest wall into a hard basin below. The jet hits the basin and is redirected horizontally. With sufficient tailwater, a hydraulic jump is initiated. Otherwise, the flow continues horizontally in a supercritical mode until the specific force of the tailwater is sufficient to force the jump. Energy is dissipated through turbulence in the hydraulic jump. Size the basin immediately downstream of the vertical wall to contain the supercritical flow and the erosive turbulent zone (see Figure 9-23).

Vertical drops are not appropriate where:

- Fish passage is needed,
- Design flow (over the length of the drop) exceeds 500 cfs or a unit discharge of 35 cfs/ft,
- Net drop height is greater than 2 feet, or
- The stream is boatable or there are other concerns related to in-channel safety.

1. The design approach uses the unit discharge in the main and low-flow channel to determine separately the water surface profile and jump location in these zones.

   (Chow 1959) presents the hydraulic analysis for the "Straight Drop Spillway."

The drop number, D_n, is defined as:

$$D_n = \frac{q^2}{\left(gY_f^3\right)}$$ Equation 9-8

Where:

q = unit discharge (cfs/ft)

Y_f = height from the crest to the basin floor (ft)

g = acceleration of gravity = 32.2 ft/sec^2

For hydraulic conditions at a point immediately downstream of where the nappe hits the basin floor, the following variables are defined as illustrated in Figure 9-23:

$$\frac{L_d}{Y_f} = 4.3D_n^{0.27}$$

$$\frac{Y_p}{Y_f} = 1.0D_n^{0.22}$$

$$\frac{Y_1}{Y_f} = 0.54D_n^{0.425}$$

$$\frac{Y_2}{Y_f} = 1.66D_n^{0.27}$$

Drop Number for a Vertical Drop:

The drop number, D_n is a function of the unit discharge and vertical distance between the crest of the drop and the basin floor. From this value, the following can be determined:

- Location of the impingement,
- Depth of the pool under the nappe,
- Flow depth just downstream of the point of impingement,
- Sequent depth required to force the hydraulic jump

These values are necessary to properly place boulders (or baffles) for dissipation as well as determine the length of the basin.

Where:

Y_f = height from the crest to the basin floor (ft)

L_d = length from the crest wall to the point of impingement of the jet on the floor or the nappe length (ft)

Y_p = pool depth under the nappe just downstream of the crest (ft)

Y_1 = flow depth on the basin floor just below where the nappe contacts the basin (ft)

Y_2 = tailwater depth (sequent depth) required to cause the jump to form at the point evaluated (ft)

In the case where the tailwater does not provide a depth equivalent to or greater than Y_2, the jet will wash downstream as supercritical flow until its specific force is sufficiently reduced to allow the jump to occur. This requires the designer to also check normal depth just downstream of the drop to ensure that it is equal or greater than Y_2.

Determination of the distance to the hydraulic jump, D_j, requires a separate water surface profile analysis for the main and low-flow zones (See Section 2.3.6 for additional guidance). Any change in tailwater affects the stability of the jump in both locations.

The hydraulic jump length, L_j, is approximated as 6 times the sequent depth, Y_2. Where tailwater provides a depth equivalent to or greater than Y_2, the design basin length, L_b, includes nappe length, L_d, and 60% of the jump length, L_j. (The subscripts "m" and "l" in Equations 9-8 and 9-9 refer to the main and low-flow zones, respectively). Where the tailwater is not sufficient to force the jump at the point of impingement, the distance from this point to the jump must be added to the basin length in the below equations.

At the main channel zone:

$$L_{bm} = L_{dm} + 60\% \left(6Y_{2m}\right)$$ Equation 9-9

At the low- flow zone, without boulders to break up the jet:

$$L_{bl} = L_{dl} + 60\% \left(6Y_{2l}\right)$$ Equation 9-10

1. Caution is advised regarding the higher unit flow condition in the low-flow zone. Large boulders and meanders in the low-flow zone of the basin may help dissipate the jet and may reduce the extent of armoring downstream along the low-flow channel. When large boulders are used as baffles in the impingement area of the low-flow zone, the low-flow basin length L_{bl}, may be reduced, but not less than L_{bm}. Boulders should project into the flow 0.6 to 0.8 times the critical depth. They should be located between the point where the nappe hits the basin and no closer than 10 feet from the basin end.

2. The basin floor elevation should be designed as depressed or free-draining similar to the stilling basin for stepped grouted boulder drop structures. A depressed basin adds to the effective tailwater depth for jump control. The basin is typically constructed of grouted boulders (24-inch minimum). The stilling basin must be evaluated for seepage uplift (Section 2.4) and other hydraulic forces.

3. Use a sill at the end of the stilling basin to assist in causing the hydraulic jump to form in the basin. Soil riprap or void filled riprap should be used downstream of the sill to minimize any local scour caused by the lift over the sill.

4. Use caution to avoid boulder placement such that flow impinges the channel side slopes of the basin.

5. Determine crest wall and footer dimensions by conventional structural methods. Underdrain requirements should be determined from seepage analysis.

6. Seepage uplift conditions require evaluations for each use. Complete a seepage analysis to provide for control and weight/size of components (see Section 2.4).

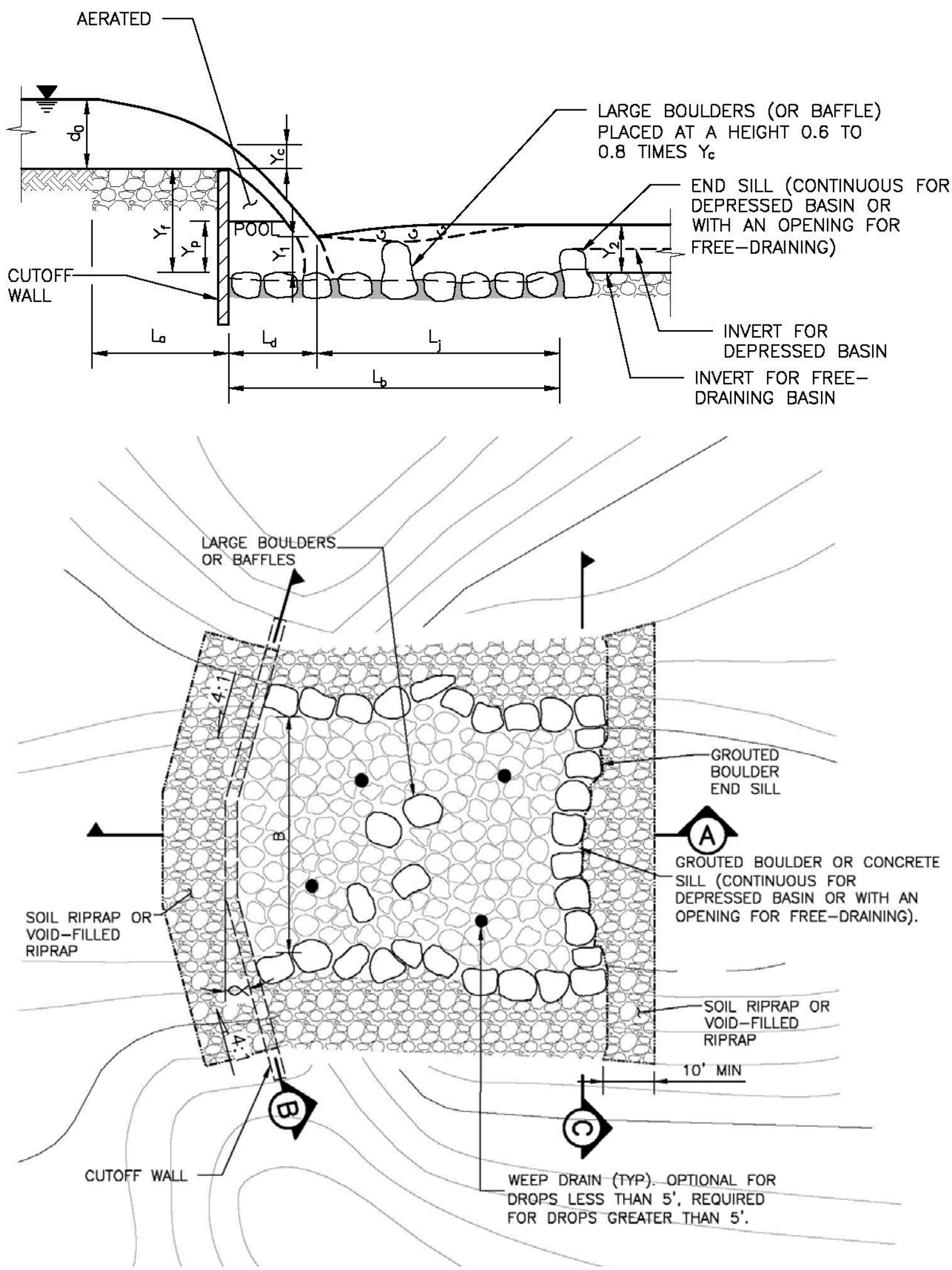


Figure 9-24. Example vertical drop structure plan

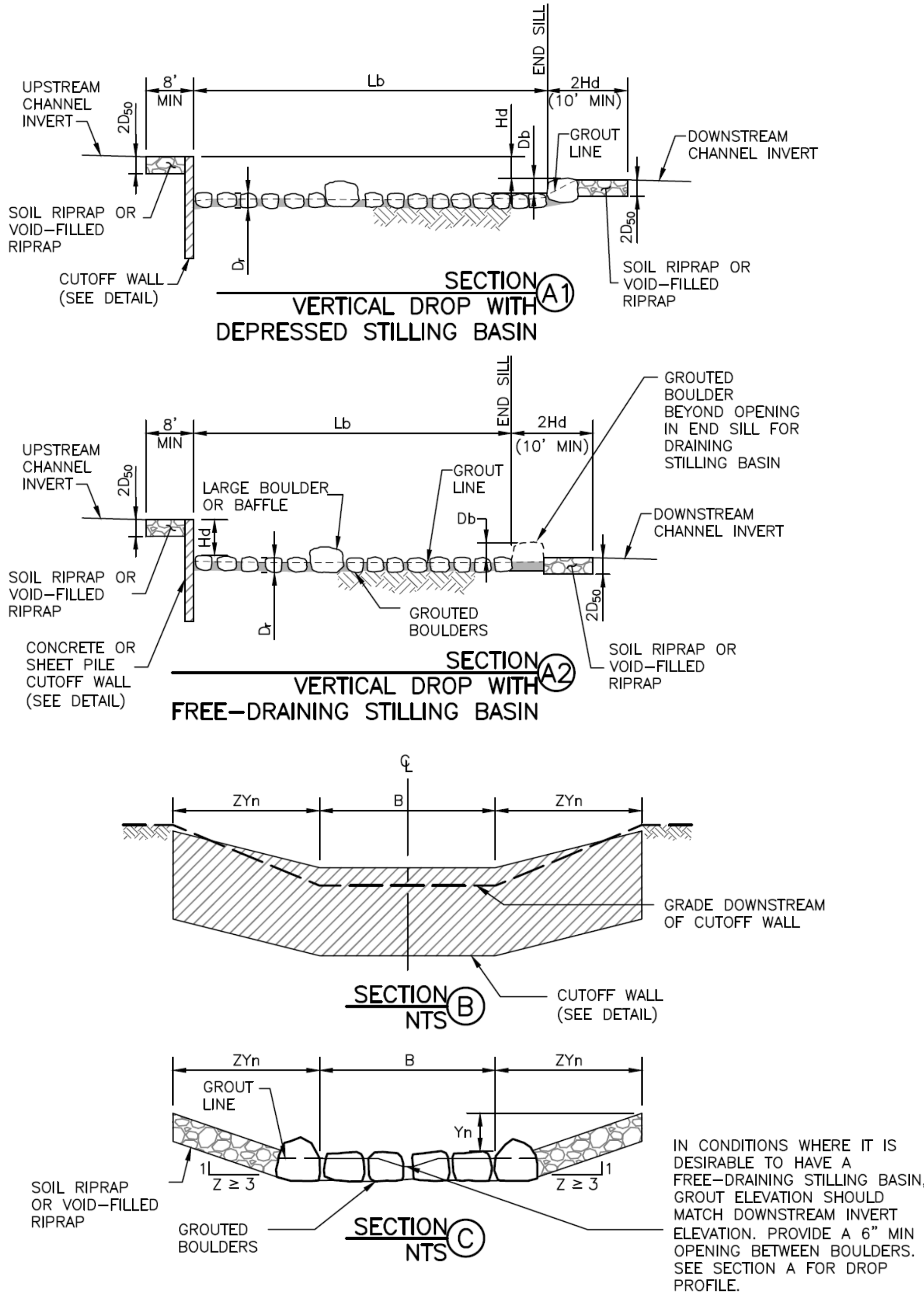


Figure 9-25. Example vertical drop structure sections

2.9 Low-flow Drop Structures and Check Structures

If a channel has not yet experienced significant erosion and degradation, but may degrade in the future, a number of options provide a level of reinforcement against future degradation. One approach is to install a standard drop structure and then backfill it to be mostly buried in the near term, but ready to handle additional grade difference as the channel invert lowers over time. Other approaches include:

- Low-flow Drop Structures. Low-flow drop structures are small structures designed to provide control points and establish stable bed slopes within the low-flow channel. Erosion of the low-flow channel, if left uncontrolled, can cause degradation and destabilization of the entire channel. Low-flow drop structures must be tied securely into the banks of the low-flow channel and take advantage of backwater from downstream drop structures to reduce the likelihood of circumventing, also known as flanking, or “end-around” erosion as flow converges back to the low-flow channel from the main channel overbanks below the drop structure. Low-flow drop structures in themselves do not address erosion potential in the overbank areas outside of the low-flow channel. See the criteria in the Open Channel chapter when evaluating the stability of the existing channel. Note that the low-flow channel is referred to as the bankfull channel in other parts of this manual.

- Check Structures with follow-up field observation program. Check structure construction typically consists of driving sheet pile to a 10-foot depth and capping it with concrete or filling an excavated narrow trench (12” minimum width) with concrete (if soil and groundwater conditions permit trenching to a depth of six feet). Only specify concrete check structures where soils permit excavation of a narrow trench. Never over-excavate to form concrete checks. Extend the walls laterally as necessary to contain the 5- to 10-year flow (depending on local criteria), but no less than 2 feet above the top of the low-flow channel banks. This will reduce the risk of side cutting. Space check structures so that there is no more than a 3-foot net drop from the crest of the check to the projected downstream invert based on the estimated long-term equilibrium slope. Figure 9-26 illustrates sheet piling and concrete check structures and a typical concrete cap for sheet piling check structures. Additional protection (e.g., soil riprap or void-filled riprap) downstream of the check structure may be appropriate based on scour potential. Consider soil type, longitudinal slope, and other site-specific considerations when evaluating scour potential.

  If a local government allows check structures, a follow-up field observation program is required to identify checks where erosion has exposed the face of the check structure, creating a significant drop in elevation from crest of the check to the elevation immediately downstream of the check. This vertical distance should not exceed 3 feet. Rehabilitative maintenance improvements may be necessary to install stable downstream erosion protection and convert the check to a drop structure (e.g., a grouted stepped boulder drop structure). Soil riprap placed downstream of the check structure can help as an interim condition to ensure the vertical distance does not exceed 3 feet. Vertical differences in excess of 3 feet present a fall hazard during dry weather and can increase potential for an overly retentive hydraulic during wet weather.

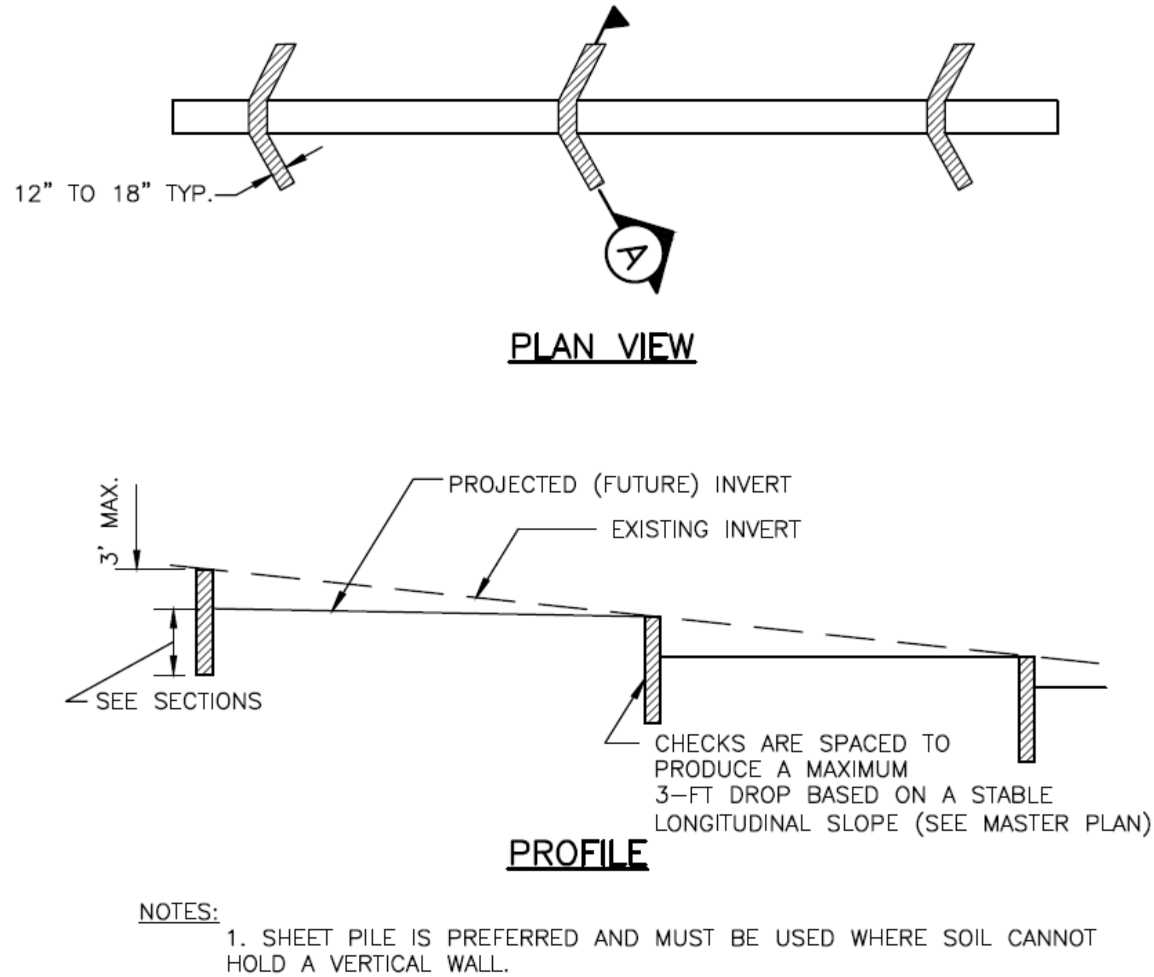


Figure 9-26. Check structure details (Part 1 of 3)

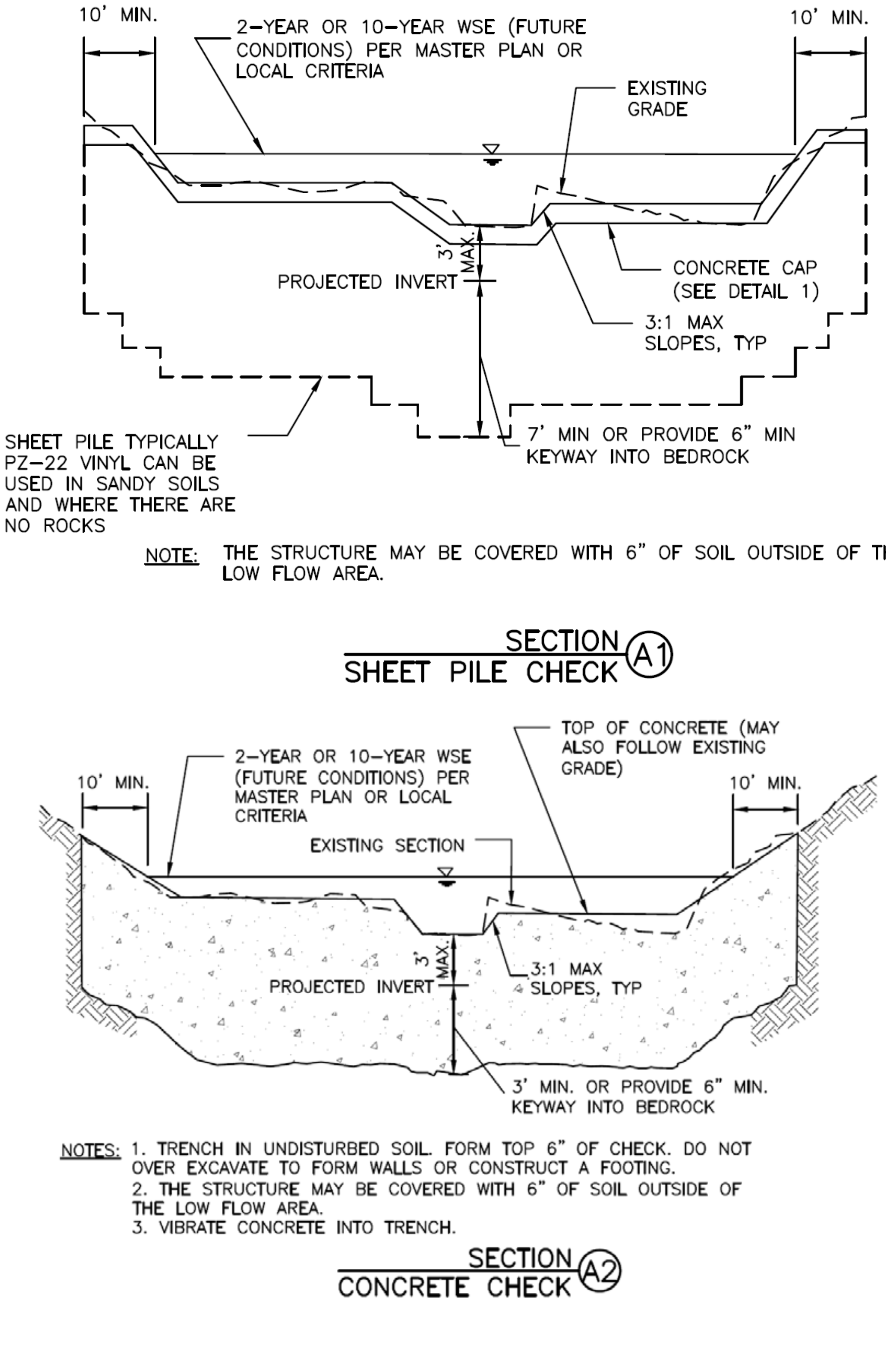


Figure 9-27. Check structure details (Part 2 of 3)

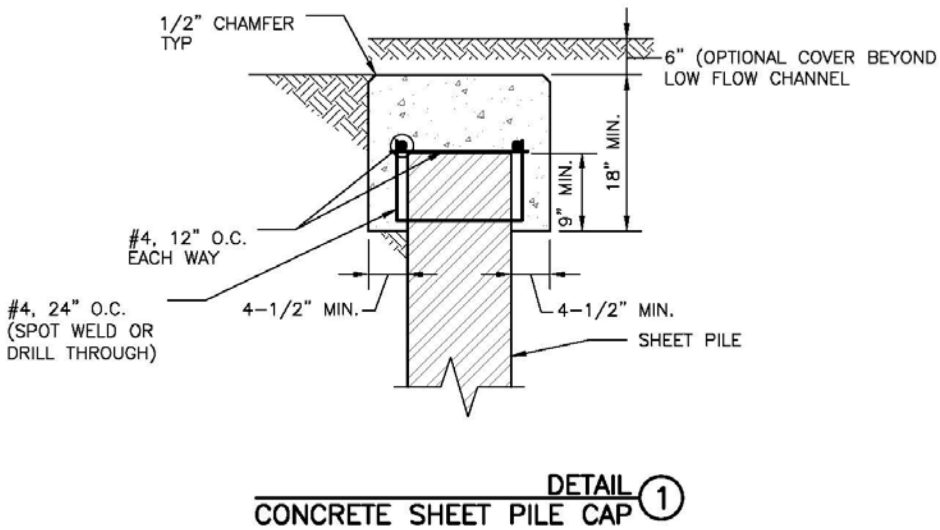


Figure 9-28. Check structure details (Part 3 of 3)

3.0 Pipe Outfalls and Rundowns

Pipe outlets represent a persistent problem due to concentrated discharges and turbulence of flow reaching this point of transition in an open channel. Too often, the designer focuses efforts on a culvert inlet and its sizing with outlet hydraulics receiving only passing attention. Appropriate pipe end treatment and downstream erosion protection at pipe outfalls is critical to protect the structural integrity of the pipe and to maintain the stability of the adjacent slope. Further discussion regarding appropriate treatment at pipe outfalls is included in the following sections.

Photograph 9-28. Pipe outfalls are recommended over rundowns due to the high failure rate of rundowns.

The use of rundowns to convey storm runoff down a channel bank is discouraged due to their high rate of failure and the resulting maintenance and repair burden. Instead, use a pipe to convey runoff to a point just above the channel invert (normally 1 foot for small receiving streams or ponds and up to 2 feet for large receiving channels).

3.1 Pipe End Treatment

Pipe end treatment consists of a flared end section, toe wall, headwall, wingwall or combination of treatments to protect the outfall from failure and provide a stable transition from hard to soft conveyance elements. Further discussion regarding these treatments follows.

Photograph 9-29. Pipe end failure resulting in loss of the flared end section

3.1.1 Flared-End Sections and Toe Walls

Flared end sections may be installed on both the inlet and outlet ends of culverts or storm drain systems. Erosion is likely at the outlet and possible at the inlet. Construction of a concrete toe wall (cutoff) is will protect the culvert from damage if inlet or outlet protection fails. At the outlet, provide scour protection including cutoff wall and use joint fasteners immediately upstream of the outlet. Protection at the upstream end can also help control seepage in the storm drain trench. See the Culverts chapter for discussion on inlet improvements.

Concrete toe walls include a footing and stem wall as shown in Figure 9-29 although the footing is optional for pipes 48 inches or less. Freezing depth should dictate the depth of the wall. The depth shown in the details represents freezing depth in the UDFCD region. Included is a design table for pipes 18 to 72 inches. The wall length shown allows an approximate 3(H):1(V) final ground slope from the flared-end section invert to the top outside edge of wall. Note that for large diameter flared-end sections, the wall lengths are quite large. It may be advantageous to use a combination headwall/wingwall approach or consider incorporating boulders for pipes larger than 36 inches in diameter. Always evaluate public safety including the need for pedestrian railing where a potential fall of 36 inches or more as possible. Along with the toe wall, install two joint fasteners between the flared end section and the last pipe section. Install these roughly at the ten o’clock and two o’clock positions and trim joint fastener threads flush with the interior bolts. Left untrimmed, these can catch debris and reduce pipe capacity. Joint fasteners are not necessary for flared end sections on the entrance of culverts or storm drains.

Figures 9-29 and 9-30 are applicable to both ends of a culvert or storm drain system. It is the design engineer’s responsibility to assess the need for a cutoff wall. Factors to consider include:

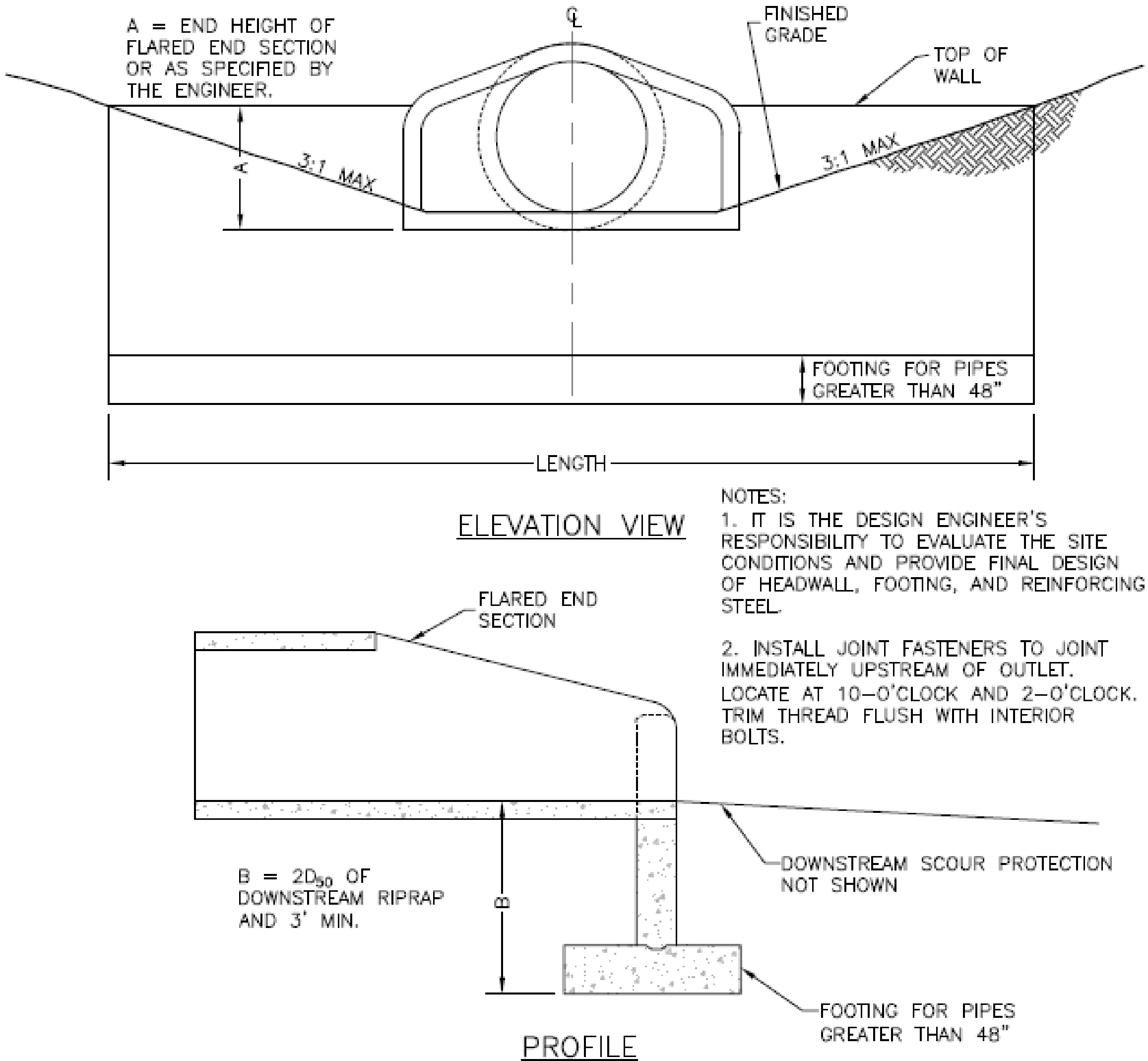
- The slope of the culvert or storm drain system is steep;
- The surrounding subsoils are granular or otherwise susceptible to erosion and/or piping;
- Potential for the roadway to wash out and the associated impact to public safety.

3.1.2 Concrete Headwall and Wingwalls

Concrete headwalls are an acceptable alternative to flared-end sections at pipe inlets and outlets. Figure 9-31 provides design guidance and a headwall design table for the design of a concrete headwall at a pipe inlet or outlet. When a 3(H):1(V) final ground slope from the pipe invert to the top outside edge of wall is used, the wall length can become quite long. Headwalls can be paired with wingwalls or boulders in order to reduce the overall headwall length. For 18” to 36” diameter pipes, headwalls can be paired with loosely placed boulders as shown in Figure 9-32. The addition of boulders can enhance the appearance of the end treatment and significantly reduce the wall length.

Storm drain outfalls into large river systems (e.g., the South Platte River) often require special consideration with respect to the channel bank geometry and base flow water surface elevation. Figure 9-33 provides general layout information for the construction of a headwall with wingwalls. It is the design engineer’s responsibility to evaluate the site conditions and provide final design of headwall, wingwalls, footings, and reinforcing steel.

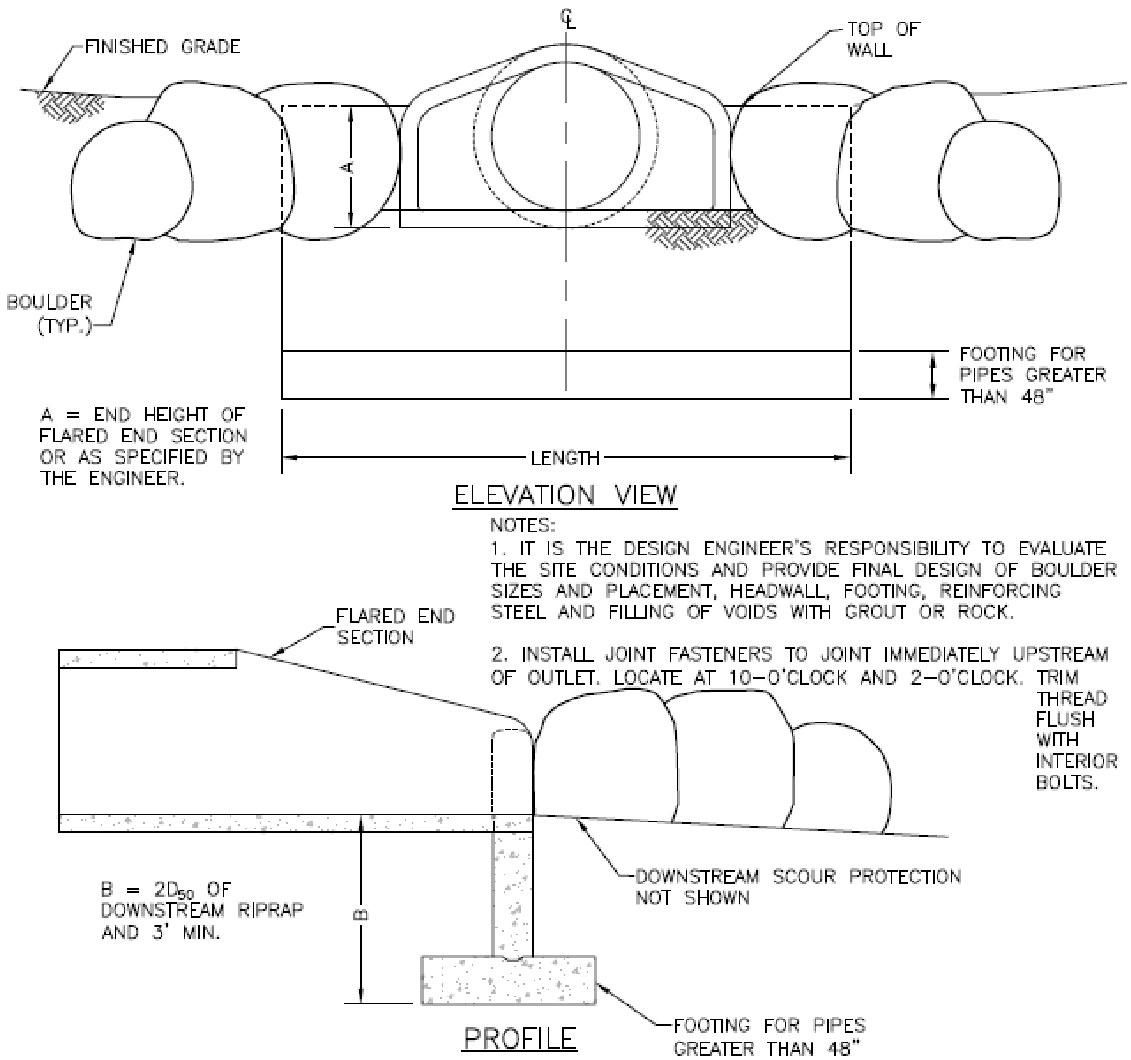
On large receiving streams, UDFCD encourages the use of wingwalls that are constructed perpendicular to the receiving channel centerline (or headwall), thereby reducing the impact to the channel overbanks. Further discussion regarding structure requirements for outfalls into large river systems is in Section 3.2.4.



HEADWALL DESIGN TABLE

PIPE SIZE	LENGTH, MIN
18"	7'–0"
24"	8'–0"
30"	10'–0"
36"	12'–6"
42"	15'–9"
48"	17'–6"
54"	19'–6"
60"	21'–6"
66"	22'–6"
72"	24'–0"

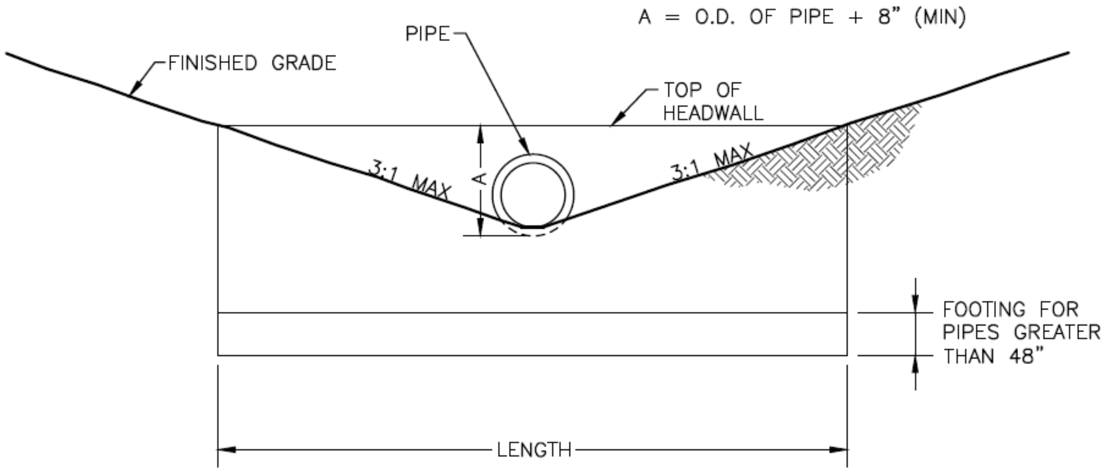
Figure 9-29. Flared end section (FES) headwall concept



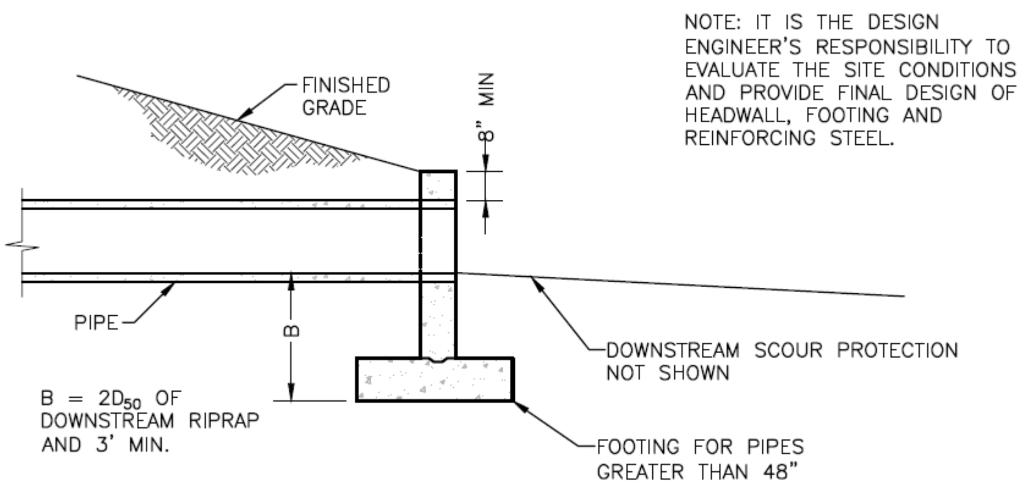
TOEWALL DESIGN TABLE

PIPE SIZE	LENGTH, MIN.
18"	7'–0"
24"	8'–0"
30"	10'–0"
36"	12'–0"
42"	12'–6"
48"	13'–0"
54"	13'–6"
60"	14'–0"
66"	14'–6"
72"	15'–0"

Figure 9-30. Flared end section (FES) headwall concept



ELEVATION VIEW

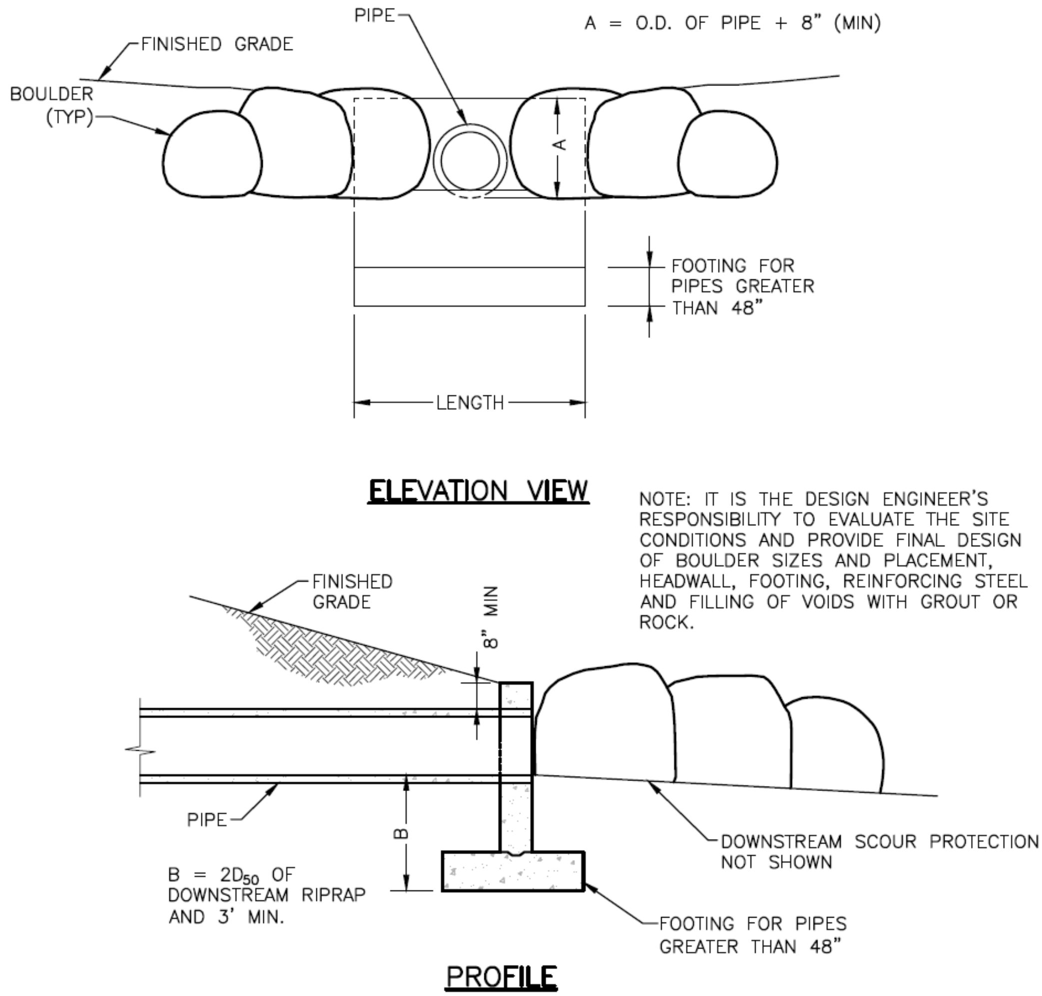


PROFILE

HEADWALL DESIGN TABLE

PIPE SIZE	LENGTH, MIN
18"	15'–9"
24"	19'–6"
30"	23'–4"
36"	27'–0"
42"	30'–9"
48"	34'–6"

Figure 9-31. Pipe headwall concept



HEADWALL DESIGN TABLE

PIPE SIZE	LENGTH, MIN
18"	7'-0"
24"	8'-0"
30"	8'-6"
36"	9'-0"
42"	9'-6"
48"	10'-0"

PIPE HEADWALL DETAIL (WITH BOULDERS)

Figure 9-32. Pipe headwall with boulders concept

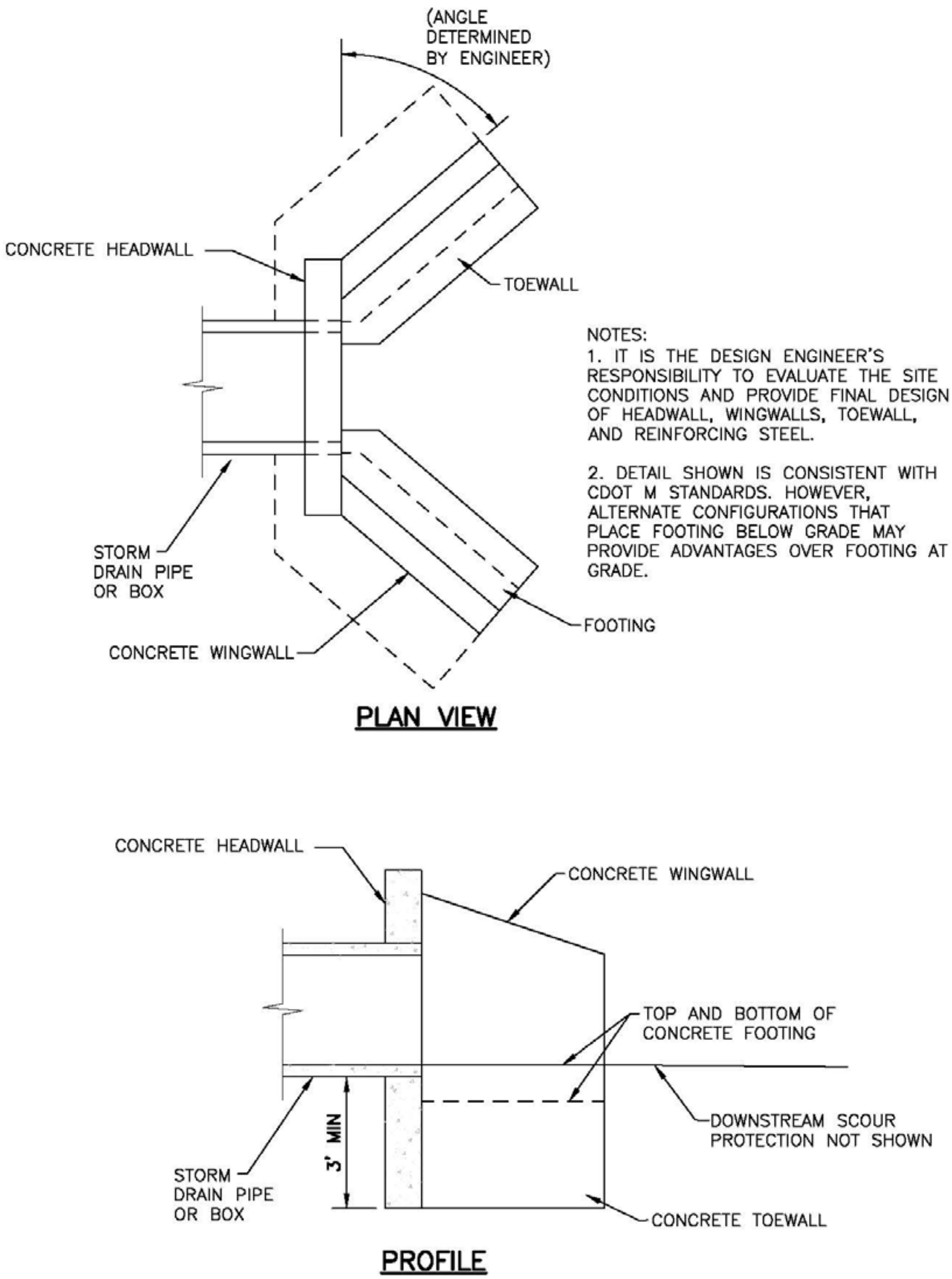


Figure 9-33. Pipe headwall/wingwall concept

3.2 Energy Dissipation and Erosion Protection

Local scour is typified by a scour hole produced at a pipe or culvert outlet. This is the result of high exit velocities, and the effects extend only a limited distance downstream. Coarse material scoured from the circular or elongated hole is deposited immediately downstream, often forming a low bar. Finer material moves farther downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow. Methods for predicting scour hole dimensions are found in the *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 1983 and 2000).

Protection against scour at outlets ranges from limited riprap placement to complex and expensive energy dissipation devices. Pre-formed scour holes (approximating the configuration of naturally formed holes) dissipate energy while providing a protective lining to the streambed.

This section addresses energy dissipation and erosion control measures that can be used to minimize or eliminate local scour at a pipe outlet. The following measures are discussed:

- Riprap Apron
- Low Tailwater Basin
- Grouted Boulders
- Impact Basin

In general, all of these measures pose risks to the public. Discourage public access and minimize the risk of falls at these structures.

Scour and Stream Degradation

Scour is typically found at culvert outlets and other isolated transitional areas within a stream. Frequently, scour holes fill in with sediment over time only to appear again during infrequent high flows.

Degradation is a phenomenon that is independent of culvert performance. Natural causes can produce a lowering of the streambed over time. Contributing factors include the slope of the stream and the size and availability of the sediment load. Degradation can also be a result of other constructed features such as upstream detention or increased watershed imperviousness. The identification of a degrading stream is an essential part of the original site investigation. Discussion of this subject is in the *Open Channels* chapter.

3.2.1 Riprap Apron

This section addresses the use of riprap for erosion protection downstream of conduit and culvert outlets. Refer to the *Open Channels* chapter for additional information on applications for and placement of riprap. Those criteria will be useful in design of erosion protection for conduit outlets. When incorporating a drop into the outfall use Figure 9-40 or 9-41.

Rock Size
The procedure for determining the required riprap size downstream of a conduit outlet is in Section 3.2.3.

Configuration of Riprap Apron
Figure 9-34 illustrates typical riprap protection of culverts at conduit outlets.

Extent of Protection
The length of the riprap protection downstream from the outlet depends on the degree of protection desired. If it is necessary to prevent all erosion, the riprap must extend until the velocity decreases to an acceptable value. The acceptable major event velocity is set at 5 ft/sec for non-cohesive soils and at 7 ft/sec for erosion resistant soils. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. The procedure recommended here assumes the rate of decrease in velocity is related to the angle of lateral expansion, θ, of the jet. The velocity is related to the expansion factor, $(1/(2\tan\theta))$, which can be determined directly using Figure 9-35 or Figure 9-36, by assuming that the expanding jet has a rectangular shape:

$$L_p = \left(\frac{1}{2\tan\theta}\right)\left(\frac{A_t}{Y_t} - W\right) \qquad \text{Equation 9-11}$$

Where:

L_p = length of protection (ft)

W = width of the conduit (ft, use diameter for circular conduits)

Y_t = tailwater depth (ft)

θ = the expansion angle of the culvert flow

and:

$$A_t = \frac{Q}{V} \qquad \text{Equation 9-12}$$

Where:

Q = design discharge (cfs)

V = the allowable non-eroding velocity in the downstream channel (ft/sec)

A_t = required area of flow at allowable velocity (ft²)

In certain circumstances, Equation 9-11 may yield unreasonable results. Therefore, in no case should L_p be less than $3H$ or $3D$, nor does L_p need to be greater than $10H$ or $10D$ whenever the Froude parameter, $Q/WH^{1.5}$ or $Q/D^{2.5}$, is less than 8.0 or 6.0, respectively. Whenever the Froude parameter is greater than these maximums, increase the maximum L_p required by ¼ D_c or ¼ H for circular or rectangular (box) culverts, respectively, for each whole number by which the Froude parameter is greater than 8.0 or 6.0, respectively.

Once L_p has been determined, the width of the riprap protection at the furthest downstream point should be verified. This dimension is labeled "T" on Figure 9-34. The first step is to solve for ϑ using the results from Figure 9-35 or 9-36:

$$\theta = \tan^{-1}\left(\frac{1}{2(\text{ExpansionFactor})}\right) \qquad \text{Equation 9-13}$$

Where:

Expansion Factor = determined using Figure 9-35 or 9-36

T is then calculated using the following equation:

$$T = 2(L_p \tan\theta) + W \qquad \text{Equation 9-14}$$

Multiple Conduit Installations

The procedures outlined in this section can be used to design outlet erosion protection for multi-barrel culvert installations by replacing the multiple barrels with a single hydraulically equivalent hypothetical rectangular conduit. The dimensions of the equivalent conduit may be established as follows:

1. Distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed; otherwise, the flow through each barrel must be computed.

2. Compute the Froude parameter $Q_i/D_{ci}^{2.5}$ (circular conduit) or $Q_i/W_iH_i^{1.5}$ (rectangular conduit), where the subscript i indicates the discharge and dimensions associated with an individual conduit.

3. If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit.

4. Make the height of the equivalent conduit, H_{eq}, equal to the height, or diameter, of the selected individual conduit.

5. The width of the equivalent conduit, W_{eq}, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q/W_iH_{eq}^{1.5}$.

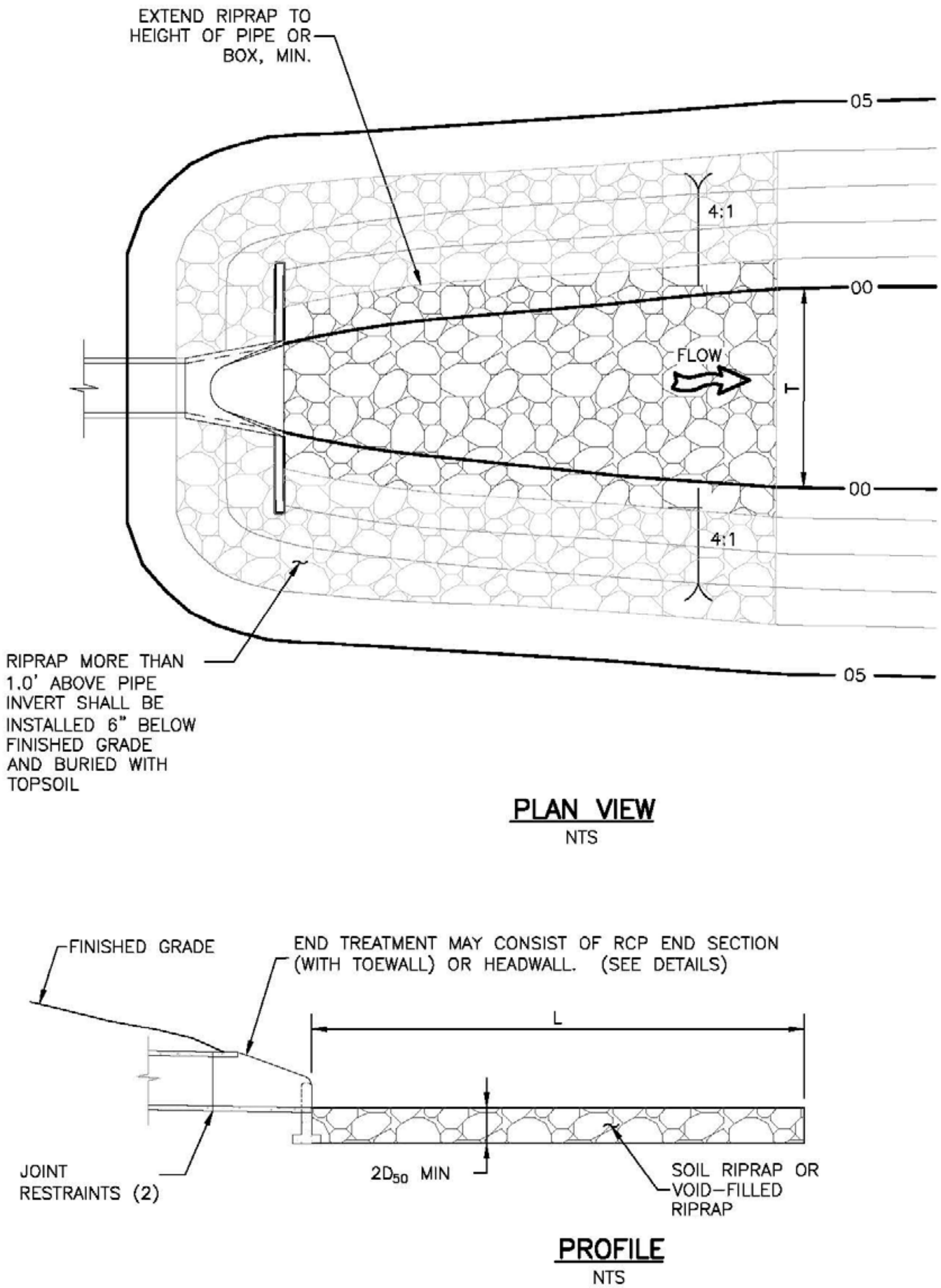


Figure 9-34. Riprap apron detail for culverts in-line with the channel

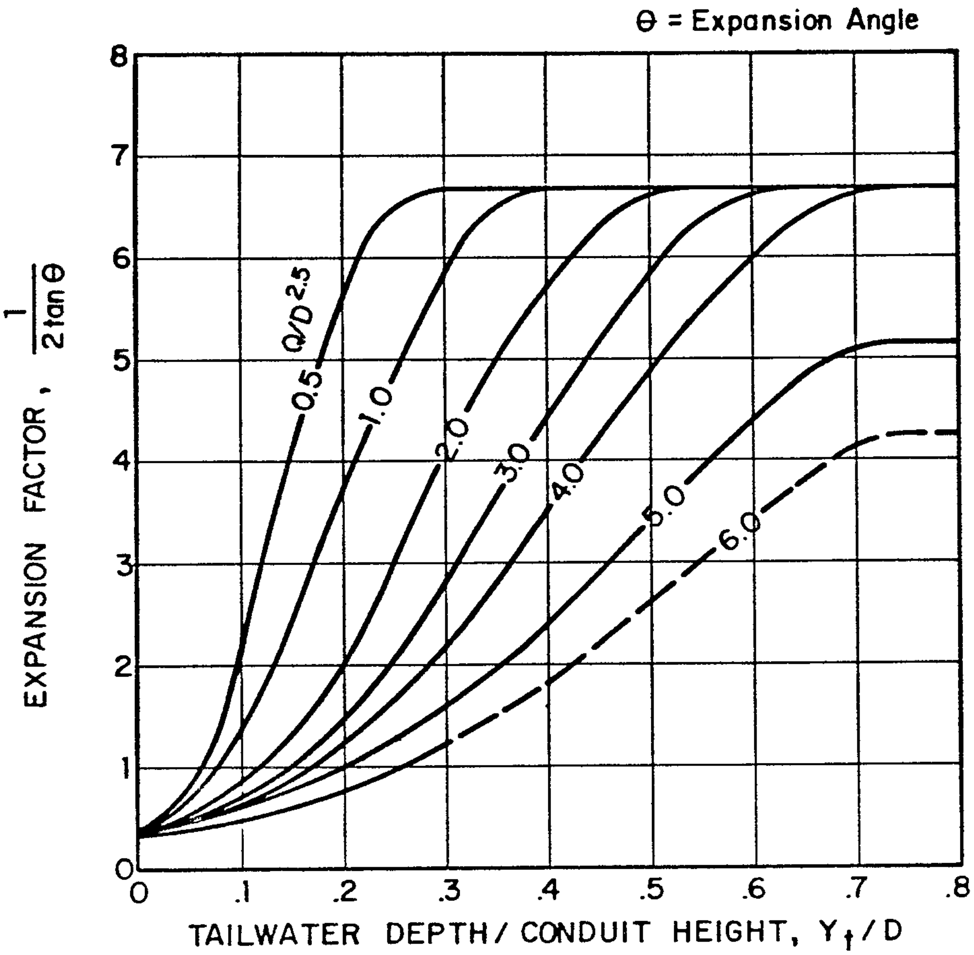


Figure 9-35. Expansion factor for circular conduits

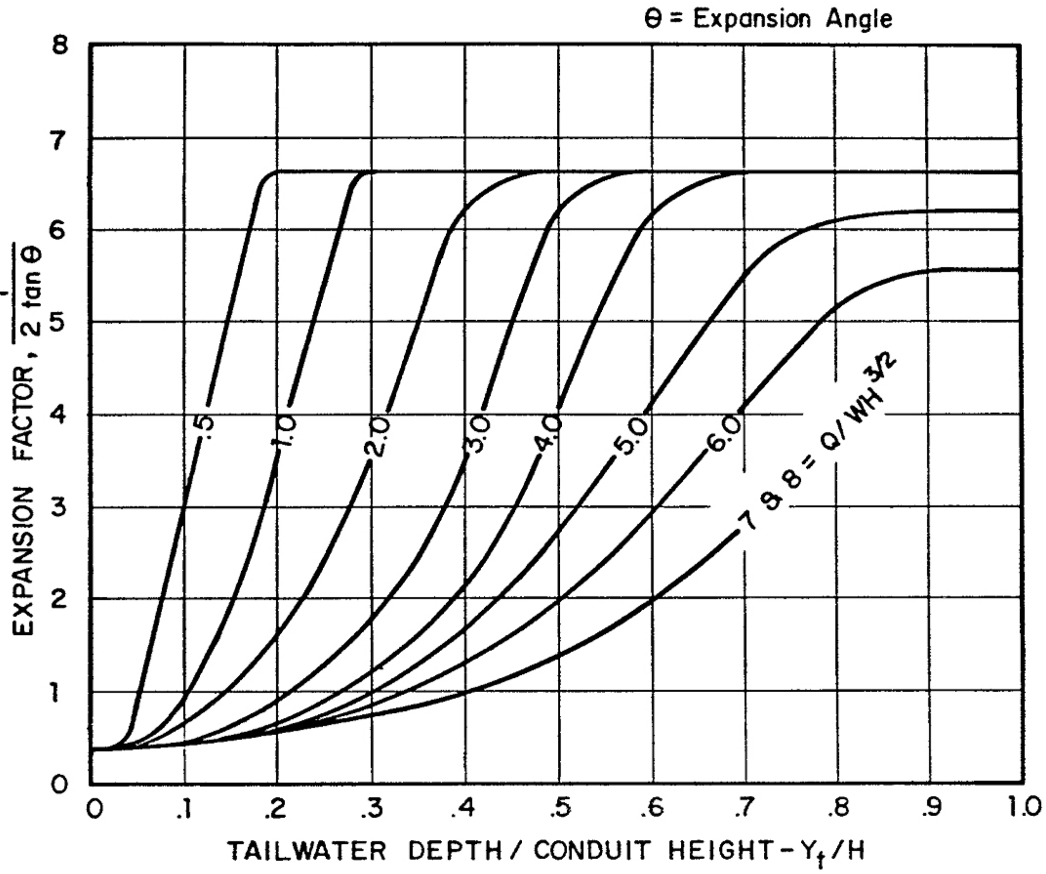


Figure 9-36. Expansion factor for rectangular conduits

3.2.2 Low Tailwater Basin

The design of low tailwater riprap basins is necessary when the receiving channel may have little or no flow or tailwater at time when the pipe or culvert is in operation. Figure 9-37 provides a plan and profile view of a typical low tailwater riprap basin.

By providing a low tailwater basin at the end of a storm drain conduit or culvert, the kinetic energy of the discharge dissipates under controlled conditions without causing scour at the channel bottom.

Low tailwater is defined as being equal to or less than ⅓ of the height of the storm drain, that is:

$$y_t \leq \frac{D}{3} \quad \text{or} \quad y_t \leq \frac{H}{3}$$

Where:

y_t = tailwater depth at design flow (feet)

D = diameter of circular pipe (feet)

H = height of rectangular pipe (feet)

Rock Size
The procedure for determining the required riprap size downstream of a conduit outlet is in Section 3.2.3.

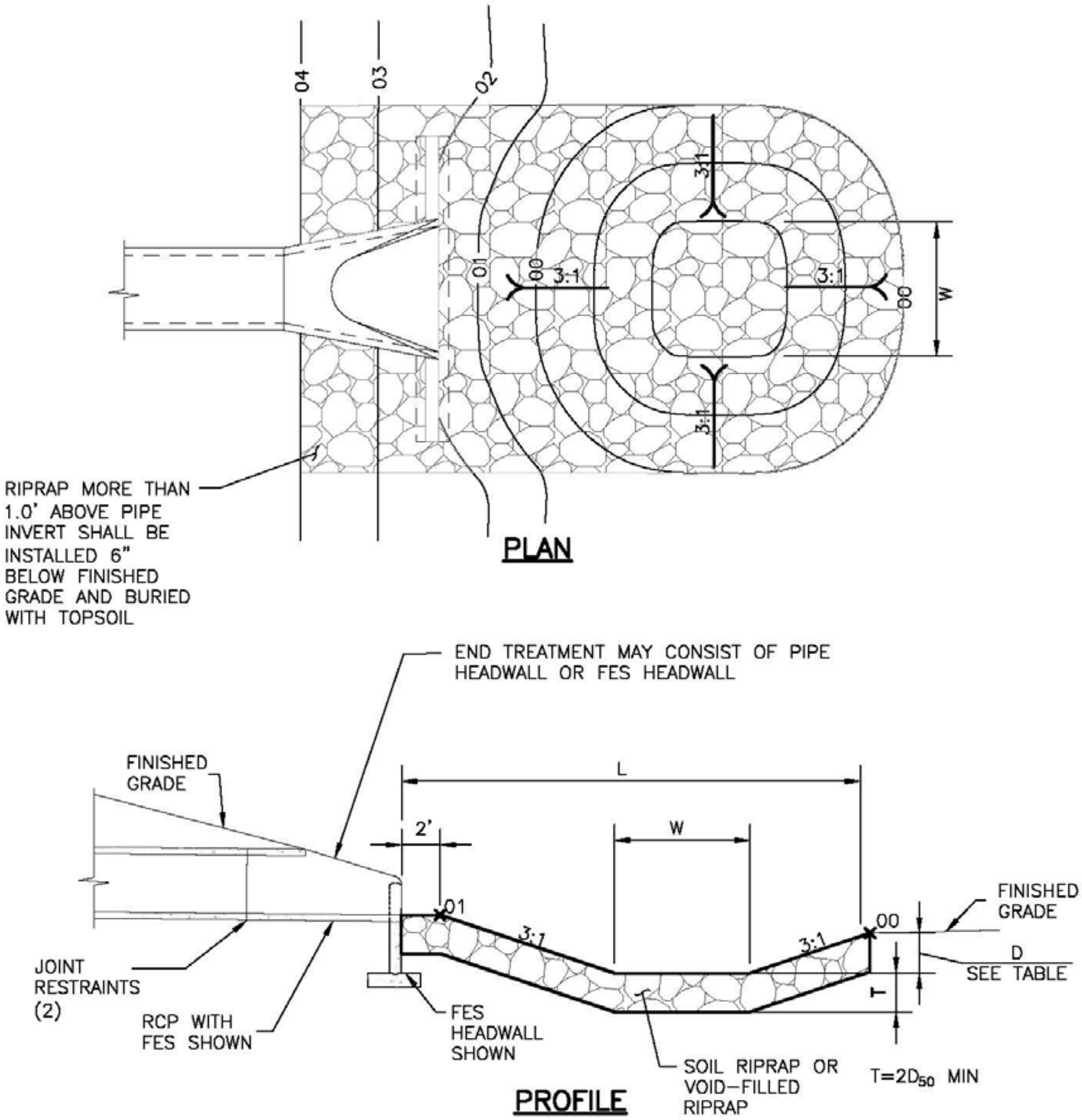
After selecting the riprap size, the minimum thickness of the riprap layer, *T*, in feet, in the basin is defined as:

$$T = 2D_{50}$$ Equation 9-15

Basin Geometry
Figure 9-37 includes a layout of a standard low tailwater riprap basin with the geometry parameters provided. The minimum length of the basin (L) and the width of the bottom of the basin (W1) are provided in a table at the bottom of Figure 9-37. All slopes in the low tailwater basin shall be 3(H):1(V), minimum.

Other Design Requirements
Extend riprap up the outlet embankment slope to the mid-pipe level, minimum. It is recommended that riprap that extends more than 1 foot above the outlet pipe invert be installed 6 inches below finished grade and buried with topsoil.

Provide pipe end treatment in the form of a pipe headwall or a flared-end section headwall. See Section 3.1 for options.



PIPE SIZE OR BOX HEIGHT	D	W*	L
18" – 24"	1'–0"	4'	15'
30" – 36"	1'–6"	6'	20'
42" – 48"	2'–0"	7'	24'
54" – 60"	2'–6"	8'	28'
66" – 72"	3'–0"	9'	32'

* IF OUTLET PIPE IS A BOX CULVERT WITH A WIDTH GREATER THAN W, THEN W = CULVERT WIDTH

Figure 9-37. Low tailwater riprap basin

3.2.3 Rock Sizing for Riprap Apron and Low Tailwater Basin

Scour resulting from highly turbulent, rapidly decelerating flow is a common problem at conduit outlets. The following section summarizes the method for sizing riprap protection for both riprap aprons (Section 3.2.1) and low tailwater basins (Section 3.2.2).

Use Figure 9-38 to determine the required rock size for circular conduits and Figure 9-39 for rectangular conduits. Figure 9-38 is valid for $Q/D_c^{2.5}$ of 6.0 or less and Figure 9-39 is valid for $Q/WH^{1.5}$ of 8.0 or less. The parameters in these two figures are:

1. $Q/D^{1.5}$ or $Q/WH^{0.5}$ in which Q is the design discharge in cfs, D_c is the diameter of a circular conduit in feet, and W and H are the width and height of a rectangular conduit in feet.

2. Y_t/D_c or Y_t/H in which Y_t is the tailwater depth in feet, D_c is the diameter of a circular conduit in feet, and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet, use $Y_t/D_t = Y_t/H = 0.40$ when using Figures 9-38 and 9-39.

3. The riprap size requirements in Figures 9-38 and 9-39 are based on the non-dimensional parametric Equations 9-16 and 9-17 (Steven, Simons, and Watts 1971 and Smith 1975).

Circular culvert:

$$d_{50} = \frac{0.023Q}{Y_t^{1.2} D_c^{0.3}} \qquad \text{Equation 9-16}$$

Rectangular culvert:

$$d_{50} = \frac{0.014H^{0.5}Q}{Y_t W} \qquad \text{Equation 9-17}$$

These rock size requirements assume that the flow in the culvert is subcritical. It is possible to use Equations 9-16 and 9-17 when the flow in the culvert is supercritical (and less than full) if the value of D_c or H is modified for use in Figures 9-38 and 9-39. Note that rock sizes referenced in these figures are defined in the *Open Channels* chapter. Whenever the flow is supercritical in the culvert, substitute D_a for D_c and H_a for H, in which D_a is defined as:

$$D_a = \frac{(D_c + Y_n)}{2} \qquad \text{Equation 9-18}$$

Where the maximum value of D_a shall not exceed D_c, and

$$H_a = \frac{(H + Y_n)}{2}$$ Equation 9-19

Where the maximum value of H_a shall not exceed H, and:

D_a = parameter to use in place of D in Figure 9-38 when flow is supercritical (ft)

D_c = diameter of circular culvert (ft)

H_a = parameter to use in place of H in Figure 9-39 when flow is supercritical (ft)

H = height of rectangular culvert (ft)

Y_n = normal depth of supercritical flow in the culvert (ft)

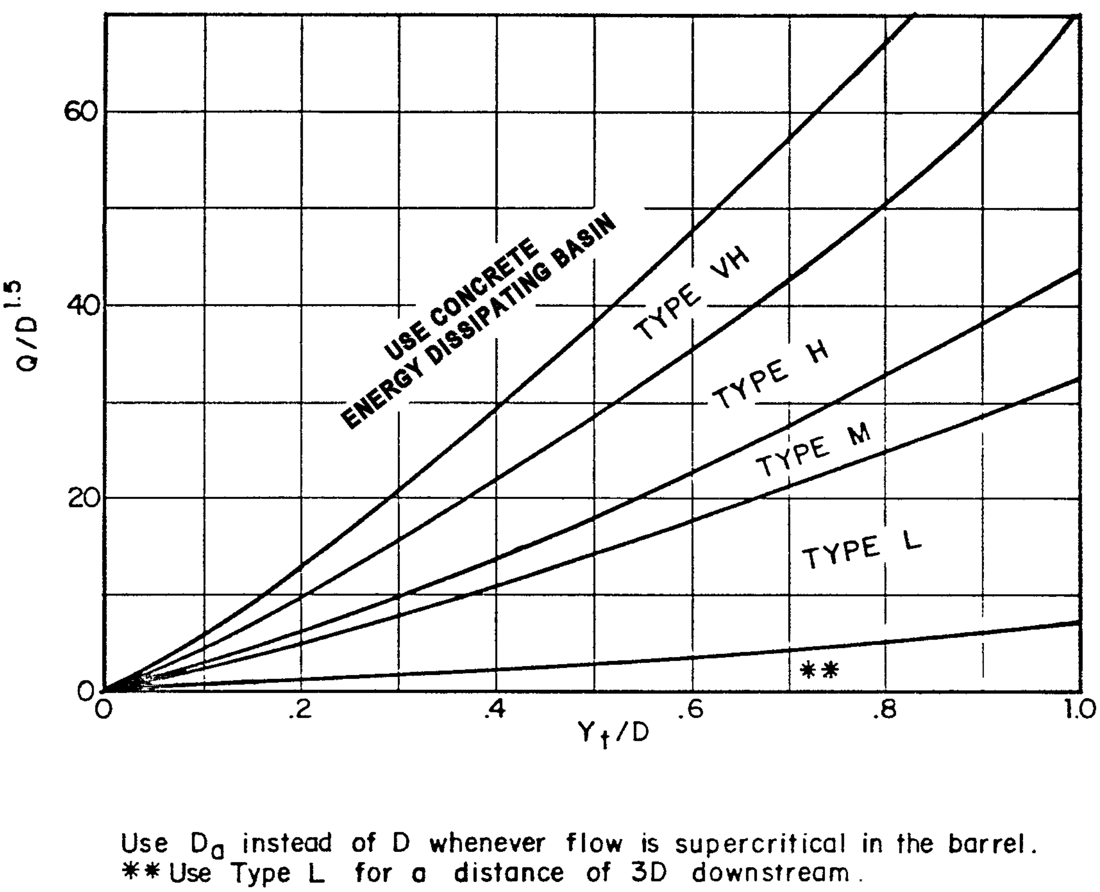


Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for Q/D2.5 ≤ 6.0)

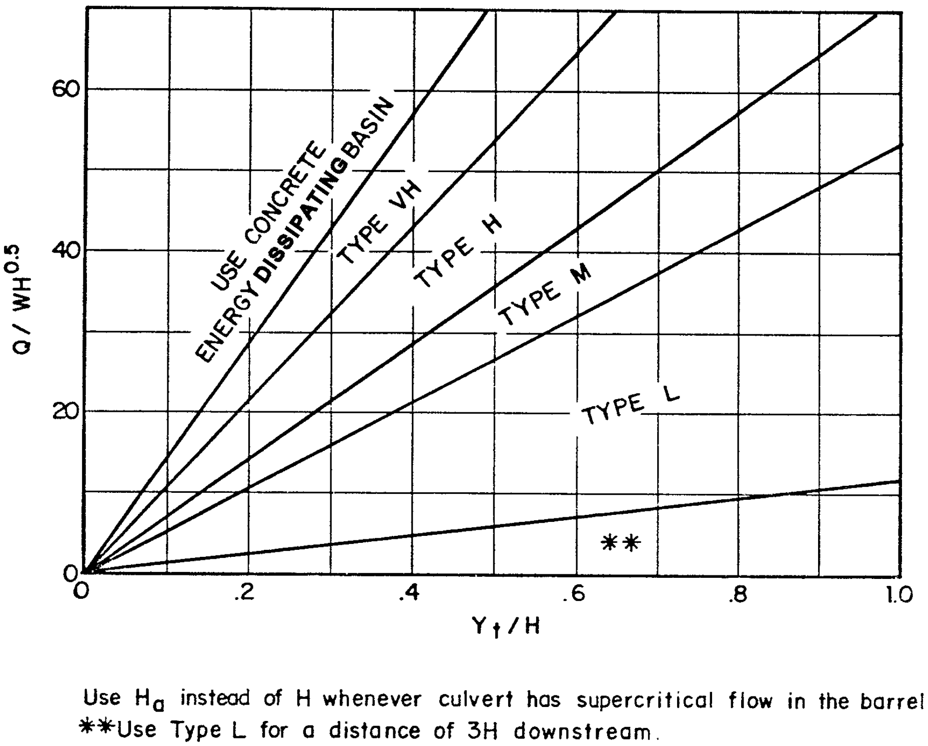


Figure 9-39. Riprap erosion protection at rectangular conduit outlet (valid for Q/WH1.5 ≤ 8.0)

3.2.4 Outfalls and Rundowns

A grouted boulder outfall or "rundown" dissipates energy and provides erosion control protection. Grouted boulder outfalls are most commonly used in large rivers like the South Platte. Figure 9-40 provides a plan view and cross section for a standard grouted boulder rundown. See the grouted boulder drop profiles (A1, A2, and A3) in Figure 9-12 for site specific profile options, (i.e., depressed or free-draining basin for use with a stable downstream channel or with no basin for use in channels subject to degradation). Figure 9-41 provides a plan view of the same structure for use when the structure is in-line with the channel. Evaluate the following when designing a grouted boulder outfall or rundown:

- Minimize disturbance to channel bank
- Determine water surface elevation in receiving channel for base flow and design storm(s)
- Determine flow rate, velocity, depth, etc. of flow exiting the outfall pipe for the design storm(s)
- Evaluate permitting procedures and requirements for construction adjacent to large river system.

Use the criteria presented in Section 2.6 for grouted boulder drop structures as a reference for guidance in the design of a grouted boulder outfall. Those criteria for grout depth, side slopes, and boulder placement also apply to grouted boulder outfalls and rundowns.

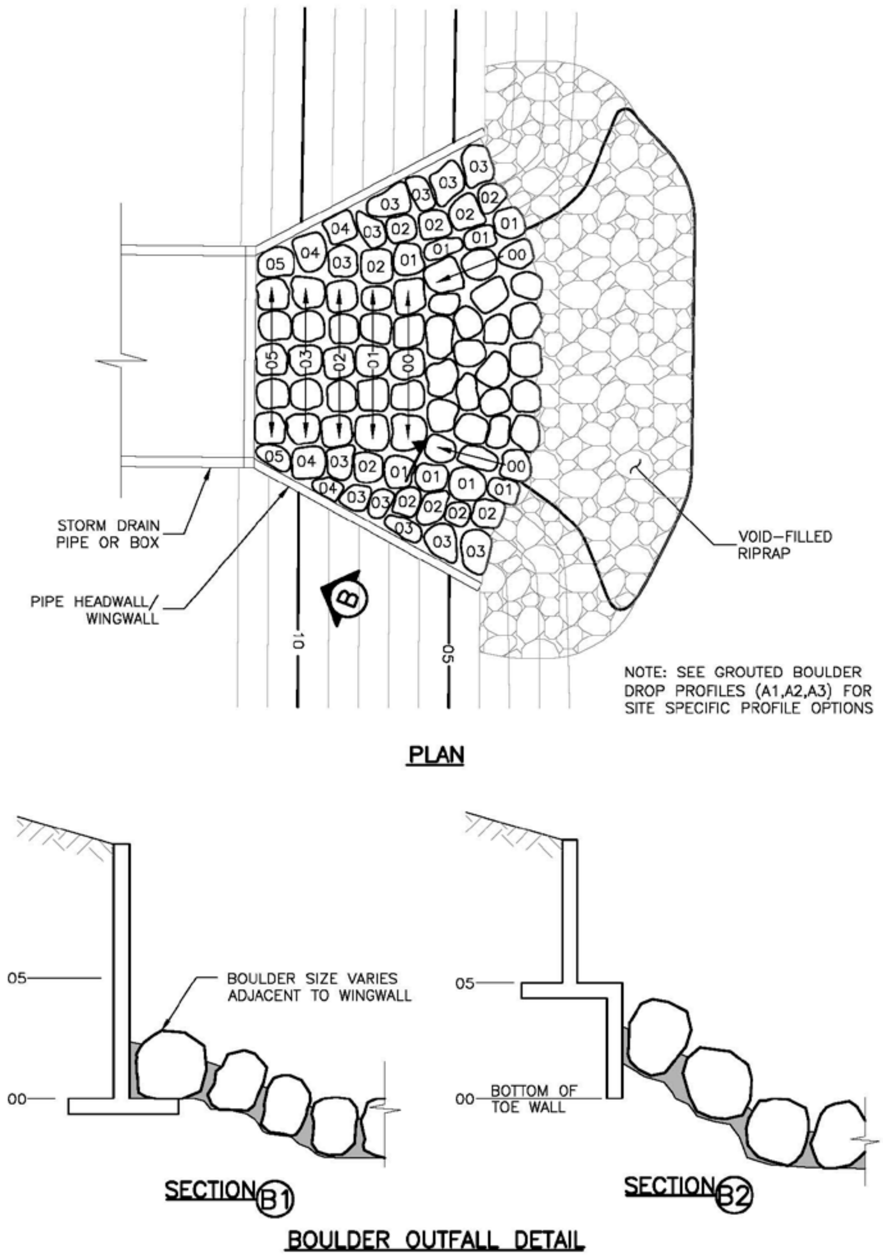


Figure 9-40. Boulder outfall detail

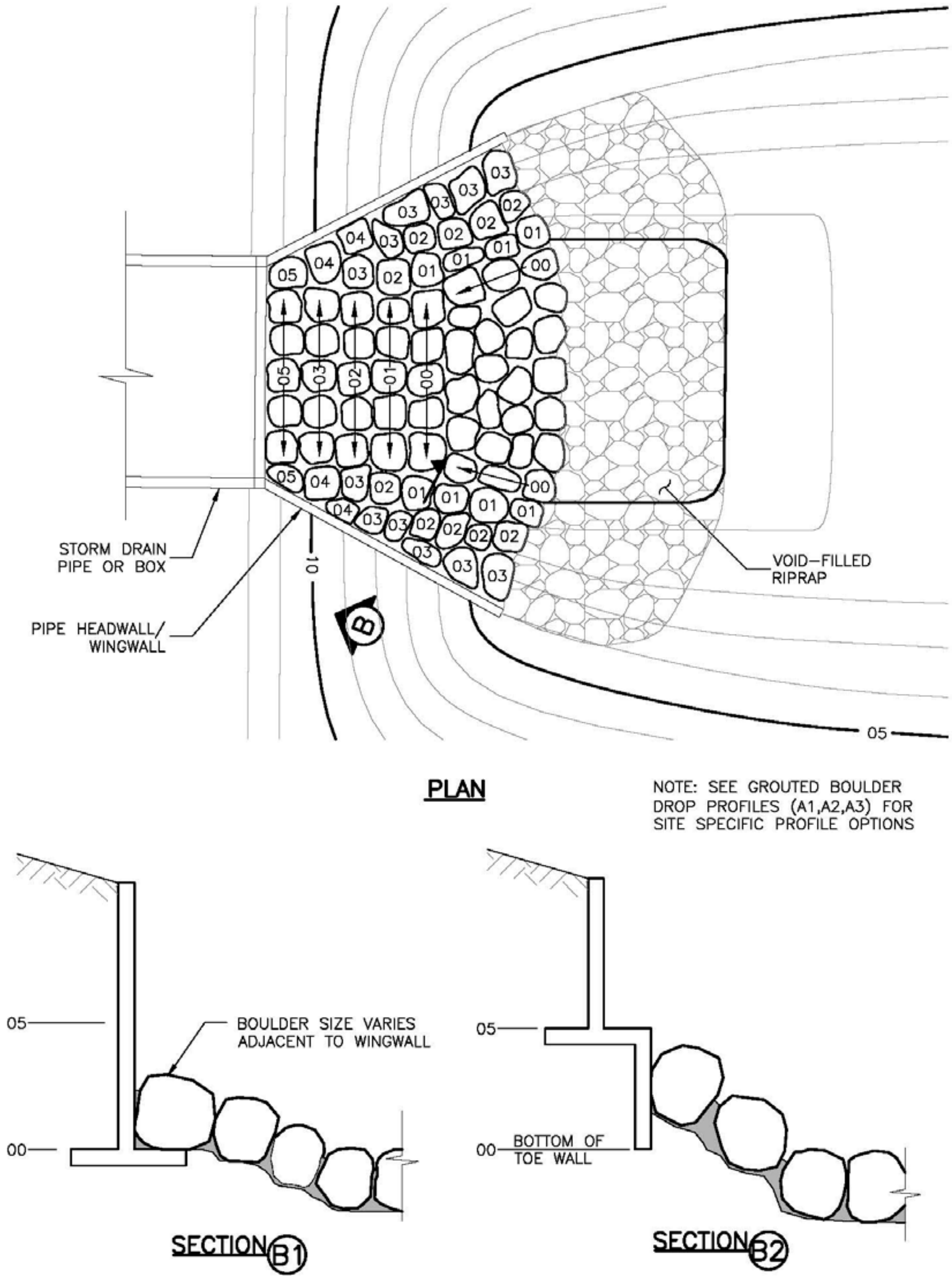


Figure 9-41. Boulder outfall detail (in-line with channel)

Energy dissipation or stilling basin structures are required to minimize scour damages caused by high exit velocities and turbulence at conduit outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets, as discussed in Section 3.2.1, is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical. Reinforced concrete outlet structures are suitable for a wide variety of site conditions. In some cases, they are more economical than larger rock basins, particularly when long-term costs are considered. The impact basin is an “all-emcompassing” structure that does not require a separate design for the pipe end treatment.

Any outlet structure must be designed to match the receiving stream conditions. The following steps include an analysis of the probable range of tailwater and bed conditions that can be anticipated including degradation, aggradation, and local scour.

Use of concrete is often more economical due to structure size or local availability of materials. Initial design selection should include consideration of a conduit outlet structure if any of the following situations exist:

- High-energy dissipation efficiency is required, where hydraulic conditions approach or exceed the limits for alternate designs (see the *Open Channels* chapter);
- Low tailwater control is anticipated; or
- Site conditions, such as public use areas, where plunge pools and standing water are unacceptable because of safety and appearance, or at locations where space limitations direct the use of a concrete structure.

Impact Basins for Small Outlets

Figures 9-43 and 9-44 provide design layout for circular outlets up to 48 inches in diameter. Unlike the Type VI impact basin used for large outlets, the modified basin does not require sizing for flow under velocities recommended in the *Streets, Inlets, and Storm Drains* chapter. However, use of this detail is limited to exit velocities of 18 feet per second or less. For larger conduits and higher exit velocities, use the Type VI impact basin.

Impact Basins for Large Outlets

Conduits with large cross-sectional areas are for significant discharges and often with high velocities requiring special hydraulic design at their outlets. Here, dam outlet and spillway terminal structure technology is appropriate (USBR 1987). Type II, III or VI (USBR nomenclature) stilling basins, submerged bucket with plunge basin energy dissipators and slotted-grating dissipators can be considered when appropriate to the site conditions. For instance, a plunge basin may have applicability where discharge is to a retention pond or a lake. Alternate designs of pipe exit energy dissipators provided in this chapter can be matched to a variety of pipe sizes, pipe outlet physical configurations, and hydraulic conditions.

Most design standards for an impact stilling basin are based on the USBR Type VI basin, often called “impact dissipater” or conduit “outlet stilling basin.” This basin is a relatively small structure that is very efficient in dissipating energy without the need of tailwater. The original hydraulic design reference (Biechley 1971) is based on model studies. Additional structural design details are provided by Aisenbrey, et al. 1974; and Peterka 1984.

The Type VI basin was originally designed to operate continuously at the design flow rate. However, it is applicable for use under the varied flow conditions of stormwater runoff. The USBR Type VI Impact Basin design configuration is shown in Figure 9-43, which consists of an open concrete box attached directly to the conduit outlet. The width, *W*, is a function of the Froude number and can be determined using Figure 9-46. The sidewalls are high enough to contain most of the splashing during high flows and slope down to form a transition to the receiving channel. The inlet pipe is vertically aligned with an overhanging L-shaped baffle such that the pipe invert is not lower than the bottom of the baffle. The end check height is equal to the height under the baffle to produce tailwater in the basin. UDFCD modified this USBR structure to provide a means of draining the structure to improve maintenance conditions and avoid development of mosquito habitat. Low-flow modifications have not been fully tested to date. Avoid compromising the overall hydraulic performance of the structure.

Multiple Conduit Entry to an Impact Basin

Where two or more conduits of different sizes outlet in close proximity to each other, a composite structure can be constructed to eliminate common walls. This can be somewhat awkward since each basin "cell" must be designed as an individual basin with different height, width, etc. Where feasible, a more economical approach is to combine storm drains at a manhole or vault and bring a single, combined pipe to the outlet structure.

When using the modified Type VI impact basin for two side-by-side pipes of the same size, the two pipes may discharge into a single basin. In this scenario, increase the width of the basin by a factor of 1.5. When the flow is different for the two conduits, the width of the basin is based on the pipe carrying the higher flow. For the modified impact basin shown in 9-43, add 1/2 D space between the pipes and to each outside pipe edge when two pipes discharge into the basin to determine the width of the headwall and extent the width of the impact wall to match the outside edges of the two pipes. The effect of mixing and turbulence of the combined flows in the basin was not modeled.

The remaining structure dimensions are based on the design width of a separate basin *W*. If the two pipes have different flow, the combined structure is based on the higher Froude number. Install handrails, access control grating, or a hinged rack around the open basin areas where safety is a concern.

General Design Procedure for Type VI Impact Basin

1. Calculate the Froude number. Determine the design hydraulic cross-sectional area inside the pipe at the outlet. Determine the effective flow velocity, *V*, at the same location in the pipe. Assume the depth of flow (D), is equal to the square root of the flow area inside the pipe at the outlet.

$$\text{Froude number} = \frac{V}{(gD)^{1/2}}$$

2. Place the entrance pipe horizontally at least one pipe diameter equivalent length upstream from the outlet. For pipe slopes greater than 15 degrees, the horizontal length should be a minimum of two pipe diameters.

3. Determine the basin width, *W*, by entering the Froude number and effective flow depth into Figure 9-40. The remaining dimensions are proportional to the basin width according to Figure 9-39. No not oversize the basin width. Larger basins become less effective as the inflow can pass under the baffle.

4. Design structure wall thickness, steel reinforcement, and anchor walls (underneath the floor) using accepted structural engineering methods. Note the baffle thickness, t_b, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis. Hydraulic forces on the overhanging baffle may be approximated by determining the hydraulic jet force at the outlet:

$$F_j = 1.94\, V_{out}\, Q \text{ (force in pounds)} \qquad \text{Equation 9-20}$$

Q = maximum design discharge (cfs)
V_{out} = velocity of the outlet jet (ft/sec)

Provide type "M" soil riprap or void-filled riprap in the receiving channel from the end check to a minimum distance equal to the basin width.

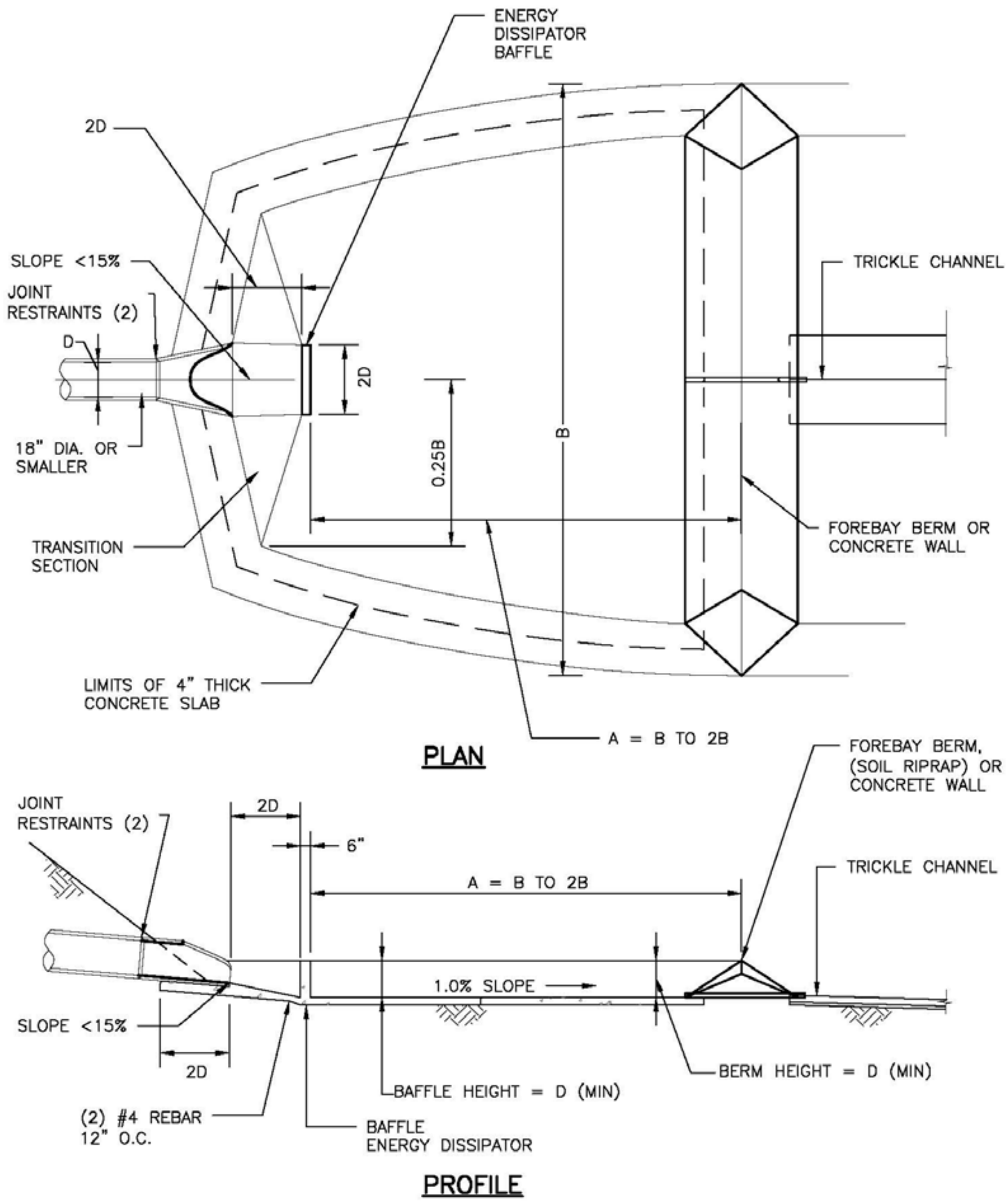


Figure 9-42. Impact stilling basin for pipes smaller than 18" in diameter

(Source: City and County of Denver 2006)

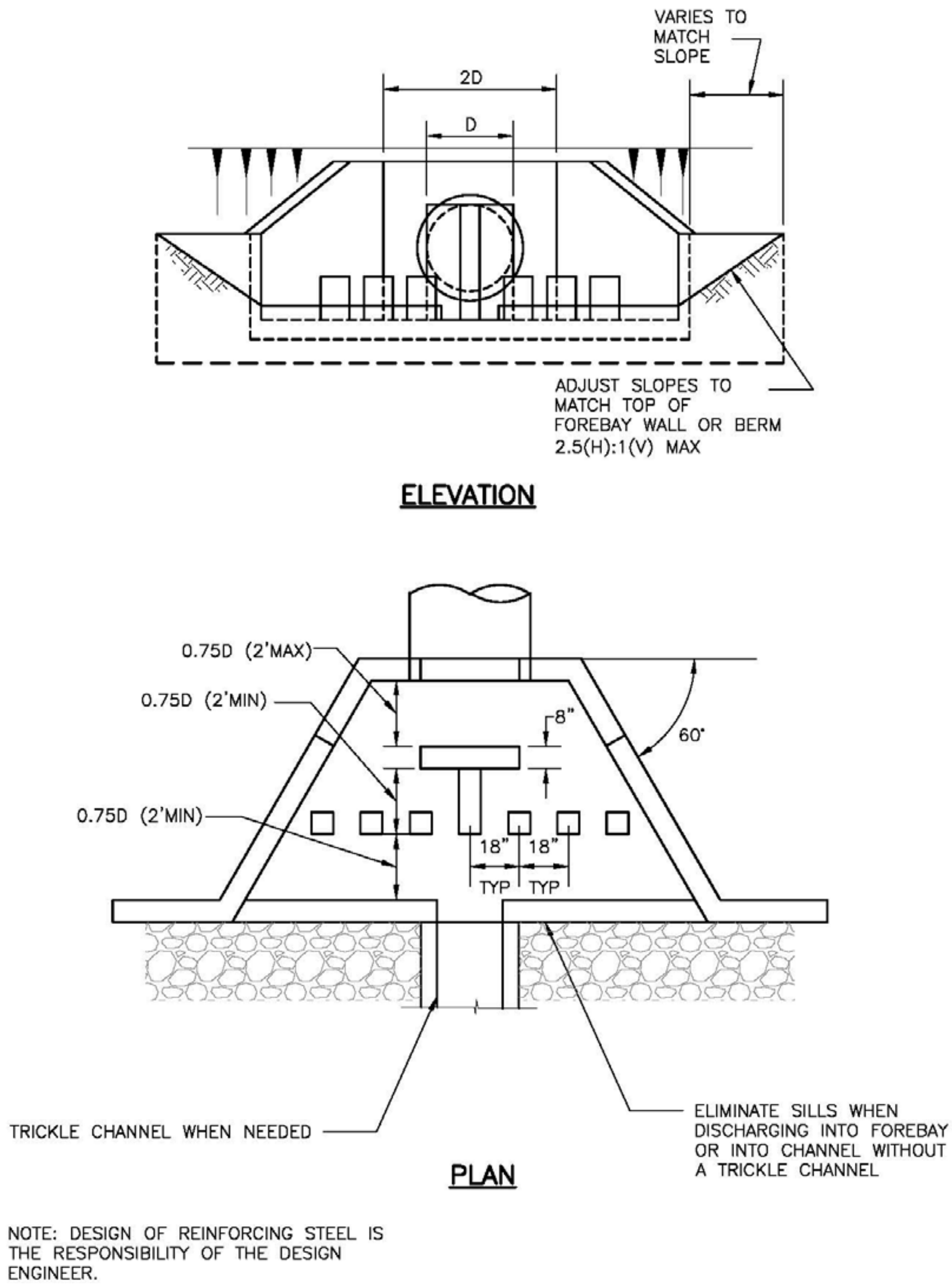


Figure 9-43. Modified impact stilling basin for conduits 18” to 48” in diameter (Part 1 of 2)

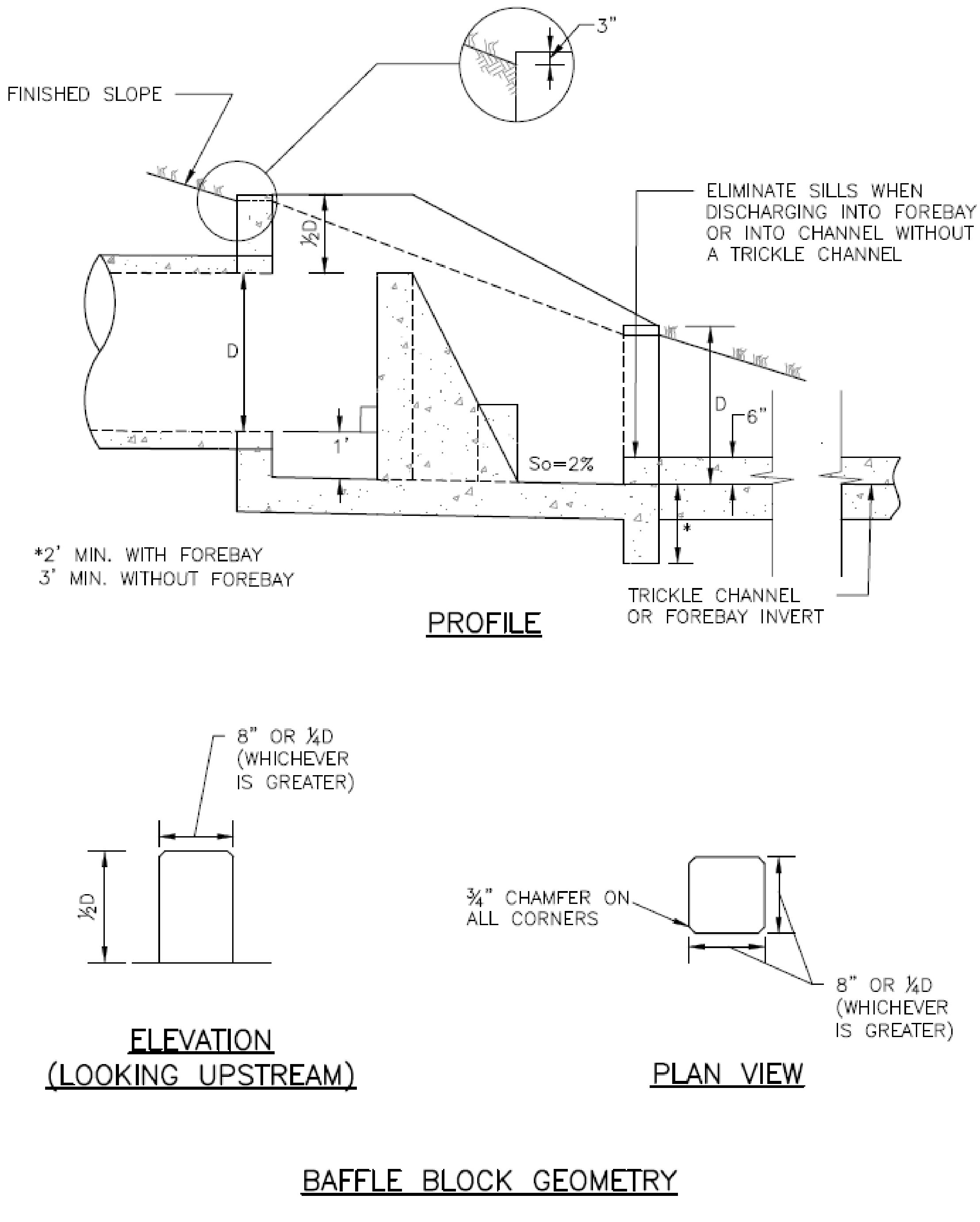


Figure 9-44. Modified impact stilling basin for conduits 18” to 48” in diameter (Part 2 of 2)

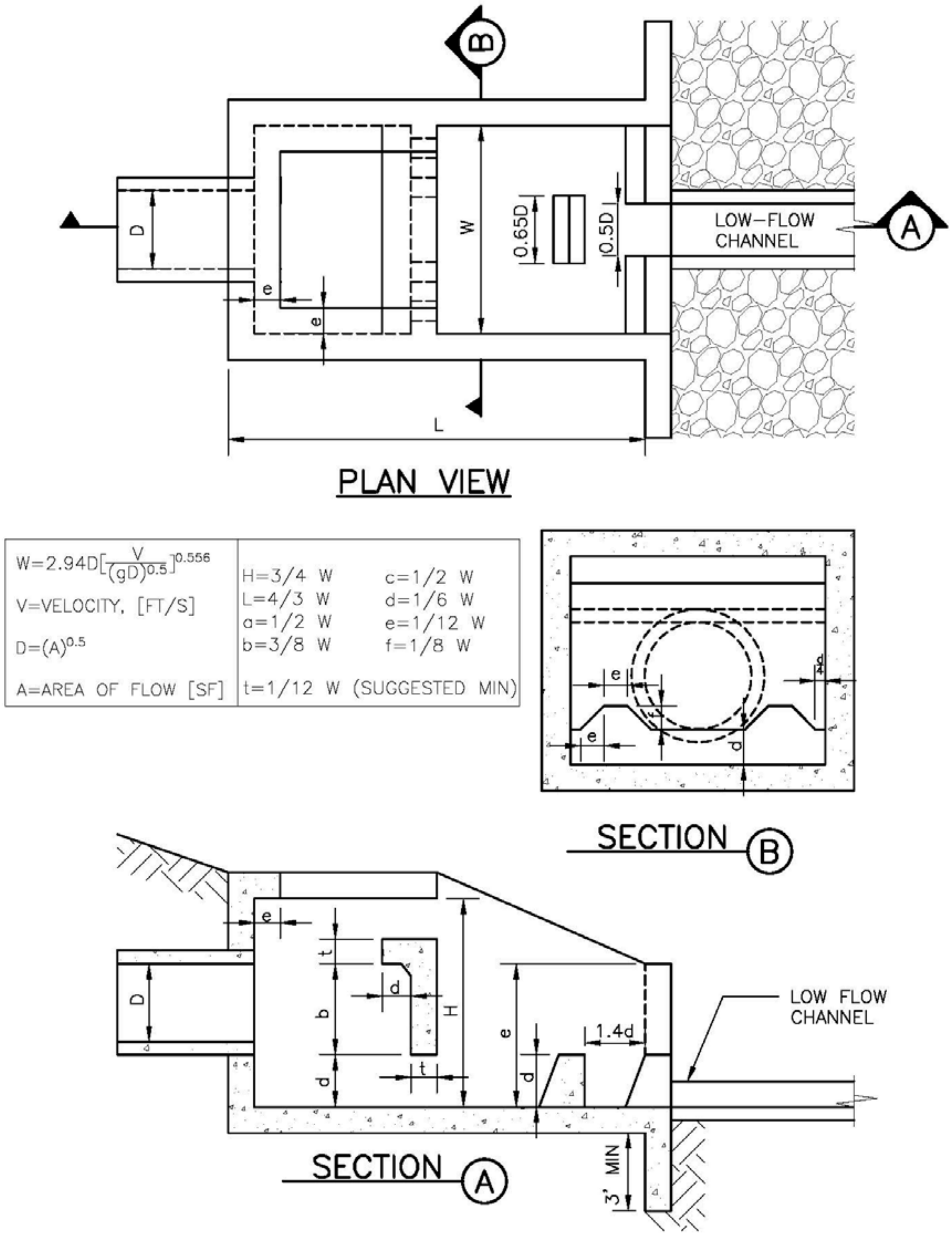
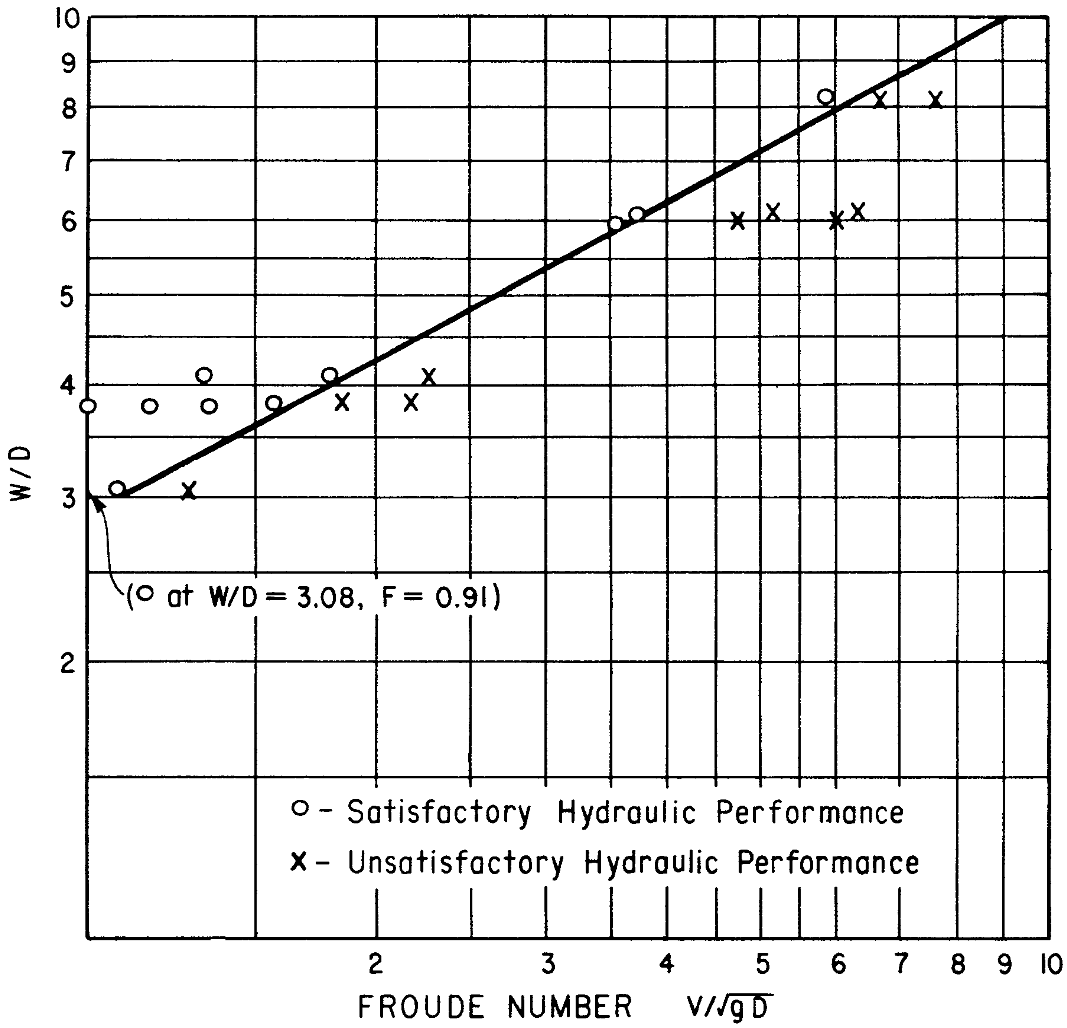


Figure 9-45. UDFCD modified USBR type VI impacts stilling basin (general design dimensions)



"W" is the inside width of the basin.
"D" represents the depth of flow entering the basin and is the square root of the flow area at the conduit outlet.
"V" is the velocity of the incoming flow.
The tailwater depth is uncontrolled.

Figure 9-46. Basin width diagram for the USBR type VI impact stilling basin

3.2.5 Rundowns

Rundowns are used to convey storm runoff from the bank of a channel to the invert of an open channel. Rundowns can also convey runoff from streets and parking lots into channels or storage facilities. The use of rundowns is discouraged due to their high rate of failure and resulting unsightly structures that become a maintenance burden. The preferred alternative is to spread flows over the embankment using a level spreader. See the Grass Buffer Fact Sheet located in Chapter 4 of Volume 3 for guidance on level spreaders. If the flow is too great to be distributed and conveyed down the slope of an open channel, use a pipe to convey flows closer to the invert of the stream or use a drop structure. For both of these options, provide adequate erosion protection at the downstream end.

In the case when a rundown is the only viable option, use the following design criteria.

Design Flow
The rundown should be designed to carry the full design flow of the tributary area upstream (see *Runoff* chapter), or 1 cfs (assuming critical depth) with freeboard, whichever is greater.

Cross Section
Construct the rundown with grouted boulder invert and edge treatment. The top of edge treatment should be flush with proposed grade. Ensure a minimum of 1 foot of freeboard from the calculated design flow depth from the invert to the top of the grouted boulders. Do not use riprap or soil riprap rundowns as they frequently fail.

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Appendix A. Force Analysis for Grade Control Structures

Each component of a drop structure has forces acting upon it that require evaluation. This section describes the general forces, with the exception of forces on riprap for which the reader is referred to Isbash 1936; Oliver 1967; Smith 1975; Smith and Strung 1967; Stevens 1976; Taggart 1984; Abt 1986 and 1987; WittIer and Abt 1988; Maynord and Ruff 1987; Richardson 1988; and LSA 1986 and 1989. It is worth noting that the boulders are subject to all of the usual forces plus the hydrodynamic forces of interflow through voids and related pressure fluctuations. A complete presentation of forces acting on riprap and boulders is not presented herein. Forces are described here, as they would apply to sloping grouted boulder and reinforced concrete drop structures.

The various criteria for structural slab thicknesses given for each type of drop structure have generally taken these forces into consideration. It is the user's responsibility to determine the forces involved.

Five location points are of concern. Point 1 is downstream of the toe, at a location far enough downstream to be beyond the point where the deflection (turning) force of the surface flow occurs. Point 2 is at the toe where the turning force is encountered. Point 3 is variable in location to reflect alternative drain locations. When a horizontal drain is used, Point 3 is at a location where the drain intercepts the subgrade of the structure. Point 4 is approximately 50% of the distance along the drop face. Point 5 is at a point underneath the grout layer at the crest and downstream of the cutoff wall.

Point 3 is usually the critical pressure location, regardless of the drain orientation. In some cases, Point 1 may also experience a low safety factor when shallow supercritical flow occurs, such as when the jump washes downstream.

Seepage uplift is often an important force controlling structure stability. Weep drains, the weight of the structure, and the water on top of the structure counteract uplift. The weight of water is a function of the depth of flow. Thus, greater roughness will produce deeper flow resulting in greater weight.

Shear Stress

The normal shear stress equation is transformed for unit width and the actual water surface profile by substituting S_e, the energy grade line slope for S_o, the slope of the drop face.

$$\tau = \gamma y S_e \qquad \text{Equation A-1}$$

Where:

τ = shear stress (lbs/ft^2)

γ = specific weight of water (lbs/ft^3)

y = depth of water at analysis point (ft)

Buoyant Weight of Structure

Each design should take into consideration the volume of grout and rock or reinforced concrete and the density of each. In the case of reinforced concrete, 150 pounds per cubic foot can be used as the specific weight (or 88 pounds per cubic foot net buoyant weight). Specific weight of rock is variable depending on the nature of the material.

Impact, Drag and Hydrodynamic Lift Forces
Water flowing over the drop will directly impact any abrupt rock faces or concrete structure projections into the flow. Technically, this is considered as a type of drag force, which can be estimated by equations found in various references. Impact force caused by debris or rock is more difficult to estimate because of the unknown size, mass, and time elapsed while contact is made. Therefore, it is recommended that a conservative approach be taken with regard to calculating water impact (drag force), which generally will cover other types of impact force. Specialty situations, where impact force may be significant, must be considered on an individual basis. In addition, boulders and riprap are subject to hydrodynamic lift forces (Urbonas 1968) that are caused by high velocities over the top of the stones and the zones of separation they create, resulting in significant reduction in pressure on the top while hydrostatic pressure remains unchanged at the stone's bottom.

Turning Force
A turning force impacts the basin as a function of slope change. Essentially, this is a positive force countering uplift and causes no great stress in the grouted rock or reinforced concrete. This force can be estimated as the momentum force of the projected jet area of water flowing down the slope onto the horizontal base and calculating the force required to turn the jet.

Friction
With net vertical weight, it follows that there would be a horizontal force resisting motion. If a friction coefficient of 0.5 is used and multiplied by the net weight, the friction force to resist sliding can be estimated.

Frost Heave
This value is not typically computed for the smaller drop structures anticipated herein. However, the designer should not allow frost heave to damage the structure and, therefore, frost heave should be avoided and/or mitigated. In reinforced concrete, frost blankets, structural reinforcing, and anchors are sometimes utilized for cases where frost heave is a problem. If gravel blankets are used, then the seepage and transmission of pressure fluctuations from the hydraulic jump are critical.

Seepage Uplift Pressure
As explained previously, uplift pressure and seepage relief considerations are critical to structural stability and usually of greater concern than the forces described above. There can be troublesome pressure differentials from either the upstream or downstream direction when there is shallow supercritical flow on the drop face or in the basin. One may consider an upstream cutoff to mitigate this problem. Weep locations with proper seepage control may be provided. For high drop structures (i.e., > 6 feet), more than one row of weep holes may be necessary.

A prudent approach is to use a flow net or other type of computerized seepage analysis to estimate seepage pressures and flows under a structure.

Dynamic Pressure Fluctuations
Laboratory testing (Toso 1986; Bowers and Toso 1988) has documented that the severe turbulence in a hydraulic jump can pose special problems often ignored in hydraulic structures. This turbulence can cause significant positive and negative pressure fluctuations along a structure. The key parameter is the coefficient of maximum pressure fluctuation, $C_{p\text{-}max}$, which is in terms of the velocity head of the supercritical flow just upstream of the jump:

$$C_{p-\max} = \frac{\Delta P}{\left(\frac{V_u^2}{2g}\right)} \qquad \text{Equation A-2}$$

Where:

ΔP = pressure deviation (fluctuation) from mean (ft)
V_u = incident velocity (just upstream of jump) (ft/sec)
g = acceleration of gravity (ft/sec^2)

Effectively, C_P is a function of the Froude number of the supercritical flow. The parameter varies as a function of X, which is the downstream distance from the beginning of the jump to the point of interest.

Dynamic Pressure Fluctuation Example

A good example of this is when an entire sloping face of a drop is underlain by a gravel seepage blanket. The gravel could be drained to the bottom of the basin or other locations where the jump will occur. In such a case, the positive pressure fluctuations could be transmitted directly to the area under the sloping face, which then could destabilize the structure since there would not be sufficient weight of water over the structure in the area of shallow supercritical flow.

Table 9-6 presents recommended $C_{p\text{-}max}$ positive pressure values for various configurations. When the Froude number for the design case is lower than those indicated, the lowest value indicated should be used (do not reduce on a linear relationship) for any quick calculations. The values can be tempered by reviewing the C_p graphs, a few of which are given in Figures A-1 and A-2. Note that the graphs are not maximum values but are the mean fluctuation of pressure. The standard deviation of the fluctuations is also indicated, from which the recommended $C_{p\text{-}max}$ values were derived.

Figure A-1 illustrates positive and negative pressure fluctuations in the coefficient, C_p, with respect to the location where the jump begins at the toe. Figure A-2 presents the positive pressure fluctuation coefficient where the jump begins on the face.

For the typical basin layouts given and where the drains are at the toe and connect directly to the supercritical flow, these pressure fluctuations should not be of great concern. However, when drains discharge to the jump zone and could transfer pressure fluctuations to areas under supercritical flow, pressure fluctuations are of concern.

Table 9-6. Nominal limit of maximum pressure fluctuations within the hydraulic jump (Toso 1986)

Jump Condition	Froude Number	Suggested Maximum C_p
0° slope, developed inflow (boundary layer has reached surface)	3.0	1.0
30° slope, toe of jump at base of chute[1]	3.8	0.7
30° slope, toe of jump on chute[1]	3.3	0.8

[1] Velocity head increased by elevation difference between toe of jump and basin floor, namely, depth at the drop toe.

Overall Analysis
All of the above forces can be resolved into vertical and horizontal components. The horizontal components are generally small (generally less than 1 psi) and capable of being resisted by the weight of the grout, rock, and reinforced concrete. When problems occur, they are generally the result of a net vertical instability.

The overall (detailed) analysis should include reviews of the specific points along the drop structure and the overall drop structure geotechnical and structural stability. All steps of this detailed analysis are not necessary for design of drop structures along modest capacity grass-lined channels, provided that the design is developed using the guidelines and configurations presented in the following simplified analysis approach section and that other USDCM criteria are met. The critical design factors are seepage cutoff and relief and pressure fluctuations associated with the hydraulic jump that can create upward forces greater than the weight of water and structure over the point of interest. Underflow can easily lift a major slab of rock and grout and, depending upon the exposure, the surface flow could cause further weakening, undermining, or displacement. Generally, a 30-pound net downward safety allowance should be provided, and 60 pounds is preferred. An underdrain is generally needed to prevent hydrostatic uplift on the stones.

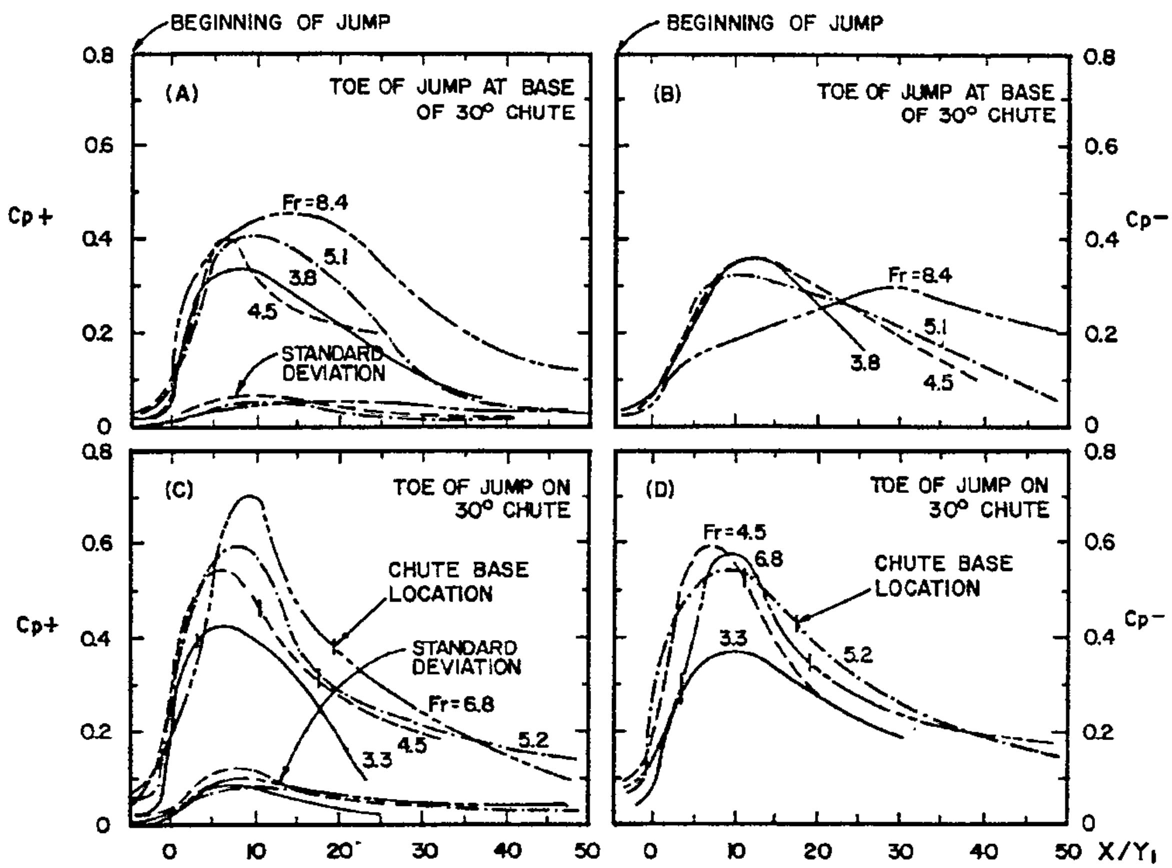

Figure A-1. Coefficient of pressure fluctuation, Cp, at hydraulic jump

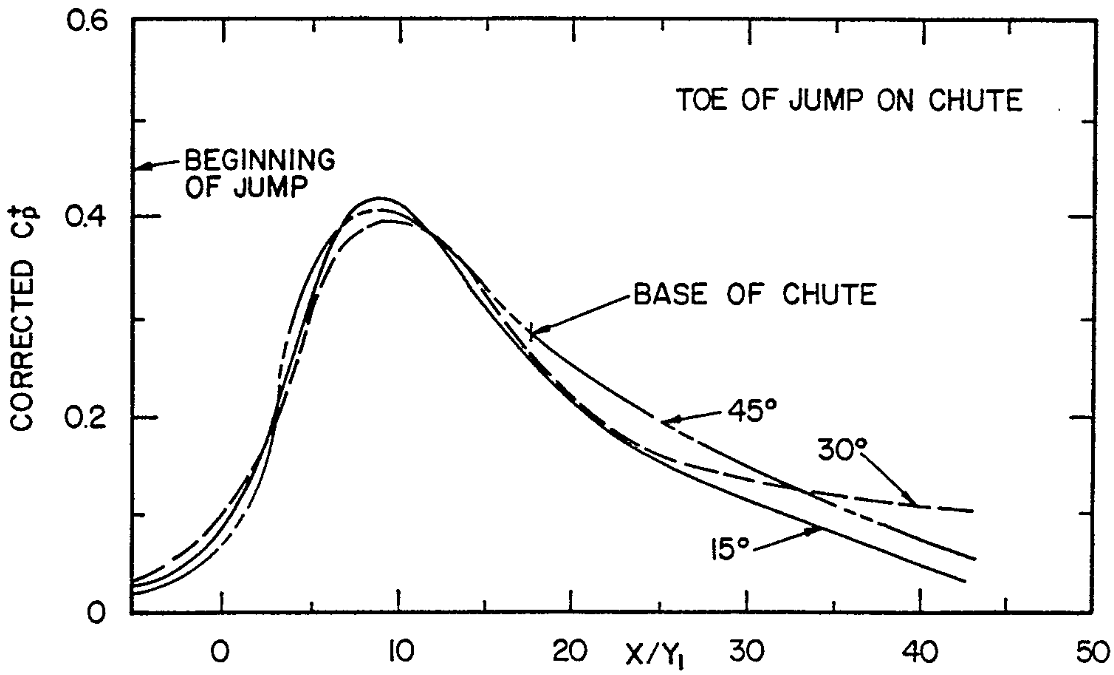

Figure A-2. Coefficient of pressure fluctuation, Cp, normalized for consideration of slope and jump beginning slope