HYDRAULIC STRUCTURES

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1.0 USE OF STRUCTURES IN DRAINAGE

1.1 Introduction

Hydraulic structures are used to guide and control water flow velocities, directions and depths, the elevation and slope of the streambed, the general configuration of the waterway, and its stability and maintenance characteristics.

Careful and thorough hydraulic engineering is justified for hydraulic structures. Consideration of environmental, ecological, and public safety objectives should be integrated with hydraulic engineering design. The proper application of hydraulic structures can reduce initial and future maintenance costs by managing the character of the flow to fit the environmental and project needs.

Photograph HS-1—Denver's Harvard Gulch Flood Control Project introduced the baffle chute drop structure to urban flood control in 1966. Vegetation and time have made the structure part of the city's urban poetry.

Hydraulic structures include transitions, constrictions, channel drops, low-flow checks, energy dissipators, bridges, bends, and confluences. Their shape, size, and other features vary widely for different projects, depending upon the discharge and the function to be accomplished. Hydraulic design procedures must govern the final design of all structures. These may include model testing for larger structures when the proposed design requires a configuration that differs significantly from known documented guidelines or when questions arise over the character of the structure being considered.

This chapter deals with structures for drainage and flood control channels, in contrast to dam spillways or specialized conveyance systems. Specific guidance is given on drop structures for channels that match the District's guidelines for grass-lined and riprap-lined channels as given in the MAJOR DRAINAGE chapter of this *Manual*. In addition, guidance is provided for the design of energy dissipaters at conduit outlets. Sections on bridges, transitions, and constrictions primarily refer to other sources for more extensive design information.

Photograph HS-2—The Clear Creek I-25 vertical concrete drop structure was a "drowning machine" until it was retrofitted by CDOT with a 10:1 downstream face. (Photograph taken before retrofit.)

1.2 Channels Used for Boating

There are streams in the District in which rafting, canoeing, kayaking, and other water-based recreational activities occur. Design and construction of hydraulic structures in these waterways require a standard of care consistent with common sense safety concerns for the public that uses them. The ultimate responsibility for individual safety still resides with the boating public and their prudent use of urban waterways.

It is reasonable to retain a whitewater boating specialist to assist in the design criteria for a hydraulic structure on a boatable stream. In particular, reverse rollers are to be avoided (USACE 1985).

1.3 Channel Grade Control Structures

Grade control structures, such as check structures and drop structures, provide for energy dissipation and thereby result in a mild slope in the upstream channel reaches. The geometry at the crest of these structures can effectively control the upstream channel stability and, to an extent, its ultimate configuration.

A drop structure traverses the entire waterway, including the portion that carries the major flood. A check structure is similar, but is constructed to stabilize the low-flow channel (i.e., one carrying the minor or lesser flood) in artificial or natural drainageways. It crosses only the low-flow portion of the waterway or floodplain. During a major flood, portions of the flow will circumvent the check. Overall channel stability is maintained because degradation of the low-flow channel is prevented. Typically, the 2-year flows are contained in the protected zone so that the low-flow channel does not degrade downward, potentially undermining the entire waterway.

1.4 Wetland Channel Grade Control

Wetland channels, whether low-flow channels or from bank to bank, require modest slopes not exceeding about 0.3%. Grade control structures are often required for stability. Due to the environmental nature of the wetlands, the grade control structures are planned and designed to be compatible with a wetland environment. Wetland channels do not need a trickle channel, but where used, the trickle channel should not lower the wetland water table more than 12 inches.

1.5 Conduit Outlet Structures

Design criteria given in this chapter are for structures specifically designed to dissipate flow energy at conduit outlets to the open waterway. These types of structures are typically located at storm sewer outlets. Design criteria for culverts and storm sewers that discharge in-line with the receiving channel are described in the MAJOR DRAINAGE chapter of this *Manual*.

1.6 Bridges

Bridges have the advantage of being able to cross the waterway without disturbing the flow. However, for practical, economic, and structural reasons, abutment encroachments and piers are often located within the waterway. Consequently, the bridge structure can cause adverse hydraulic effects and scour potential that must be evaluated and addressed as part of each design project.

1.7 Transitions and Constrictions

Channel transitions are typically used to alter the cross-sectional geometry, to allow the waterway to fit within a more confined right-of-way, or to purposely accelerate the flow to be carried by a specialized high velocity conveyance. Constrictions can appreciably restrict and reduce the conveyance in a manner that is either detrimental or beneficial. For example, a bridge, box culvert, or constriction may increase the upstream flooding by encroaching too far into the floodplain conveyance, whereas in another situation a hydraulic control structure can be employed to purposely induce an upstream spill into an off-stream storage facility.

1.8 Bends and Confluences

General considerations for lined channels and conduits are discussed in the MAJOR DRAINAGE chapter of this *Manual*. Additional emphasis is added herein for certain situations. Channels and conduits that produce supercritical flow may require special structural or design considerations. This discussion is limited since these types of structures are generally associated with hydraulic performance exceeding the recommended criteria for grass-lined channels. Extensive study, specialized modeling and/or analysis may be required for these situations.

On the other hand, confluences are commonly encountered in design. Relative flow rates can vary disproportionately with time so that high flows from either upstream channel can discharge into the downstream channel when it is at high or low level. Depending on the geometry of the confluence, either condition can have important consequences, such as supercritical flow and hydraulic jump conditions, and result in the need for structures

1.9 Rundowns

A rundown is used to convey storm runoff from high on the bank of an open channel to the low-flow channel of the drainageway or into a detention facility. The purpose is to control erosion and head cutting from concentrated flow. Without such rundowns, the concentrated flow will create erosion.

1.10 Energy Dissipation

The energy of moving water is known as kinetic energy, while the stored energy due to elevation is potential energy. A properly sloped open channel will use up the potential energy in a uniform manner through channel roughness without the flow being accelerated. A grade control structure (i.e., drop and check) converts potential energy to kinetic energy under controlled conditions. Selection of the optimum spacing and vertical drop is the work of the hydraulic engineer. Many hydraulic structures deal with managing kinetic energy—to dissipate it in a reasonable manner, to conserve it at structures such as transitions and bridges, or occasionally to convert kinetic to potential energy using a hydraulic jump. Thus, managing energy involves understanding and managing the total energy grade line of flowing water.

1.11 Maintenance

Urban drainage facilities should not be built if they cannot be properly maintained on a long-term basis. This means that suitable access must be provided, a maintenance plan must be developed and funded, and the drainage facilities must be maintained in accordance with public works standards.

1.12 Structure Safety and Aesthetics

The design of structures must consider safety of flood control workers and the general public, especially when multiple uses are intended. Regulations and interpretations vary from community to community and may change with time. There are some inherent safety risks in any waterway that have to be recognized by the public, designers, and government officials. General suggestions are given in regard to safety; however, the designer must use a reasonable standard of care for the particular structure being designed or retrofitted that includes evaluation of present or likely future public access and uses such as recreation. The designer should give special consideration to structures located in waterways where boating is likely to occur. These structures need to be designed to avoid known hazards, such as reverse rollers (Leutheusser and Birk 1991), often referred to by some as “keepers.”

Aesthetic appearance of structures in urban areas is also important. Structures can be designed with various configurations, different materials, and incorporation of adjacent landscaping to produce a pleasing appearance and good hydraulic function and to enhance the environmental and ecological character of the channel and floodplain. The incorporation of wetland vegetation, native grasses, and shrubs into the design adds to their aesthetics and provides erosion control and water quality functions.

Photograph HS-3—Stepped grouted sloping boulder drop structures such as in Denver’s Bible Park can be safe, aesthetic, and provide improved aquatic habitat besides performing their primary hydraulic function of energy dissipation.

2.0 CHANNEL GRADE CONTROL STRUCTURES (CHECK AND DROP STRUCTURES)

2.1 Planning for the Future

Channel grade control structures (typically check structures and drop structures) should be designed for future fully developed basin conditions. In the use of a natural channel, the effects of future hydrology and potential down cutting must be included so that the natural channel is properly stabilized. Urbanization will create a base flow that, over time, will cause down cutting if not managed with grade control structures.

"Drop structures" are broadly defined. They establish a stable stream grade and hydraulic condition. Included are structures built to restore damaged channels, those that prevent accelerated erosion caused by increased runoff, and grade control drops in new channels. Drop structures provide special hydraulic conditions that allow a drop in water surface and/or channel grade. The supercritical flow may go through a hydraulic jump and then return to subcritical flow.

The focus of these criteria is on channel drops with primary emphasis on grass-lined channels. Check structures may be used to stabilize the natural low-flow channel in an unmodified floodplain. Thus, check structures also require additional consideration of the wider major flood path extending around the structure abutments.

Specific design guidance is presented for the following basic categories of drop structures: baffle chute drops (BCD), grouted sloping boulder drops (GSB), and vertical hard basin drops (VHB).

All drop structures should be evaluated after construction. Bank and bottom protection and adjustments may be needed when secondary erosion tendencies are revealed. It is advisable to establish construction contracts and budgets with this in mind. Use of standardized design methods for the types of drops suggested herein will reduce the need for secondary design refinements.

The design of the drop structure crest and provisions for the trickle or low-flow channel directly affect the ultimate configuration of the upstream channel. A shallow and/or dispersed trickle configuration will tend to result in some aggradation and a wetter channel bottom than might be associated with a wetland channel bottom. However, the wetland channel design would not contain a trickle channel because the low flows would be spread out uniformly across the entire channel bottom.

A higher unit flow will pass through the trickle or low-flow area than will pass through other portions of the channel cross section. This situation must be considered in design to avoid destabilization of the drop and the channel.

2.1.1 Outline of Section

The following section provides guidelines to aid in the selection of alternate types of drops, particularly those used for grass-lined channels. Drops for boatable channels are described separately.

Much of the section is oriented toward hydraulic design and criteria for drop structures. There are two levels of analysis given. One level of hydraulic analysis is “detailed.” All steps that are important are described, along with design aids. The other level is “simplified.” Layouts of typical drops, particularly the crest configuration and related channel, are given which result in grass-lined channel hydraulic performance at the maximum depths and velocities normally allowed by the District for these types of channels. The use of these charts allows a quicker start, but certain steps from the “detailed” analysis will still be necessary, particularly the effects of greater unit flows in the low-flow or trickle channel area.

Hydraulic analysis sections are followed by further details appropriate to each of the types of drops that are recommended for grass-lined channels and boatable channel drops. Then, further information on seepage analysis, construction concerns, and low-flow channel structures is given.

Photograph HS-4—This grade control structure on the South Platte River was a hazard to the boating public until it was retrofitted by the CDOT. Here, a rescue is supervised by Colorado Governor Richard Lamm who was enjoying a rafting trip with friends and the Denver Water Rescue Team.

2.1.2 Boatable Channels

Channels that are known to be boatable, either now or that will be in the future, and those others that are classified by the Colorado Water Quality Control Commission for Class 1 or 2 Recreation, but are not presently judged to be boatable, should have hydraulic structures designed with public safety as a special consideration. The designer should not set the stage for hazardous hydraulics that would trap a boater, such as at a drop structure having a reverse roller that may develop as the hydraulic jump becomes submerged.

Designs for boatable channels, grade control structures, and low-head dams have to prevent the development of submerged hydraulic jumps, have a gently sloped or stepped downstream face, and not have a deep stilling basin that would encourage the creation of a submerged hydraulic jump. One design approach is to direct the hydraulic momentum at the bottom of the drop at a relatively flat angle to help prevent a reverse roller. A downstream face on a drop having large grouted boulders and high roughness that is sloped at 10(H) to 1(V) has been used successfully on several projects along the South Platte River and on Clear Creek, permitting safe passage of boaters as the move over them.

Drop structures or low-head dams in boatable channels should incorporate a boat chute designed in accordance with carefully planned components that are consistent with recreational requirements for boater safety. Often, physical model studies are used to verify the efficacy of the proposed design.

Hydraulic structures on boatable channels should not create obstructions that would pin a canoe, raft or kayak, and sharp edges should be avoided.

2.1.3 Grass and Wetland Bottom Channels

Structures for grass and wetland bottom (i.e., non boatable) channels are described in detail on the following pages and are represented by a variety of choices and shapes to suit the particular site and related hydraulics.

Based on experience, the sloped drop has been found to be more desirable than the vertical wall drop with a hardened energy dissipation basin. Vertical drops can create a reverse roller and backflow eddies that have been know to trap boaters. Because of boater and public safety concerns, vertical drops are less desirable than sloping drops in urban areas. Other disadvantages of a vertical drop include the turbulence and erosive effect of the falling water on the drop structure, necessitating high maintenance.

It is desirable to limit the height of most drops to 3 to 5 feet to avoid excessive kinetic energy and to avoid the appearance of a massive structure, keeping in mind that the velocity of falling water increases geometrically with the vertical fall distance. If vertical drops are use, it is best to limit their height to 3 feet.

2.1.4 Basic Approach to Drop Structure Design

The basic approach to design of drop structures includes the following steps:

1. Determine if the channel is, or will be, a boatable channel. If boatable, the drop or check structure should use a standard of care consistent with adequate public safety to provide for boater passage.

2. Define the representative maximum channel design discharge (often the 100-year) and other discharges appropriate for analysis, (e.g., low or trickle flows and other discharges expected to

occur on a more frequent basis) which may behave differently. All channels need to be designed for stability by limiting their erosion and degradation potential and for longevity by analyzing all the effects on channel stability at levels of flow, including the 100-year flood.

3. Approximate the channel dimensions and flow parameters including longitudinal slope. Identify the probable range of drop choices and heights with the aid of Figure HS-1.

4. Select drop structure alternatives to be considered for grass-lined or other channel types (see Section 2.2).

5. Decide if channel performance at maximum allowable criteria (i.e., velocity, depths, etc.) for grass-lined channels is practical or desirable. If not, or if the design flow is over 7,500 cfs, go to step 6; otherwise, the simplified design charts in Section 2.3.3 may be used to size the basic configuration of the crest. The designer should review the precautions given and the limits of application with respect to site conditions. Then the crest section and upstream channel transition will need to be refined for incorporation of the trickle or low-flow channel. This requires review of the upstream water surface profile and the supercritical flow downstream of the crest through the dissipation zone of the drop. Under conditions of a submerged jump due to a high tailwater elevation, steps to mitigate the reverse roller should be evaluated. If measures are taken to provide baffles or large boulders to break up the jet, then extensive analysis of the trickle zone hydraulics is not necessary. The steps involved are discussed in Section 2.3. Then go to step 7.

6. For refined analysis and optimal design of grass-lined channel drop structures, use the “detailed” hydraulic analysis in Section 2.3.2.

7. Perform soils and seepage analyses as necessary to obtain foundation design information.

8. In the case of drops for grass-lined channels, comply with the minimum specific criteria and follow the guidelines for the recommended types of drops (baffle chute, vertical hard basin, and grouted sloping boulder) presented in Section 2.3.4. Otherwise, provide a complete hydraulic analysis documenting the performance and design for the type of drop or other type of channel being considered. For channels with alluvial beds that present an erosion/degradation risk, a complete stability and scour analysis should be completed, accompanied by a geotechnical investigation and seepage analysis.

9. Use specific design criteria and guidelines to determine the final drop structure flow characteristics, dimensions, material requirements, and construction methods.

10. Obtain necessary environmental permits, such as a Section 404 permit.

2.2 Drop Selection

The primary concerns in selection of the type of drop structure should be functional hydraulic performance and public safety. Other considerations include land uses, cost, ecology, aesthetics, and maintenance, and environmental permitting.

Table HS-1 presents information to assist in the selection of appropriate drop structures applicable for various situations. Generally, the drops in any group are shown in order of preference. Comparative costs are often close. However, on-site conditions, such as public safety, and aesthetics may weight the selection of a drop structure type. Whenever public access is likely to occur, fencing not withstanding, the use of sloping drops is preferred for safety reasons over the use of vertical ones.

Table HS-1—Non-Boatable Drop Structure Selection for 3- to 5-Foot High Drops and Flows of 0 to 15,000 cfs

1. Easy or limited public access; downstream degradation likely. a) Grouted sloping boulder drop with toe imbedded in the stream bed b) Baffle chute drop
2. Limited public access; downstream degradation not likely. a) Grouted sloping boulder drop b) Vertical hard basin drop c) Baffle chute drop
3. Easy public access; downstream degradation not likely. a) Grouted sloping boulder drop b) Baffle chute drop

From an engineering design standpoint, there are two fundamental systems of a drop structure: the hydraulic surface-drop system and the foundation and seepage control system. The material components that can be used for the foundation and seepage control system are a function of on-site soils and groundwater conditions. The selection of the best components for design of the surface drop system is essentially independent of seepage considerations and is based on project objectives, channel stability, approach hydraulics, downstream tailwater conditions, height of drop, public safety, aesthetics, and maintenance considerations. Thus, foundation and seepage control system considerations are discussed separately. One factor that influences both systems is the extent of future downstream channel degradation that is anticipated. Such degradation can destroy a drop structure if adequate precautions are not provided.

2.3 Detailed Hydraulic Analysis

2.3.1 Introduction

Analysis guidelines are discussed in this section to assist the engineer in addressing critical hydraulic and seepage design factors. For a given discharge, there is a balance between the crest base width, upstream and downstream flow velocities, the Froude number in the drop basin, and the location of the jump. These parameters must be optimized for each specific application.

There are two levels of analysis possible. The first involves detailed analysis of all hydraulic conditions and leads to an optimal design for each structure. The concepts involved are described herein, and numerous references are available for more detailed information. The second level of analysis is a simpler approach that is based on configurations that will be adequate at the limits of permissible grass-lined channel criteria as described in Section 2.4.

There are two general categories of drops: sloping and vertical. For safety reasons, vertical drops should be avoided under urban conditions for public safety reasons. Performance of vertical or smooth sloping drops into a hard basin is relatively well documented. Their hydraulic analysis is briefly described herein. The design criteria for other drops such as vertical plunge pools and baffle chutes is based on empirical data and model studies.

2.3.2 Crest and Upstream Hydraulics

After preliminary channel layout has indicated probable drop location and heights (see the MAJOR DRAINAGE chapter for guidance, including the design spreadsheet UD-Channels), analysis and design begins with review of the crest section at the top of the drop. As flow passes through critical depth near the crest, upstream hydraulics are separated from downstream. Usually, the key task here is to determine critical depth at the crest based on the entire section. The critical flow state needs to be verified to ascertain that the downstream tailwater does not submerge the crest and effectively controls the hydraulics above the crest. If the downstream tailwater controls, then the structure must still be evaluated as a check for the peak discharge and as a drop at lower flows, if appropriate.

With control at the drop crest, water surface profile computations are used to establish the upstream abutment and bank heights. Computations should include a transition head loss, typically ranging from 0.3 (modest transitions in grass-lined channels) to 0.5 (channels approaching abrupt constrictions) times the change in velocity head across the transition (see Section 5.2), and allowance for the end contraction where the flow may effectively separate from the abutment end walls. Refer to Section 5.0 and standard hydraulic references for guidance (Chow 1959, Rouse 1949, and USACE 1994).

2.3.3 Water Surface Profile Downstream of the Crest

2.3.7.1 Critical Depth Along a Drop Structure.

Although this discussion concerns the hydraulics below the crest of a drop structure, the fundamental analysis of this hydraulics is established by the crest conditions. Main, low-flow and trickle channel regions are considered separately. Although the actual location of critical depth can vary according to the channel, transition, and drop geometry, the assumption is made that critical depth occurs at the crest, in a horizontal straight line across the crest section.

The assumption of critical flow conditions across the crest is illustrated conceptually by the diagrams in Figures HS-2 and the corresponding energy level across the section. At any point across the crest, the velocity is a function of the critical depth at that point. This causes a higher unit discharge applied to the trickle channel zone than across the main channel flow area. Figure HS-2 also illustrates that the water surface and energy grade line profiles will be different at the trickle (or low-flow) portions of the section than in the main channel flow zones and the forces exerted by flow on individual boulders on the sloping face of the drop.

2.3.7.2 Hydraulic Analysis.

After review of the crest and upstream hydraulics, the analysis proceeds to the supercritical flow and the hydraulic jump downstream. It is here that the designer should give special consideration to the potential of reverse rollers and avoid them in boatable channels and, where practicable, in grass-lined channels. Little flow dispersal from the trickle or low-flow zone to the main zone occurs through the supercritical portion of the drop. (Flow expansion is more likely downstream of the jump.) Therefore, unit discharge determined at the crest for either the trickle channel or the main portion of the drop is assumed to remain constant. The required basin length varies between these zones. Baffle chutes are the only type of drop where this distinction is not significant because the baffles break up the flow patterns and spread the flow more evenly over the width of the channel.

With the exception of baffle chute drops, separate analysis should be performed to evaluate the main drop and trickle or low-flow channel zones, as follows:

Critical depth, Y_c, is determined for the entire section area. The subscript ($_t$) or ($_m$) is added to refer to the trickle or low-flow zone or main channel zone, respectively. For example, in the main channel zone:

$$Y_{cm} = El_c - El_m \qquad \text{(HS-1)}$$

Similarly, in the trickle or low-flow channel zone:

$$Y_{ct} = El_c - El_t \qquad \text{(HS-2)}$$

in which:

El_c = critical water surface elevation

El_m = elevation of the main channel at the drop crest

El_t = elevation of the trickle or low-flow channel at the drop crest

The remaining hydraulic parameters, such as critical velocity, V_c (ft/sec), energy grade line, *EGL*, and unit discharge, q(cfs/ft), are determined separately for the main and trickle or low-flow channel zones by equations of the form:

$$V_c = (gY_c)^{1/2} \quad \text{(HS-3)}$$

$$EGL = Y_c + \frac{V_c^2}{2g} + El_m (or\ El_t) \text{ at the drop crest} \quad \text{(HS-4)}$$

$$q = Y_c^{3/2} g^{1/2} \quad \text{(HS-5)}$$

where g is the acceleration of gravity, and each parameter would have the subscript ($_m$) or ($_t$) as appropriate for the main, trickle, or low-flow channel zone.

Water surface profiles for the drawdown along the slope of a sloping drop and through the basin may be calculated using the "Standard Step Method" (Chow 1959), or any equivalent method suitable for unit discharge computations. For baffle chutes and vertical drops, individual methods are given in later subsections. It is necessary to plot the energy grade line to assure calculations are reasonable.

2.3.7.3 Manning's *n* for Concrete, Boulders and Grouted Boulders.

Depending on the type of materials and the relative depth, the appropriate roughness parameters should be used in computations. Table HS-2 and Figure HS-3 it refers to for grouted boulders, give the recommended Manning's roughness values and 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). Normal equations typically used for riprap do not apply to boulders and grouted boulders because of their near-uniform size and because the voids may be completely or only partially filled with grout. The roughness coefficient taken from Figure HS-3 varies with the depth of flow relative to the size of the boulders and the depth of grout used to lock them in place. Stepped grouted rock placement is another method that can be used to increase roughness and reduce velocities over the face of the drop.

Table HS-2—Suggested Approximate Manning's Roughness Parameter at Design Discharge for Sloping Drops

Smooth concrete	0.011 to 0.013
Stepped concrete where step heights equal 25% of nape depth	0.025*
Grouted Boulders	See Fig. HS-3

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

2.3.7.4 Avoid Low Froude Number Jumps in Grass-Lined Channels.

Low Froude number hydraulic jumps with longer areas of hydraulic instability are common in grass-lined channel applications. Baffles and rock placements that create turbulence and dissipate energy along the face of the drop are recommended to help counteract the adverse effects of low Froude number jumps and the associated tendency to carry residual energy and waves for extended distances downstream.

2.3.4 Hydraulic Jump Location

The water surface profile analysis starts at the crest and works downstream to analyze supercritical flow. Separate analysis for the low-flow, trickle, and main channels includes the review of hydraulic jumps. In the case of a baffle chute, no jump will occur because the baffles are constantly breaking up the flow, preventing supercritical flow. Examination of tailwater conditions is still important for a baffle chute to evaluate riprap and basin layout.

To determine the location of the hydraulic jump, a tailwater elevation has to be established by water surface profile analysis that starts from a downstream control point and works upstream to the drop basin. This backwater analysis is based upon entire cross sections for the downstream waterway. The hydraulic jump, in either the low-flow, trickle channel, or the main drop, will begin to form where the unit specific force of the downstream tailwater is greater than the specific force of the supercritical flow below the drop. Special consideration must be given to submerged hydraulic jumps because it is here that reverse rollers are most common. For submerged jumps, the resulting downstream hydraulics should be evaluated (Cotton 1995).

The determination of the jump location is usually accomplished through the comparison of specific force between supercritical inflow and the downstream subcritical flow (i.e., tailwater) conditions:

$$F = \left(\frac{q^2}{gy}\right) + \left(\frac{y^2}{2}\right) \tag{HS-6}$$

in which:

F = specific force (ft^2)

q = unit discharge (determined at crest, for low-flow, trickle, and main channel zones) (cfs/ft)

y = depth at analysis point (ft)

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

The depth, y, for downstream specific energy determination is the tailwater water surface elevation minus the ground elevation at the point of interest, which is typically the main basin elevation or the trickle channel invert (if the jump is to occur in the basin). The depth, for the upstream specific energy (supercritical flow), is the supercritical flow depth at the point in question.

Note that on low drops, the jump may routinely submerge the crest or may occur on the face of the drop. Refer to Little and Daniel (1981), Little and Murphey (1982), Chow (1959), USACE (1994), and Peterka (1984) for these cases.

The jump at sloping drops typically begins no further downstream than the drop toe. In vertical drops, the jump should begin where the jet hits the floor of the basin. This is generally accomplished in the main drop zone by depressing the basin to a depth nearly as low as the downstream trickle channel elevation. This will provide drainage for the basin.

2.3.5 Jump and Basin Length

The un-submerged jump length is typically between 3.6 and 6 times the tailwater depth, depending on the Froude number. For most cases, a basin length of 5 to 6 times the tailwater depth is the most advisable. A longer basin length is advisable for erosive soils or depending on the nature of the jump. Typically, at least 60% of the jump length is rock lined or otherwise reinforced. For baffle chute drops and vertical drops, basin dimensions are empirically derived.

In the trickle or low-flow channel alignment, the jump will tend to wash further downstream of the toe, and additional mitigation is recommended such as extending the basin length and/or providing baffles or large boulders that will break up the jet and dissipate energy.

2.3.6 Seepage Analysis

Subgrade erosion caused by seepage and structure failures caused by high seepage pressures or inadequate mass are of critical concern. These factors are important in the design and must be analyzed; otherwise, the structure might fail.

Seepage analysis can range from hand-drawn flow nets to computerized groundwater flow modeling. Advanced geotechnical field and laboratory testing techniques may be used to confirm the accepted permeability values where complicated seepage problems are anticipated. Several flow net analysis programs are currently available that are suitable for this purpose.

A minimal approach is Lane's Weighted Creep method. It can be used to determine dimensions or cutoff improvements that would provide an adequate seepage length. It should only be used as a guideline and, when marginal conditions or complicated geological conditions exist, a more precise analysis should be used. The involvement of a geotechnical engineer will often be necessary. Lane's method is given later in this section.

2.3.7 Force Analysis

Each component of a drop has forces acting upon it that require evaluation. This subsection describes the general forces, except 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), Wittler 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 drops. Additional information on forces on baffle blocks is presented in the baffle chute subsection, and this information may also be useful to extrapolate for large boulders used as baffles in grouted boulder drops.

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

Figure HS-3 illustrates the forces involved for a grouted sloping boulder drop, which is similar to other sloping concrete drops or baffle chutes. 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 slope. 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, the greater the roughness, the deeper the flow condition and the greater the weight.

2.3.7.1 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, and the drop slope.

$$\tau = \gamma y S_e \tag{HS-7}$$

in which:

τ = shear stress (lbs/ft^2)

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

y = depth of water at analysis point (ft)

2.3.7.2 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.

2.3.7.3 Impact, Drag and Hydrodynamic Lift Forces

Water flowing down 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. One should compare calculated drag force results with the forces shown later for baffle chute blocks (Section 2.5). 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.

2.3.7.4 Tur ning 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.

2.3.7.5 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.

2.3.7.6 Frost Heave

This value is not typically computed for the smaller drops 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.

2.3.7.7 Seepage Uplift Pressure

As explained previously, uplift pressure and seepage relief considerations are extremely important 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 slope 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 drops (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.

2.3.7.8 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.

A good example of the problem can be envisioned by a situation in which the entire sloping face of the 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.

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:

$$C_{p-\max} = \frac{\Delta P}{\left(\frac{V_u^2}{2g}\right)} \tag{HS-8}$$

in which:

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.

Table HS-3 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 HS-4 through HS-6. 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 HS-4 illustrates positive and negative pressure fluctuations in the coefficient, C_p, with respect to the location where the jump begins at the toe. Figure HS-5 presents the positive pressure fluctuation coefficient where the jump begins on the face. Figure HS-6 illustrates how the pressure fluctuations vary in a U.S. Bureau of Reclamation (USBR) Type II or III basin.

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 HS-3—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*	3.8	0.7
30° slope, toe of jump on chute*	3.3	0.8
30° slope, with Type II basin (USBR)	5.0	0.7
30° slope with Type III basin (USBR)	5.0	1.0

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

2.3.7.9 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 and the overall drop structure geotechnical and structural stability. All steps of this detailed analysis are not necessary for design of drops 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 District 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 as shown in detail 2 of Figure HS-7D to prevent hydrostatic uplift on the stones.

2.4 Simplified Drop Structure Designs for District's Grass-Lined Channels

2.4.1 Introduction and Cautions

As previously mentioned, there is a balance between the crest shape chosen, upstream channel stability, and the configuration of the drop downstream which will result in reasonable or optimal energy dissipation. Further, there is usually a single configuration of drop crest, upstream channel slope, and base width that will result in an acceptable drop structure performance for grass-lined channels designed using the District's criteria described in the MAJOR DRAINAGE chapter.

This subsection presents simplified relationships that provide basic configuration and drop-sizing parameters that may be used when the District's maximum allowable velocity and depth criteria for grass-lined channels are used.

Design guidance presented in this section is developed for channels that operate at the brink of maximum criteria (i.e., approximately having unit discharge of 25 cfs/ft for erosive soil and 35 cfs/ft for erosion resistant soils and Froude Number $\leq$ 0.8). They do not consider channel curvature, effects of other hydraulic structures, or unstable beds, all of which require detailed analysis. They do provide guidelines for initial sizing and reasonableness checking, but are not a substitute for comprehensive hydraulic analysis in the context of the entire waterway.

2.4.2 Applicability of Simplified Channel Drop Designs

This section presents guidelines and analysis steps and specific minimum design criteria for two types of drops. Grouted sloping boulder drops and vertical hard basin drops are the only two types of drops for which these simplified design procedures may be utilized when used in grass-lined channels. Other designs are available, but they are more limited in application and require an individual analysis. Regardless of the type of drop used, it should never be located within or immediately downstream of a curve in a channel. Namely, locate all drops on a tangent and not on a curve of a channel.

Photograph HS-5—Example of stepped downstream face for a sloping boulder drop structure. Note dissipation of energy at each step for low flow.

2.4.3 Simplified Grouted Sloping Boulder Drop Design

This type of structure has gained acceptance in the Rocky Mountain 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. There is no maximum height limit; however, the rock sizing procedure is more complex than the simplified procedures and details provided by Figures HS-7A, HS-7B and HS-7C for GSB drops 6-feet or less in height.

For typical channels the drop is designed with a hydraulic jump dissipator basin, although some energy loss is incurred due to the roughness of the grouted rock slope. In sandy soil channels the design provides for a scour at the toe and does not require an energy-dissipating basin. Structure integrity and containment of the erosive turbulence within the basin area are the main design objectives.

Photograph HS-6—Detail of the grouted sloping boulder drop with a trickle channel section creating the sight and sound of cascading water.

Construct boulder drops using uniform-height boulders with a minimum height specified in Table HS-4. Grout all boulders to a depth of 1/2 or 1/3 of their height through the approach, sloping face, and basin areas, except at the upstream crest where it needs to extend the full depth of the rock in order to provide stability of the approach channel. Figures HS-7A, HS-7B and HS-7C illustrate the general configuration of three types of GSB drops; one for a channel with a trickle channel (Figure HS-7A), one for one with a low-flow channel (Figure HS-7B) and one for channels in erosive soils or unstable conditions. (Figure HS-7C). Requirements for the grout, riprap and boulders are specified in the MAJOR DRAINAGE chapter of this *Manual*. Adequate seepage control with underdrains is important for a successful design whenever drop height exceeds 5-feet.

The following outlines the fundamental design steps and guidelines.

1. Hydraulics should be completed as described in Section 2.3 whenever the drop height exceeds 6 feet. Otherwise, use critical depth to size the boulders, using the boulder sizing procedure described below.
2. Grouted boulders must cover the crest and cutoff and extend downstream through the energy-dissipating basin when there is one, or through the imbedded toe of the drop when not present.
3. The vertical cutoff should be located at the upstream face of the crest, at a minimum depth of $0.8H_d$ or 4 feet, whichever is deeper. Evaluate specific site soils for use in seepage analysis and foundation suitability.

Photograph HS-7—An overall view of the drop structure from the previous page is illustrated here to emphasize the opportunities available for creating an attractive urban hydraulic setting for the riparian corridor.

4. The trickle or low-flow channel should extend through the drop crest section. Downstream, the trickle or low-flow channel protection should extend past the main channel protection, or large boulders and curves in the trickle or low-flow channel can be used in the basin area to help dissipate the energy.
5. Grout thickness, D_g, and rock thickness, D_r, should be determined based upon a minimum safety surplus net downward force of 30 pounds. The rocks must be carefully placed to create a stepped appearance, which helps to increase roughness. Minimum criteria for the simplified design process are referred to in step 8, below.
6. The main stilling basin should be depressed 1 to 2 feet deep in order to stabilize the jump. A row

of boulders should be located at the basin end to create a sill transition to the downstream invert elevation. It is advisable to bury riprap for a distance of 10 feet downstream of the sill to minimize any erosion that may occur due to secondary currents.

When the drop is located in sandy soils and in channels with lesser stability, the stilling basin is eliminated and the sloping face extended to where the top of the boulders are five feet (5') below the projected (i.e., after accounting for downstream degradation) downstream channel's invert.

7. Do not use longitudinal slopes steeper than 4:1. Longitudinal slopes flatter than 4:1 improve appearance and safety while steeper slopes reduce structural stability. With high public usage, very flat longitudinal slopes (i.e., flatter than 8H:1V) help to mitigate reverse roller formation at higher tailwater depths that can cause submerged hydraulic jump formation and create "keepers".
8. Simplified design criteria are provided in Table HS-4 for grouted sloping boulder drops. These criteria are valid only where the channel flow conditions meet the minimum criteria recommended in the MAJOR DRAINAGE chapter.

Table HS-4—Grouted Sloping Boulder Drops: Minimum Design Criteria for Grass-Lined Channels Meeting the District's Maximum Depth and Velocity Criteria

Design Parameter	Drop Height (H_d) 6 Feet or Less	Drop Height (H_d) Greater Than 6 Feet
Maximum longitudinal slope	4H to 1V	4H to 1V
Minimum boulder depth	Use V_c to size*	Use V_n to size***
Grout thickness—D_g	½ to $^{1}/_{3}$ D_r except at the upstream crest of the structure where full grout depth is needed	½ D_r to $^{1}/_{3}$ D_r except at the upstream crest of the structure where full grout depth is needed
Basin depression	1 to 2 feet (*see Step 6 above for sandy/unstable channels*)	Do sequential depth analysis
Grouted boulder approach—L_a	5 feet (min.)	8 feet
Basin length—L_b** Erosive (sandy channel) Non-erosive	20 feet (*see Step 6 above for sandy/unstable channels*) 15 feet	20 feet (*also see Step 6 above for sandy/unstable channels*) 15 feet
Basin width—B	Same as crest width (*see Step 6 above for sandy/unstable channels*)	
Trickle and low-flow zone provisions	Install large boulders in center basin zone to break up high flow stream (*see Step 6 above for sandy/unstable channels*)	
Trickle zone protection width below drop	$3b_1$ or b^2 (whichever is smaller; see Figure HS-7)	
Other provisions	A buried riprap zone should be installed for $2H_d$ (10 feet minimum) downstream of the basin (*see Step 6 above for sandy channels*) Do not locate a drop within a channel curve or immediately downstream of one.	

* Use critical velocity in low-flow and main channels to size boulders.

** Use drawdown velocity at H_d to size low-flow and main channel section boulders.

Sizing of boulders for the simplified grouted sloping boulder procedure is based on the following:

1. This procedure can be used only for channels designed using the specified maximum velocities and depths for grass-lined channels in this *Manual* (see the MAJOR DRAINAGE chapter).

2. For drops of 6-feet or less in height, one can use UD-Channels Spreadsheet to find the 100-year critical velocities in the low-flow and the main channels to size boulders for each section.

For drops greater than 6-feet in height, a detailed design procedure has to be used consisting of the following:

1. Determine the critical velocities using drawdown calculations to establish the 100-year flow depth at the toe of the drop.

   a. For a composite channel, find critical velocity, V_c, for the channel cross-section segment outside the low-flow section.

   b. For a composite channel, find critical velocity, V_{mc}, for the low-flow channel cross-section segment.

   c. For a simple trapezoidal or wetland bottom channel, find critical velocity, V_c, for the channel cross section.

2. Calculate rock-sizing parameter, R_p, for the channel cross-section segment outside the low-flow section or for a simple trapezoidal channel section using the critical velocity estimated for this segment of the cross section:

$$R_p = \frac{V_c S^{0.17}}{(S_s - 1)^{0.66}}$$

in which: S = longitudinal slope along direction of flow in ft/ft

S_s = Specific gravity of the rock. Assume 2.55 unless the quarry certifies higher specific gravity.

3. Calculate rock-sizing parameter, R_{pL}, for the channel cross-section segment within the low-flow section using the critical velocity for drops 6-feet in height (the draw-down velocity estimates at bottom of the drop for taller structures):

$$R_{pL} = \frac{V_{mc} S^{0.17}}{(S_s - 1)^{0.66}} \qquad \text{(HS-9)}$$

4. Select minimum boulder sizes for the cross-section segments within and outside the low-flow channel cross-section from Table HS-5. If the boulder sizes for the low-flow channel and the

overbank segments differ, decide to use only the larger sized boulders throughout the entire structure, or to specify two sizes, namely, one for the low-flow channel and the other for the overbank segments of the cross section. Consider the complexity of specifying two different sizes on the design drawings and in the construction of the structure before deciding.

Regardless of the design procedure used above, all boulders shall be grouted in accordance with the specifications Figure HS-8. All grouted boulders outside of the low-flow channel shall be buried with topsoil to a depth of no less than 4 inches (6 inches or more preferred for successful grass growth) above the top of the highest boulder and the surface vegetated with native grasses on the overbank bench and native grasses and dry-land shrubs on the overbank channel's side slopes.

Table HS-5—Boulder Sizes for Various Rock Sizing Parameters

	Ungrouted Boulders		**Grouted Boulders ***	
Rock Sizing Parameter, R_p	Minimum Dimensions of Boulder, D_r	Boulder Classification	Minimum Dimensions of Boulder, D_r	Boulder Classification
Less than 4.50	18 inches	B18	18 inches	B18
4.50 to 4.99	24 inches	B24	18 inches	B18
5.00 to 5.59	30 inches	B30	24 inches	B24
5.60 to 6.39	36 inches	B36	30 inches	B30
6.40 to 6.99	42 inches	B42	36 inches	B36
7.00 to 7.49	48 inches	B48	42 inches	B42
7.50 to 8.00	n/a	n/a	48 inches	B48

* Grouted to no less than ½ the height (+1"/- 0"), no more than ⅓ (+0"/- 1") of boulder height.

2.4.4 Vertical Hard Basin Drops

The vertical hard basin drops include a wide variety of structure designs, but they are not generally recommended for use in urban areas because of concerns for public safety, during wet and dry weather periods. In addition, vertical hard basin drops are to be avoided due to impingement energy, related maintenance and turbulent hydraulic potential (ASCE and WEF 1992). Whenever used, it is recommended their drop height, upstream invert to downstream channel invert, be limited to 3-feet.

The hydraulic phenomenon provided by this type of drop is a jet of water that overflows the crest wall into the basin below. The jet hits the hard 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. The basin is sized to contain the supercritical flow and the erosive turbulent zone. Figure HS-9 shows a vertical drop with a grouted boulder basin. The rock-lined approach length ends abruptly at a structural retaining crest wall that has trickle channel section.

Photograph HS-8—A vertical hard basin drop structure can be an effective tool for controlling grade, but its use in urban areas is not generally not recommended because of public safety concerns and aesthetics.

Basic design steps are as follows:

1. The design approach uses the unit discharge in the main and trickle channel to determine separately the water surface profile and jump location in these zones. The overall jump hydraulic problems are the same as previously described.

   Chow (1959) presents the hydraulic analysis for the "Straight Drop Spillway." Add subscript ($_t$) for the trickle channel area and subscript ($_m$) for the main channel area in the following equations. The drop number, D_n, is defined as:

   $$D_n = \frac{q^2}{\left(gY_f^3\right)} \qquad \text{(HS-10)}$$

   in which:

   q = unit discharge (cfs/ft)

   Y_f = effective fall 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 HS-10:

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

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

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

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

in which:

Y_f = effective fall 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. Determination of the distance to the hydraulic jump, D_j, requires a separate water surface profile analysis for the main and low-flow zones as described herein for sloping drops. Any change in tailwater affects the stability of the jump in both locations.

2. The hydraulic jump length, L_j, is approximated as 6 times the sequent depth, Y_2. The design basin length, L_b, includes nappe length, L_d, the distance to the jump, D_j, and 60% of the jump length, L_j. (The subscripts "m" and "t" in Equations HS-11 and HS-12 refer to the main and trickle channel zones, respectively.)

   At the main channel zone:

$$L_{bm} = L_{dm} + D_{jm} + 60\% \left(6Y_{2m}\right) \quad \text{(HS-11)}$$

   At the trickle channel flow zone, without baffles or boulders to break up the jet:

$$L_{bt} = L_{dt} + D_{jt} + 60\% \left(6Y_{2t}\right) \tag{HS-12}$$

3. Caution is advised regarding the higher unit flow condition in the low-flow zone. Large boulders and meanders in the trickle zone of the basin may help dissipate the jet and may reduce downstream if riprap extended 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_{bt}, 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 nape hits the basin and no closer than 10 feet from the basin end.

4. The basin floor elevation should be depressed in depth, and variable with drop height. Note that the basin depth adds to the effective tailwater depth for jump control. The basin can be constructed of concrete or grouted rock. Use of either material must be evaluated for hydraulic forces and seepage uplift.

   There should be a sill at the basin end to bring the invert elevation to that of the downstream channel and sidewalls extending from the crest wall to the sill. The sill is important in causing the hydraulic jump to form in the basin. Buried riprap should be used downstream of the sill to minimize any local scour caused by the lift over the sill.

5. Caution is advised to avoid flow impinging on the channel side slopes of the basin.

6. Crest wall and footer dimensions should be determined by conventional structural methods. Underdrain requirements should be determined from seepage analysis.

7. Seepage uplift conditions require evaluations for each use. Thus, seepage analysis should be completed to provide for control and weight/size of components (see Section 2.6).

8. Simplified design criteria are provided in Table HS-6 for vertical hard basin (grouted boulder) drops. These criteria are valid only where the channel flow conditions meet the criteria in the MAJOR DRAINAGE chapter of this *Manual* and the drop does not exceed 3-feet in height.

9. Drops with reinforced concrete basins will have slab thickness and drop lengths that vary somewhat from the simplified design in Item 8 above, depending upon hydraulic and seepage considerations.

Table HS-6—Vertical Drops With Grouted Boulder Basin: Simplified Design Criteria for Small Vertical Drops in Grass-Lined Channels Meeting District Criteria

Design Parameter	**Criterion**
Maximum Drop Height	3 feet, invert to invert
Boulder size—D_r*	18 inch minimum dimension
Grout thickness—D_g	10 inches**
Basin depression—B (see Figure HS-10)	1.5 ft
Basin length—L_b (see Figure HS-10)	25 ft
Approach length—L_a	10 ft buried riprap
Trickle flow zone provisions	Install large boulder or baffles in center zone to break up high flow stream, or apply separate water surface analysis
Other provisions	A buried riprap zone should be installed for 10 ft minimum downstream of the drop basin Consider the possible hazard to public when selecting this type of drop for use in urban areas.

* Boulder size refers to the minimum dimension of all boulders measured in any direction.

** Bury all grouted boulders on side slopes by filling all gaps and depressions to top of boulders with lightly compacted topsoil and capping with at least 4 inches of top soil; however, capping it with 6 to 12 inches of topsoil will insure a much more robust conditions the native grasses to be seeded on the soil cap.

2.5 Baffle Chute Drops

The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of the drop, commonly referred to as baffled apron or baffle chute drops. There are references such as *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka 1984) and *Design of Small Canal Structures* (Aisenbrey, et al. 1978) that should be used for the design of these structures. A baffle chute drop was constructed on Harvard Gulch that can be inspected for long-term performance (Wright 1967).

The hydraulic concept involves flow repeatedly encountering obstructions (baffle blocks) that are of a nominal height equivalent to critical depth. The excess energy is dissipated through the drop by the momentum loss associated with reorientation of the flow. A minimum of four rows of baffle blocks is recommended to achieve control of the flow and maximum dissipation of energy. Guidelines are given for sizing and spacing the blocks. Designing for proper approach velocities is critical to structure performance. One advantage of this type of drop is that it does not require tailwater control. However, the designer does need to consider local flow and scour patterns in the transition back to the channel.

Optimal performance occurs for a unit discharge of 35 to 60 cfs/ft of chute width, which happens to be a well-matched design for the District's grass-lined channel criteria. Refer to Rhone (1977) for guidance on higher unit discharge and entrance modifications to address backwater effects.

Photograph HS-9—Close-up of the inside workings of a baffle chute drop after more than three decades of service.

The typical design consists of upstream transition walls, a rectangular approach chute, a sloping apron of 2:1 or flatter slope that has multiple rows of baffle blocks and downstream transition walls. The toe of the chute extends below grade and is backfilled with loose rock to prevent undermining of the structure by eddy currents or minor degradation of the downstream channel. This rock will rearrange to establish a stable bed condition and produce additional stilling action. The structure is effective without tailwater; however, tailwater reduces scour at the toe. Grouted and concrete basins have been used at the transition to the downstream trickle and main channels. The structure also lends itself to a variety of soils and foundation conditions.

There are fixed costs associated with the upstream transition walls, crest approach section, downstream transition walls and a minimum length of sloping apron (for four baffle rows). Consequently, the baffle chute becomes more economical with increasing drop height.

The potential for debris accumulation and subsequent maintenance must be considered. Caution is advised regarding streams with heavy debris flow because the baffles can become clogged, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet on the downstream channel. Baffle chute drops are best suited for grass-lined channels and should not be used for boatable streams.

The basic design criteria and details are given in Figure HS-11 (adapted from Peterka 1984). Remaining structural design parameters must be determined for specific site conditions. Recommended design procedures are as follows:

1. Determine the maximum inflow rate and the design unit discharge, $q = \frac{Q}{W}$.

2. An upstream channel transition section with vertical wingwalls constructed 45 degrees to the flow direction causes flow approaching the rectangular chute section to contract. It is also feasible to use walls constructed at 90 degrees to the flow direction. In either configuration, it is important to analyze the approach hydraulics and water surface profile. Often, the effective flow width at the critical cross section is narrower than the width of the chute opening due to flow separation at the corners of the abutment (see Section 5.0).

3. The entrance transition should be followed by a rectangular flow alignment apron, typically 5 feet in length. The upstream approach channel velocity, V, should be as low as practical and less than critical velocity at the control section of the crest. Figure HS-11 gives the USBR-recommended chute entrance velocity. In a typical grass-lined channel, the entrance transition to the rectangular chute section will produce the desired upstream channel velocity reduction. The chute elevation (shown in Figure HS-11) should only be above the channel elevation when approach velocities cannot be controlled by the transition. Extra measures to prevent upstream aggradation are required with the raised crest configuration.

4. Normally, the baffles should be sized at height, H, equal to 0.8 times critical depth at peak flow. The chute face slope should be 2:1 for most cases but may be reduced for low drops or where a flatter slope is desirable. For unit discharge applications greater than 60 cfs/ft, the baffle height may be based on two-thirds of the peak flow; however, the chute sidewalls should be designed for peak flow (see Step 8 below).

   Baffle block widths and spaces should equal approximately $1.5H$ but not less than H. Other baffle block dimensions are not critical hydraulically. The spacing between the rows of baffle block should be H times the slope ratio. For example, a 2:1 slope makes the row spacing equal to $2H$ parallel to the chute floor. The baffle blocks should be constructed with the upstream face normal to the chute floor.

5. Four rows of baffle blocks are required to establish full control of the flow. At least 1½ rows of baffles should be buried in riprap where the chute extends below the downstream channel grade. Rock protection, assumed here as Type M riprap, should continue from the chute outlet to a minimum distance of approximately $4H$ at a riprap layer depth of 2.0 feet to prevent eddy currents from undermining the walls. Additional rows of baffles may need to be buried below grade to allow for downstream channel degradation. Determine if the downstream channel grade has been stabilized to determine how many rows of baffles may need to be buried.

6. The baffle chute wall height (measured normal to the floor slope) should be 2.4 times the critical depth based on peak discharge. The wall height will contain the main flow and most of the splash. The designer of the area behind the wall should consider that some splash may occur, but extensive protection measures are not required.

7. Determine upstream transition and apron sidewall height as required by backwater analysis. Lower basin wingwalls generally should be constructed normal to the chute sidewalls at the chute outlet to prevent eddy current erosion at the drop toe. These transition walls should be of a height equal to the channel normal depth in the downstream channel plus 1 foot and length sufficient to inhibit eddy current erosion.

8. The trickle flow channel should be maintained through the entrance transition apron, approach, and crest sections. It may be routed between the first row of baffle piers. The trickle channel should start again at the basin rock zone that should be slightly depressed and then graded up to transition into the downstream channel to focus the low flows into the trickle channel. Figure HS-12 illustrates one method of designing the trickle channel through the crest.

9. The conventional design shown in Figure HS-11 results in the top elevation of the baffles being higher than the crest, which causes a backwater effect upstream. Figure HS-12 may be used to estimate the extent of the effect and to determine corrective measures such as increasing the upstream freeboard or widening the chute. Note that blocks projecting above the crest will tend to produce upstream sediment aggradation. Channel aggradation can be minimized by the trickle channel treatment suggested in Step 8.

   Another means of alleviating these problems is by using the Fujimoto entrance developed by the USBR and illustrated in Figure HS-12. The upper rows of baffles are moved one row increment downstream. The important advantage of this entrance is that there is not a backwater effect of the baffles. The serrated treatment of the modified crest begins disrupting the flow entering the chute without increasing the headwater. More importantly, this configuration provides a level crest control. The designer may either bring the invert of the upstream trickle channel into this crest elevation, widening the trickle channel as it approaches the crest, or he or she may have a lower trickle channel and bring it through the serrated crest similar to Step 8.

10. Concrete walls and footer dimensions should be determined by conventional structural methods. Cutoffs and underdrain requirements should be determined by seepage analysis discussed earlier in this chapter.

11. The hydraulic impact forces on the baffles should be determined to allow the structural engineer to size adequate reinforcing steel. Figure HS-12 may be used as a guideline. The structural engineer should apply a conservative safety factor.

2.6 Seepage Control

2.6.1 Seepage Analysis Methods

The **preferred deterministic methodology** for seepage analysis is the use of manual and computerized flow net analyses. It is used to quantify groundwater flows, pressures, and critical gradients under hydraulic structures. Flow net analysis can quantify the effects of multiple strata of different soil media and complex geometries and situations. Full decryption of flow net analysis is beyond the scope of this *Manual* and the user is referred to Cedergren (1967), USBR (1987) and Taylor (1967) for more information and instruction in the use of flow net analysis techniques.

At an absolute minimum and as a first order of estimation, Lane's Weighted Creep Method (CWM) can be used to identify probable seepage problems, evaluate the need for control measures, and roughly estimate uplift forces. It is not as definitive as the above-mentioned flow net analysis. The CWM technique was originally proposed by E.W. Lane in 1935. This method has been deleted, however, in the 1987 revision of *Design of Small Dams* (USBR 1987), possibly indicating greater use of flow net and computer modeling methods or for other reasons that we do not know about. Although Lane's method is relatively well founded, it should be used as a guideline, and when marginal conditions or complicated geological conditions exist, the more sophisticated flow-net analysis should be used. 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, L_v (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:

$$C_W = \frac{\left(\frac{L_H}{3} + L_V\right)}{H_S} \qquad \text{(HS-13)}$$

   in which:

   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 HS-13 should be used along with Equation HS-14 to check the short path between the bottom of the vertical cutoffs.

$$C_{W2} = \frac{\left(L_{V-US} + \quad 2L_{H-C} + L_{V-DS}\right)}{H_S} \tag{HS-14}$$

in which:

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 HS-14 similar to Equation HS-13. Seepage is controlled by increasing the total seepage length such that C_W or C_{W2} is raised to the value listed in Table HS-7. Soils tests must be conducted during design and confirmed during construction.

6. The upward pressure to be used in design may be estimated by assuming that the drop in uplift pressure from headwater to tailwater along the contact line of the dam and foundation is proportional to the weighted-creep distance.

Table HS-7—Lane's Weighted Creep: Recommended 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

2.6.2 Foundation/Seepage Control Systems

Table HS-8 presents some typical foundation conditions and systems that are often used for various drop heights. For each condition, cutoff types are listed in general order of preference for guidance purposes only. As a general rule, it is not recommended that groundwater flow cutoffs not be installed at the downstream ends of drop structures. Their presence 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 during construction, during dominant low flows, during flood flows, during design flows and other critical loading scenarios. The soils/foundation engineer combines 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 in selection and sizing of structural components.

Table HS-8—General Cutoff Technique Suitability

	Drop Height (ft)			
Soil Conditions	2	4	8	12
Sands and gravel over bedrock with sufficient depth of material to provide support—groundwater prevalent	SP[1] CTc CTf	SP[1] CTc/ST CTf/CTI	Sp/SwB[1] ST	Sp/SwB[1] ST
Sands and gravel with shallow depth to bedrock—groundwater prevalent	CTc CW SP[2]	CTc/ST CW SP[2]	ST CW SP[2]	ST CW SwB[2]
Sands and gravel with large depths to bedrock—groundwater prevalent	SP CTc	SP CTc/ST	SP ST	SP/SwB ST
Sands and gravel, no groundwater, or water table normally below requirement (for variation caused by depth to bedrock, see first case)	SP CTf/CTI CW	SP CTI CW	SP CTI	SP/SwB CTI
Clay (and silts)—medium to hard	CTc	CT	CT	CT
	CW	Reduce length for difficult backfill conditions		
	CTI/CTf	Only for local seepage zones/silts		
	ST	Expensive—for special problems		
Clays (and silts)—soft to medium with lenses of permeable material—groundwater present	CP CTc	SP CTc	SP CTc/ST	SP/SwB ST
Clay (and silts)—soft to medium with lenses of permeable material (may be moist but not significant groundwater source)	SP CTc CTf CW	SP CTc CTI CW	SP CTc/ST CTI CW	SP/SwB ST CTI CW

[1] Consider scour in sheet pile support.

[2] Excavate into bedrock and set into concrete.

<u>Legend</u>:

- SP Sheet pile
- SwB Sheet pile with bracing and extra measures
- CTc Cutoff trench backfilled with concrete
- ST Slurry trench; similar to CTc, but trench walls are supported with slurry and then later replaced with concrete or additives that provide cutoff
- CW Cutoff wall; conventional wall, possibly with footer, backfilled; note that the effective seepage length should generally be decreased because of backfill
- CTI Cutoff trench with synthetic liner and fill
- CTf Cutoff trench with clay fill

Photograph HS-10—Boatable channels of the District waterways provide enjoyment to a wide variety of citizens. The South Platte River example in this photograph provides an easily accessible boating experience.

2.7 Simplified Minimum Design Approach for Boatable Channels

Due to the fact that a special standard of care for the design of drops and low-head dams on boatable channels is required, the following design approach for boatable channels is limited to suggestions for the experienced hydraulic structure designer once the channel has been determined to be a boatable one.

1. Contact reliable whitewater boating experts to discuss general design objectives and boater safety concerns.

2. Select maximum height of individual drops—generally 4 feet. If they are more than 4 feet, a physical hydraulic model may be necessary.

3. Determine basic drop characteristics to be compatible with public safety and recreational boating. Suggestions are as follows:

    - Use a Froude number, F_r, less than 1.5 at the toe of the drop.
    - Avoid reverse rollers under all conditions of flow.
    - Assess stability of the structure taking into account expected downstream channel degradation.
    - Consider the slope of the downstream face of a sloping drop; 10(H) to 1(V) is common.

Photograph HS-11—Unprotected urban channels can experience bank erosion and degradation when established design criteria are not used. The invert of pipe used to be at invert of channel before degradation occurred.

- Provide boat chute with pilot rocks for routine boat passage of drop.
- Do not use an energy dissipating basin; instead, continue the sloping surface at least 5 feet below the downstream thalweg of the stream.
- Provide adequate warning signs and portage area.
- Use grouted sloping boulder or appropriately sized large ungraded sloping boulder structure.
- Consider vertical cutoff walls at the upstream end for seepage control.

4. Obtain peer review on the preliminary design.
5. Allow for follow-up rock adjustment after completion, especially for boat chutes.

2.8 Construction Concerns: Grass-Lined Channels

The selection of a drop or a grade control check and its foundation may be tempered by construction difficulty, access, material delivery, etc. Some of the important concerns are discussed below, although this is by no means an exhaustive list of the concerns possible for every site and situation.

2.8.1 Foundation/Seepage Control

Initial items that are especially important are site water control and foundation conditions. A common problem is destabilization of the foundation soils by rapid local dewatering of fine-grained, erosive soils, or soils with limited hydraulic conductivity. Often the preferred method is continuous pumping rates at perimeter locations (or well points) that allow the entire construction area to remain stable. Appropriate water control techniques for use during construction of a drop structure should be presented to the contractor. Diversion berms should be designed with planned berm failure points to avoid flooding of drop-structure sites during construction.

The actual subgrade condition with respect to seepage control assumptions must be inspected and field verified. The engineer who established the design assumptions and calculated the required cutoffs should inspect the cutoff for each drop 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, then the cutoff trench may need to be deepened, or a different cutoff type may need to be implemented. Obviously, soil testing is an advisable precaution to minimize changes and avoid failures.

2.8.2 Baffle Chute Construction

There are numerous steps necessary in the construction of a baffle chute, but a contractor usually easily controls them. For quality control and inspection there are consistent, measurable, and repeatable standards to apply.

Baffle chutes are highly successful as far as hydraulic performance is concerned and are straightforward to construct. Steel, formwork, concrete placement and finish, and backfill generally require periodic inspection. Potential problems include foundation integrity, riprap quality control, water control, and the finish work with regard to architectural and landscape treatments. Formwork, form ties, and seal coatings can leave a poor appearance if not done properly.

2.8.3 Vertical Hard Basin Construction

Foundation and seepage concerns are critical with regard to the vertical wall. Poor construction and seepage control can result in sudden failure. The use of caissons or piles can mitigate this effect. Put in comparative terms with the baffle chute, seepage problems can result in displacement of the vertical wall with no warning, where the box-like structure of the baffle chute may experience some movement or cracking, but not total failure, and thus allow time for repairs.

The quality control concerns and measures for vertical basins are the same as for baffle chutes. The subsoil condition beneath the basin is important to insure that the stilling basin concrete or grouted rock bottom is stable against uplift pressures.

2.8.4 Sloping Grouted Boulder Construction

The sloping grouted boulder drops require significant construction control efforts in the field. Seepage analysis is required to determine a compatible combination of cutoff depth, location of weep and toe drains, and the thickness of grouted rock layer. The greatest danger 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.

Individual boulders should be larger in diameter than the grout layer so that the contractor and the inspector can verify the grout depth and have grout placed directly to the subgrade. The best balance appears to have the grout thickness set at 1/2 the boulder height, but no more than 2/3 boulder height, and to have an overall mass sufficient to offset uplift, plus a safety factor. Limiting grout thickness also improves the overall appearance of the grouted boulder structure.

The condition of the subgrade, adequate seepage control, and sub-drainage of the seepage flow are all critical. There is a tendency to disturb the subgrade during rock placement, leaving a potential piping route. This should be controlled by good subgrade preparation, careful rock placement, and removal of loose materials. Absolutely no granular bedding or subgrade fill using granular materials should be used to prevent conditions that will cause piping. Problems with rock density, durability and hardness are of concern and can vary widely for different locations. The rock should be inspected at regular intervals to meet minimum physical dimensions, strengths, durability and weights as defined in the specifications.

For aesthetic reasons, it is recommended that the grouted boulders above the low flow section and on the banks be covered with local soils, topsoil and revegetated.

2.9 Low-Flow Check and Wetland Structures

Urbanization causes more frequent and sustained flows, and therefore the trickle/low-flow channel and wetlands become more susceptible to erosion even though the overall floodplain may remain stable and able to resist major flood events. Erosion of the low-flow channel, if left uncontrolled, can cause degradation and destabilization of the entire channel. Low-flow grade-control check structures are designed to provide control points and establish stable bed slopes within the base flow channel. They should be used to limit longitudinal slope of the channel to about 0.3% to 0.5% and as described in the MAJOR DRAINAGE chapter. Low-flow check structures are not appropriate along incised floodplains and may not be economical for very steep channels, where higher drop structures may be needed.

Grouted sloping boulder and vertical hard basin designs can be adapted for use as check structures after considering (1) stable bed slopes for the unlined trickle or low-flow channel and (2) potential overflow erosion during submergence of the check structure and where flow converges back from the main channel sides or below the check structure.

The basic design steps for low-flow grade-control check structures include the following:

1. Determine a stable slope and configuration for the low-flow zone. For unlined channels, discharges from full floodplain flow to the dominant discharge should first be considered. The dominant discharge is more fully explained in sediment transport texts (Richardson 1988; Shen 1971; Simons 1977; Simons, Li and Associates 1982; and Muessetter 1983). It is generally defined as the flow that represents the average or equilibrium conditions controlling the channel bed. In the Denver region, the dominant discharge is typically the 2-year flood. Numerous references (Chow 1959; SCS 1977; and above references) cite information on permissible

velocities. The range of stable longitudinal slopes for non-rock lined major drainageways in the Denver area is between 0.003 ft/t and 0.005 ft/ft. Two exceptions to this range exist, one is for larger streams and the South Platte River, where it can be much flatter, and the other is for steep waterways with small tributary catchments of relatively low imperviousness, where the final stable slopes can be steeper.

2. The configuration of the low-flow zone and number and placement of the check structures must be reviewed. A good rule is to have the check structures spaced so the drop does not exceed 3-feet after the downstream channel has degraded to the projected stable longitudinal slope.

One type of check structure that can be used to stabilize low-flow channels within relatively stable channels is the control check (see Figure HS-13a and Figure HS-13b). This type of a check structure can be constructed by filling an excavated narrow trench (12' minimum) with concrete if soil and groundwater conditions permit trenching to a depth of 6 feet, or by driving a concrete capped sheet piles to 10 foot depth when trenching is not possible.

Extend the cutoff walls into the main channel banks a minimum of 10 feet and make sure it rises sufficiently to contain the 5- to 10-year flow (depending on local criteria), but no less than 2-feet above the approach channel (outside the trickle flow section) to avoid side cutting.

Wetland channel check structures will typically do not have a trickle channel. When building check structures for wetland bottom channels, place riprap downstream of the cutoff wall to dissipate the kinetic energy when downstream backwater is low so as to avoid deep scour hole downstream.

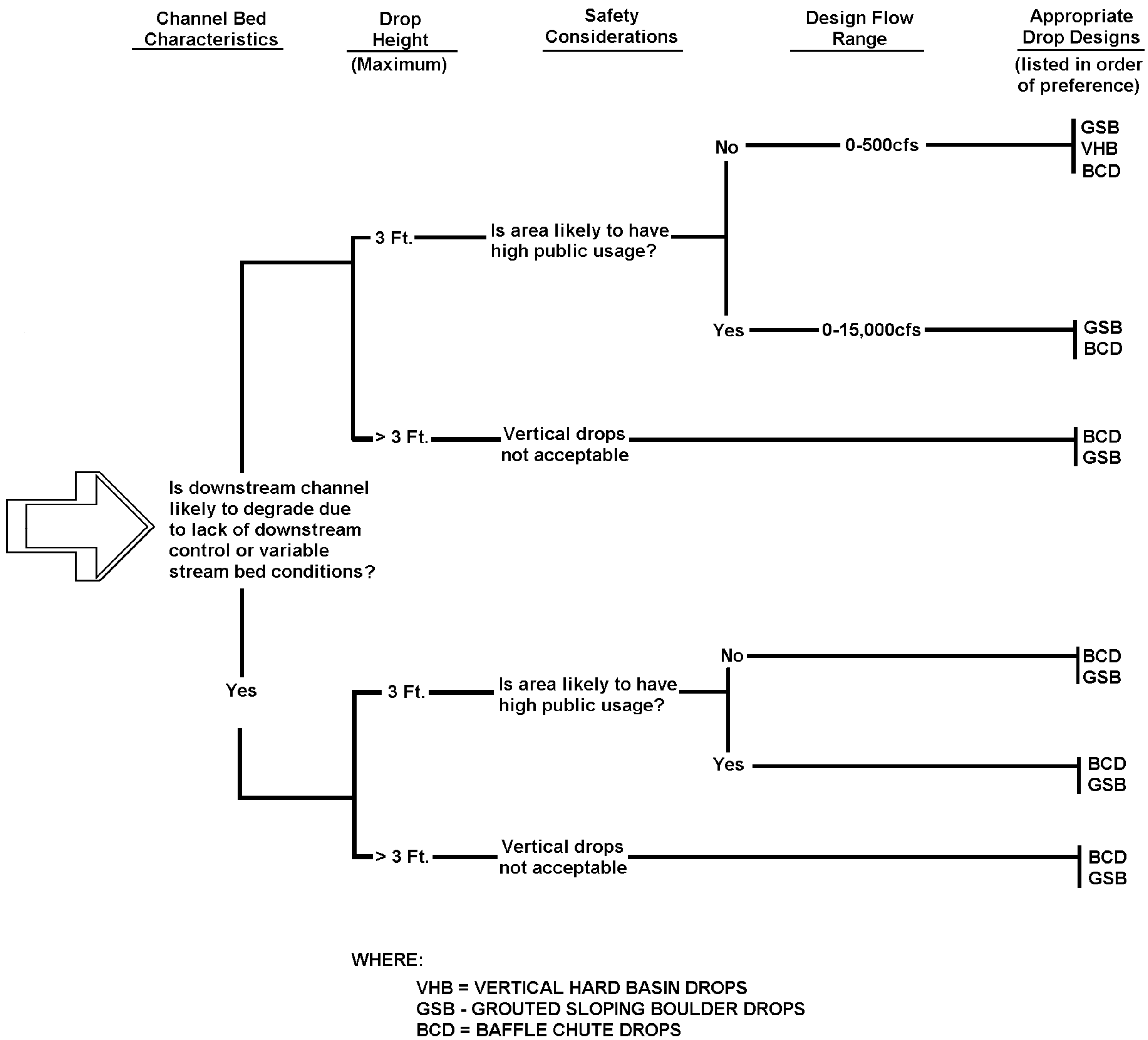


Figure HS-1—Probable Range of Drop Choices and Heights

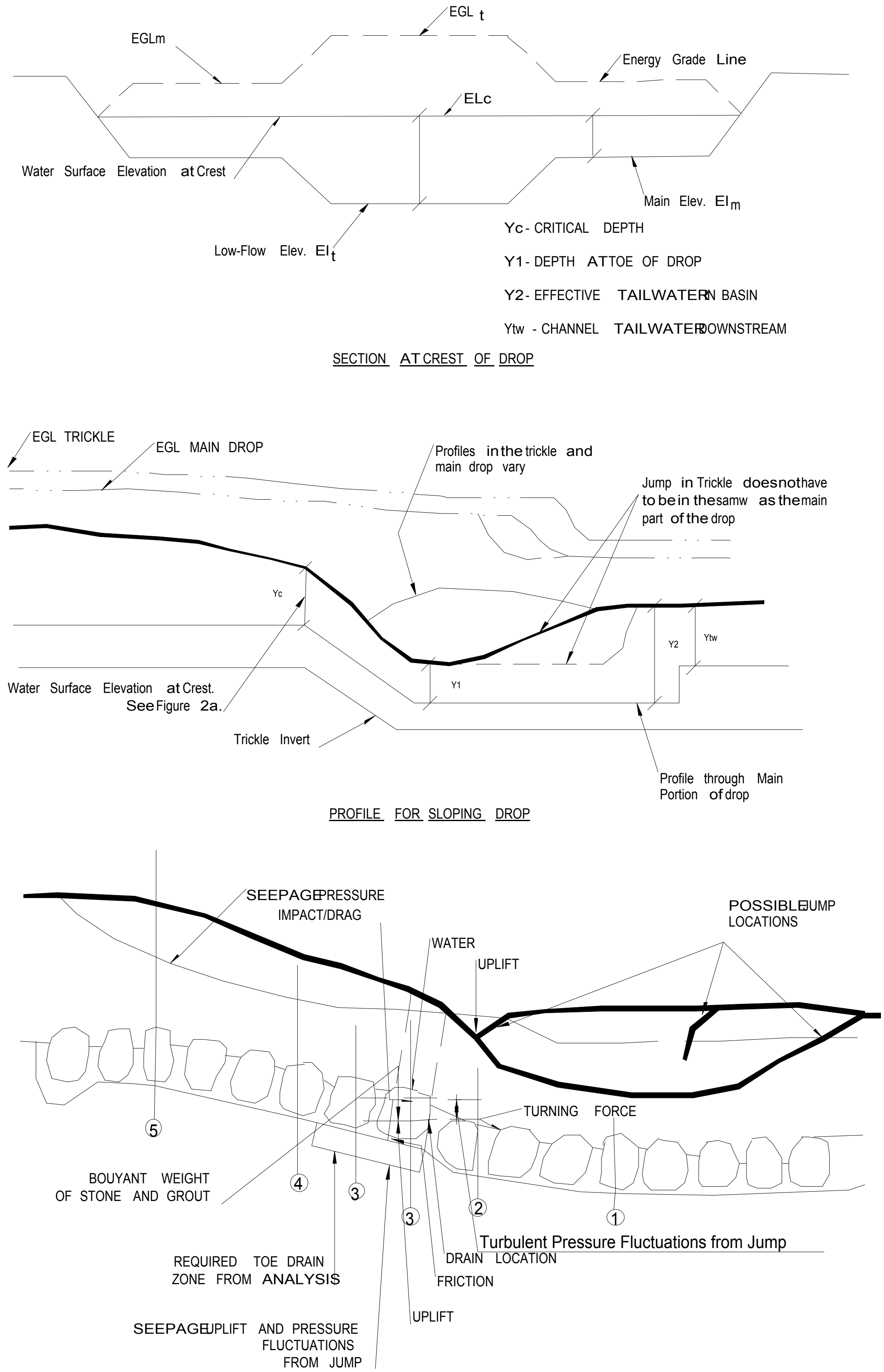
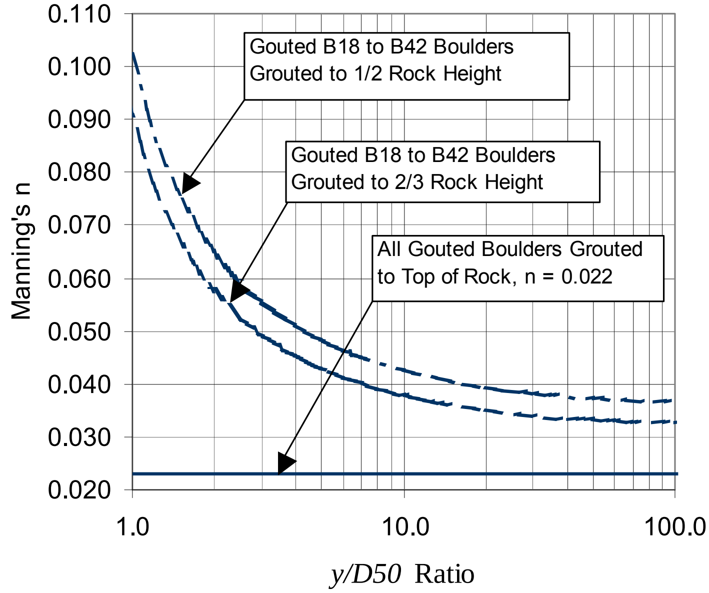


Figure HS-2—Hydraulic Analysis and Typical Forces at Sloping Boulder Drops



The following equations may be used to find the recommended Manning's n as a function of flow depth over height of the boulders, $y/D50$ represented by the above two curves:

When the upper one-half (+/- 1") of the rock depth (height) is left ungrouted, the equation for n is:

$$n_{18''-42''(1/2)} = \frac{0.086 \cdot y^{0.17}}{\ln(1.64 \cdot y)}$$

Upper limit: $n \leq 0.15$ for above equation

When the upper one-third (+/- 1") of the rock depth (height) is left ungrouted, the equation for n is:

$$n_{18''-42''(2/3)} = \frac{0.086 \cdot y^{0.17}}{\ln(2.46 \cdot y)}$$

Upper limit: $n \leq 0.12$ for above equation

In both, y = depth of flow above top of rock, in feet

When rock is grouted to the top of the rock, Manning's is a constant $n = 0.022$.

Note that grouting only the lower ½ of the rock on the sloping face of the drop has 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.

Figure HS-3—Recommended Manning's *n* for Flow Over B18 to B42 Grouted Boulders

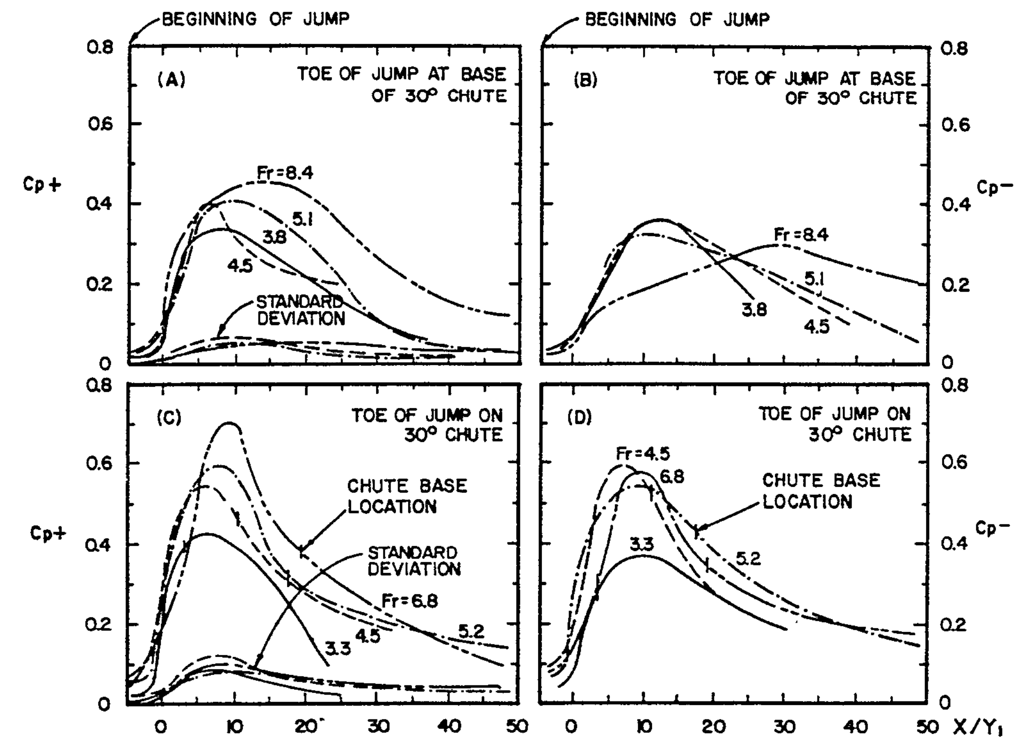


Figure HS-4—Coefficient of Pressure Fluctuation, C_p, at a Hydraulic Jump

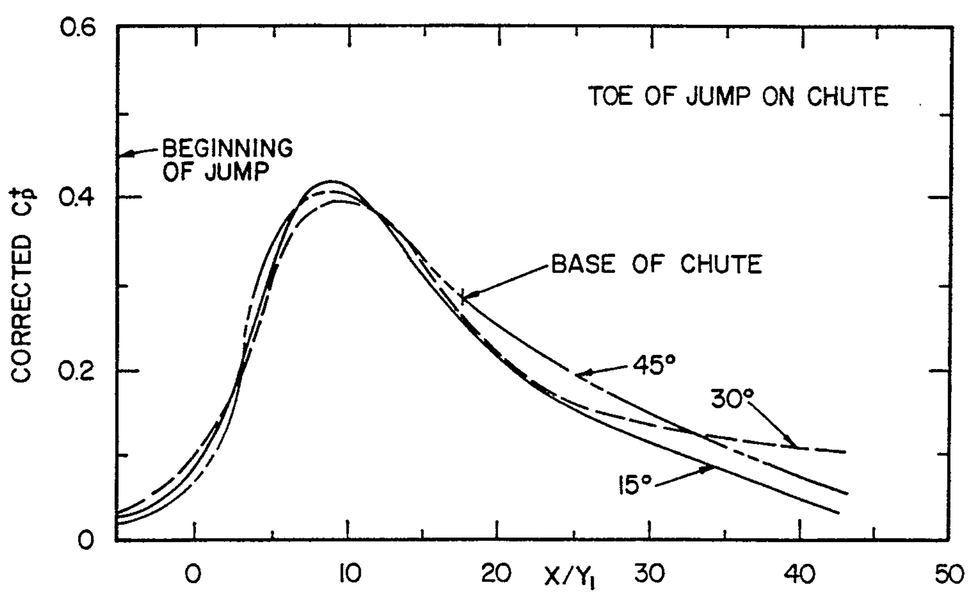


Figure HS-5—Pressure Fluctuation Coefficient, C_p, Normalized for Consideration of Slope and Jump Beginning on Slope

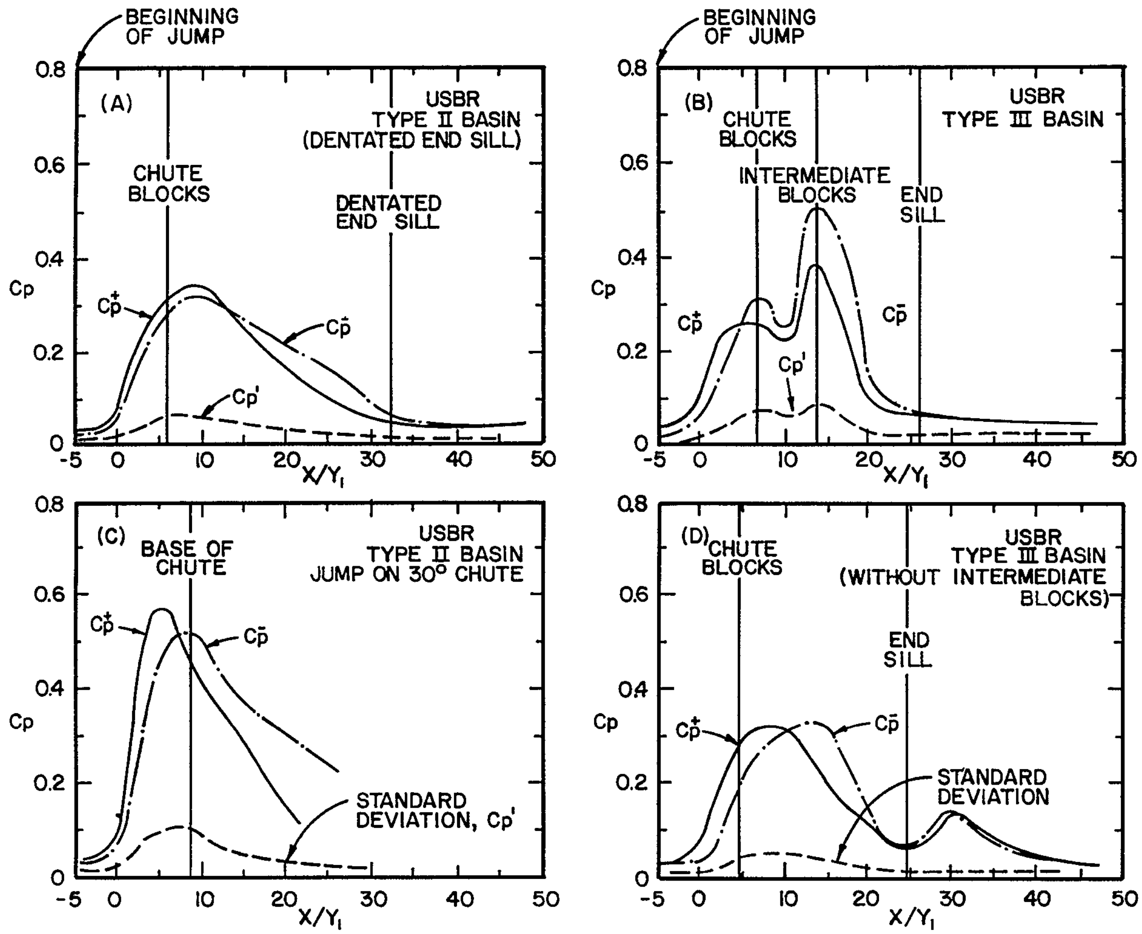


Figure HS-6—Coefficient of Pressure Fluctuation, C_p, in a Jump on a USBR II or III Basin

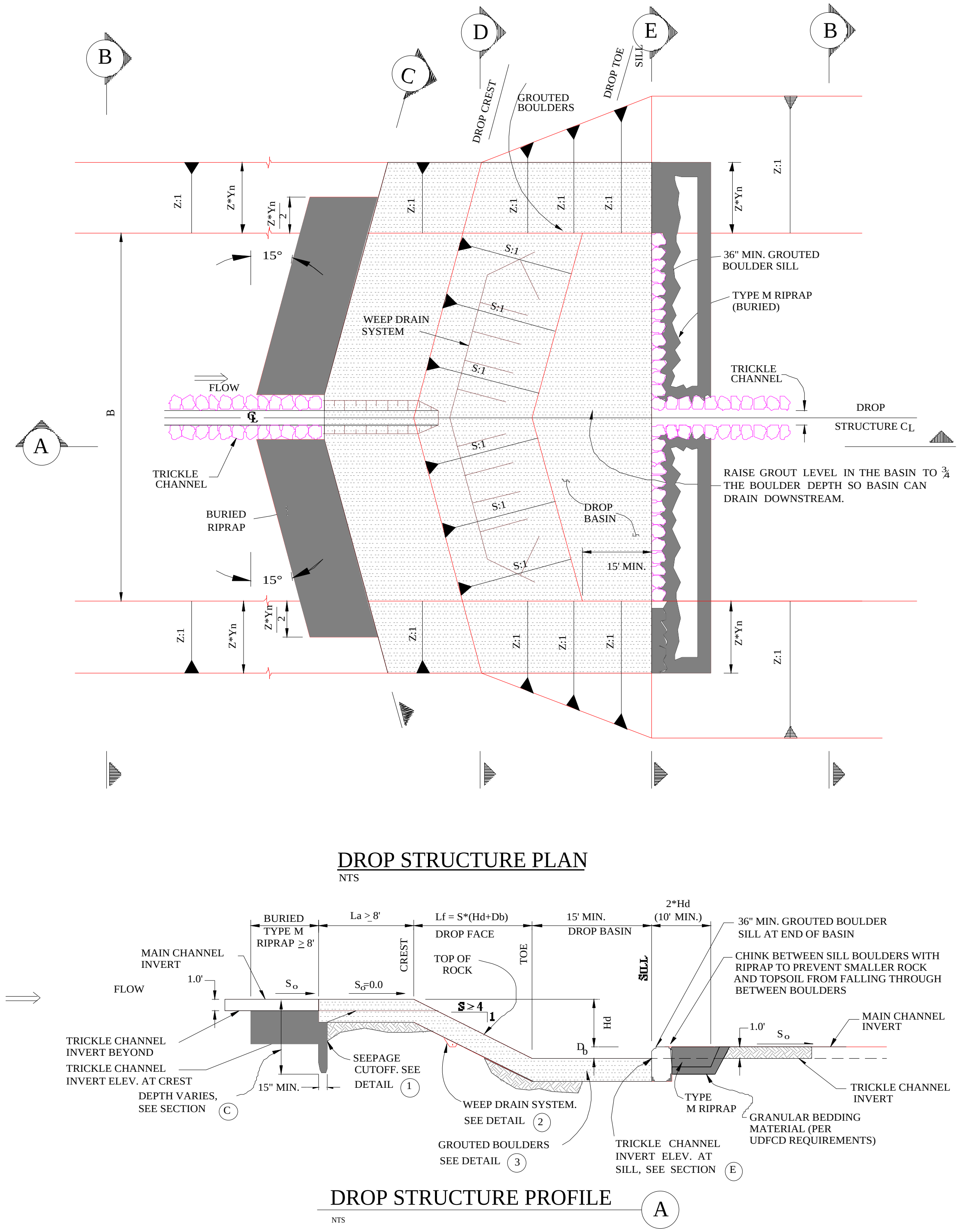


Figure HS-7A—Grouted Sloping Boulder Drop with Trickle Channel for Stabilized Channels in Erosion Resistant Soils (Figure 1 of 2)

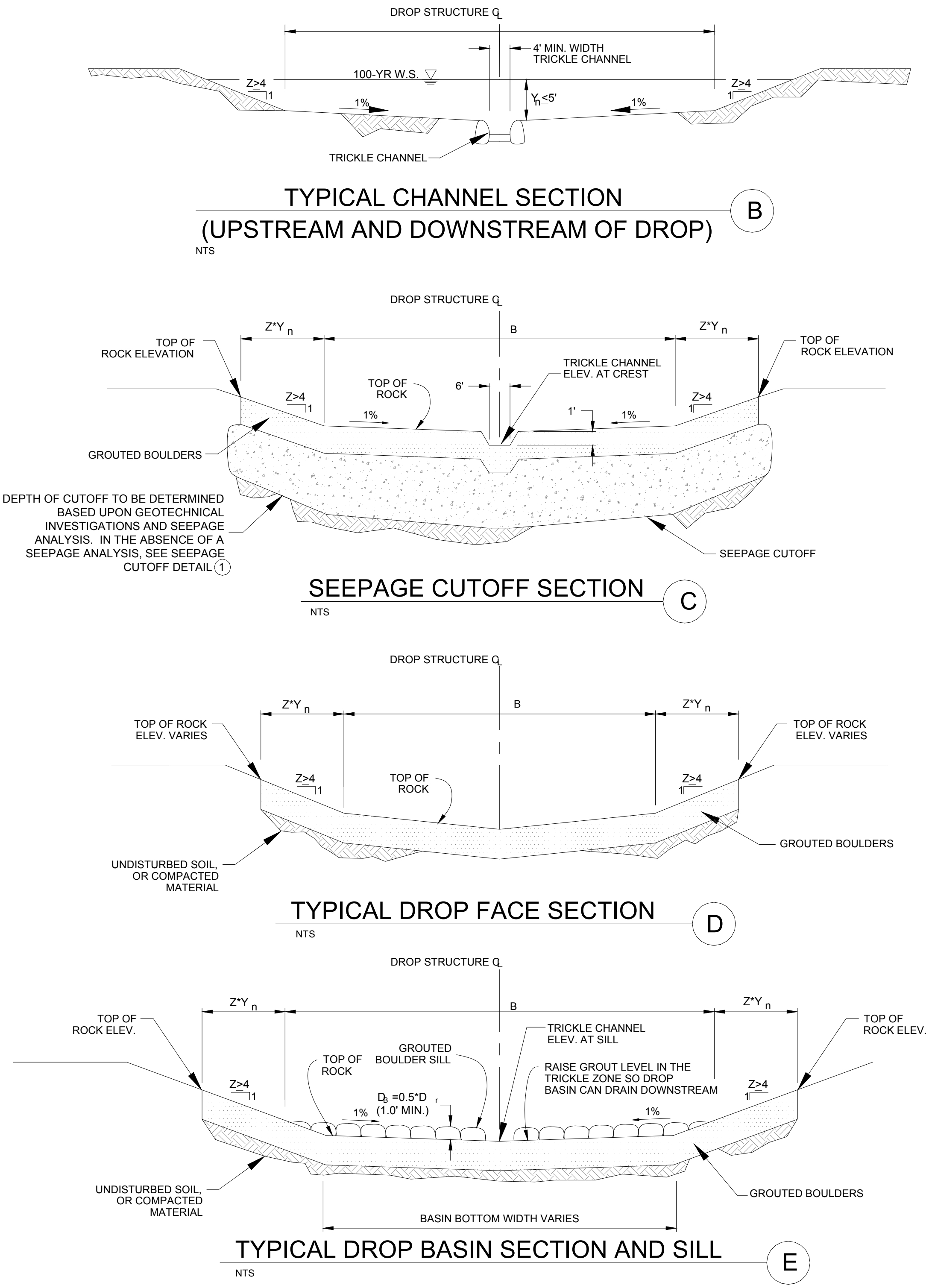


Figure HS-7A— Grouted Sloping Boulder Drop with Trickle Channel for Stabilized Channels and Erosion Resistant Soils (Figure 2 of 2)

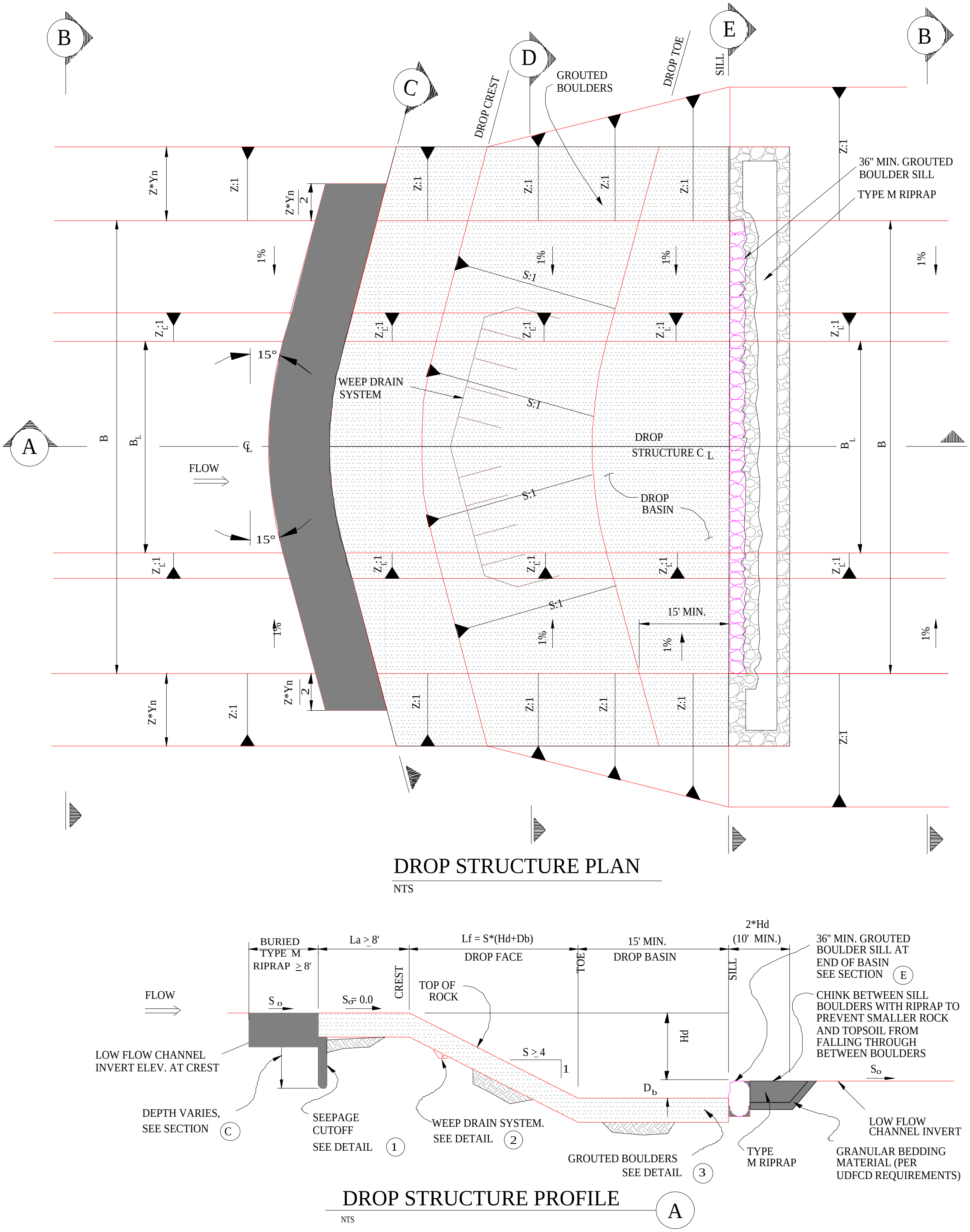
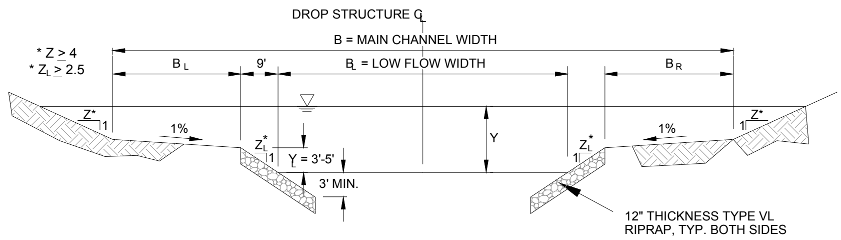


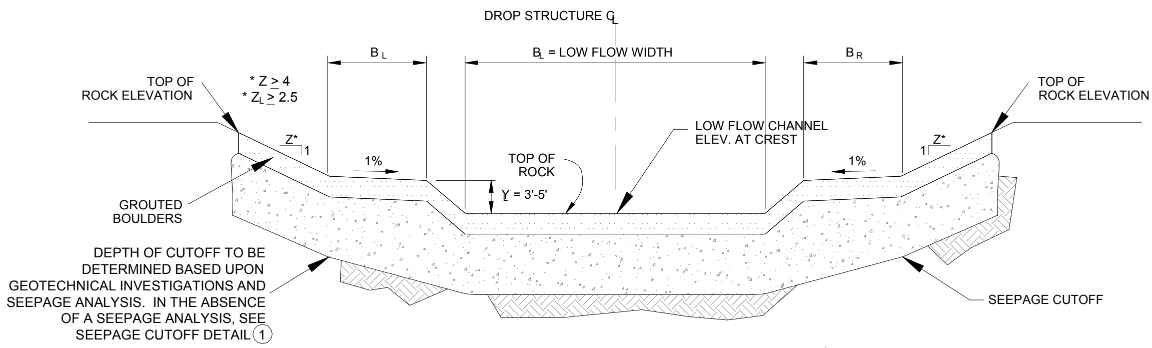
Figure HS-7B—Grouted Sloping Boulder Drop With Low-Flow Channel for Stabilized Channels in Erosion Resistant Soils (Figure 1 of 2)



TYPICAL CHANNEL SECTION
(UPSTREAM AND DOWNSTREAM OF DROP

NTS

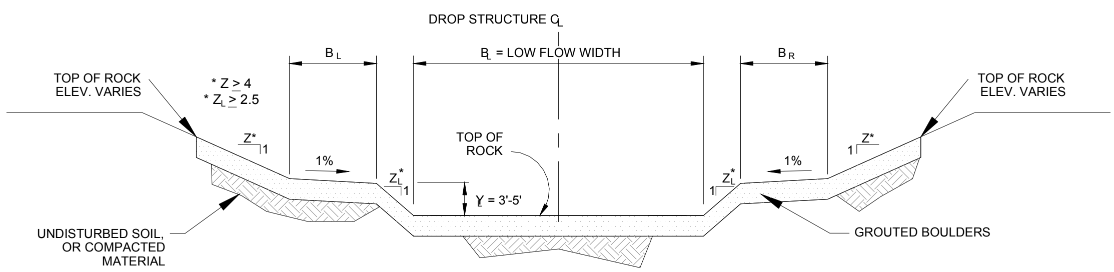
B



SEEPAGE CUTOFF SECTION

NTS

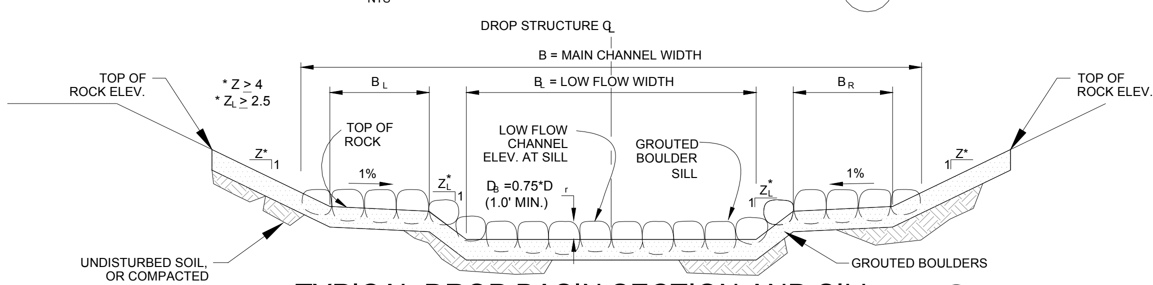
C



TYPICAL DROP FACE SECTION

NTS

D



TYPICAL DROP BASIN SECTION AND SILL

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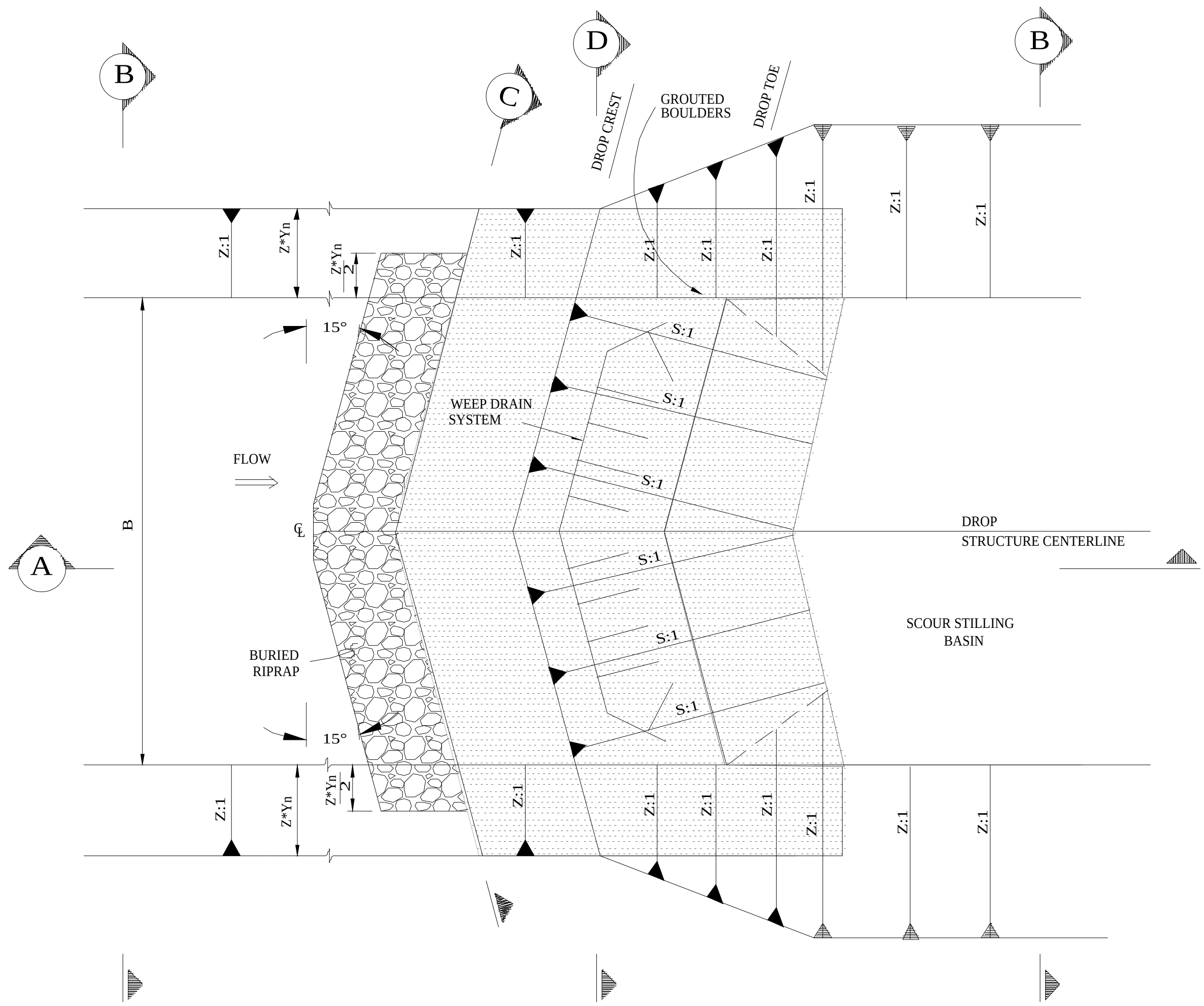
Figure HS-7B— Grouted Sloping Boulder Drop With Low-Flow Channel For Stabilized Channels and Erosion Resistant Soils (Figure 2 of 2)



DROP STRUCTURE PLAN
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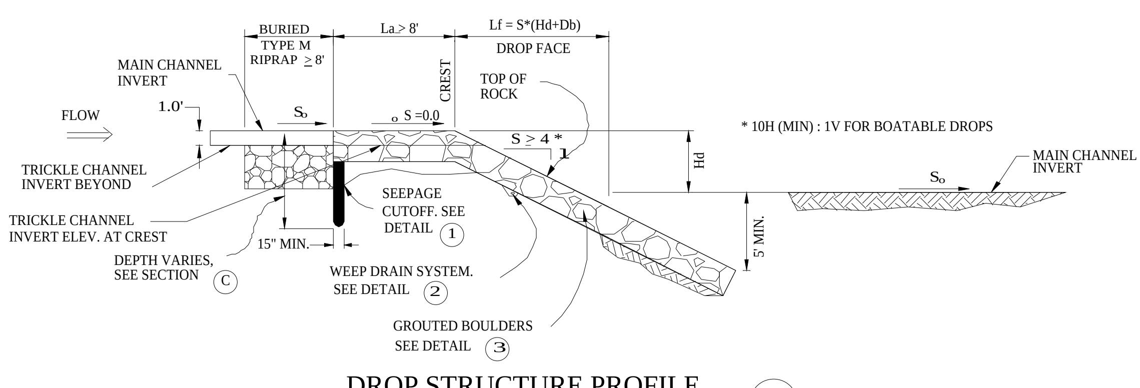

DROP STRUCTURE PROFILE (A)
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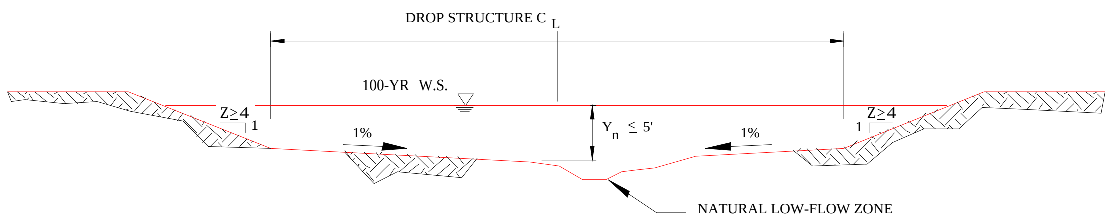
Figure HS-7C—Grouted Sloping Boulder Drop for Unstable Channels in Erosive Soils (Figure 1 of 2)



TYPICAL CHANNEL SECTION
(UPSTREAM AND DOWNSTREAM OF DROP) B

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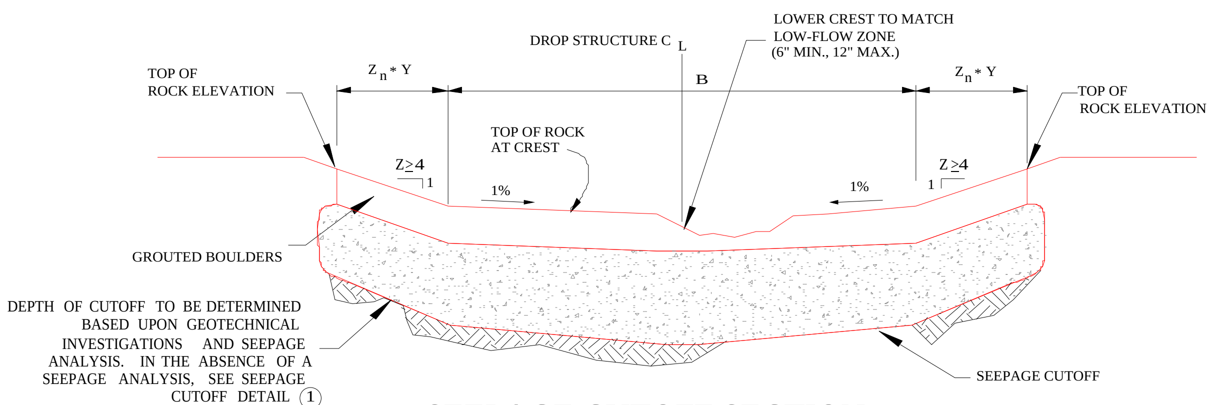

SEEPAGE CUTOFF SECTION C

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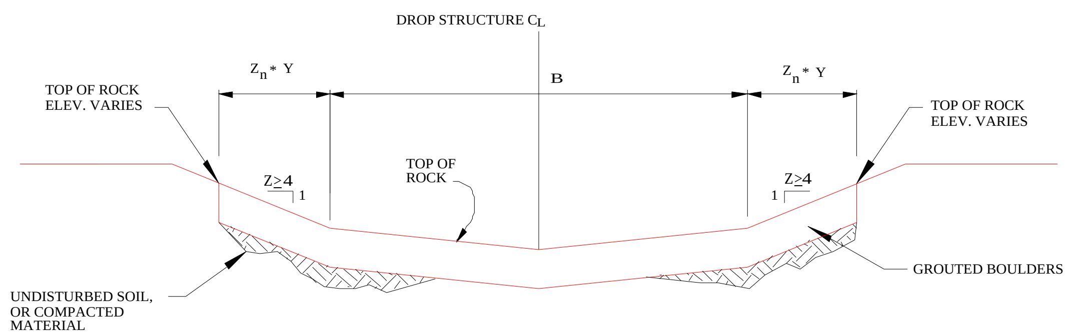

TYPICAL DROP FACE SECTION D

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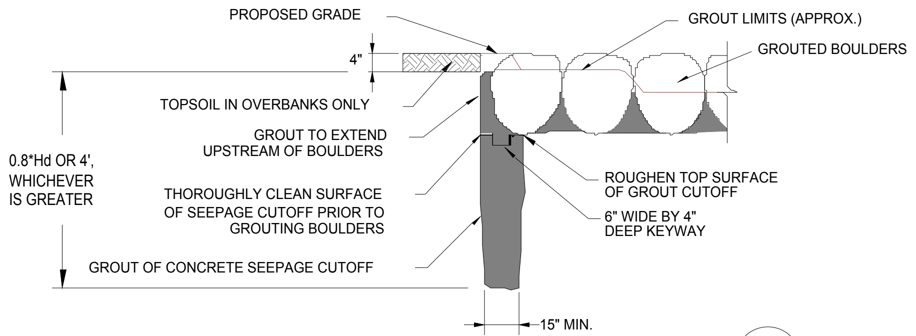
Figure HS-7C— Grouted Sloping Boulder Drop for Unstable Channels in Erosive Sandy Soils. (Figure 2 of 2)



SEEPAGE CUTOFF DETAIL 1

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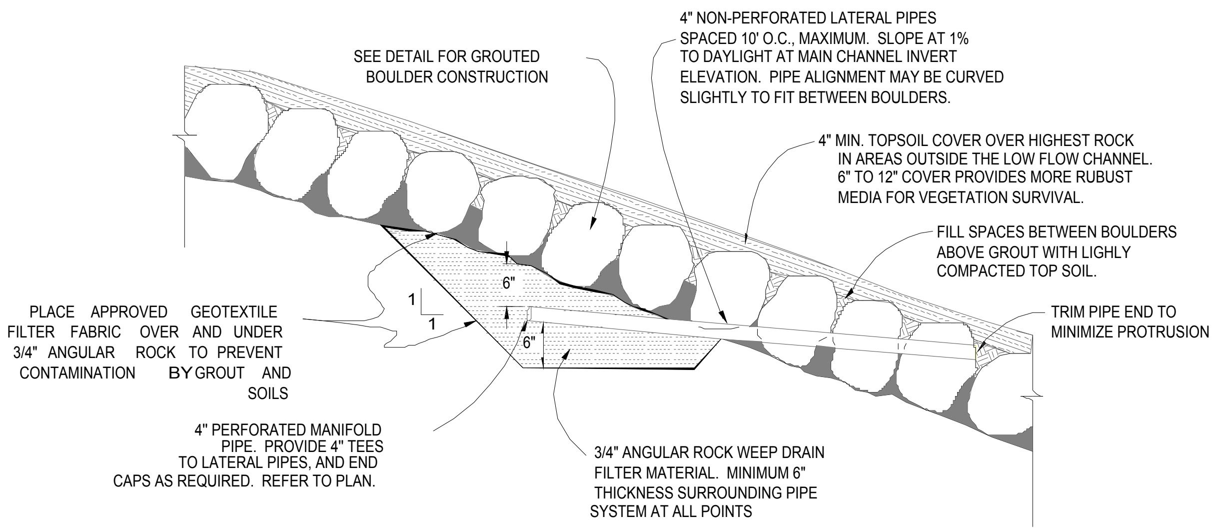

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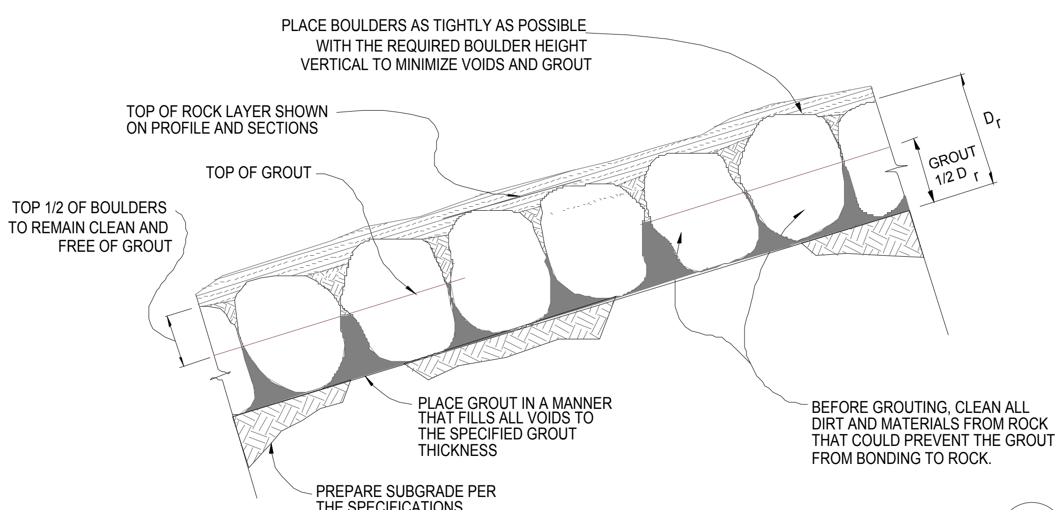
WEEP DRAIN SYSTEM DETAILS 2

USE ONLY IN DROPS HIGHER THAN 5 FEET



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GROUTED BOULDER PLACEMENT DETAIL 3

Figure HS-7D— Grouted Sloping Boulder Drop Details.

GROUT NOTES

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. For Type A grout, the aggregate shall be comprised of 70% natural sand (fines) and 30% $\frac{3}{8}$ -inch rock (coarse).
5. For Type B grout, the aggregate shall be comprised of $\frac{3}{4}$ -inch maximum gravel, structural concrete aggregate.
6. Type B grout shall be used in streams with significant perennial flows.
7. The grout slump shall be 4-inches to 6-inches.
8. Air entrainment shall be 5.5%-7.5%.
9. To control shrinkage and cracking, 1.5 pounds of Fibermesh, or equivalent, shall be used per cubic yard of grout.
10. Color additive in required amounts shall be used when so specified by contract.

Placement Specifications

1. All Type A grout shall be delivered by means of a low pressure (less than 10 psi) grout pump using a 2-inch diameter nozzle.
2. All Type B grout shall be delivered by means of a low pressure (less than 10 psi) concrete pump using a 3-inch diameter 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 grout fills, while vibrating grout into place using a pencil vibrator.
4. After grout placement, exposed boulder faces shall be cleaned with a wet broom.
5. All grout between boulders shall be treated with a broom finish.
6. All finished grout surfaces shall be sprayed with a clear liquid membrane curing compound as specified in ASTM C-309.
7. 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.
8. Clean Boulders by brushing and washing before grouting.

Figure HS-8—Specifications and Placement Instructions for Grout in Sloping Boulder Drops.

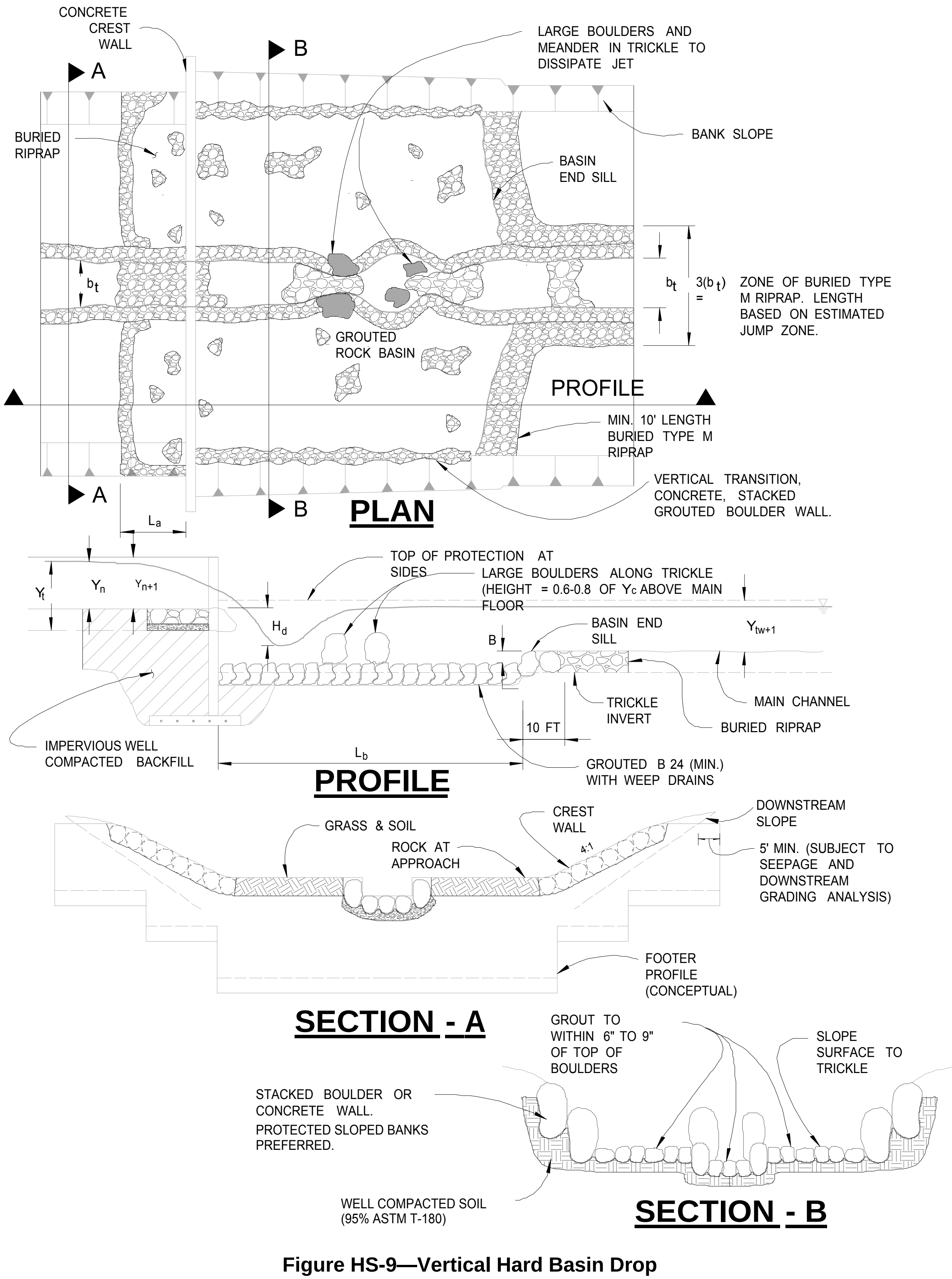


Figure HS-9—Vertical Hard Basin Drop

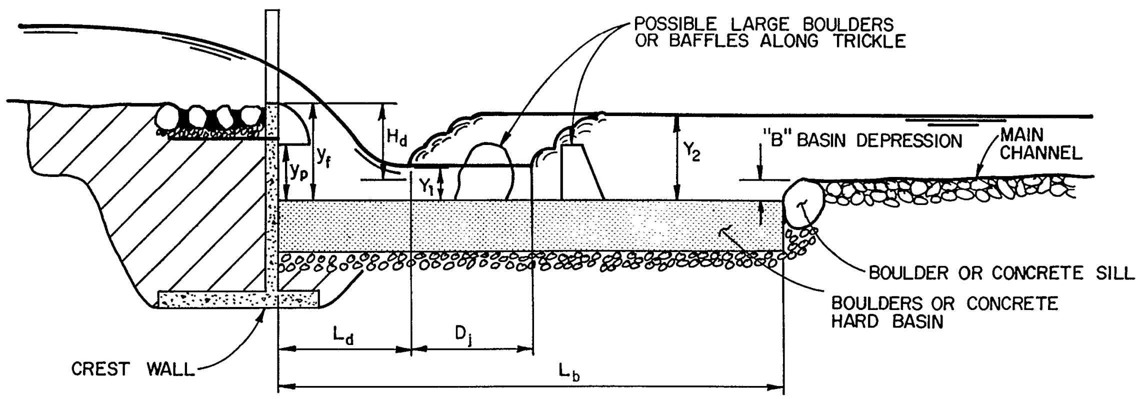
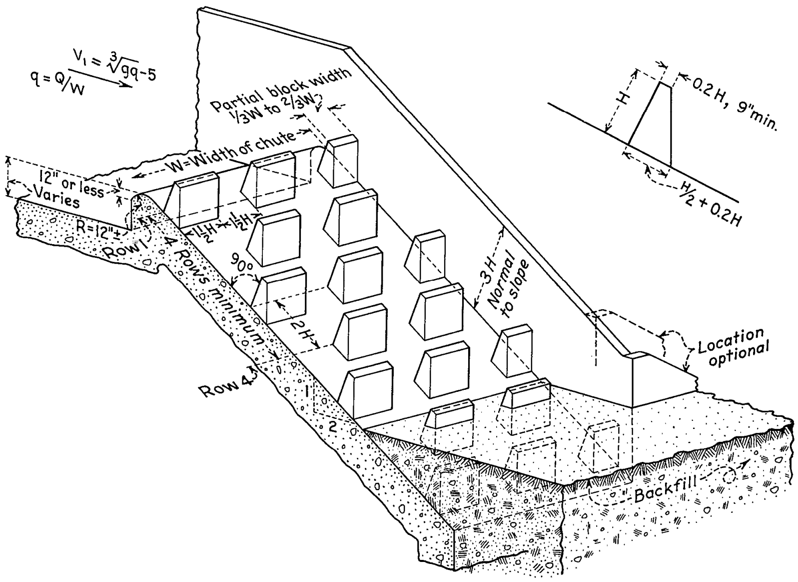
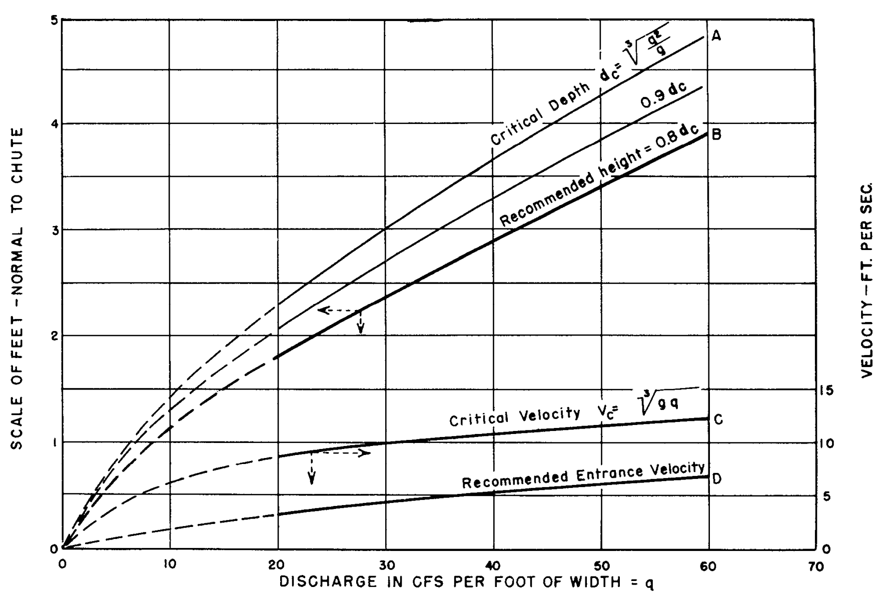


Figure HS-10—Vertical Drop Hydraulic System



(A) USBR ISOMETRIC



(B) DESIGN CRITERIA

Figure HS-11—Baffle Chute Drop Standard USBR Entrance

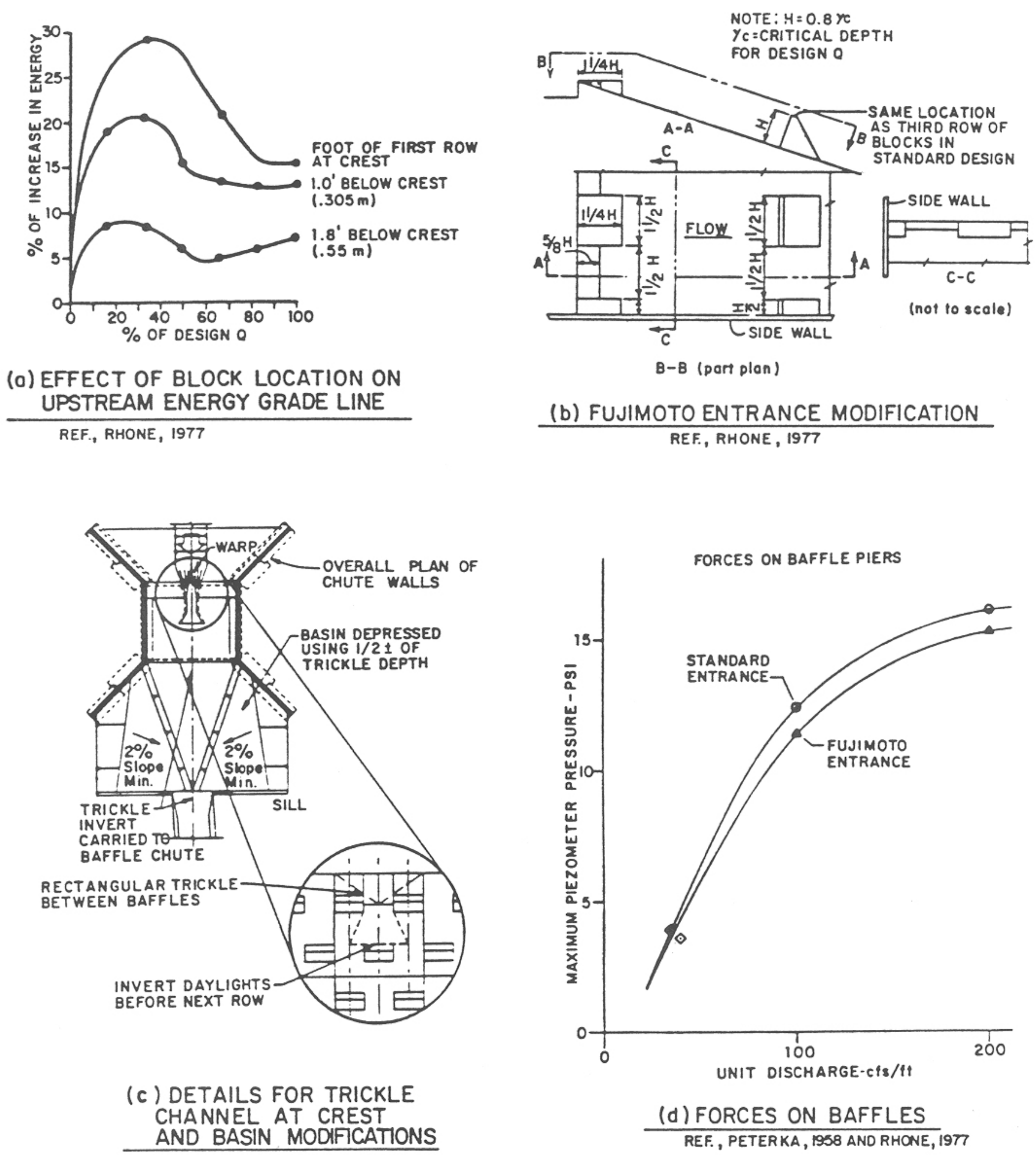
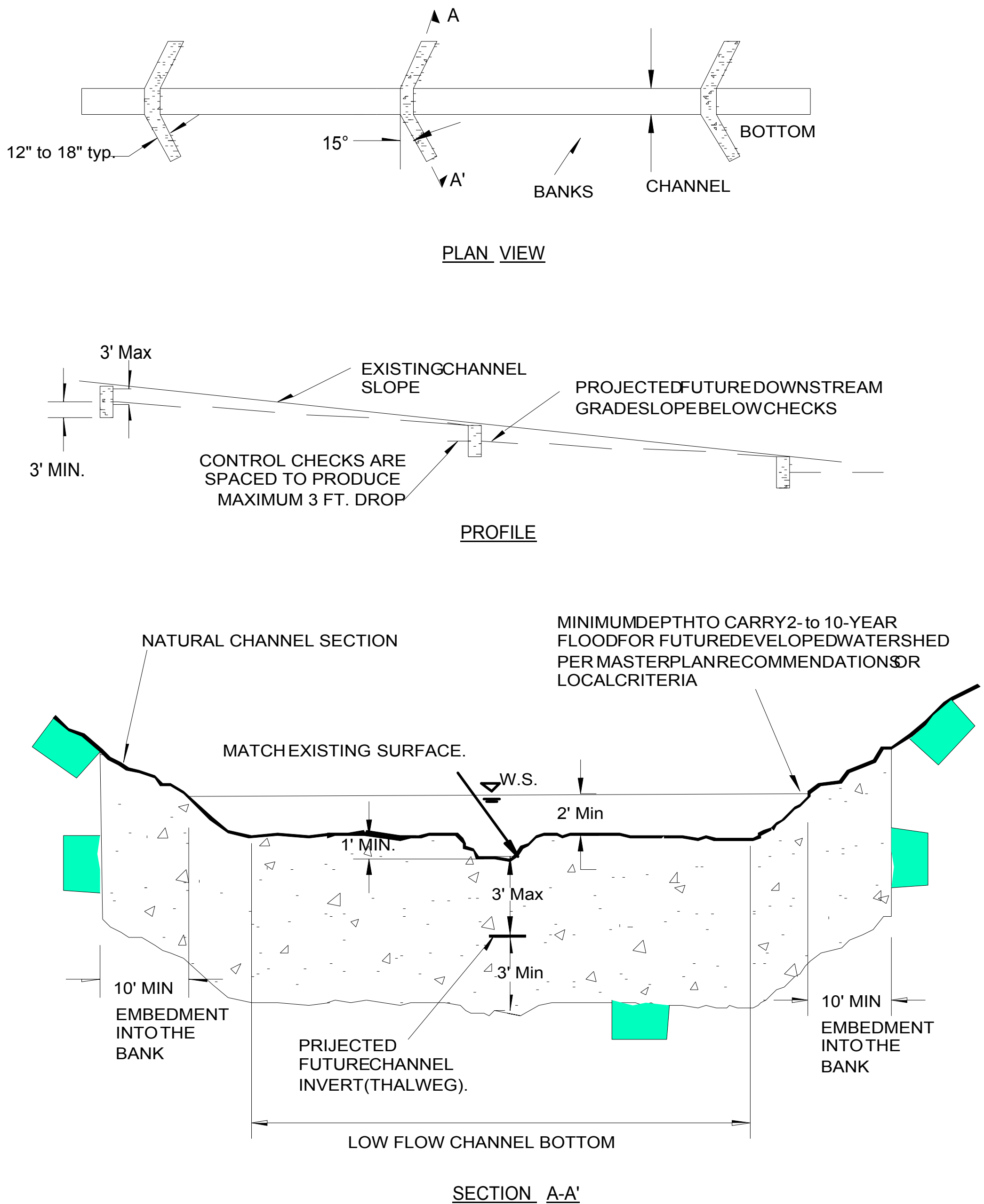


Figure HS-12—Baffle Chute Crest Modifications and Forces



NOTES:
1. TRENCH IN UNDISTURBED SOIL. DO NOT EXCAVATE TO FORM WALLS.
2. POUR AND VIBRATE CONCRETE INTO TRENCH.
3. IF SOIL CANNOT BE TRENCHED WITHOUT WALLS COLLAPSING, USE SHEET PILE CHECK SHOWN IN FIGURE HS-13b.

Figure HS-13a—Control Check for Stable Floodplain – Concrete Wall

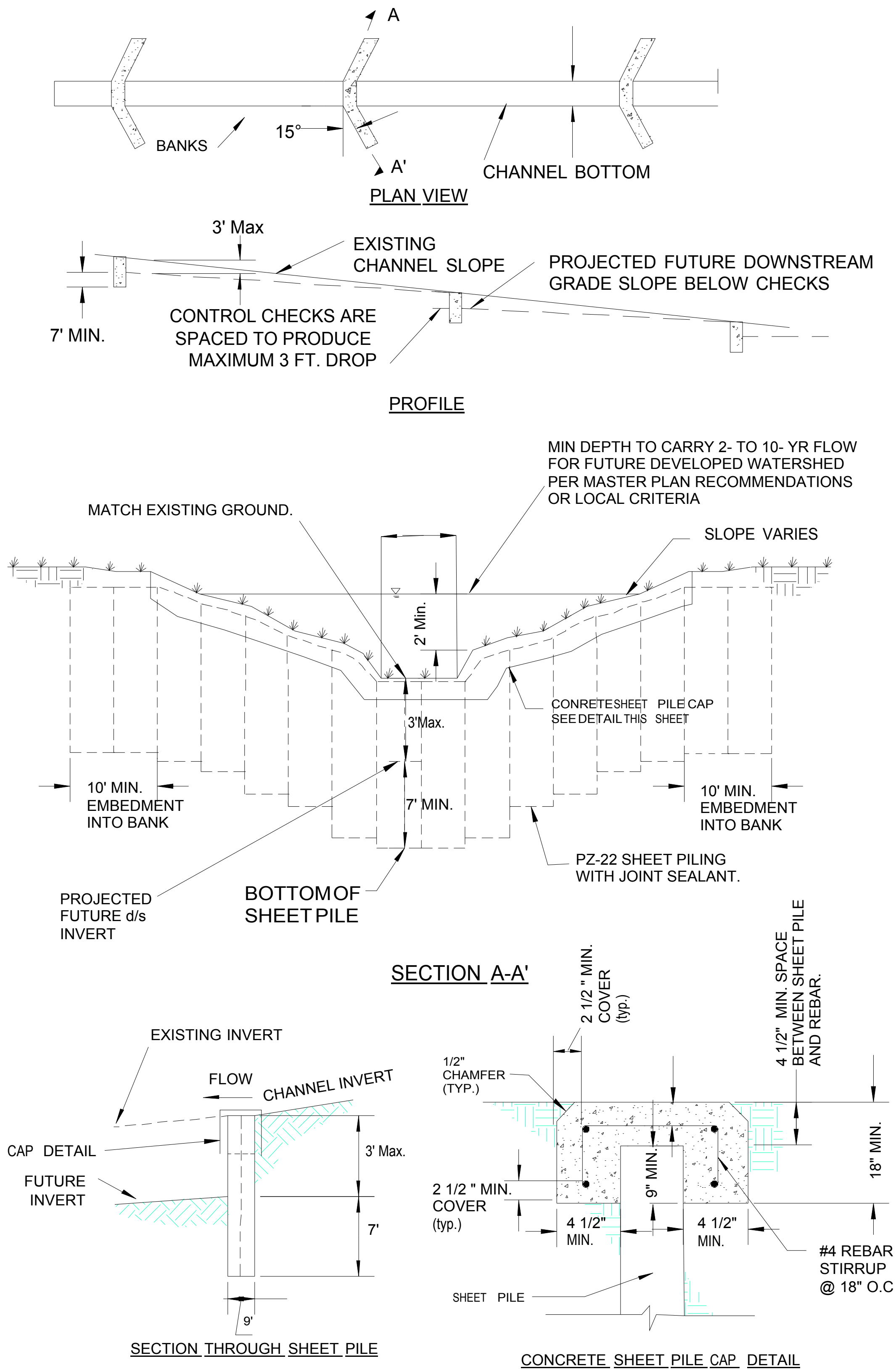


Figure HS-13b—Control Check for Stable Floodplain – Sheet Piling Type

3.0 CONDUIT OUTLET STRUCTURES

3.1 General

Energy dissipation or stilling basin structures are required to minimize scour damages caused by high exit velocities and turbulence at conduit outlets. Similarly, culverts nearly always require special consideration at their 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 (see the MAJOR DRAINAGE chapter) 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.

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.

Hydraulic concepts and design criteria are provided in this section for an impact stilling basin and adaptation of a baffle chute to conduit outlets. 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: (1) high-energy dissipation efficiency is required, where hydraulic conditions approach or exceed the limits for alternate designs (see the MAJOR DRAINAGE chapter); (2) low tailwater control is anticipated; or (3) 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.

Longer conduits with large cross-sectional areas are designed 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 IV 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 wet detention pond or a lake. Alternate designs of pipe exit energy dissipators are provided in this *Manual* that can be matched to a variety of pipe sizes and pipe outlet physical and hydraulic settings.

3.2 Impact Stilling Basin

Most design standards for an impact stilling basin are based on the USBR Type VI basin, often called “impact dissipator” or conduit “outlet stilling basin”. This basin is a relatively small structure that is very efficient energy in dissipating energy without the need of tailwater. The original hydraulic design reference by 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 use of this outlet basin is limited only by structural and economic considerations.

Energy dissipation is accomplished through the turbulence created by the loss of momentum as flow entering the basin impacts a large overhanging baffle. At high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then redirected under the baffle to the open basin and out to the receiving channel. A check at the basin end reduces exit velocities by breaking up the flow across the basin floor and improves the stilling action at low to moderate flow rates.

The generalized, slightly modified, USBR Type IV Impact Basin design configuration is shown in Figure HS-14, 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 HS-15. 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. The alternate end transition (at 45 degrees) is recommended for grass-lined channels to reduce the downstream scour potential.

The impact basin can also be adapted to multiple pipe installations. Such modifications are discussed later, but it should be noted that modifications to the design may affect the hydraulic performance of the structure. Model testing of designs that vary significantly from the standard is recommended.

3.2.1 Modified Impact Basins for Smaller Outlets

For smaller pipe outlets a modified version of the USBR Type IV Impact Basin is suggested in this *Manual*. Figure HS-16a provides a design layout for circular outlets ranging in size from 18-inches to 48-inches in diameter and Figure HS-16b for pipes 18-inches in diameter and smaller. The latter was added for primary use as an outlet energy dissipator upstream of forebays of small extended detention basins, sand filters and other structural best management practices requiring energy dissipation at the end of the pipe delivering water to the BMP facility.

Unlike the Type IV impact basin, the modified basins do not require sizing for flow under normal stormwater discharge velocities recommended for storm sewers in this *Manual*. However, their use is limited to exit velocities of 18 feet per second or less. For larger conduits and higher exit velocities, it is recommended that the standard Type IV impact basin be used instead.

3.2.2 Low-flow Modifications

The standard design will retain a standing pool of water in the basin bottom that is generally undesirable from an environmental and maintenance standpoint. As a result, the standard USBR design has been modified herein for urban applications to allow drainage of the basin bottom during dry periods. This situation should be alleviated where practical by matching the receiving channel low-flow invert to the basin invert. A low-flow gap is extended through the basin end check wall. The gap in the check should be as narrow as possible to minimize effects on the check hydraulics. This implies that a narrow and deeper (1½- to 2-foot) low-flow channel will work better than a shallow and wide gap section.

For the modified impact basin illustrated in Figure HS-16a, the downstream geometry recognizes the need for a trickle channel and also provides for a modification when this structure is used upstream of a forebay in an Extended Detention Basin or other BMP requiring energy dissipation at the entrance.

Low-flow modifications have not been fully tested to date. Caution is advised to avoid compromising the overall hydraulic performance of the structure. Other ideas are possible including locating the low-flow gap at one side (off center) to prevent a high velocity jet from flowing from the pipe straight down the low-flow channel. The optimal configuration results in continuous drainage of the basin area and helps to reduce the amount of siltation.

3.2.3 Multiple Conduit Installations

Where two or more conduits of different sizes outlet in proximity, 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 possible, a more economical approach is to combine storm sewers underground, at a manhole or vault, and bring a single, combined pipe to the outlet structure.

When using a Type IV impact basin shown in Figure HS-14 for two side-by-side pipes of the same size, the two pipes may discharge into a single basin. If the basin's design width for each pipe is W, the combined basin width for two pipes would be $1.5W$. When the flow is different for the two conduits, the design width W is based on the pipe carrying the higher flow. For the modified impact basin shown in Figure HS-16, 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 has not been model tested to date.

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 when designing the Type IV basins. Use of a handrail is suggested around the open basin areas where safety is a concern. Access control screens or grating where necessary are a separate design consideration. A hinged rack has been used on a few projects in the District.

3.2.4 General Design Procedure for Type IV Impact Basin

1. Determine the design hydraulic cross-sectional area just inside the pipe, at the outlet. Determine

the effective flow velocity, V, at the same location in the pipe. Assume depth $D = (A_{sect})^{1/2}$ and compute the Froude number = $\dfrac{V}{(gD)^{1/2}}$

2. The entrance pipe should be turned 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 HS-15. The remaining dimensions are proportional to the basin width according to Figure HS-14. The basin width should not be oversized since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.

4. Structure wall thickness, steel reinforcement, and anchor walls (underneath the floor) should be designed using accepted structural engineering methods. Note that 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 determination of the hydraulic jet force at the outlet:

   F_j = 1.94 $V_{out} Q_{des}$ (force in pounds) (HS-15)

   Q_{des} = maximum design discharge (cfs)

   V_{out} = velocity of the outlet jet (ft/sec)

5. Type “M” rock riprap should be provided in the receiving channel from the end check to a minimum distance equal to the basin width. The depth of rock should be equal to the check height or at least 2.0 feet. Rock may be buried to finished grades and planted as desired.

6. The alternate end check and wingwall shown in Figure HS-14 are recommended for all grass-lined channel applications to reduce the scour potential below the check wall.

7. Ideally, the low-flow invert matches the floor invert at the basin end and the main channel elevation is equal to the top of the check. For large basins where the check height, d, becomes greater than the low-flow depth, dimension d in Figure HS-14 may be reduced by no more than one-third. It should not be reduced to less than 2 feet. This implies that a deeper low-flow channel (1.5 to 2.0 feet) will be advantageous for these installations. The alternate when d exceeds the trickle flow depth is that the basin area will not drain completely.

8. A check section should be constructed directly in front of the low-flow notch to break up bottom flow velocities. The length of this check section should overlap the width of the low flow and its dimension is shown in Figure HS-14.

3.3 Pipe Outlet Rundowns

3.3.1 Baffle Chute Rundown

The baffle chute developed by the USBR (1958) has also been adapted to use at pipe outlets. This structure is well suited to situations with large conduit outfalls and at outfalls to channels in which some future degradation is anticipated. As mentioned previously, the apron can be extended at a later time to account for channel degradation. This type of structure is only cost effective if a grade drop is necessary below the outfall elevation.

Figure HS-17 illustrates a general configuration for a baffled outlet application for a double box culvert outlet. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to reduce the flow velocity at the chute entrance. The remaining hydraulic design is the same as for a standard baffle chute using conditions at the crest to establish the design. The same crest modifications are applicable to allow drainage of the approach depression, to reduce the upstream backwater effects of the baffles, and to reduce the problems of debris accumulation and standing water at the upstream row of baffles.

Flow entering the chute should be well distributed laterally across the width of the chute. The velocity should be below critical velocity at the crest of the chute. To insure low velocities at the upstream end, it may be necessary to provide a short energy dissipating pool. The sequent or conjugate depth in the approach basin should be sized to prevent jump sweep-out, but the basin length may be considerably less than a conventional hydraulic jump basin since its primary purpose is only to reduce the average entrance velocity. A basin length of twice the sequent depth will usually provide ample basin length. The end check of the pool may be used as the crest of the chute as shown in Figure HS-17.

3.3.2 Grouted Boulder Chute Rundown

Another option for rundowns at outlets of larger pipes is to use a grouted boulder rundown illustrated in Figure 18. This type of rundown has been used successfully for several large storm sewers entering the South Platte River. It is critical that the details shown in Figure 18 be strictly followed and the grout and the actual filling of spaces between the boulders with grout closely adhere to the recommendations for grouted boulders provided in the Major Drainage Chapter of this *Manual*.

If the exit velocities of the pipe exceeds 12 feet per second, an approach chute for the baffle chute rundown described above should be considered and provided. If this approach chute is lined with grouted boulders in a manner called for in the Major Drainage Chapter, the stilling basin sill can be eliminated.

3.4 Low Tailwater Riprap Basins at Pipe Outlets

3.4.1 General

The design of low tailwater riprap basins for storm sewer pipe outlets and at some culvert outlets is necessary when the receiving or downstream channel may have little or no flow or tailwater at time when the pipe or culvert is in operation. Design criteria are provided in Figures HS-19a through HS-20c.

3.4.2 Objective

By providing a low tailwater basin at the end of a storm sewer conduit or culvert, the kinetic energy of the discharge is dissipated under controlled conditions without causing scour at the channel bottom. Photograph HS-12 shows a fairly large low tailwater basin.

3.4.3 Low Tailwater Basin Design

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

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

in which:

y_t = tailwater depth at design

D = diameter of circular pipe (ft)

H = height of rectangular pipe (ft)

3.4.3.1 Finding Flow Depth and Velocity of Storm Sewer Outlet Pipe

The first step in the design of a scour protection basin at the outlet of a storm sewer is to find the depth and velocity of flow at the outlet. Pipe-full flow can be found using Manning's equation.

$$Q_{full} = \frac{1.49}{n} A_{full} \left(R_{full}\right)^{2/3} S_o^{1/2} \qquad \text{(HS-16a)}$$

Then and the pipe-full velocity can be found using the continuity equation.

$$V_{full} = Q_{full} / A_{full} \qquad \text{(HS-16a)}$$

The normal depth of flow, d, and the velocity in a conduit can be found with the aid of Figure HS-20a and Figure HS-20b. Using the known design discharge, Q, and the calculated pipe-full discharge, Q_{full}, enter Figure HS-20a with the value of Q/Q_{full} and find d/D for a circular pipe of d/H for a rectangular pipe.

Compare the value of d/D (or d/H) with the one obtained from Figure HS-20b using the Froude parameter.

$$Q/D^{2.5} \quad \text{or} \quad Q/\left(wH^{1/5}\right) \qquad \text{(HS-16a)}$$

Choose the smaller of the two (d/D or d/H) ratios to calculate the flow depth at the end of the pipe.

$$d = D(d/D) \quad \text{or} \quad d = H(d/H) \qquad \text{(HS-16b)}$$

Again, enter Figure HS-19a using the smaller d/D (or d/H) ratio to find the A/A_{full} ratio. Then,

$$A = \left(A/A_{full}\right)A_{full} \qquad \text{(HS-16c)}$$

Finally,

$$V = Q/A \qquad \text{(HS-16d)}$$

In which for Equations 16a through 16d above:

A_{full} = cross-sectional area of the pipe (ft^2)

A = area of the design flow in the end of the pipe (ft^2)

n = Manning's n for the pipe full depth

Q_{full} = pipe full discharge at its slope (cfs)

R = hydraulic radius of the pipe flowing full, ft [R_{full} = D/4 for circular pipes, R_{full} = A_{full}/(2H + 2w) for rectangular pipes, where D = diameter of a circular conduit, H = height of a rectangular conduit, and w = width of a rectangular conduit (ft)]

S_o = longitudinal slope of the pipe (ft/ft)

V = design flow velocity at the pipe outlet (ft/sec)

V_{full} = flow velocity of the pipe flowing full (ft/sec)

3.4.3.2 Riprap Size

For the design velocity, use Figure HS-20c to find the size and type of the riprap to use in the scour protection basin downstream of the pipe outlet (i.e., B18, H, M or L). First, calculate the riprap sizing design parameter, P_d , namely,

$$P_d = \left(V^2 + gd\right)^{1/2} \qquad \text{(HS-16e)}$$

in which:

V = design flow velocity at pipe outlet (ft/sec)

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

d = design depth of flow at pipe outlet (ft)

Photograph HS-12—Upstream and downstream views of a low tailwater basin in Douglas County protecting downstream wetland area. Burying and revegetation of the rock would blend the structure better with the adjacent terrain.

When the riprap sizing design parameter indicates conditions that place the design above the Type H riprap line in Figure HS-20, use B18, or larger, grouted boulders. An alternative to a grouted boulder or loose riprap basin is to use the standard USBR Impact Basin VI or one of its modified versions, described earlier in this Chapter of the *Manual*.

After the riprap size has been selected, the minimum thickness of the riprap layer, T, in feet, in the basin is set at:

$$T = 1.75 D_{50} \qquad \text{(HS-17)}$$

in which:

D_{50} = the median size of the riprap (see Table HS-9.)

Table HS-9—Median (i.e., D_{50}) Size of District's Riprap/Boulder

Riprap Type	D_{50}—Median Rock Size (inches)
L	9
M	12
H	18
B18	18 (minimum dimension of grouted boulders)

3.4.3.3 Basin Length

The minimum length of the basin, L, in Figure HS-19, is defined as being the greater of the following:

for circular pipe: $L = 4D$ or $L = (D)^{1/2}\left(\dfrac{V}{2}\right)$ (HS-18) for rectangular pipe: $L = 4H$ or $L = (H)^{1/2}\left(\frac{V}{2}\right)$ (HS-19)

in which:

L = basin length

H = height of rectangular conduit

V = design flow velocity at outlet

D = diameter of circular conduit

3.4.3.4 Basin Width

The minimum width, W, of the basin downstream of the pipe's flared end section is set as follows:

for circular pipes: $W = 4D$ (HS-20)

for rectangular pipe: $W = w + 4H$ (HS-21)

in which,

W = basin width (Figure HS-19)

D = diameter of circular conduit

w = width of rectangular conduit

3.4.3.5 Other Design Requirements

All slopes in the pre-shaped riprapped basin are 2H to 1V.

Provide pipe joint fasteners and a structural concrete cutoff wall at the end of the flared end section for a circular pipe or a headwall with wingwalls and a paved bottom between the walls, both with a cutoff wall that extends down to a depth of:

$$B = \frac{D}{2} + T \quad \text{or} \quad B = \frac{H}{2} + T \qquad \text{(HS-22)}$$

in which,

B = cutoff wall depth

D = diameter of circular conduit

T = Equation HS-17

The riprap must be extended up the outlet embankment's slope to the mid-pipe level.

3.5 Culvert Outlets

Photograph HS-13—Culvert outlets when left unprotected cause downstream erosion. The designer's job is not complete until provisions are made to protect the outlet. Use of vegetated soil-riprap would blend this structure better into the natural landscape.

Culvert outlets represent a persistent problem because of concentrated discharges and turbulence that are not fully controlled prior to the flow reaching the standard downstream channel configuration described in the Major Drainage Chapter of this *Manual*. Too often the designer's efforts are focused on the culvert inlet and its sizing with outlet hydraulics receiving only passing attention. Culvert design is not complete until adequate attention is paid to the outlet hydraulics and proper stilling of the discharge flows.

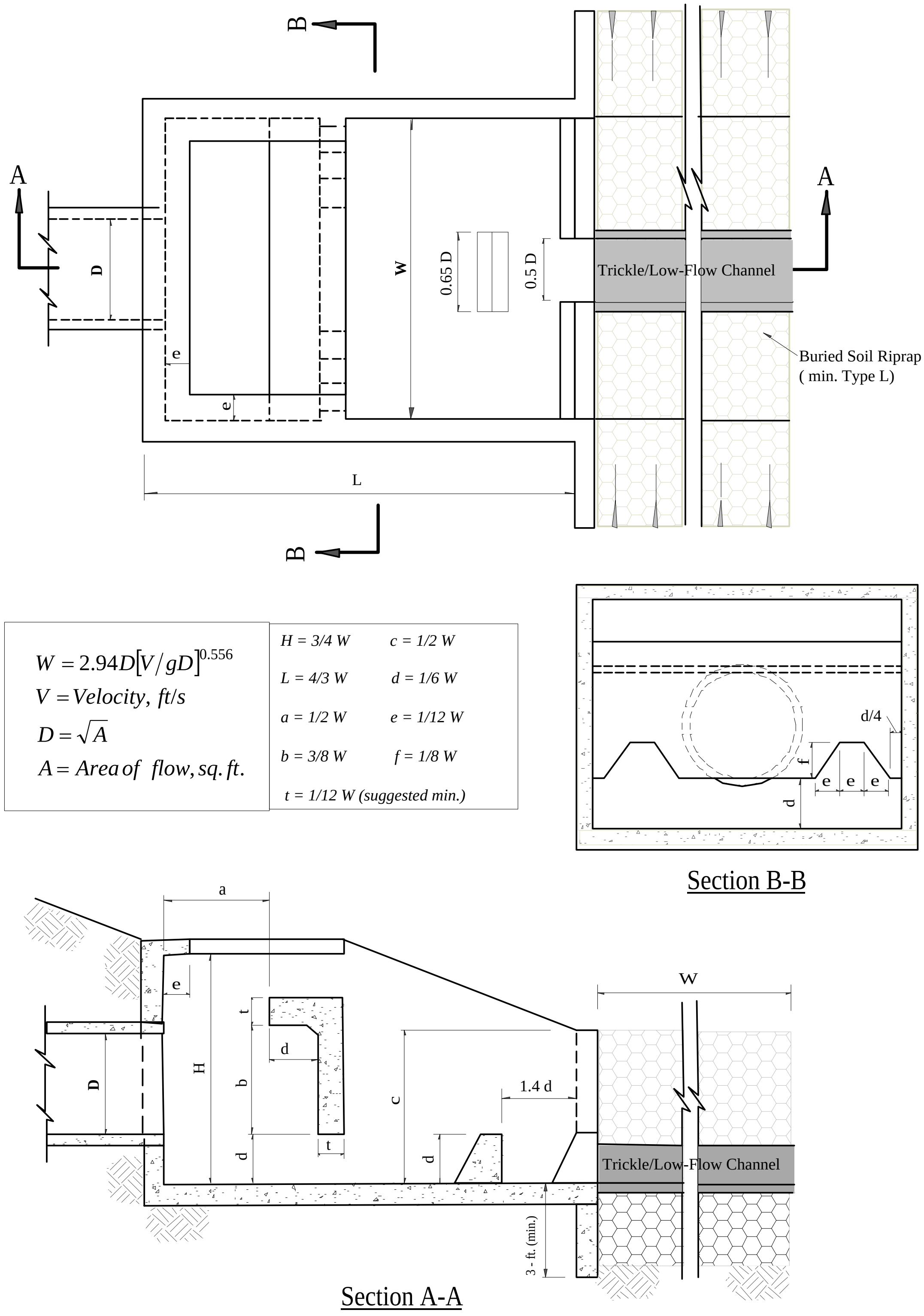
Culvert outlet energy dissipator and flow spreading may require special structures downstream of the culvert outlet to limit local scour, general stream degradation, and troublesome head cutting. Some of the techniques described in Sections 3.2, 3.3 and 3.4 may be applied at culver outlets as well if the downstream channel and/or tailwater conditions so indicate.

Local scour is typified by a scour hole at the pipe's outlet. High exit velocities cause this, and the effects extend only a limited distance downstream. Coarse material scoured from the hole is deposited immediately downstream, often forming a low bar. Finer material is transported further 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 flow when there is minimal tailwater depth at the outlet and not necessarily when the flow is highest. Methods for predicting scour hole dimensions are found in HEC No. 14 (Corry, et al. 1975) and need to be applied using a range of possible tailwater depth conditions during different design storms or floows.

General storm degradation, or head cutting, is a phenomenon independent of culvert performance. Natural causes produce a lowering of the streambed over time. The identification of a degrading stream is an essential part of the original site investigation. However, high-energy discharges from a culvert can often cause stream degradation for a limited in distance downstream. Both scour and steam degradation can occur simultaneously at a culvert outlet.

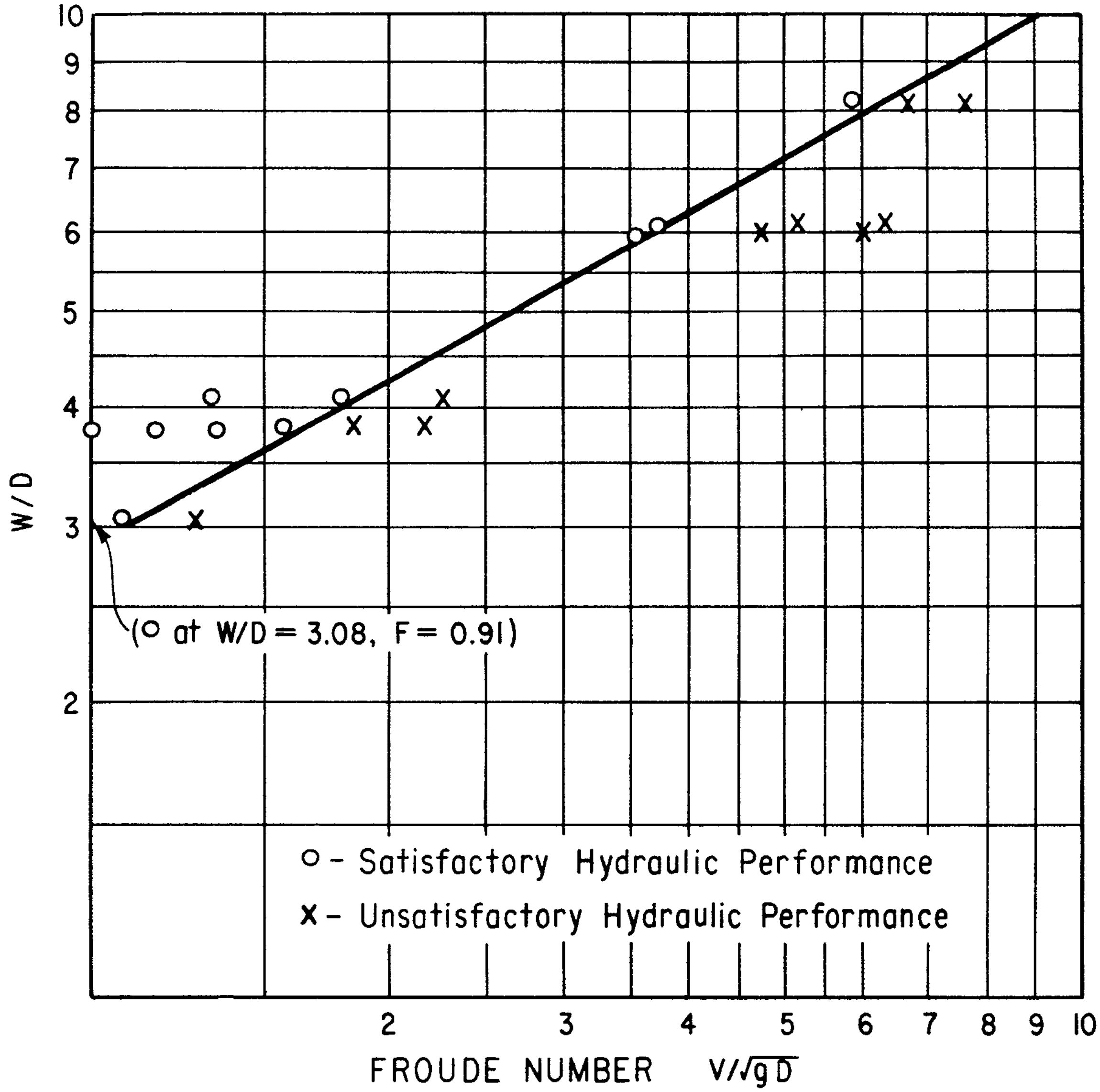
Various measures described in HEC No. *14* and in this *Manual* listed below need to be considered to protect the downstream channel or stream and control culvert outlet flow. It is beyond the scope of this *Manual* to provide detailed information about all available controls in HEC No. 14, but the District encourages the proper application and design as appropriate for the specific site.

1. Colorado State University rigid boundary basin
2. Tumbling flow rectangular section
3. Increased resistance—box culverts
4. Roughness elements—circular culverts
5. USBR Type II
6. USBR Type III
7. USBR Type IV
8. Contra Costa
9. Hook-type energy dissipator
10. Straight drop structure
11. Riprap basins
12. Channel check and drop structures and other energy dissipating and control structures described earlier in this Chapter
13. Use of properly anchored flared end sections – see Figure HS-19a



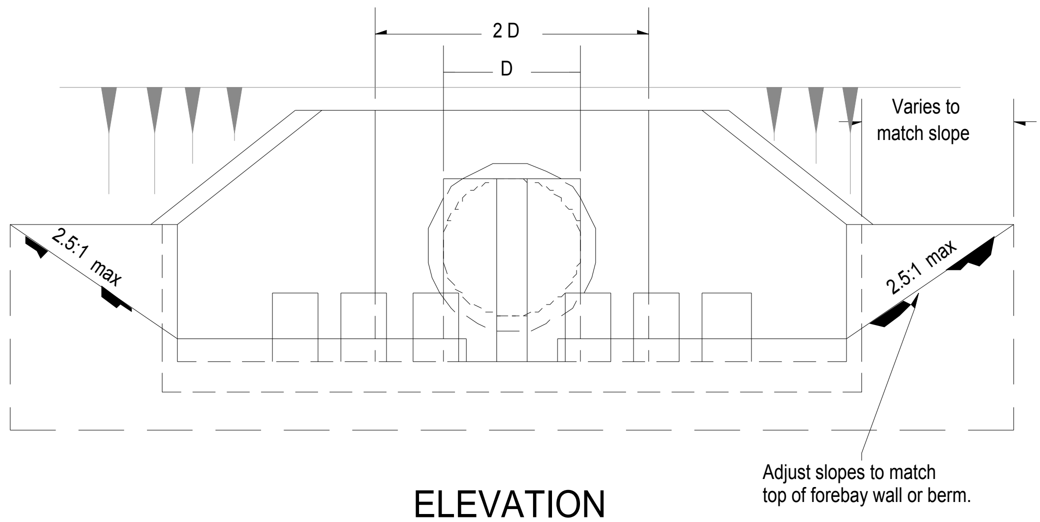
USGS Impact Stilling Basin Modified by UDFCD February 2004

Figure HS-14—General Design Dimensions for a USBR Type VI Impact Stilling Basin

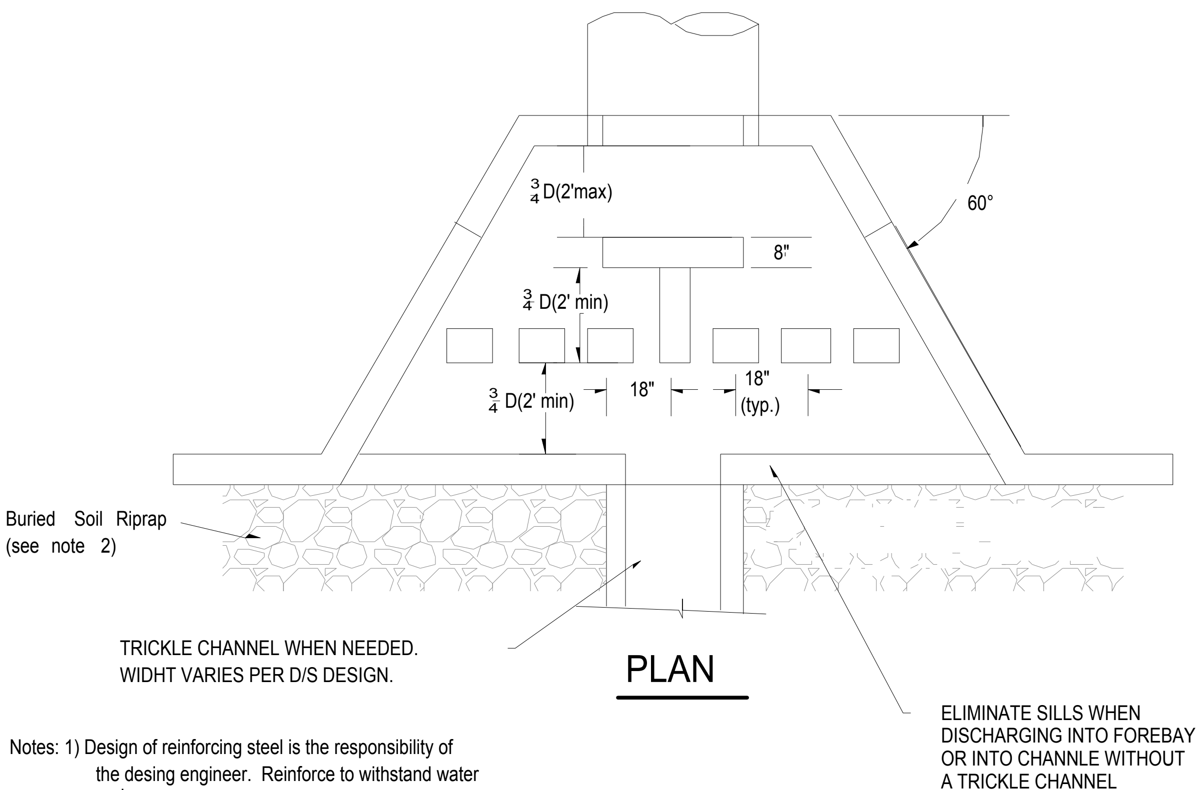


"W" is the inside width of the basin.
"D" represents the depth of flow entering the basin and is the square root of the flow area at the conduit outlet.
"V" is the velocity of the incoming flow.
The tailwater depth is uncontrolled.

Figure HS-15—Basin Width Diagram for the USBR Type VI Impact Stilling Basin)



ELEVATION



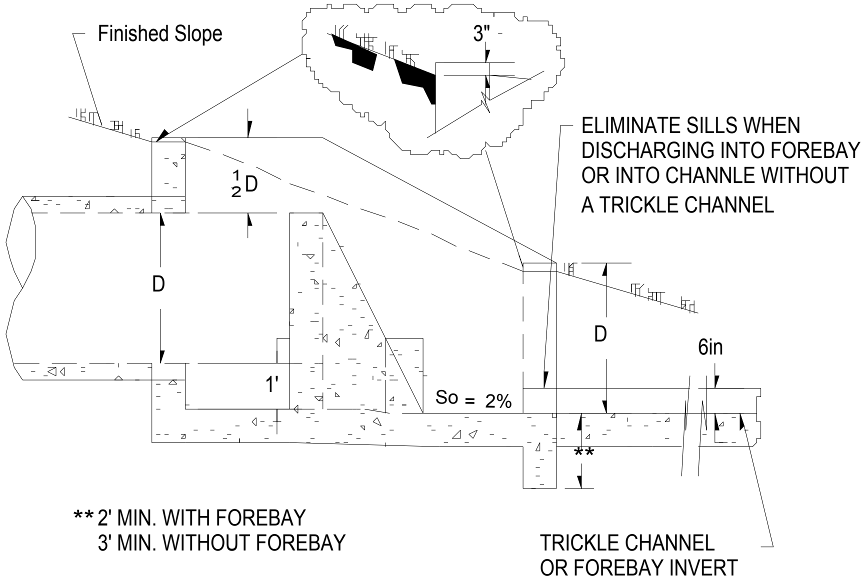
PLAN

Notes: 1) Design of reinforcing steel is the responsibility of the desing engineer. Reinforce to withstand water earh pressueres.

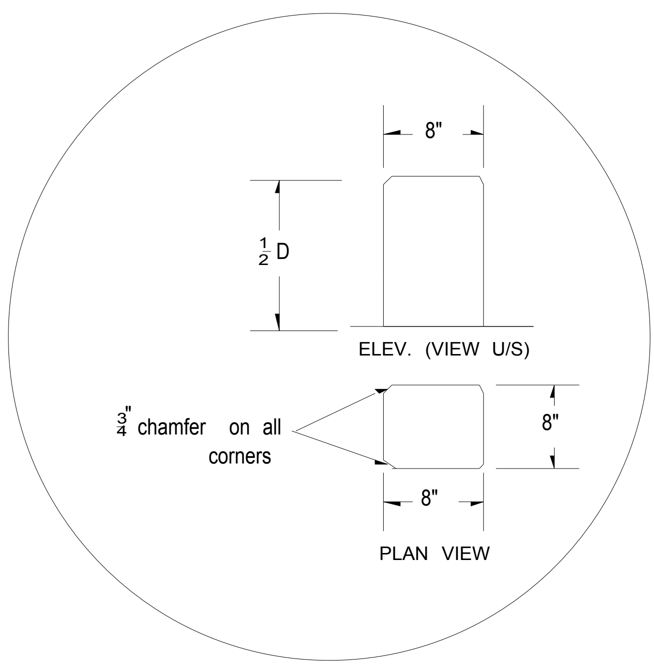
2) When dischargning into channel and not forebay install Type M buried soil riprap for a distance of 3*D downstream of structure.

Figure HS-16a Modified Impact Stilling Basin for Conduits 18" to 48" in Diameter

(Sheet 1 of 2)



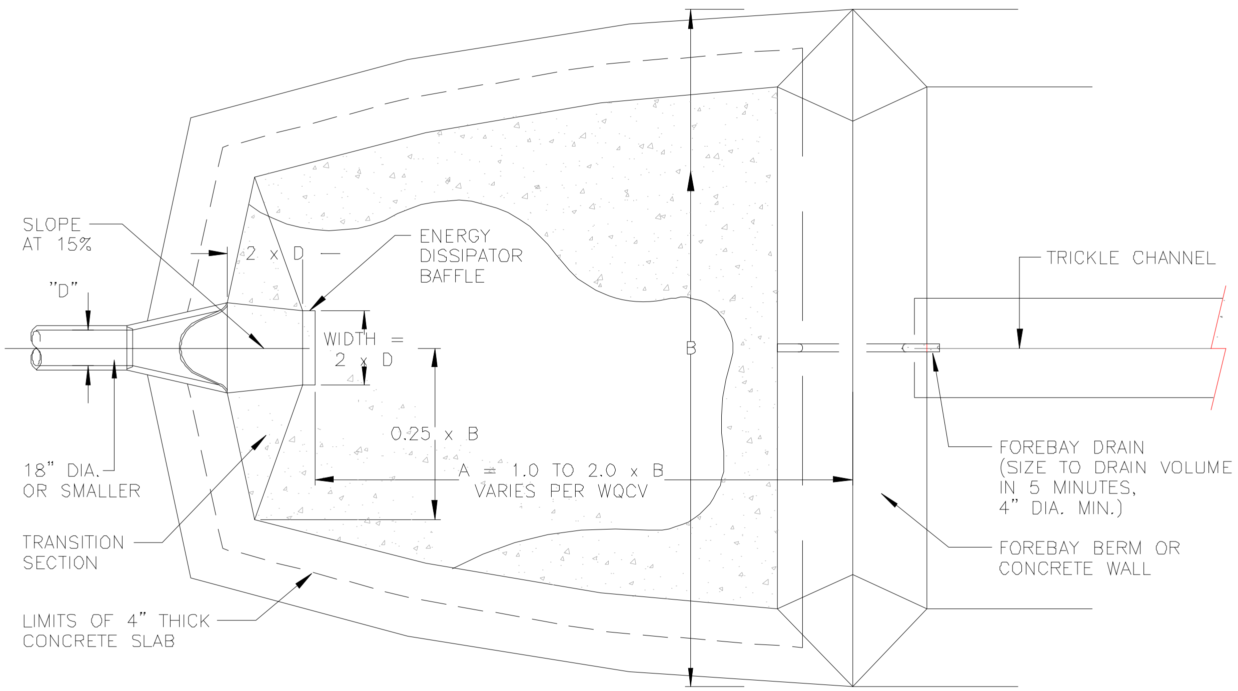
PROFILE



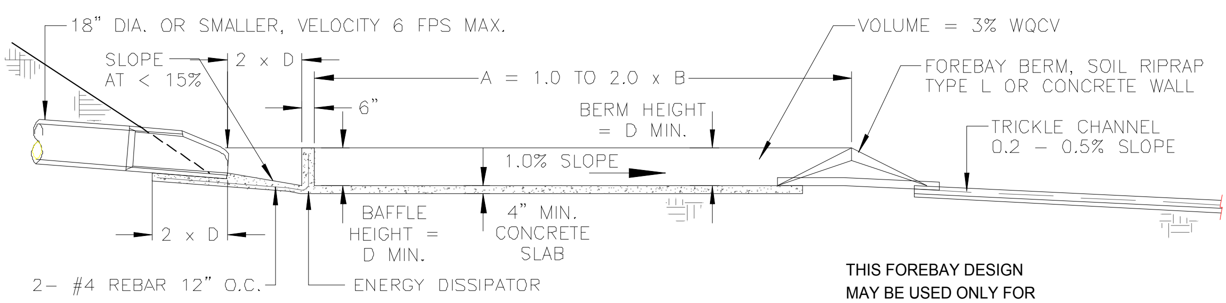
BAFFLE BLOCK GEOMETRY

Figure HS-16a. Modified Impact Stilling Basin for Conduits 18" to 48" in Diameter

(Sheet 2 of 2)



PLAN
NOT TO SCALE



THIS FOREBAY DESIGN MAY BE USED ONLY FOR OUTLETS PIPES 18" AND SMALLER

PROFILE
NOT TO SCALE

This figure courtesy of the City and County of Denve

Figure HS-16b. Impact Stilling Basin for Pipes Smaller than 18” in Diameter Upstream of Forebays.

(Courtesy: Technical and Design Criteria, City and County of Denver, 2006)

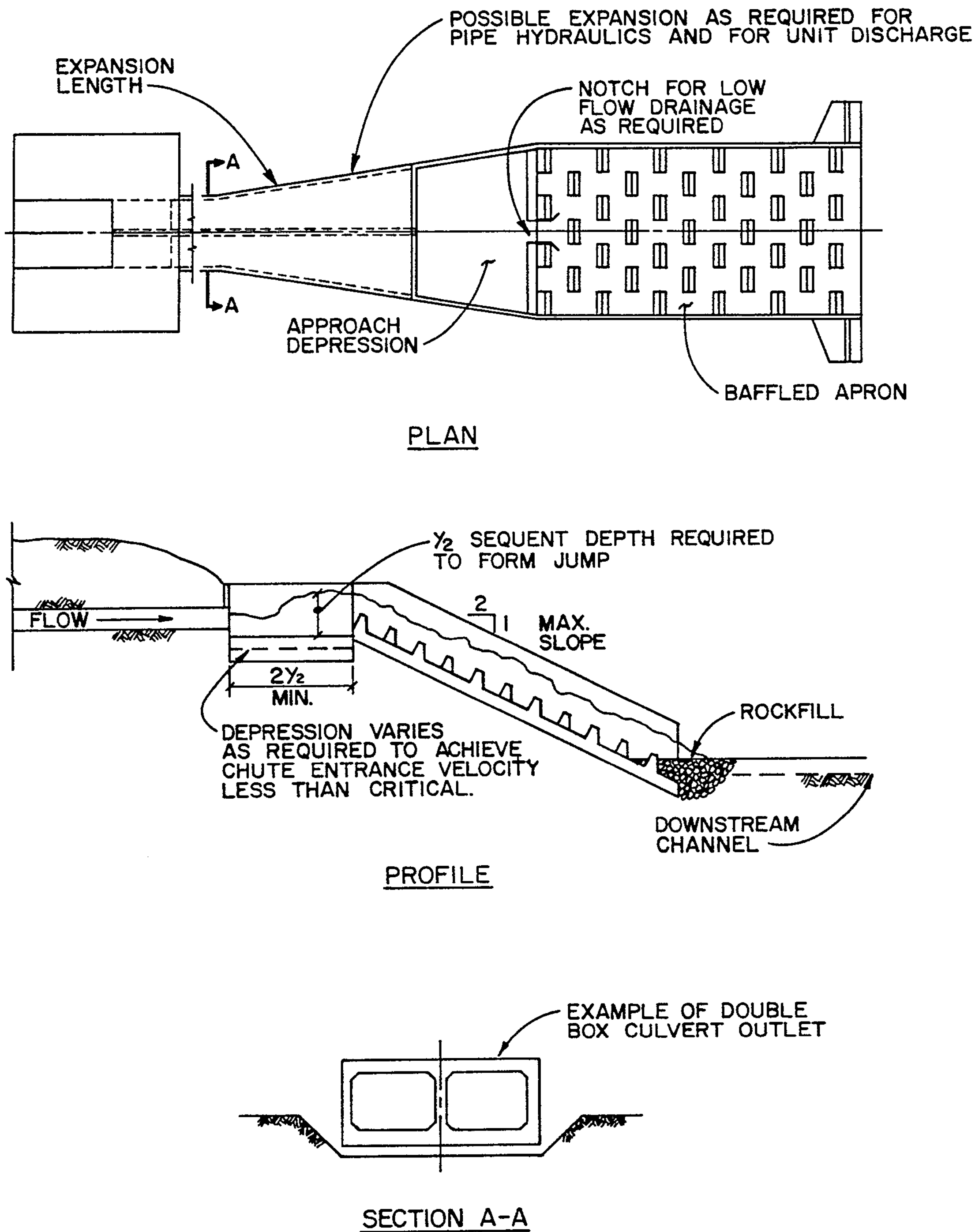


Figure HS-17—Baffle Chute Pipe Outlet

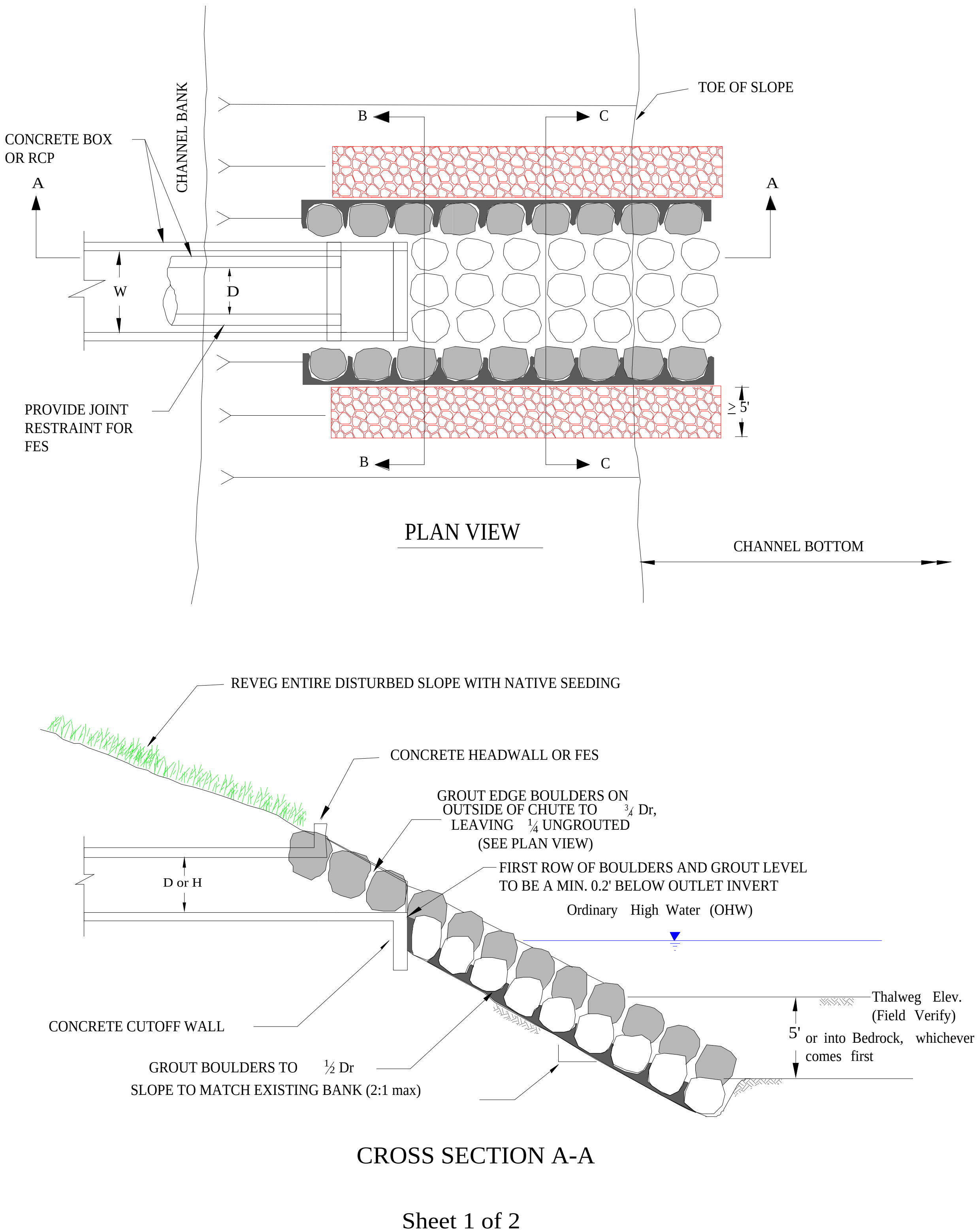
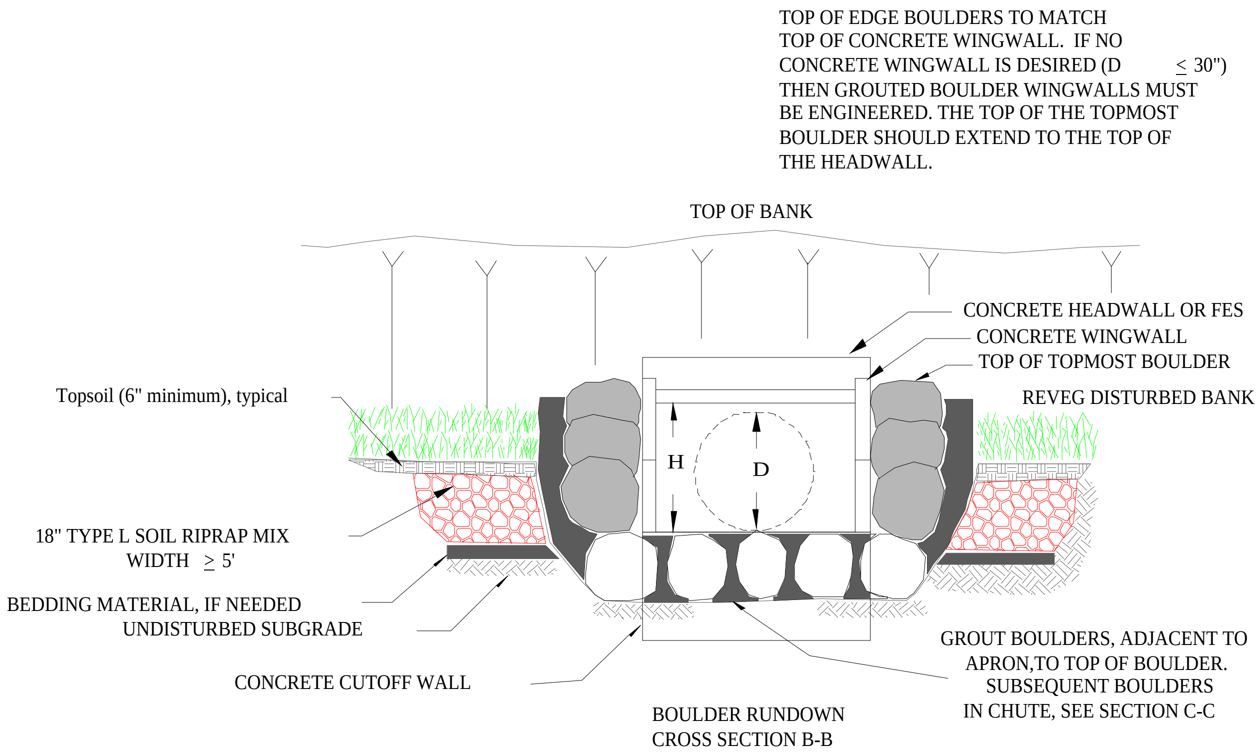
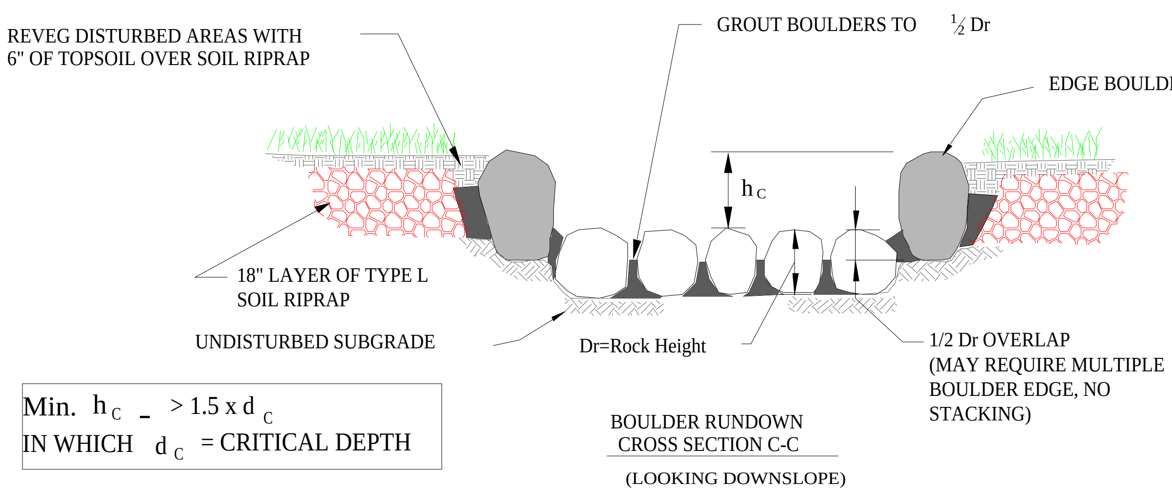


Figure HS-18—Grouted Boulder Rundown

(Sheet 1 of 2)

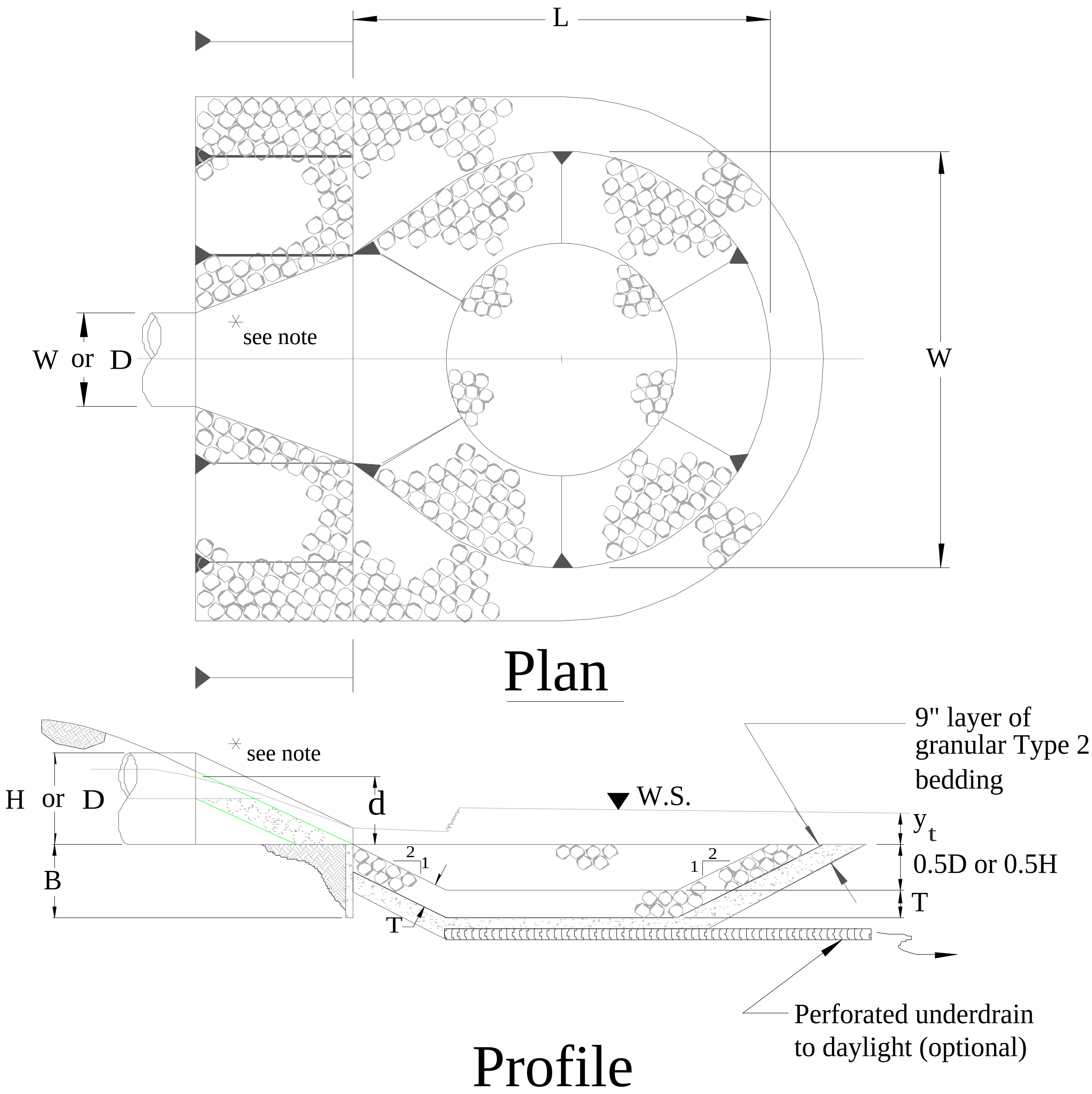




Sheet 2 of 2

Figure HS-18a—Grouted Boulder Rundown

(Sheet 2 of 2)



*Note: For rectangular conduits use a standard design for a headwall with wingwalls, paved bottom between the wingwalls, with an end cutoff wall extending to a minimum depth equal to B

**Figure HS-19—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—
Low Tailwater Basin at Pipe Outlets**
(Stevens and Urbonas 1996)

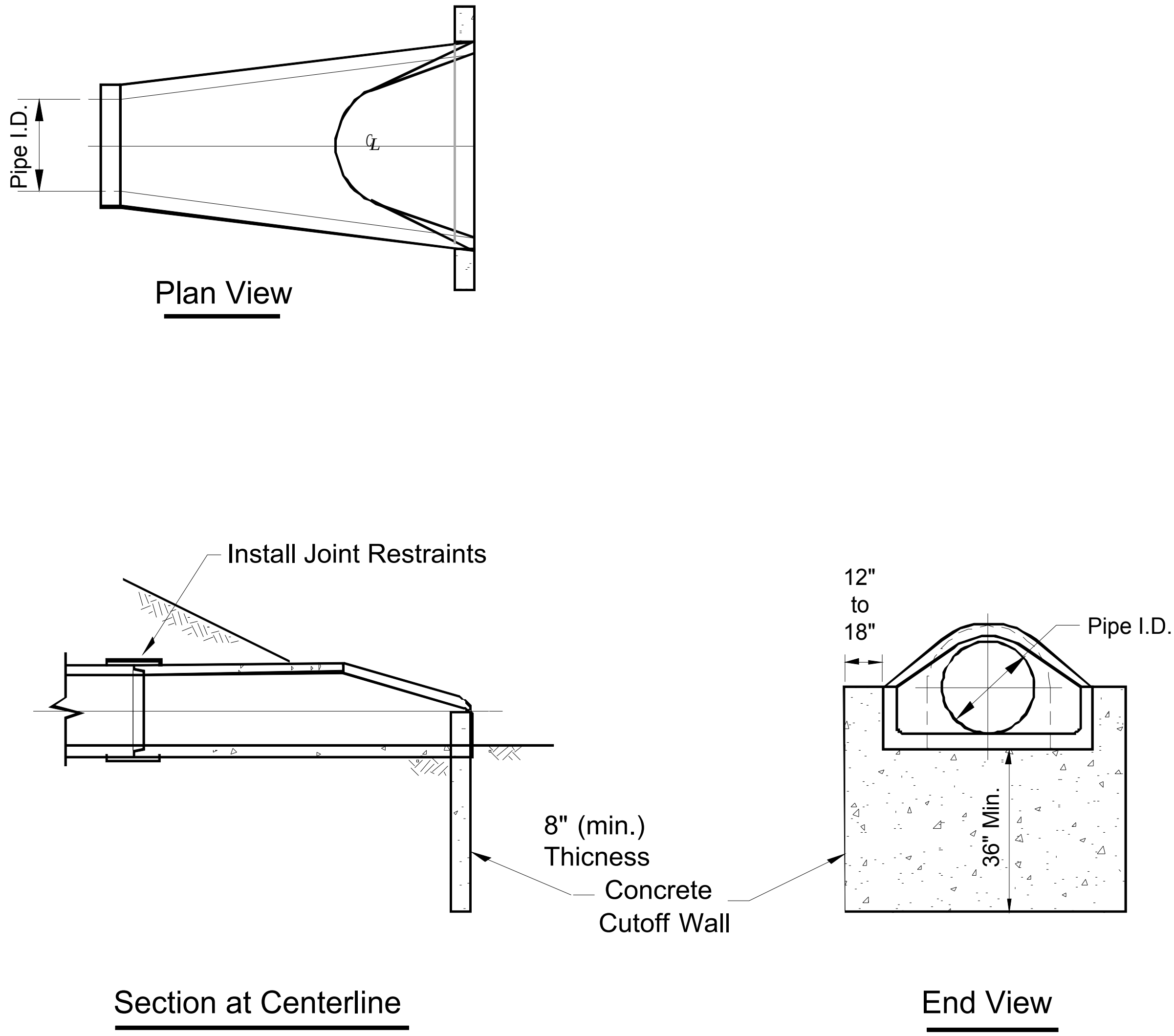


Figure HS-19a—Concrete Flared End Section with Cutoff Wall for all Pipe Outlets

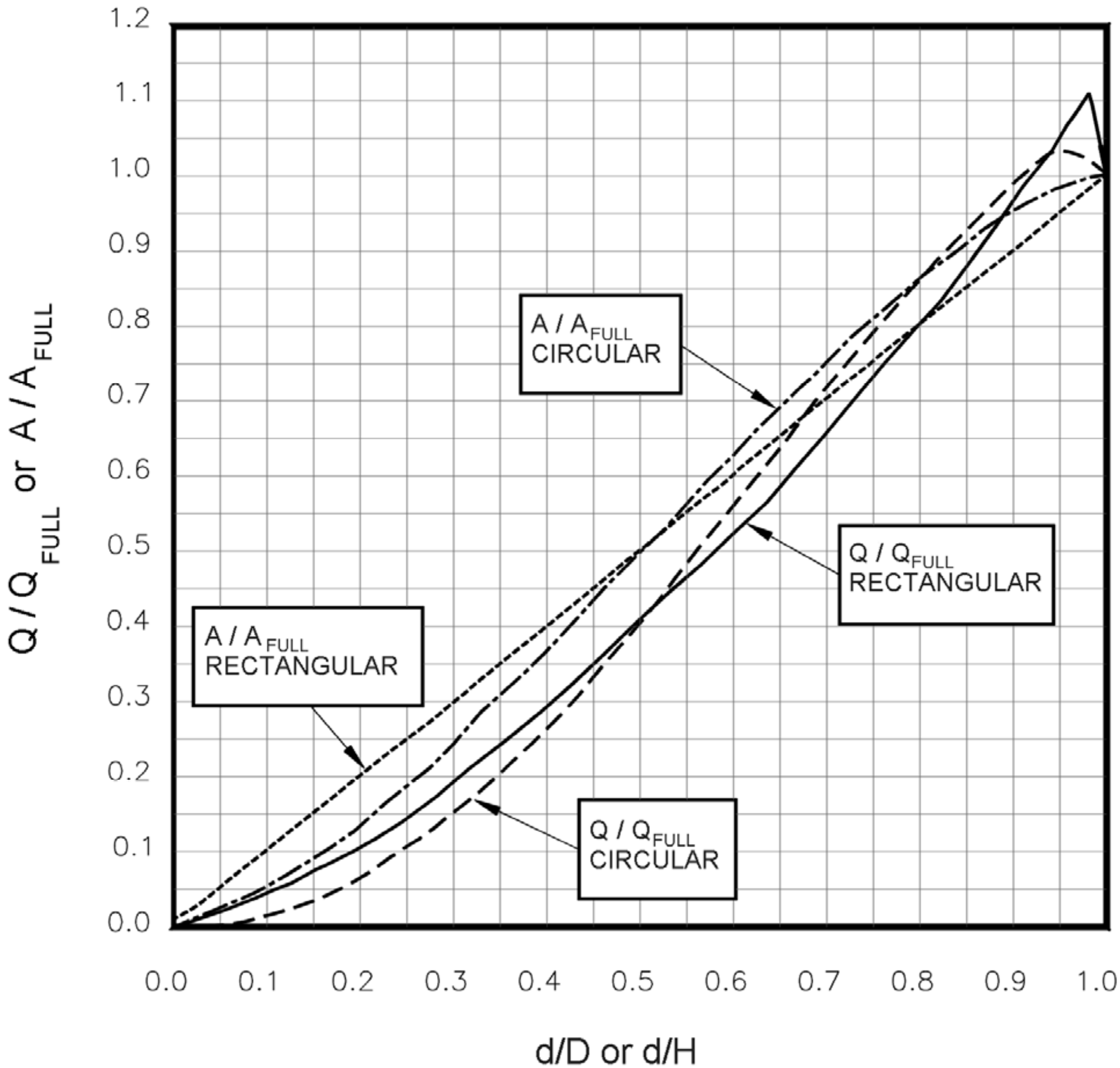


Figure HS-20a—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets— Discharge and Flow Area Relationships for Circular and Rectangular Pipes
(Ratios for Flow Based on Manning's *n* Varying With Depth)
(Stevens and Urbonas 1996)

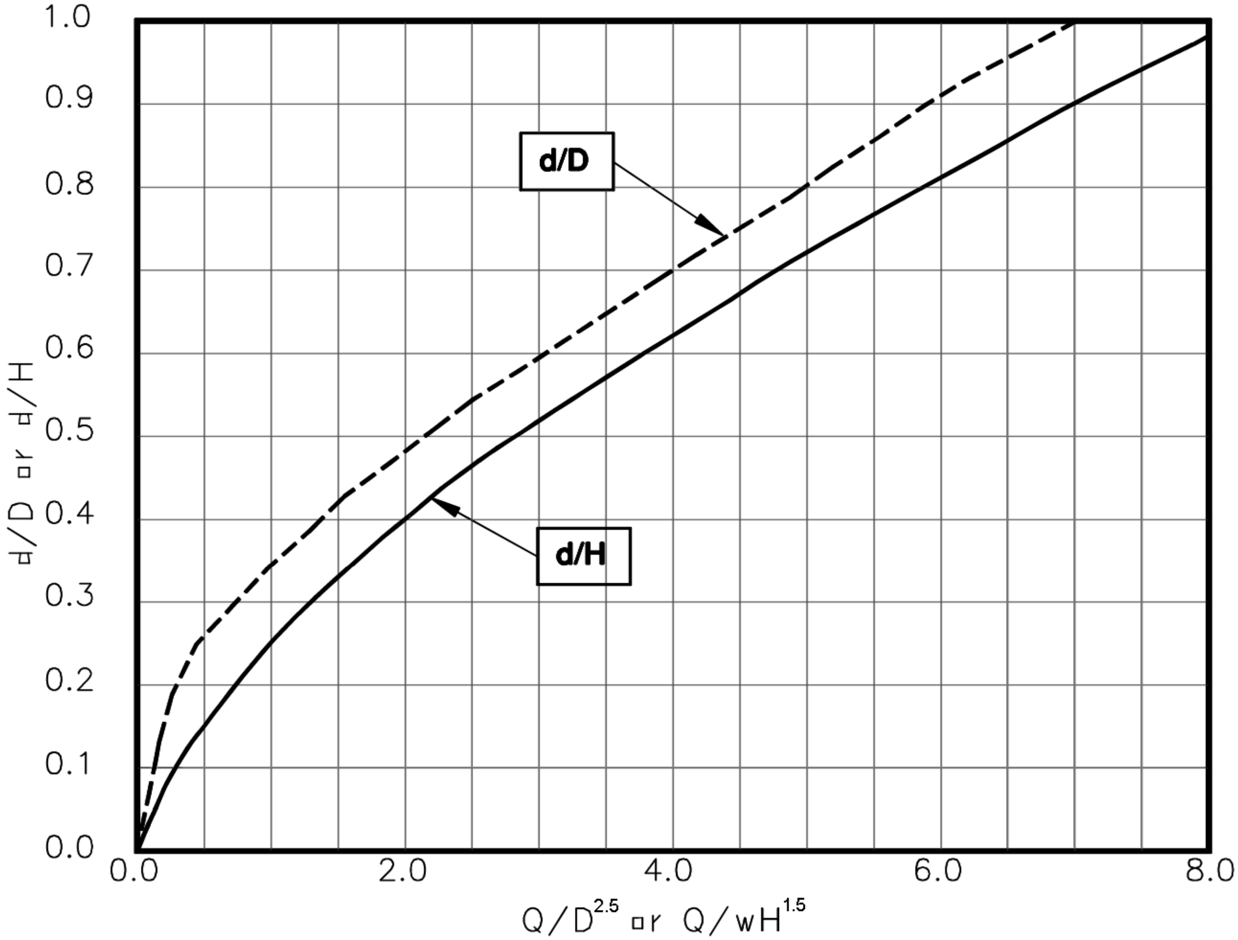


Figure HS-20b—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—

Brink Depth for Horizontal Pipe Outlets

(Stevens and Urbonas 1996)

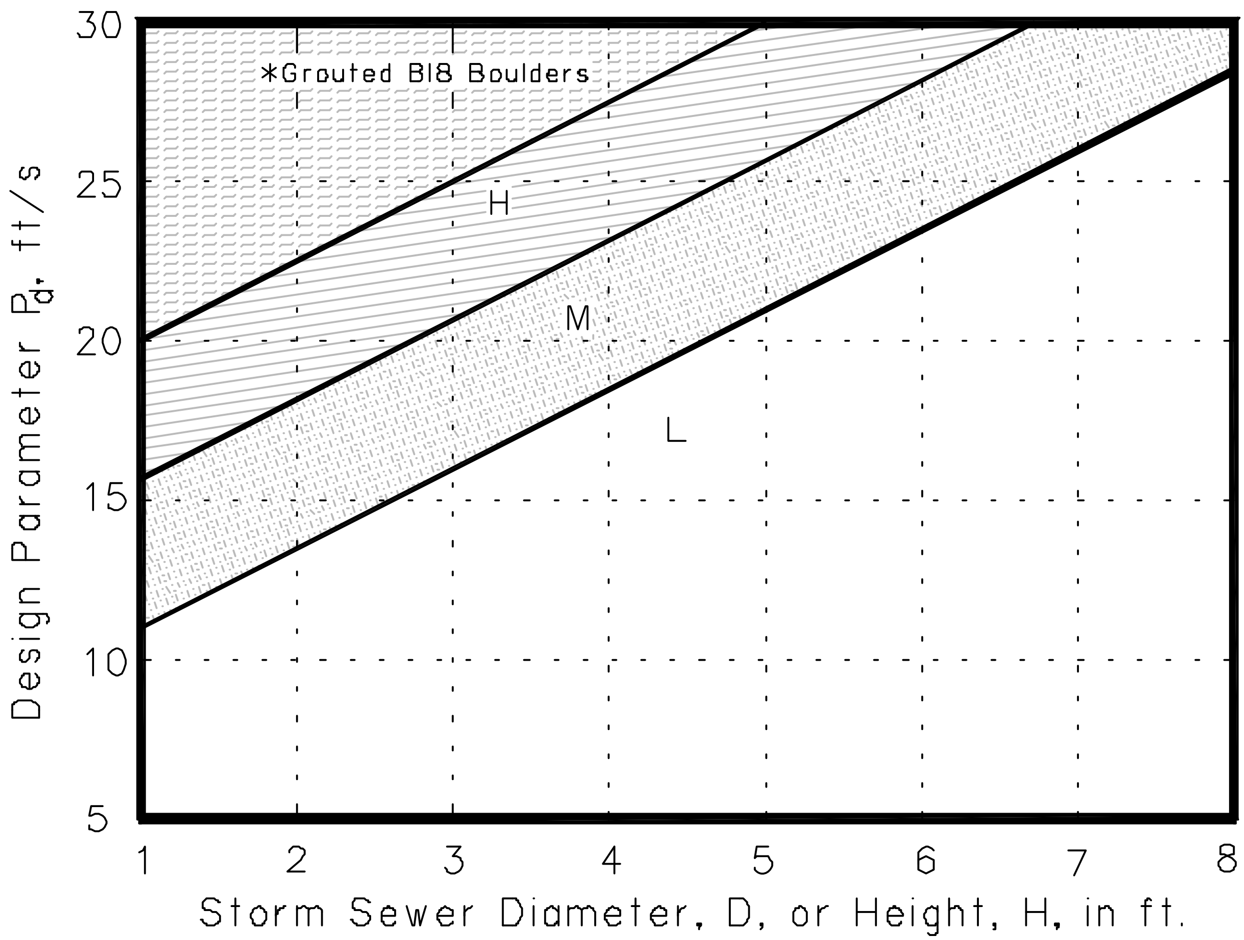


Figure HS-20c—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—

Riprap Selection Chart for Low Tailwater Basin at Pipe Outlet

(Stevens and Urbonas 1996)

4.0 BRIDGES

There are extensive manuals on bridges that are available and should be used in bridge hydraulic studies and river stability analysis. Some of the best include:

1. *Hydraulics of Bridge Waterways* Hydraulic Design Series No. 1 (FHWA 1978). This is a good basic reference.

2. *Highway in the River Environment* (Richardson 1988 draft with appendices and 1974). This is particularly good for hydraulics, geomorphology, scour, and degradation.

3. *Design Manual for Engineering Analysis of Fluvial Systems for the Arizona Department of Water Resources* (LSA 1985). This is a prime reference on hydraulics and the three-level sediment transport analysis, with examples.

Photograph HS-14—A stable channel at bridges is important and includes caring for the stream downstream of the bridge as shown here on Cherry Creek.

4. *Hydraulic Analysis Location and Design of Bridges* Volume 7 (AASHTO 1987). This is a good overview document.

5. *Technical Advisory on Scour at Bridges* (FHWA 1988). This presents information similar to references 2, 3, and 4 above, but in a workbook format, and perhaps oversimplified.

Bridges are required across nearly all open urban channels sooner or later and, therefore, sizing the bridge openings is of paramount importance. Open channels with improperly designed bridges will either have excessive scour or deposition or not be able to carry the design flow.

4.1 Basic Criteria

Bridge openings should be designed to have as little effect on the flow characteristics as reasonable, consistent with good bridge design and economics. However, in regard to supercritical flow with a lined channel, the bridge should not affect the flow at all—that is, there should be no projections into the design water prism that could create a hydraulic jump or flow instability in form of reflecting and standing waves.

4.1.1 Design Approach

The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the major storm runoff. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge should be determined. In urban cases this should not exceed a backwater effect of more than 6 to 12 inches.

Velocities through the bridge and downstream of the bridge must receive consideration in choosing the bridge opening. Velocities exceeding those permissible will require special protection of the bottom and banks.

For supercritical flow, the clear bridge opening should permit the flow to pass under unimpeded and unchanged in cross section.

4.1.2 Bridge Opening Freeboard

The distance between the design flow water surface and the bottom of the bridge deck will vary from case to case. However, the debris that may be expected must receive full consideration in setting the freeboard. Freeboard may vary from several feet to minus several feet. There are no general rules. Each case must be studied separately. In larger waterways, streams and on rivers where large floating debris is likely, at least a 3-foot freeboard during a 100-year flood should be considered.

Bridges that are securely anchored to foundations and designed to withstand the dynamic forces of the flowing water might, in some cases, be designed without freeboard.

4.2 Hydraulic Analysis

The hydraulic analysis procedures described below are suitable, although alternative methods such as FHWA HY-4 or HEC-RAS are acceptable, as well.

The design of a bridge opening generally determines the overall length of the bridge. The length affects the final cost of the bridge. The hydraulic engineering in the design of bridges has more impact on the bridge cost than does the structural design. Good hydraulic engineering is necessary for good bridge design (FHWA 1978, Richardson 1974 and 1988).

The reader is referred to *Hydraulics of Bridge Waterways* (U.S. Bureau of Public Roads 1978) for more guidance on the preliminary assessment approach described below. In working with bridge openings, the designer may use the designation shown in Figure HS-21.

4.2.1 Expression for Backwater

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been reestablished, as shown in Sections 1 and 4, respectively, of Figure HS-21. The expression is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical.

The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_1^* = (K^*)\left(\frac{(V_{n2})^2}{2g}\right) + \propto 1\left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{V_{n2}{}^2}{2g} \qquad \text{(HS-23)}$$

in which:

h_1^* = total backwater (ft)

K^* = total backwater coefficient

$\propto 1 = \dfrac{qv^2}{QV_1^2}$ = kinetic energy coefficient

A_{n2} = gross water area in constriction measured below normal stage (ft^2)

V_{n2} = average velocity in constriction or Q/A_{n2} (ft/sec). The velocity V_{n2} is not an actual measurable velocity but represents a reference velocity readily computed for both model and field structures.

A_4 = water area at Section 4 where normal stage is reestablished (ft^2)

A_1 = total water area at Section 1 including that produced by the backwater (ft^2)

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

To compute backwater by Equation HS-23, it is necessary to obtain the approximate value of h_1^* by using the first part of the equation:

$$h_1^* = (K^*)\left(\frac{V_{n2}^2}{2g}\right) \qquad \text{(HS-24)}$$

The value of A_1 in the second part of Equation HS-23, which depends on h_1^*, can then be determined.

This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $\frac{V_{n2}^2}{2g}$. Equation HS-24 may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$, where M = bridge opening ratio

$V_{n2} < 7$ ft/sec

$(K*)\left(\frac{V_{n2}^2}{2g}\right) < 0.5$ ft

If values meet all three conditions, the backwater obtained from Equation HS-24 can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use Equation HS-23 in its entirety. The use of the guides is further demonstrated in the examples given in FHWA (1978) that should be used in all bridge design work.

4.2.2 Backwater Coefficient

The value of the overall backwater coefficient $K*$, which was determined experimentally, varies with:

1. Stream constriction as measured by bridge opening ratio, M.
2. Type of bridge abutment: wingwall, spill through, etc.
3. Number, size, shape, and orientation of piers in the constriction.
4. Eccentricity, or asymmetric position of bridge with the floodplains.
5. Skew (bridge crosses floodplain at other than 90 degree angle).

The overall backwater coefficient $K*$ consists of a base curve coefficient, K_b, to which are added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of $K*$ is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

4.2.3 Effect of M and Abutment Shape (Base Curves)

Figure HS-22 shows the base curve for backwater coefficient, K_b, plotted with respect to the opening ratio, M, for several wingwall abutments and a vertical wall type. Note how the coefficient K_b increases with channel constriction. The several curves represent different angles of wingwalls as can be identified by the accompanying sketches; the lower curves represent the better hydraulic shapes.

Figure HS-23 shows the relation between the backwater coefficient, K_b, and M for spill-through abutments for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. Figures HS-22 and HS-23 are "base curves" and K_b is referred to as the "base curve coefficient." The base curve coefficients apply to normal crossings for specific abutment shapes but do not include the effect of piers, eccentricity, or skew.

4.2.4 Effect of Piers (Normal Crossings)

The effect on the backwater from introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, has been assigned the letter J. In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure HS-24. The procedure is to enter Chart A, Figure HS-24, with the proper value of J and read ΔK and obtain the correction factor σ from Chart B, Figure HS-24, for opening ratios other than unity. The incremental backwater coefficient is then

$$\Delta K_p = \Delta K \sigma \tag{HS-25}$$

The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing but should be increased if there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20% higher than those shown for bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figures HS-22 or HS-23)} + \Delta K_p \text{ (Figure HS-24)} \tag{HS-26}$$

4.3 Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.

2. Determine the stage of the stream at the bridge site for the design discharge.

3. Plot representative cross section of stream for design discharge at Section 1, if not already done under Step 2. If the stream channel is essentially straight and the cross section substantially uniform in the vicinity of the bridge, the natural cross section of the stream at the bridge site may be used for this purpose.

4. Subdivide the above cross section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient, n, to each subsection. Careful judgment is necessary in selecting these values.

5. Compute conveyance and then discharge in each subsection.

6. Determine the value of the kinetic energy coefficient.

7. Plot the natural cross section under the proposed bridge based on normal water surface for design discharge and compute the gross water area (including area occupied by piers).

8. Compute the bridge opening ratio, M, observing modified procedure for skewed crossings.

9. Obtain the value of K_b from the appropriate base curve.

10. If piers are involved, compute the value of J and obtain the incremental coefficient, ΔK_p.

11. If eccentricity is severe, compute the value of eccentricity and obtain the incremental coefficient, ΔK_e (FHWA 1978).

12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain the incremental coefficient, ΔK_s, for proper abutment type.

13. Determine the total backwater coefficient, K^*, by adding incremental coefficients to the base curve coefficient, K_b.

14. Compute the backwater by Equation HS-23.

15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in *Hydraulics of Bridge Waterways* (FHWA 1978).

4.4 Inadequate Openings

The engineer will often encounter existing bridges and culverts that have been designed for storms having return periods less than 100 years. In addition, bridges will be encountered which have been improperly designed. Often the use of the orifice formula will provide a quick determination of the adequacy or inadequacy of a bridge opening:

$$Q_m = C_b A_b \sqrt{2gH_{br}} \tag{HS-27}$$

or

$$H_{br} = 0.04\left(\frac{Q_m}{A_b}\right)^2 \tag{HS-28}$$

in which:

Q_m = the major storm discharge (cfs)

C_b = the bridge opening coefficient (0.6 assumed in Equation HS-27)

A_b = the area of the bridge opening (ft^2)

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

H_{br} = the head, that is the vertical distance from the bridge opening center point to the upstream water surface about $10H$ upstream from the bridge, where H is the height of the bridge, in feet. It is approximately the difference between the upstream and downstream water surfaces where the lower end of the bridge is submerged.

These expressions are valid when the water surface is above the top of the bridge opening.

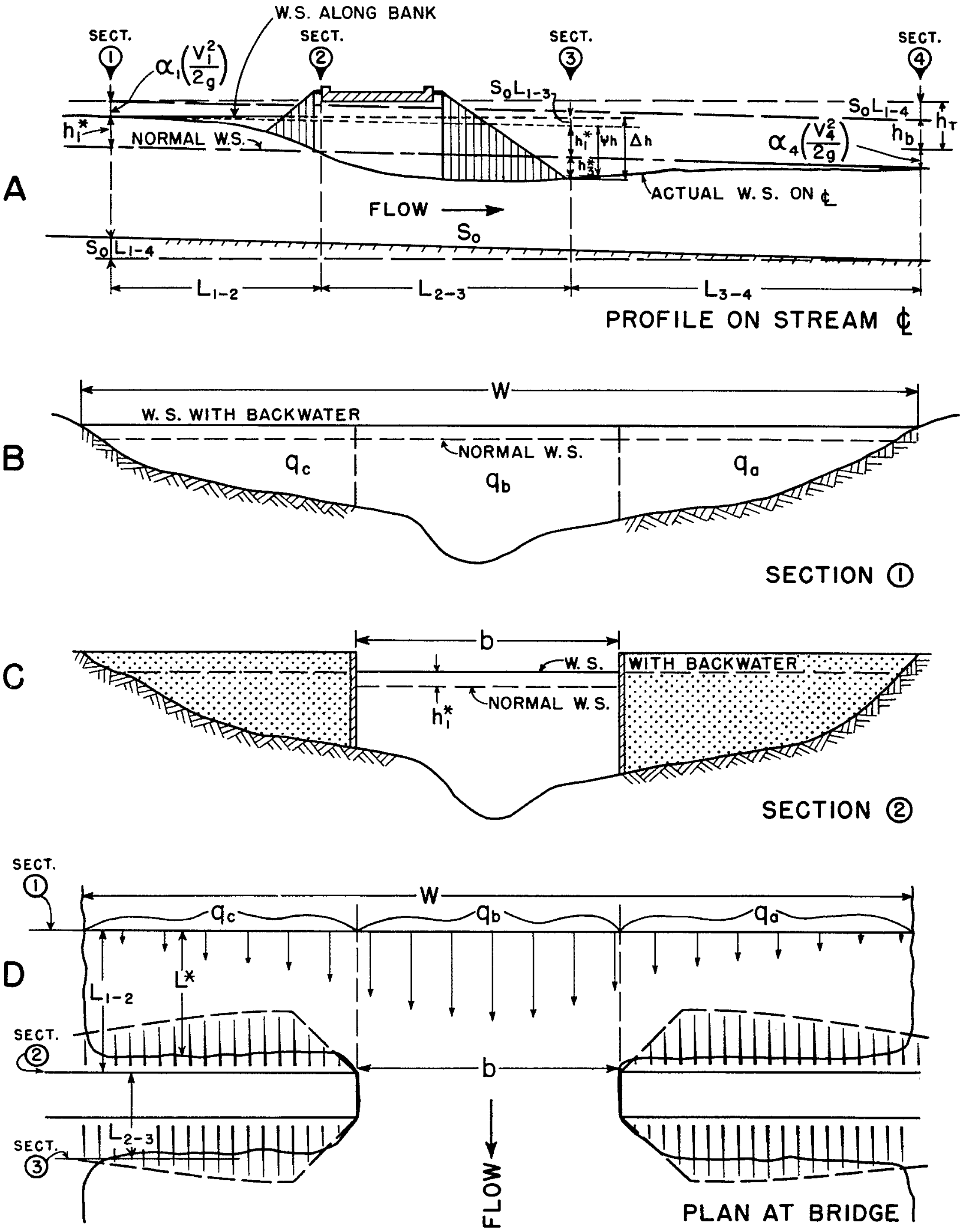


Figure HS-21—Normal Bridge Crossing Designation

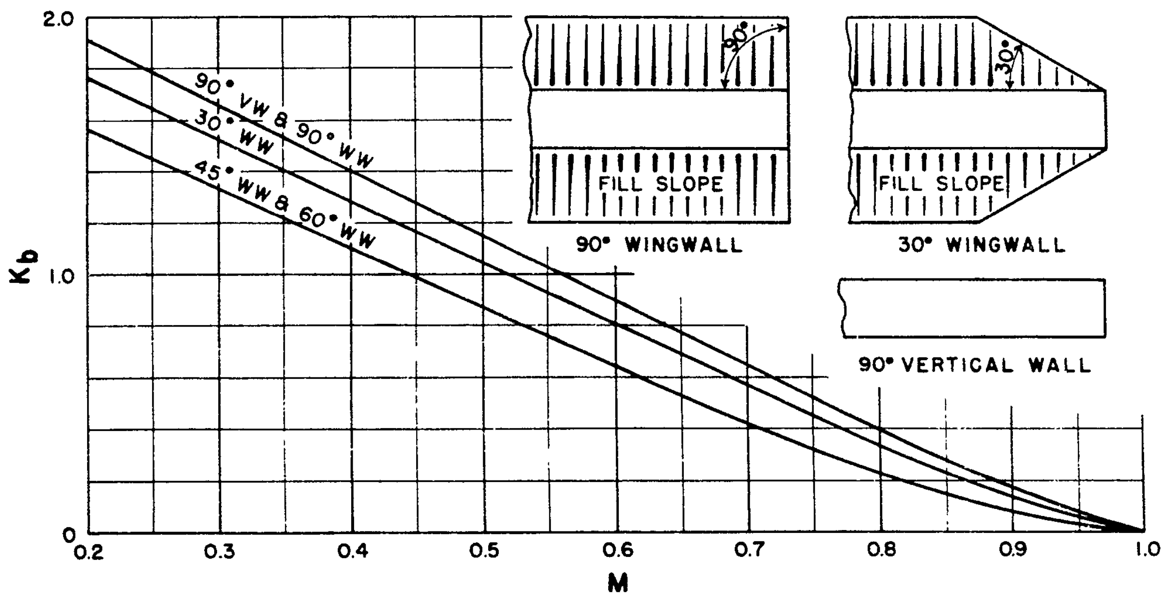


Figure HS-22—Base Curves for Wingwall Abutments

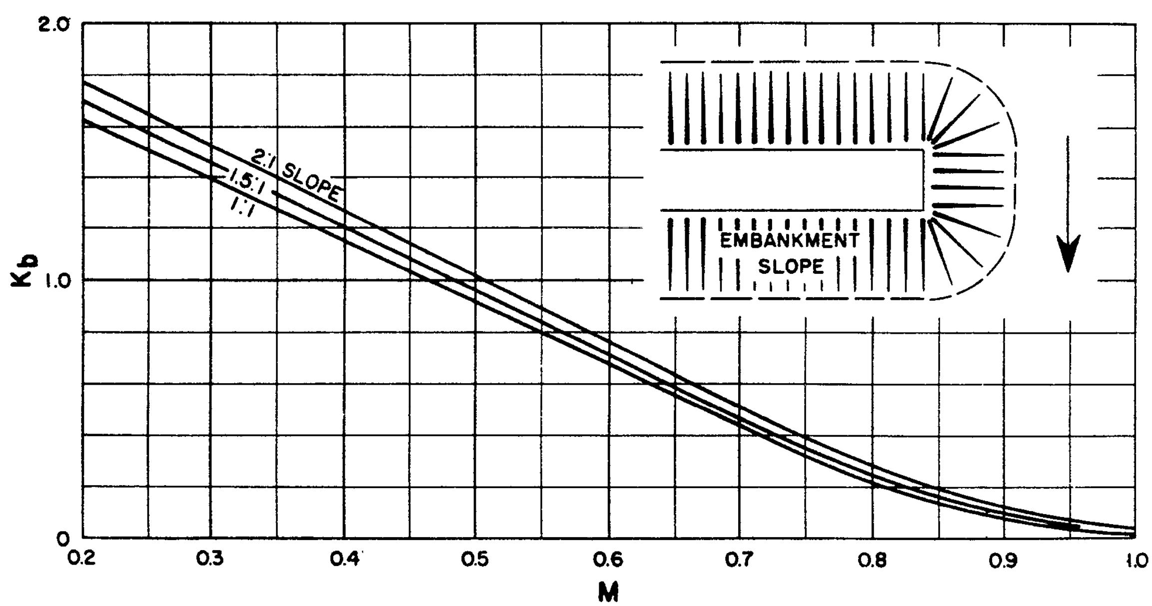


Figure HS-23—Base Curves for Spillthrough Abutments

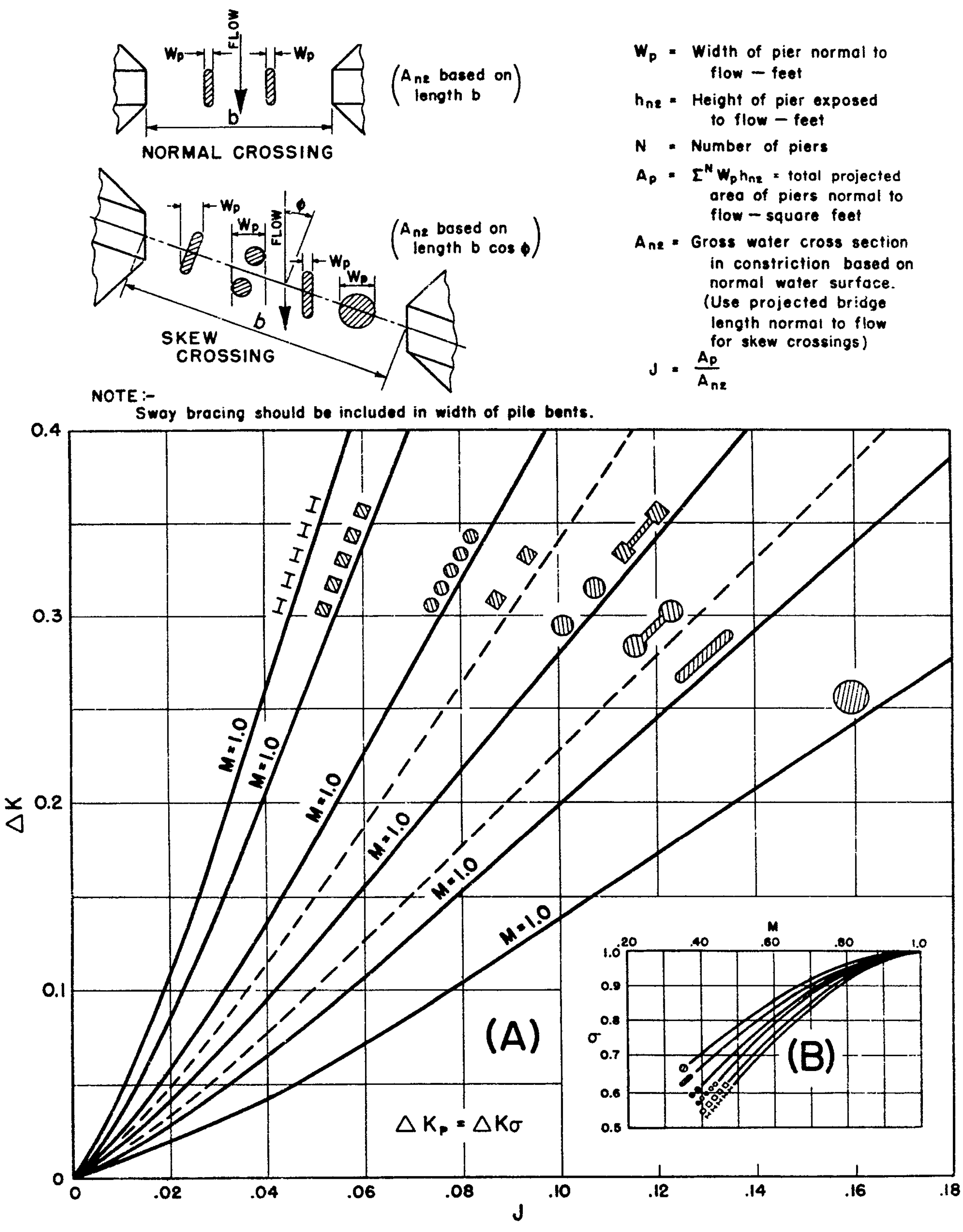


Figure HS-24—Incremental Backwater Coefficient for Pier

5.0 TRANSITIONS AND CONSTRICTIONS

5.1 Introduction

The purpose of this section is to outline typical design procedures for transition and constriction structures that are commonly encountered in the District's flood control and drainage projects. There are numerous references that can be useful for detailed analysis of different project objectives or site conditions (Rouse 1949, Chow 1959, USACE 1970 and 1982, FHWA 2000, SCS 1977). This topic is also addressed in MAJOR DRAINAGE, under riprap-lined channels.

5.2 Transition Analysis

5.2.1 Subcritical Transitions

Transitions for subcritical flow frequently involve localized structures or bank lining configurations that allow change in the cross section and produce a water surface profile based on gradually varied flow. The energy lost through a transition is a function of the friction, eddy currents and turbulence. The intent is often to minimize friction losses and/or erosional tendencies. Examples include transitions between trapezoidal and rectangular sections, modest transitions at bridges where little change takes place in the cross section, or slight encroachments into a channel to allow for utilities. Transitions can be handled with various structures, including concrete facilities (Figure HS-25) and riprap-lined channel reaches (see MAJOR DRAINAGE).

Standard water surface profile analysis is applied, with the addition of an energy loss at the transition. The loss is expressed as a function of the change in velocity head occurring across the contraction or expansion transition (from upstream to downstream locations). Figure HS-25 illustrates some of these transitions with basic design guidelines. Loss coefficients shown in Table HS-10 are applied to the difference in velocity head, as shown in Equation HS-29.

Analysis of transitions requires careful water surface profile analysis including verification of effective channel hydraulic controls. It is not uncommon to have a transition that is first thought to be performing in a subcritical mode, subsequently found to produce a supercritical profile with a hydraulic jump.

Energy Loss (ft) = Coefficient $(h_{v1} - h_{v2})$

in which:

$$\left(h_{v1} - h_{v2}\right) = \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad \text{(HS-29)}$$

V_1 = flow velocity upstream of transition

V_2 = flow velocity downstream of transition

Table HS-10—Subcritical Transition Energy Loss Coefficients

	Contraction	Expansion
Less than 4 inches between centerline and tangent lines	0.00	0.00
Less than 12.5 degrees between centerline and tangent lines	0 to 0.10	0 to 0.10
Warped type	0.10	0.20
Cylindrical quadrant type	0.15	0.25
Modest transitions	0.30	0.50
Straight line type	0.30	0.50
Square ended type	0.30+	0.75

5.2.2 Supercritical Transition Analysis

Supercritical transitions are beyond the scope of this *Manual* and require special analysis when used. The configuration of a supercritical transition is entirely different than subcritical transitions. Improperly designed and configured supercritical transitions can produce shock wave patterns which result in channel overtopping and other hydraulic and structural problems.

5.3 Constriction Analysis

5.3.1 Constrictions With Upstream Subcritical Flow

There are a variety of structures that are constrictions. They can include bridges, culverts, drop structures, and flow measurement devices. Constrictions of various types are used intentionally to control bed stability and upstream water surface profiles. For example, a constriction may be used to cause water to back up into or overflow into a flood storage pond.

The hydraulic distinction of constrictions is that they can cause rapidly varied flow. The upstream transition loss coefficients in Table HS-10 apply, but other factors come into play. Significant eddies can form upstream and downstream of the constriction depending upon the geometry. Flow separation will start at the upstream edge of the constriction, then the flow contracts to be narrower than the opening width. Typically, the width of contraction is 10% of the depth at the constriction for each side boundary. For example, at a typical drop with an abrupt crest contraction and assuming critical depth of 3.5 feet, the constriction on each side would be 0.35 feet or 0.7 feet total contraction from the opening width. Based on this contracted width and an assumption of critical conditions at that location, the upstream water surface profile may be computed.

In certain cases the flow regime will remain subcritical through the constriction. Chow (1959) presents guidelines developed by the U.S. Geological Survey for constrictions where the Froude number in the contracted section does not exceed 0.8. These cases are considered to be mild constrictions.

A consequence of abrupt contractions (and abrupt expansions) is that the velocities can be much higher in the center and change significantly across the constriction throat section. This results in a large energy coefficient and a further drop in water surface over what is first anticipated. This condition can produce strong eddy currents with high erosion potential. A constriction in an open channel needs to be carefully evaluated for velocity, scour, water surface, and related problems.

Constrictions used for flow depth control or flow measurement devices require a high degree of accuracy. The design information available that can be used for insuring a high degree of accuracy is limited. It is advisable to use model-tested or proven prototype layouts. As a secondary option, adjustable edge plates or other components can be provided to allow later changes at minimal cost if the constructed facilities should need refinement.

5.3.2 Constrictions With Upstream Supercritical Flow

This situation is highly complex and goes beyond the scope of this *Manual*. Possible shock waves or choked flow causing high upstream backwater or a hydraulic jump are distinct possibilities and are should be of major concern to designers. The situation is best avoided in urban channels and settings.

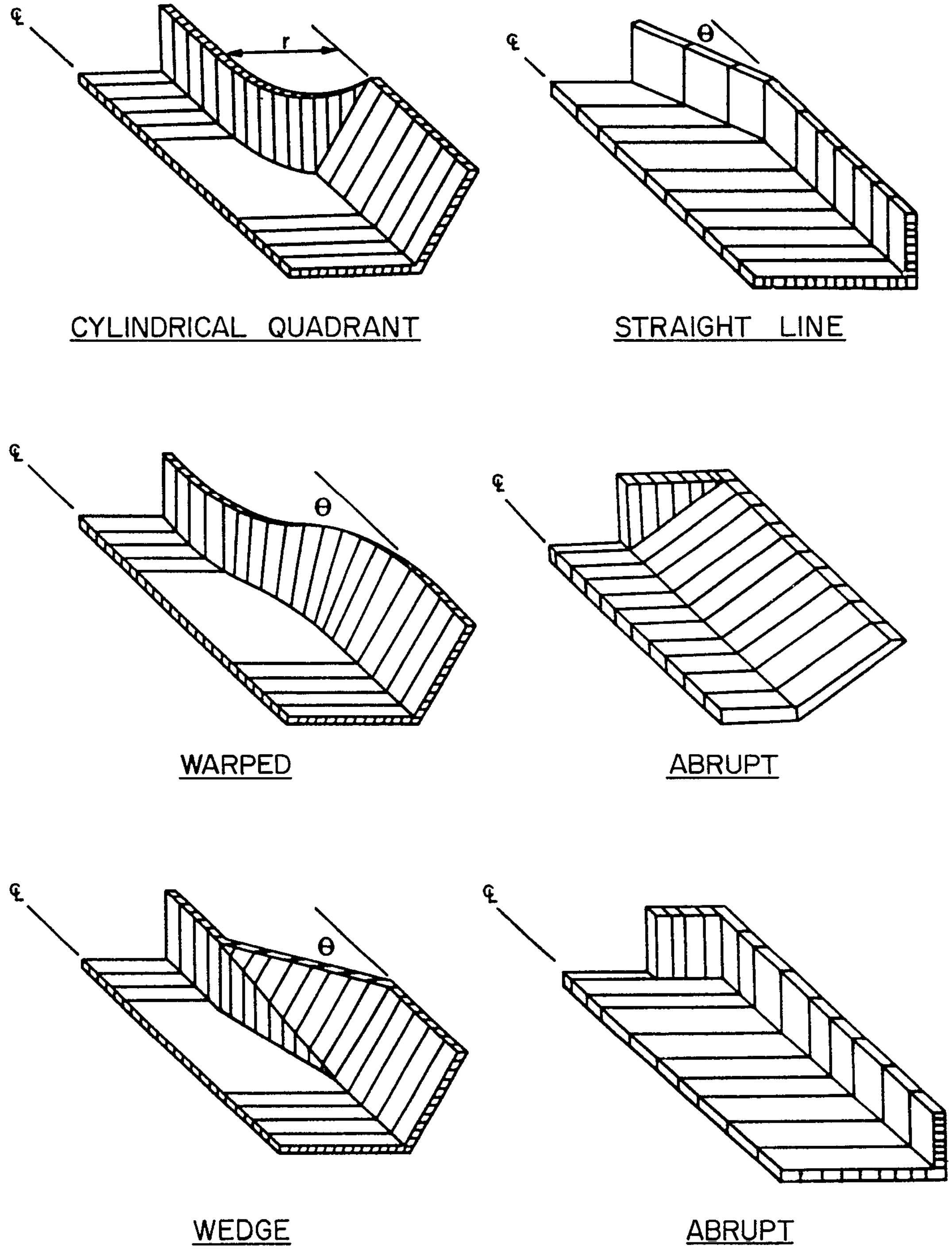


Figure HS-25—Transition Types

6.0 BENDS AND CONFLUENCES

6.1 Introduction

This section focuses on subcritical flow conditions. Because supercritical conditions can occur in various situations, a few supercritical conditions are also generally reviewed; however, supercritical flow analysis is not described in detail.

6.2 Bends

6.2.1 Subcritical Bends

Subcritical bends are required to have certain minimum curvatures described in the MAJOR DRAINAGE chapter. It is important that the engineer recognize the consequence of approaching and exceeding these criteria. Chow (1959), Rouse (1949) (see chapter by Ippen), and others illustrate flow patterns, superelevation, and backwater or flow resistance characteristics. Superelevation refers to a rise in water surface on the outer side of the bend. Effectively, the bend can behave like a contraction, causing backwater upstream and accelerated velocity zones, with high possibility of erosion on the outside of the bend and other locations. Significant eddy currents, scour, sedimentation, and loss of effective conveyance can occur on the inside of the bend.

Concrete-lined channels can be significantly affected by superelevation of the water surface. The designer should always add superelevation to the design freeboard of the channel. The equation for the amount of superelevation of the water surface, Δy that takes place is given as:

$$\Delta y = C_{se}\left(\frac{V^2 W_t}{g r_c}\right) \quad \text{(HS-30)}$$

in which:

C_{se} = coefficient, generally 0.5 for subcritical flow (see references for higher coefficients for supercritical)

V = mean channel velocity

W_t = channel top width of water surface

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

r_c = radius of the channel centerline curvature

6.2.2 Supercritical Bends

As with supercritical transitions, supercritical bend hydraulics are completely different than subcritical. Supercritical channels are not desirable in urban drainage; however, special situations occur where supercritical flows enter a curved channel. Some examples include at confluences where one channel is empty and the entering flow expands and becomes supercritical, at a sharp bend in a conduit with a slope that inherently leads to supercritical conditions, or at a channel drop that unavoidably ends up on a curve.

The main phenomenon to be aware of is shock waves, of which there are two types: positive and negative. On the outside of an angular bend, a positive shock wave will occur that results in a rise in water surface. The wave is stationary and crosses to the inside of the channel and then can continue to reflect back and forth. Where the flow passes the inside angular bend, a separation will occur, and a negative shock wave or drop in water surface will occur. This stationary negative shock wave will cross to the outside of the channel. Both shock waves will continue to reflect off the walls, resulting in a very disturbed flow pattern.

A basic control technique is to set up bend geometry to cause the positive shock wave to intersect the point where the negative wave is propagated. A bend usually requires two deflections on the outside and one bend on the inside. A beneficial aspect of the shock wave is that it turns the flow in a predictable pattern; thus, the channel walls have no more force imposed on them than that caused by the increased (or decreased) depths. This technique is described in Rouse (1949), USACE (1970), and Chow (1959).

Other control techniques include very gradual bends, super elevated floors and control sills, but these methods are generally less efficient. There is limited data on channels with sloping side banks, but it is clear there is a great tendency for shock waves to propagate up side slopes and divert flow out of the channel. Chow (1959) shows several good photographs of these problems. The SCS (1976) presents a documental report of a curved spillway on a modest flood control storage facility. During an overflow event, a shock wave pattern was produced that resulted in no flow on one side of a spillway, and great depths on the opposite.

Another problem observed at bends when channels operate under supercritical conditions is flow jumping out of the channel at the bend. When this happens, the downstream channel no longer carries the design flow and major damages to prosperities in line with the flows jumping out of the channel can and have occurred.

A special problem with long conduits used for flood control, particularly large box culverts, is that they will have an inherent tendency toward supercritical flow conditions at less than full capacity. When supercritical flow encounters bends or transitions, standing and reflective waves can occur which hit the ceiling of the culvert and can cause pressurized conditions or unstable conditions where the flow fluctuates between supercritical free surface flow and pressurized pipe flow conditions, often exacerbated by pressure variations in the pipe that can range from less than atmospheric to pressures approaching full velocity head. It is recommended that there be no bends or very gradual bends in conduits, along with air venting be provided when supercritical flows are expected in conduits, especially rectangular ones.

Use extreme caution in design anytime supercritical flow may occur and may encounter a bend or a transition.

6.3 Confluences

Some of the most difficult problems to deal with are confluences where the difference in flow characteristics may be great. When the flow enters the combined channel, the flow can diverge and drop in level if the flow capacity is suddenly increased. This can result in high velocity or unstable supercritical flow conditions with high erosion potential. When significant sediment flows exist, aggradation can occur at the confluence, resulting in loss of capacity in one or both upstream channels. The following material is adapted from *Hydraulic Design of Flood Control Channels* (USACE 1970).

6.3.1 Subcritical Flow Confluence Design

The design of channel junctions is complicated by variables such as the angle of intersection, shape and width of the channels, flow rates, and type of flow. The design of large complex junctions should be verified by model tests. The momentum equation design approach has been verified for small angles by Taylor (1944) and Webber and Greated (1966).

Figure HS-26 illustrates two types of junctions. The following assumptions are made for combining subcritical flows.

1. The side channel cross section is the same shape as the main channel cross section.
2. The bottom slopes are equal for the main channel and the side channel.
3. Flows are parallel to the channel walls immediately above and below the junction.
4. The depths are equal immediately above the junction in both the side and main channel.
5. The velocity is uniform over the cross sections immediately above and below the junction.

Assumption number 3 implies that hydrostatic pressure distributions can be assumed, and assumption number 5 suggests that the momentum correction factors are equal to each other at the reference sections.

The equation governing flow conditions for a vertical walled channel with the main channel width constant is shown in Figure HS-26(a) and the following equation:

$$\frac{Q_3^2}{gA_3} + \frac{by_3^2}{2} = \frac{Q_1^2}{gA_1} + \frac{(\cos\theta)\,Q_2^2}{gA_2} + \frac{by_1^2}{2} \tag{HS-31}$$

Or, for a vertical walled channel with the main channel width variable, Figure HS-26(b):

$$\frac{Q_3^2}{gA_3} + \frac{b_3y_3^2}{2} = \frac{Q_1^2}{gA_1} + \frac{(\cos\theta)\,Q_2^2}{gA_2} + \frac{b_3y_1^2}{2} \tag{HS-32}$$

Or, for a trapezoidal channel with the main channel width constant, Figure HS-26(a):

$$\frac{Q_3^2}{gA_3}+\left(\frac{b_1}{2}+\frac{Zy_3}{3}\right)y_3^2=\frac{Q_1^2}{gA_1}+\frac{(\cos\theta)\,Q_2^2}{gA_2}+\left(\frac{b_1}{2}+\frac{Zy_1}{2}\right)y_1^2 \qquad \text{(HS-33)}$$

Or, for trapezoidal channels with the main channel width variable, Figure HS-26(b):

$$\frac{Q_3^2}{gA_3}+\left(\frac{b_3}{2}+\frac{Zy_3}{3}\right)y_3^2=\frac{Q_1^2}{gA_1}+\frac{(\cos\theta)\,Q_2^2}{gA_2}+\left(\frac{b_3}{2}+\frac{Zy_1}{3}\right)y_1^2 \qquad \text{(HS-34)}$$

In which:

b = bottom width of the trapezoidal cross section

Z = side slope, horizontal to vertical

Momentum computations for a confluence involve a trial and error process. Starting with a known depth above or below the confluence, one iterates with an assumed depth on the unknown side of the confluence until the momentum has been balanced upstream to downstream.

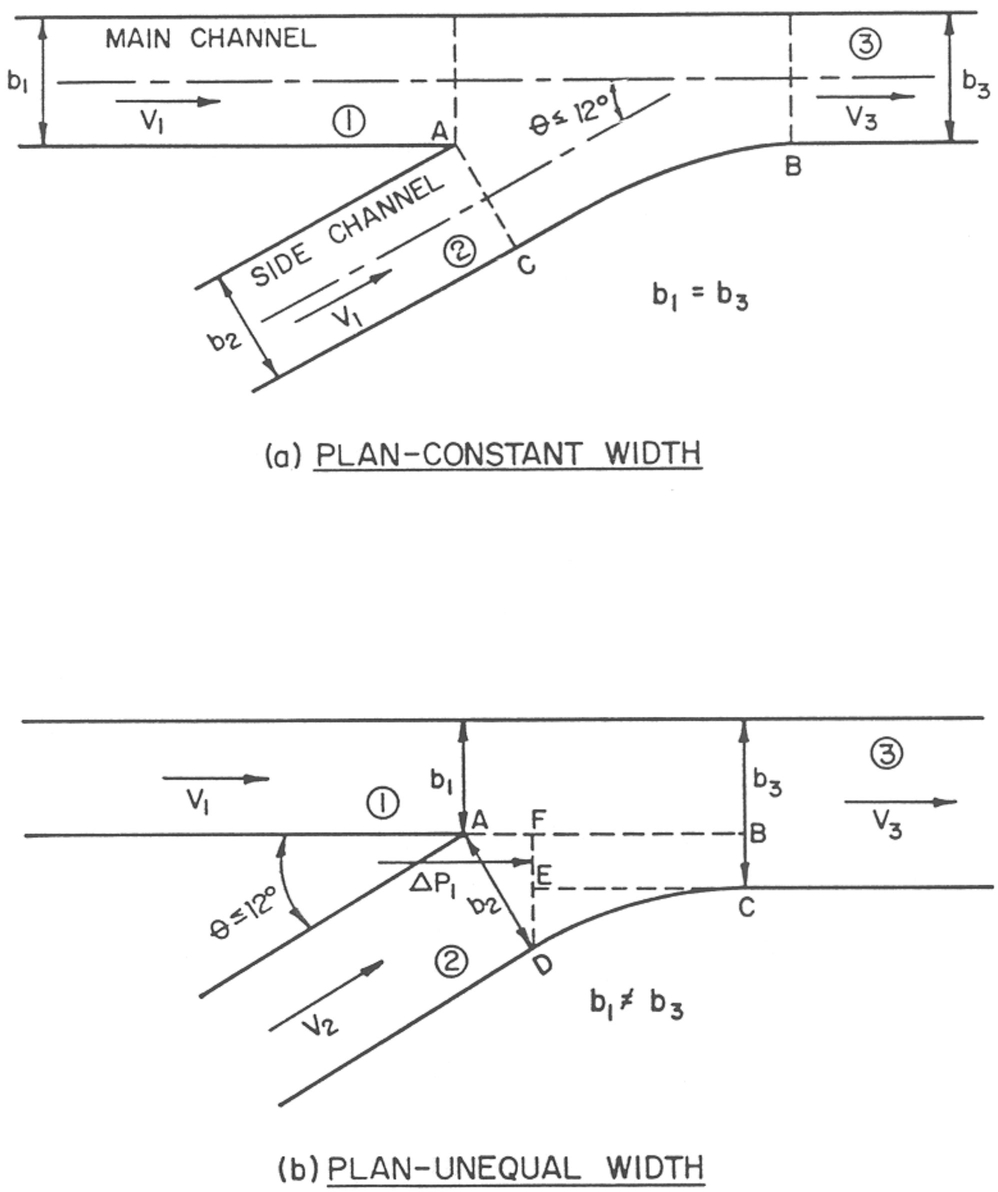


Figure HS-26—Channel Junction Definition Sketches

7.0 RUNDOWNS

A channel rundown is used to convey storm runoff from the bank of a channel to the invert of an open channel or drainageway. Rundowns can also convey runoff from streets and parking lots into channels or storage facilities. The purpose of these structures is to minimize channel bank erosion from concentrated overland flow. All too frequently, rundowns are treated as an afterthought, and receive little, if any, design attention. As a result, failure is common, resulting in unsightliness and a maintenance burden.

7.1 Cross Sections

Typical types of channel rundowns are presented in Figure HS-17, Figure HS-18 and Figure HS-27.

7.2 Design Flow

The channel rundown should be designed to carry the full design flow of the channel or storm sewer upstream of it (see the RUNOFF chapter) or 1 cfs, whichever is greater.

Photograph HS-15—A failed rundown that relied upon a geotextile membrane for stability.

7.3 Flow Depth

The maximum depth at the design flow should be equal to the calculated flow depth using drawdown calculations for the design flow plus 6 inches of freeboard. Due to the typical profile of a channel rundown beginning with a flat slope and then dropping steeply into the channel or storage facility, the design depth of flow should be the computed critical depth for the design flow.

7.4 Outlet Configuration for Trickle Channel

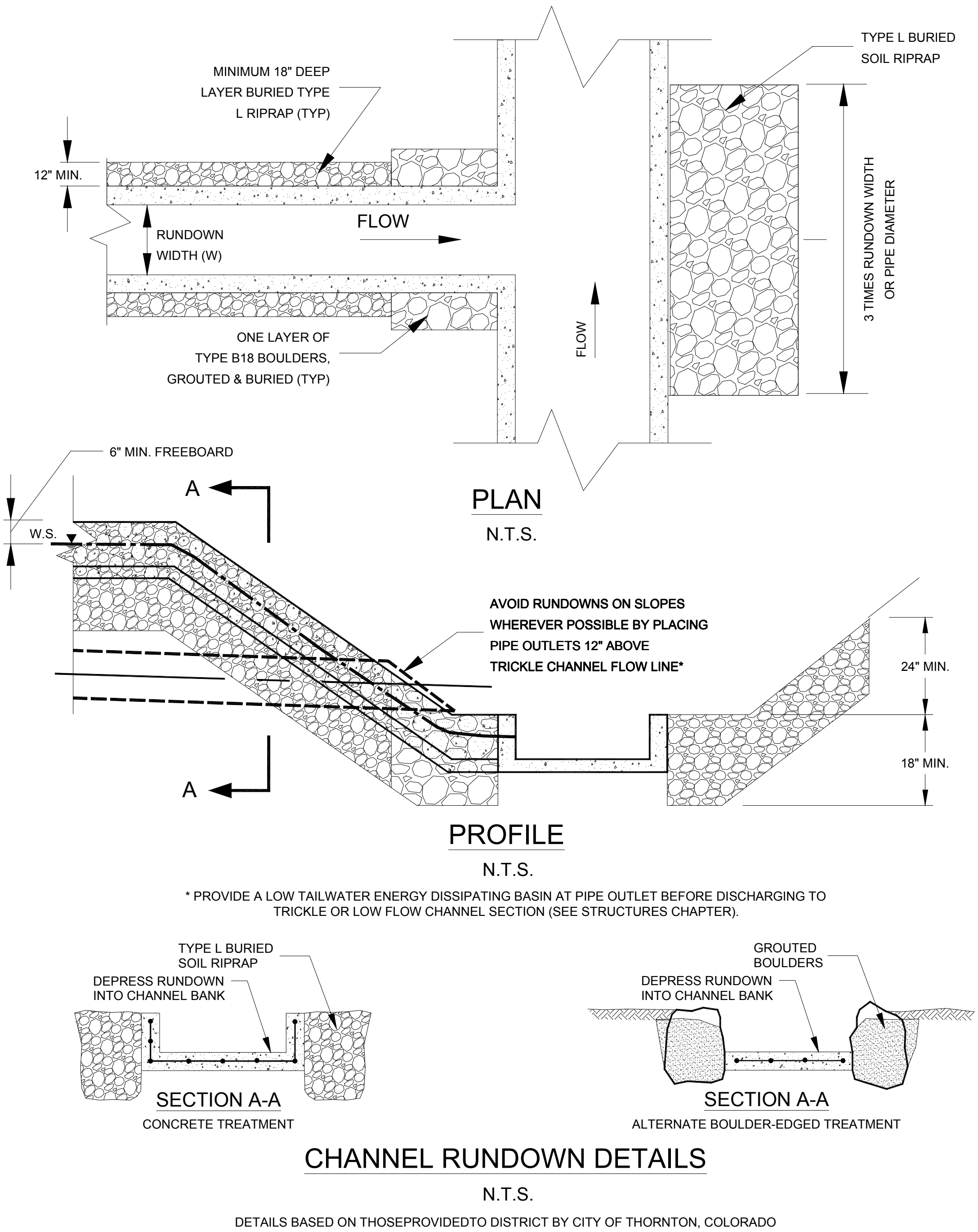
The channel rundown outlet should enter the drainageway at the trickle channel flow line. Erosion protection of the opposite channel bank should be provided by a layer of buried, grouted, Type B18 boulders. The width of this riprap erosion protection should be at least three times the channel rundown width or pipe diameter. Riprap protection should extend up the opposite bank to the minor storm flow depth in the drainageway or 2 feet, whichever is greater. Rundowns discharging into storage facilities should have comparable scour protection at the outlet, typically in the form of buried, grouted, Type B18 boulders. A forebay upstream of a trickle channel sized in accordance with Volume 3 recommendations of this *Manual* can provide this energy dissipation.

7.5 Outlet Configuration for Wetland Channel

For a wetland channel or low-flow channel, the rundown must be carried to the edge of the wetland where grouted rock is placed to dissipate the kinetic energy so that rundown discharge velocities do not cause erosion of the wetland. A low tailwater basin is also suitable for this purpose.

7.6 Grouted Boulder Rundowns

Instead of a concrete rundown, a grouted boulder rundown illustrated in Figure HS-18 can be provided. At a minimum, the width of a grouted boulder rundown should equal the width of the upstream storm sewer. The rundown depth should start at about ¾ of the height of the upstream pipe at the pipe and taper down to a depth equal to the calculated drawdown depth of water along the rundown plus 9 inches of freeboard. To find the depth of flow use Manning's n from Figure HS-3b. This will require iteration to find the value n that matches the depth of flow. Use boulders equal to at least 1/2 the height of the pipe (see boulder classifications in Table MD-8 of the MAJOR DRAINAGE chapter) grouted in accordance with the recommendations of Section 4.2.1.2 of the MAJOR DRAINAGE chapter.



* Provide a low tailwater energy dissipating basin at end of pipe before discharging to trickle or low-flow channel section.

Figure HS-27—Rundown

8.0 MAINTENANCE

8.1 General

Maintenance of structures includes removing debris, excessive vegetation and excessive sediment. Replacing or realigning stones, repairing grout and concrete, and replacing warning signs are also items of maintenance that cannot be avoided under normal conditions. Refer to the District's Maintenance Eligibility Guidelines as contained on the CD version of this *Manual* for specific guidance on maintenance provisions for many of the structures addressed in this chapter. See the District's Web site (www.udfcd.org) for the latest updates to these guidelines.

8.2 Access

During the design process, attention must be given to providing for adequate maintenance access from one or both banks in accordance with current District regulations and guidelines.

8.3 Maintenance Optimization

Structures should be designed in accordance with public works policies related to minimizing operation and maintenance requirements.

9.0 BOATABLE DROPS

9.1 Introduction

Low-head dams or drop structures on a stream that includes boating should not present undue hydraulic hazards to boaters, maintenance workers or to the public. This is why some low-head dams and drop structures are retrofitted. This section outlines the approach for use in improving recreational boater safety.

9.2 Retrofitting Existing Structures

Retrofitting low-head dams and drop structures generally includes installing a stepped or sloped downstream structure face and suitable boat chute with upstream pilot rocks; eliminating sharp edges; and providing appropriate barriers, signing and accessible portages with take-out and put-in landings. A structure that is too high for the site may be replaced with two or more structures to reduce the drop at a single location.

Retrofitting boatable low-head dams or drop structures requires specific care to insure that the retrofit meets the objective of enhancing public safety. Hydraulic model tests are common for retrofitting of low-head dams and drop structures.

9.2.1 Downstream Face

A vertical or steep downstream face of a structure to be retrofitted may be corrected with a rock face having a slope of 10(H) to 1(V). Large rock or derrick stone is often used. The engineer may select a stepped face of either concrete or stone.

9.2.2 Boat Chute

Installing a boat chute to provide passage around or over the low-head dam or drop is desirable for boatable streams, even where the total drop may be only 3 feet or less. The boat chute may be combined with a relatively flat, sloping downstream face in many instances. Pilot rocks planted upstream of the boat chute signal the entrance to the boat chute.

9.2.3 Sharp Edges

Exposed sheet piling edges, sharp concrete edges, sharp rock protuberances, and angle-iron ends should be avoided in boatable stream structures.

9.2.4 Barriers and Signing

A range of barriers may be considered for use at structures to help keep watercraft from crests, intakes, and areas of highly turbulent flow. Barriers often include buoy lines. Warning signs should be placed upstream of structures at easily visible locations.

Photograph HS-16—The unsightly and hazardous 8-foot-high Brown Ditch weir was replaced with three low-head drop structures having a 10:1 downstream slope and a boat chute. The resulting improvement by the USACE has provided for safe, enjoyable recreational boating.

9.2.5 Portages

At many hydraulic structures, portages are provided to permit beginning boaters to bypass a boat chute or to avoid a more challenging hydraulic structure. Portages have take-outs and put-ins at appropriate locations combined with suitable signing.

9.3 Safety

Retrofitting hydraulic structures on boatable streams should be undertaken with an adequate standard of care related to public safety for boating. A retrofit often includes installation of anchor points and suitable access for use by rescue personnel (Wright, et al. 1995).

10.0 STRUCTURE AESTHETICS, SAFETY AND ENVIRONMENTAL IMPACT

10.1 Introduction

Aesthetics, safety, overall integration with nearby land uses, and minimizing adverse environmental impacts are important aspects in the design of hydraulic structures. The planning, design, construction, and maintenance of hydraulic structures in an urban setting must include consideration of aesthetics, safety, and effects on the environment. Maximizing functional uses while improving visual quality and safety require good planning from the onset of the project and the coordinated efforts of owner/client, engineer, landscape architect, biologist, and planner.

10.2 Aesthetics and Environmental Impact

The combination and diversity of forms, lines, colors, and textures creates the visual experience. Material selection and placement of vegetation can provide visual character and create interesting spaces in and around hydraulic structures.

Photograph HS-17—Grouted sloping boulder drops can be built in series to create pleasing amenities and to provide stable and long-lived grade control structures.

Good planning may offer opportunities to minimize potentially adverse environmental impacts and maintain the natural habitat characteristics of the drainageway while fulfilling hydraulic, open space, and recreation requirements. As discussed in detail in the POLICY, PLANNING, and MAJOR DRAINAGE chapters, multiple uses of flood control structures, open space, and parks have proven to be an effective land use combination. Such structures as channels, overflow structures, grade controls, energy dissipators, maintenance roads, and others can blend in with the park environment.

In natural and urbanized areas, the use of vegetation for bank protection and landscape treatment is effective. Bioengineering strategies that incorporate vegetation and natural materials can improve habitat for fish and wildlife, and create a pleasant environment, as discussed in the MAJOR DRAINAGE chapter.

Plant selection and placement around structures and channel features and use of planting that reduces erosion, dissipates residual energy, and does not create debris or local scour problems are fundamental to good aesthetics and environmental quality, as well as hydraulic function. Inclusion of high-maintenance plantings and spaces with planting that are inaccessible or require extensive care are not advisable, since they may end up poorly maintained, become a nuisance, and be unattractive.

In highly developed streamside areas, concrete plazas and edge treatment can be combined to increase channel efficiencies while providing reasonable access to the waterway area. Geometric and architectural forms, hard edges, and formal arrangements of materials are generally associated with urban settings. However, all of these features require sound engineering and evaluation of the structure stability and the effects on the hydraulic characteristics of the channel. Such facilities are usually well received by the public.

A variety of materials and finishes are available for use in hydraulic structures. Concrete color additives, exposed aggregates and form liners can be used to create visual interest to otherwise stark walls. The location of expansion and control joints in combination with edges can be used to help create attractive design detailing of headwalls and abutments.

Natural materials, rock, and vegetation can be used for bank stability and erosion protection while providing unusual interest, spatial character, and diversity. The placement and type of the rock can provide poor or pleasing appearance. A stepped boulder arrangement for drops, where there is a larger top horizontal surface, is usually an appealing placement that also improves hydraulics.

10.3 Safety

Design and construction of urban drainage facilities must account for potential public safety hazards. When planning and providing for recreation within public parks and open space, safety must always be considered, and safety for the public and maintenance workers should be incorporated. The design engineer must consider the variations in hydraulic jumps as they relate to the tailwater elevation as illustrated in Figure HS-28. Some hydraulic structures and drainage features offer an invitation to play; therefore, what is constructed should be made safe and attractive. While safety, to a reasonable extent, becomes the responsibility of the user, appropriate warning signage must be used. In some instances, fencing and emergency access and egress should be provided.

Safety requirements are usually defined by local government agencies. However, case-made law may define the responsibilities of involved parties. Risk and liability are important with respect to including signs, handrails, or barriers at steep slopes or vertical drop-offs as well as other safety related features. Signage should be provided at locations where public use is intended near hydraulic structures and where hazards are not obvious to the average person. For boatable waterways the standard of care should include avoidance of hazardous hydraulics such as reverse rollers and reverse flow eddies associated with hydraulic structures. When bicycle paths are incorporated with the construction of structures, there should be adequate directional and warning signs, sight distance, and avoidance of unannounced sharp turns and dropoffs.

Photograph HS-18—Warning signs can be used to help achieve public boating safety, but signs cannot in themselves serve as a substitute for an appropriate standard of care in the design of a reasonable grade control structures on a boatable waterway.

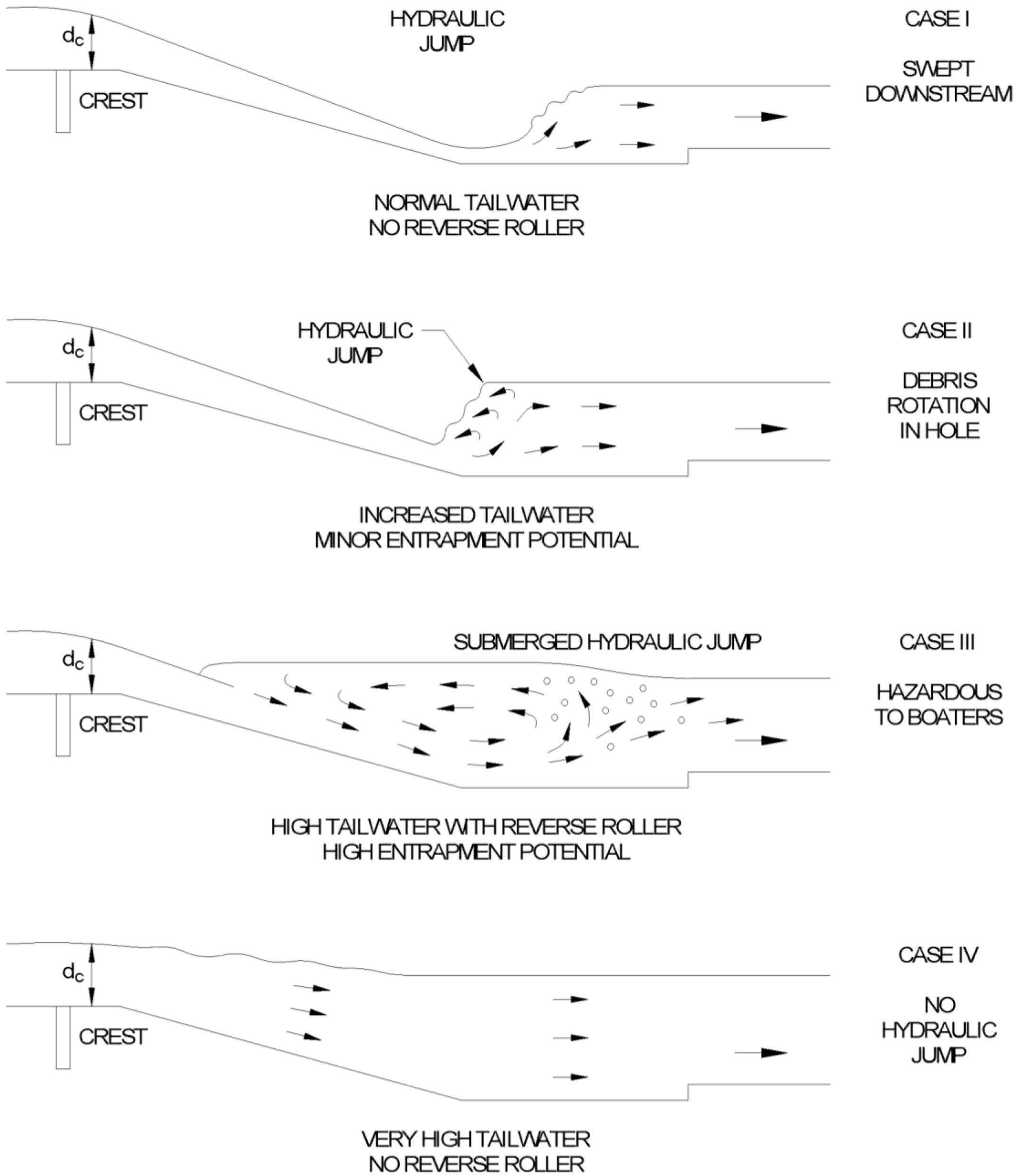


Figure HS-28—Hydraulic Jump Tailwater Stages as Related to Boating Hazards

11.0 CHECKLIST

Criterion/Requirement (Note: Before work begins in a floodplain, obtain a floodplain development permit form local jurisdiction)	✓
Drop Structures (All Types)	
Simplified design or detailed hydraulic analysis	
Soils and seepage analysis	
Environmental permits	
High public usage or low public usage	
Likely downstream degradation or no likely downstream degradation	
Critical depth at crest	
Transition head loss	
Hydraulic roughness	
Hydraulic jump length and location	
Basin length	
Seepage control (need detailed analysis or provisions for drops taller than 5 feet)	
Individual force analysis	
Trickle and low-flow zone provisions	
Sloping Drop Height > 6 feet, Use Special Design	
Sloping Drop Height $\leq$ 6 feet, Used Simplified Design	
Vertical Drop	
Rock sizing	
Boatable channel, or not	
Froude number at toe	
Reverse roller evaluation	
Portages and warning signs, with peer review	
Non-Boatable Grouted Sloping Boulder Drops	
Waterway is not boatable	
Maximum design discharge less than 7,500 cfs	
Uniform size boulders as per Table HS-4	
Drop height less than 5 feet	
Vertical cutoff minimum depth at crest of 0.8 H_d or 4 feet	
Trickle or low-flow channel through crest	
Net downward force of 30 PSF	
Stilling basin depressed 1 to 2 feet	
Drop face slope at 4:1, or flatter	
Grouted rock approach of 8 feet	
Basin length of 25 feet for erosive soils, and 20 feet for non-erosive soils	
Large boulders in center basin	
Buried downstream riprap zone 2 H_d or 10 feet	
If drop height exceeds 5 feet, detailed hydraulic analysis used (see Section 2.3)	
Vertical Hard Basin Drops	
Waterway is not boatable	
Maximum drop height of 3 feet	
Low probability for public access (public safety concern for vertical drops)	
Drop number D_n defined	
Hydraulic jump length is 6 times Y_2	
Basin floor depressed minimum 1.5 feet	
Minimum boulder size of 1.5 feet	
Grout thickness minimum 10 inches	
Basin length of 25 feet minimum	
Riprap approach length of 10 feet	

Criterion/Requirement	✓
Baffle Chute Drops	
Waterway is not debris-prone	
Waterway is not boatable	
Minimum of 4 baffle rows	
Unit discharge maximum of 60 cfs/ft	
Sloping apron of 2:1 or less	
Buried and protected toe with 1.5 baffle rows	
Baffle height of $0.8D_c$	
Wall height $2.4D_c$	
Boatable Channel Drops	
Maximum drop height of 4 feet	
Froude number at toe < 1.5	
Reverse rollers avoided	
Downstream face slope 10:1	
Pilot rocks and signing	
Suitable portage facilities	
Peer review	
Low-Flow Check and Wetland Structures	
Dominant discharge computed	
Trickle channel maximum depth of 3 feet, or 5 feet downstream of check	
Lateral overflow protection	
Trickle channel cutoff extension of 5 to 10 feet into bank	
Wetland checks extended 10 feet into bank	
Maintain upstream wetland water table	
Impact Stilling Basin Outlet Structures	
Horizontal entrance pipe	
Basin width as per Figure HS-15	
Calculate hydraulic force	
Type M riprap downstream	
Sill wall minimum of 2 feet	
Low Tailwater Basin Outlet Structures	
Riprap size as per Figure HS-20	
Minimum riprap thickness of 1.75 D_{50}	
Minimum basin length as per Equations HS-18 or HS-19	
Minimum basin width of $4D$ or $W + 4H$	
Riprap slopes of 2H to 1V	
Pipe fasteners and cutoff wall	
Culvert Outlet Energy Dissipator (Outlet Structures)	
Scour and degradation control	
Tailwater depth adequacy	
Bridges (Preliminary Assessment Only)	
Avoid scour and deposition	
Minimize hydraulic interferences	
Water surface profiles and hydraulic gradients determined	
Backwater effect less than 1 foot	
Banks and bottom protected from higher velocity flows	
Check for supercritical flow	
Adequate freeboard if debris prone	
Backwater coefficient K	
Procedure 4.3 followed for design	

Criterion/Requirement	✓
Boatable Drop Structures	
Downstream face at reasonable slope (e.g., 10H to 1V)	
Stepped face, or derrick stone	
Boat chute	
No sharp protrusions	
Pilot rocks	
Barriers if desirable	
Signing, informational and warning	
Portage with adequate signing	
Anchor points suitable for emergency rescue	
Peer review by whitewater expert	
General Items for Hydraulic Structures	
Visual quality	
Forms and lines	
Colors	
Vegetation	
Accessibility for maintenance; long-term maintenance assured	
Safety	
Public access	
Maintenance workers	
Hydraulic jump analysis with various tailwater elevations	
Signage	
Absence of reverse rollers and minimal reverse eddies	
Peer review	
Permitting	

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