22.9 Criteria for Hydraulic Design of Closed Conduits

22.9.1 General Hydraulic Criteria

Closed conduit sections (pipe, box or arch sections) will be designed as flowing full and, whenever possible, under pressure except when the following conditions exist:

a. In some areas of high sediment potential, there is a possibility of stoppage occurring in drains. In situations where sediment may be expected, the City Engineer must be consulted for a determination of the appropriate bulking factor.

b. In certain situations, open channel sections upstream of the proposed closed conduit may be adversely affected by backwater.

If the proposed conduit is to be designed for pressure conditions, the hydraulic grade line shall not be higher than the ground or street surface, or encroach on the same in a reach where interception of surface flow is necessary. However, in those reaches where no surface flow will be intercepted, a hydraulic grade line which encroaches on or is slightly higher than the ground or street surface will be acceptable provided that pressure manholes exist or will be constructed.

22.9.2 Hydraulic Grade Line Calculations

a. Determination of Control Water Surface Elevation

A conduit to be designed for pressure conditions may discharge into one of the following:

(1) A body of water such as a detention reservoir

(2) A natural watercourse or arroyo

(3) An open channel, either improved or unimproved

(4) Another closed conduit

The controlling water surface elevation at the point of discharge is commonly referred to as the control and, for pressure flow, is generally located at the downstream end of the conduit. If flow becomes unsealed, the control may be at the first grade break upstream of the point where unsealing occurs or, under certain conditions, may be farther upstream.

Two general types of controls are possible for a conduit on a mild slope, which is a physical requirement for pressure flow in discharging conduits.

a. Control elevation above the soffit elevation. In such situations, the control must conform to the following criteria:

(1) In the case of a conduit discharging into a detention facility, the control is the 10-year water surface reservoir elevation.

(2) In the case of a conduit discharging into an open channel, the control is the 10-year design water surface elevation of the channel.

(3) In the case of a conduit discharging into another conduit, the control is the design hydraulic grade line elevation of the outlet conduit immediately upstream of the confluence.

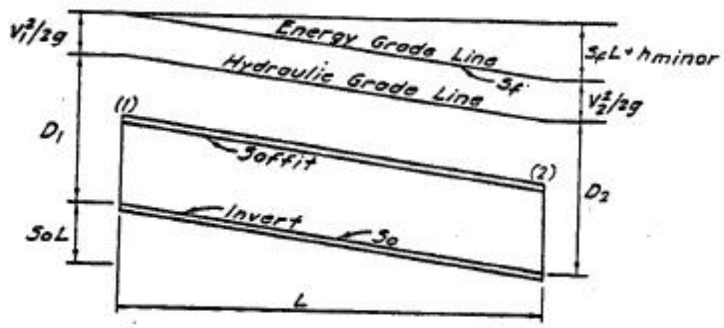
Whenever case (1) or (2) above is used, the possibility of having flow out of manholes or inlets due to discharge elevations at the 100-year level must be investigated and appropriate steps taken to prevent its occurrence.

b. Control elevation at or below the soffit elevation. The control is the soffit elevation at the point of discharge. This condition may occur in any one of the four situations described above in 2a.

22.9.3 Instructions for Hydraulic Calculations

Most procedures for calculating hydraulic grade line profiles are based on the Bernoulli equation. This equation can be expressed as follows:

$$\frac{V_1^2}{2g} + D_1 + S_o L = \frac{V_2^2}{2g} + D_2 + S_f L + h_{minor}$$



in which D = Vertical distance from invert to H.G.L.
So = Invert slope
L = Horizontal projected length of conduit
Sg = Average friction slope between Sections 1 and 2
V = Average velocity (g/A)
h_{minor} = Minor head losses

Minor losses have been included in the Bernoulli equation because of their importance in calculating hydraulic grade line profiles and are assumed to be uniformly distributed in the above figure.

When specific energy (E) is substituted for the quantity (V2/2g + D) in the above equation and the result rearranged,

$$L = \frac{E2 - E1}{S_o - S_f}$$

The above is a simplification of a more complex equation and is convenient for locating the approximate point where pressure flow may become unsealed.

22.9.3.1 **Head Losses**

(A) Friction Loss

Friction losses for closed conduits carrying storm water, including pump station discharge lines, will be calculated from the Manning equation or a derivation thereof. The Manning equation is commonly expressed as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2}$$

in which Q = Discharge, in c.f.s.

n = Roughness coefficient

A = Area of water normal to flow in ft.2

R = Hydraulic radius

S_f = Friction slope

When rearranged into a more useful form,

in which

$$S_f = \left[\frac{Qn}{1.486AR^{2/3}}\right]^2 = \left[\frac{Q}{K}\right]^2$$

in which:

$$K = \frac{1.486\ AR^{2/3}}{n}$$

The loss of head due to friction throughout the length of reach (L) is calculated by:

$$h_f = S_f L = \left[\frac{Q}{K}\right]^2 L$$

The value of K is dependent upon only two factors: the geometrical shape of the flow cross section as expressed by the quantity ($AR^{2/3}$), and the roughness coefficient (n). The values of n are shown in Section 16.

(B) Transition Loss

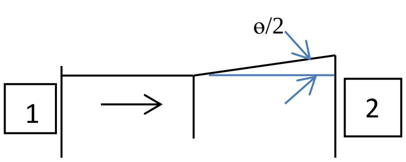
Transition losses will be calculated from the equations shown below.

For a Contraction (increasing velocity):

$H_f=K_e/2(V_2-V_1)^2/2g$

For an Expansion (decreasing velocity):

$H_f=K_e(V_2-V_1)^2/2g$



Where:

$Ke=3.50(\tan \theta/2)^{1.22}$

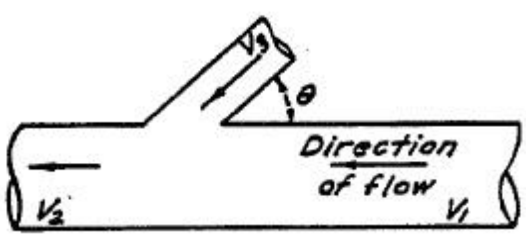
These equations are applicable when no change in Q occurs and where the horizontal angle of divergence or convergence (θ /2) between the two sections does not exceed 5 degrees 45 minutes.

Deviations from the above criteria must be approved by the City Engineer. When such situations occur, the angle of divergence or convergence (θ /2) may be greater than 5 degrees 45 minutes. However, when it is increased beyond 5 degrees 45 minutes, the above equation will give results for h_f that are too small, and the use of more accurate methods, such as the Gibson method shown, wherein $Ke=3.50(\tan \theta/2)^{1.22}$.

(C) Junction Losses

In general, junction losses are calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using either the P + M method or the Thompson equation, both of which are shown in Section 22.7. Both methods are applicable in all cases for pressure flow and will give the same results.

For the special case of pressure flow with $A_1 = A_2$ and friction neglected,



$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \frac{2A_3}{A_2} \cdot \frac{V_3^2}{2g} \cdot \cos\theta$$

(D) Manhole Loss

Manhole losses will be calculated from the equation shown below. Where a change in pipe size and/or change in Q occurs, the head loss will be calculated in accordance with Sections (2) and (3), preceding.

$H_{mh}=0.05(V^2/2g)$

(E) Bend Loss

Bend losses will be calculated from the following equations:

$$H_b=Kb(V^2/2g)$$

in which:

$$Kb=0.20(\theta/90^o)^{0.5}$$

□here θ=Central angle of bend in degrees

Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

(F) Exit Loss

Exit loss is the loss when storm drains daylight into a pond or channel, the loss associated with this condition.

$$h_{exit}=0.25(V^2/2g)$$

(G) Transition to Smaller Pipe Size

As a general rule, storm drains will be designed with sizes increasing in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the conduit may be decreased in size with the approval from the City Engineer.

22.9.4 Design Requirements for Maintenance and Access

a. Manholes

(1) Spacing

Manholes should be spaced at intervals of approximately 450 feet. Where the proposed conduit is less than 30 inches in diameter and the horizontal alignment has numerous bends or angle points, the manhole spacing should be reduced to approximately 300 feet.

The spacing requirements shown above apply regardless of design velocities. Deviations from the above criteria are subject to City Engineer approval.

(2) Location

Manholes should not be located in street intersections, especially when one or more streets are heavily traveled.

In situations where the proposed conduit is to be aligned both in easement and in street right-of-way, manholes should be located in street right-of-way, wherever possible.

Manholes should be located as close to changes in grade as feasible when the following conditions exist:

(a) When the upstream conduit has a steeper slope than the downstream conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.

(b) When transitioning to a smaller downstream conduit due to an abruptly steeper slope downstream, sediment tends to accumulate at the point of transition.

(3) Design

When the design flow in a pipe flowing full has a velocity of 20 f.p.s. or greater, or is supercritical in a partially full pipe, the total horizontal angle of divergence or convergence between the walls of the manhole and its center line should not exceed 5º45'.

b. Pressure Manholes

A pressure manhole shaft and a pressure frame and cover will be installed in a pipe or box storm drain whenever the design water surface is more than 0.2 feet above the ground surface. Pressure manholes should only be used when a non-pressure manhole solution is unavoidable.

c. Special Manholes

Special 36-inch diameter manholes or vehicular access structures will be provided when required. The need for access structures will be determined by the City Engineer during the review of preliminary plans.

d. Deep Manholes

A manhole shaft safety ledge will be provided in all instances when the manhole shaft is 20 feet or greater in depth. Installation will be in accordance with City Engineer requirements.

22.9.5 Closed Conduit Pipe Size and Slope

(A) Minimum Pipe Size

In cases where the conduit may carry significant amounts of sediment (greater than 8%), the minimum diameter of main line conduit will be 36 inches. In situations where sediment may be expected, the City Engineer will be consulted to determine the applicability of sediment criteria.

(B) Minimum Slope

The minimum slope for main line conduit will be .001 (.10 percent), unless otherwise approved by the City Engineer. Minimum flow velocity for the 10-year design flow will be 3 f.p.s.

22.9.6 Earthen Channels to Storm Drain Structures

An inlet structure will be provided for storm drains located in natural channels. The structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron or rip-rap.

The apron slope should be limited to a maximum of 2:1. Wall heights should conform to the height of the water upstream of the inlet, and be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing and a protection barrier to prevent public entry will be provided.

If trash and debris are prevalent, barriers consisting of vertical 3-inch or 4-inch diameter steel pipe at 24 inches to 36 inches on centers should be embedded in concrete immediately upstream of the inlet apron. Trash rack designs must have City Engineer approval.

22.9.7 Storm Drain Outlets to Public Earthen Arroyos and Ponds

22.9.7.1 When a storm drain outlets into an earthen arroyo, an outlet structure will be provided which prevents erosion and property damage. Fencing and a protection barrier will be provided where deemed necessary by the City Engineer.

22.9.7.2 The outlet structure shall have an end treatment and design that minimizes erosion. The following design criteria was adopted from “Urban Storm Drainage Criteria Manual Volume 2” from the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, revised April 2008.

2.1 General

Energy dissipation or stilling basin structures are required to minimize scour damage 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 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 probably 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;

(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 or retention pond.

2.2 Impact Stilling Basins

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 in dissipating energy without the need of tailwater. The original hydraulic design reference by Biechly (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. Art 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 consist 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.

2.2.1 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.

2.2.2 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 with of 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 1/2D 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 is also an alternative.

2.2.3 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 $D=(A_{sect})^{0.5}$ and compute the Froude number$=V/(gD)^{0.5}$.

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 flow) 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 overhaning 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)

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/earthern 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 flor depth is that the basin area will not drain completely.

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

2.3 Pipe Outlet Rundowns

2.3.1 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, tha 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 a the crest of the chute. To insure low velocities at the upstream end, it may be necessary t 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.

2.3.2 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 closedly adhere to the recommendations for grouted boulders.

If the exit velocities of the pipe exceeds 12 feet per second, an approach cute for the baffle chute rundown described above should be considered and provided.

2.3.4 Low Tailwater Riprap basins at Pipe Outlets

2.3.4.1 General

The design of low tailwater riprap bsins 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.

2.3.4.2 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 casuing scour at the channel bottom. Photograph HS-12 shows a fairly large low tailwater basin.

2.3.4.3 Low Tailwater Basin Design

Low tailwater is defined as being equal to or less than 1/3 of the height of the storm sewer, that is:

$Y_t \leq D/3$ or $Y_t \leq H/3$

where:

Y_t= tailwater depth at design

D=diameter of circular pipe (ft)

H=height of rectangular pipe (ft)

2.3.4.3.1 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. See Section 22.16.

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

$V_{full}=Q_{full}/A_{full}$

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 form Figure HS-20b using the Froude parameter.

$Q/D^{2.5}$ or $Q//(wH^{1/5})$

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

D=D(d/D0 or d=H(d/H)

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

$A=(A/Af_{ull})A_{full}$

Finally,

$V=Q/A$

Where for equations in this section:

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

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

See Section 22.16 for definitions of the Manning's equation.

2.3.4.3.2 Riprap Size

For the deisgn 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 (e.g. B18, H, M, or L)(check on this). First calculate the riprap sizing design parameter, P_d, namely,

$P_d=(V^2+gd)^{0.5}$

where:

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 alternate to a grouted boulder or lose riprap basin is to use the standard USBR Impact Basin Vi or one of its modified versions, described earlier in this Section.

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

$T=1.75D_{50}$

where:

D50 = the median size of the riprap (see Table HS-9)

Table HS-9

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

2.3.4.3.3 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 $L=(D)^{0.5}(V/2)$

For rectangular pipe: $L=4H$ or $L=(H)^{0.5}(V/2)$

Where:

L=basin length

H=height of rectangular conduit

V= design flow velocity at outlet

D= diameter of circular conduit

2.3.4.3.4 Basin Width

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

For circular pipes: $W=4D$

For rectangular pipe: $W= w+4H$

where:

W=basin width (Figure HS-19)

D=diameter of circular conduit

W=width of rectangular conduit

2.3.4.3.5 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 bottm between the walls, both with a cutoff wall that extends down to a depth of:

$B=D/2 + T$ or $B= H/2 + T$

where:

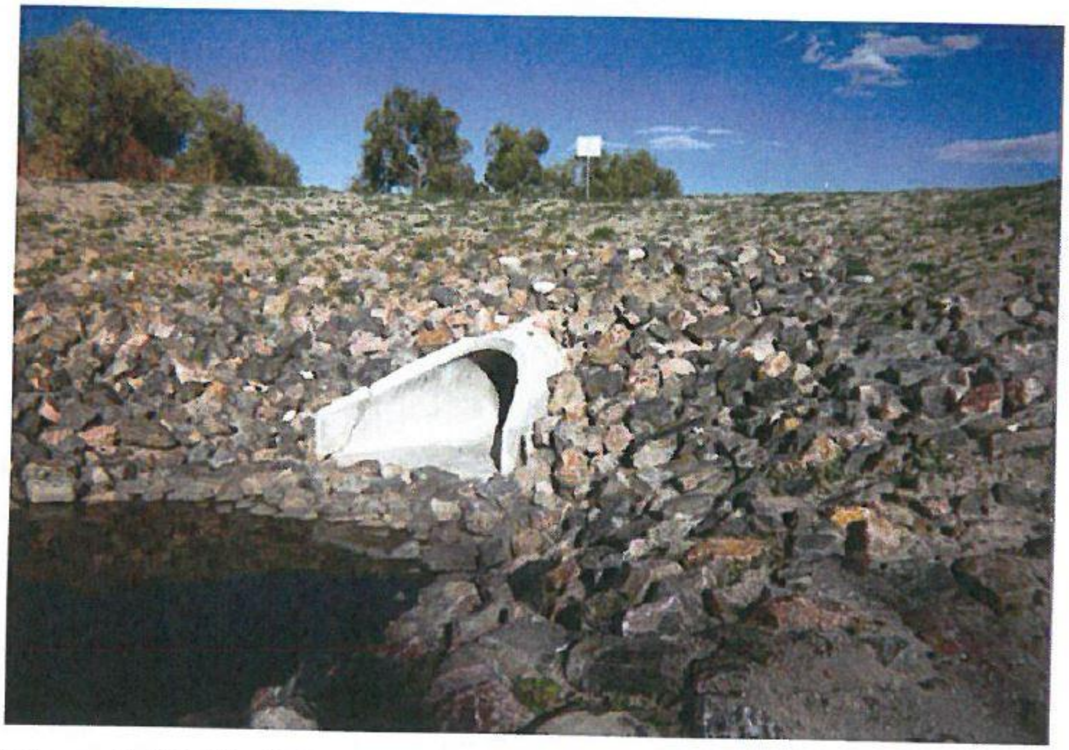
B= cutoff wall depth

D= diameter of circular conduit

$T=1.75D_{50}$

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

2.3.5 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.

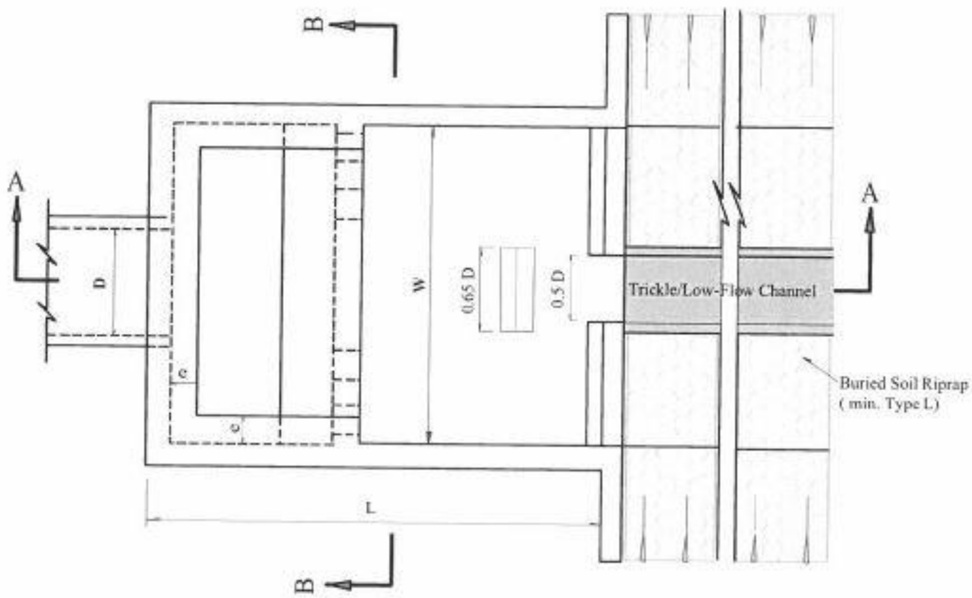
Culvert outlets represent a persistent problem because of concentrated discharges and turbulence that are not fully controlled prior to the flow reaching the standard downstream channel configuration. Too often the designer's efforts are focused on the culvert inlet and its sizing with outlet hydraulics 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 required special structures downstream of the culvert outlet to limit local scour, general stream degradation, and troublesome head cutting. Some of the techniques described earlier in this section my be applied at culvert 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 casue 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 fond 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 flows.

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 distance downstream. Both scour and stream degradation can occur simultaneously at a culvert outlet.

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



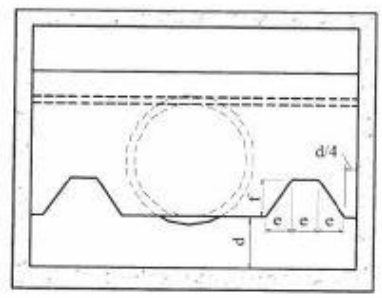
$$W = 2.94D\left[V/gD\right]^{0.556}$$

$V = Velocity,\ ft/s$

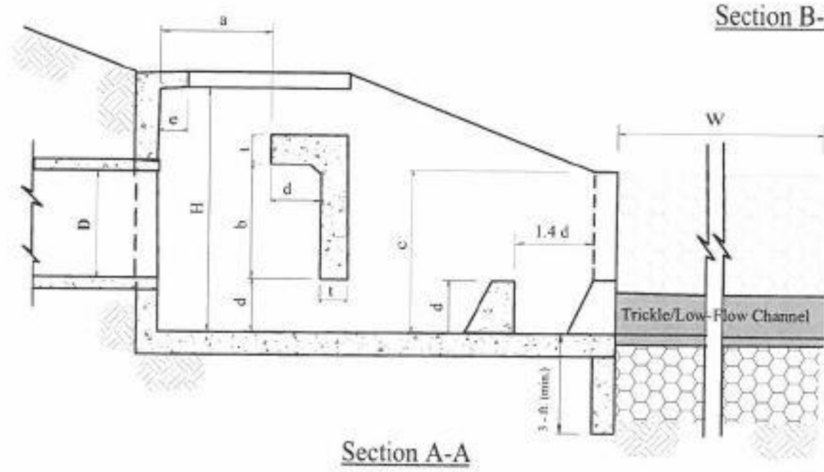
$D = \sqrt{A}$

$A = Area\ of\ flow,\ sq.ft.$

$H = 3/4\ W$	$c = 1/2\ W$
$L = 4/3\ W$	$d = 1/6\ W$
$a = 1/2\ W$	$e = 1/12\ W$
$b = 3/8\ W$	$f = 1/8\ W$
$t = 1/12\ W$ (suggested min)	



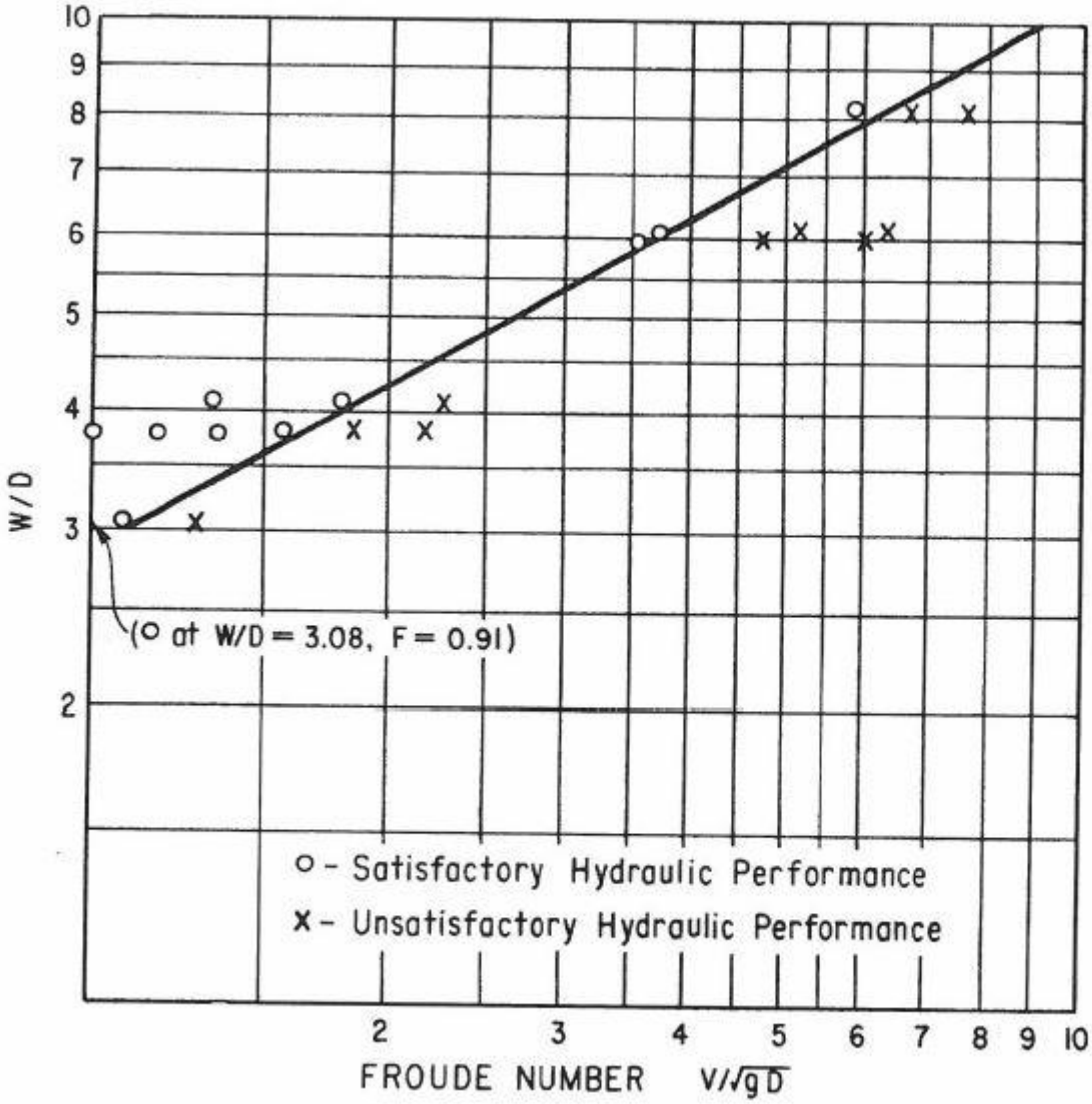
Section B-B



Section A-A

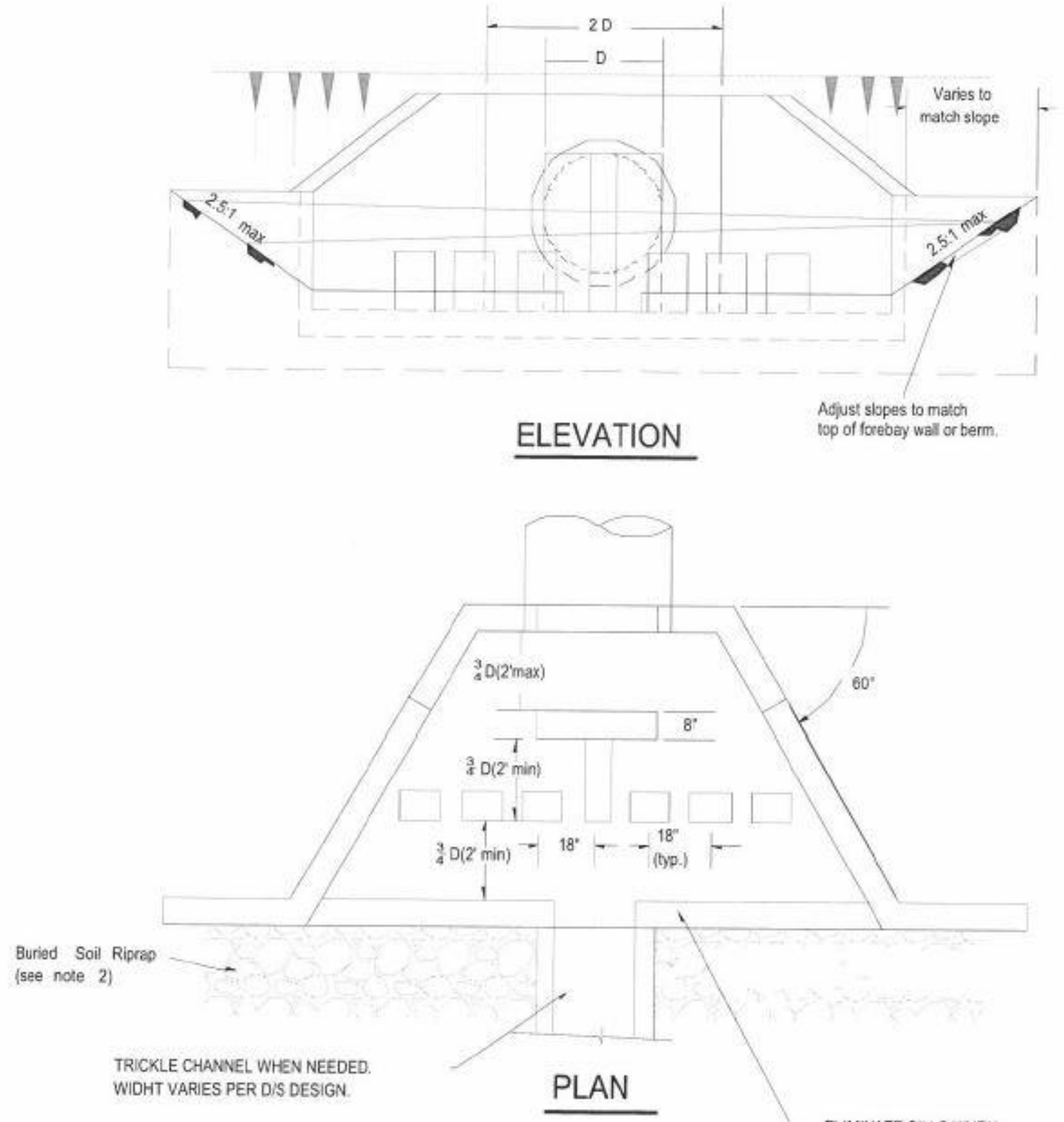
USGS Impact Stilling Basin Modified by UDFCD February 2004

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)

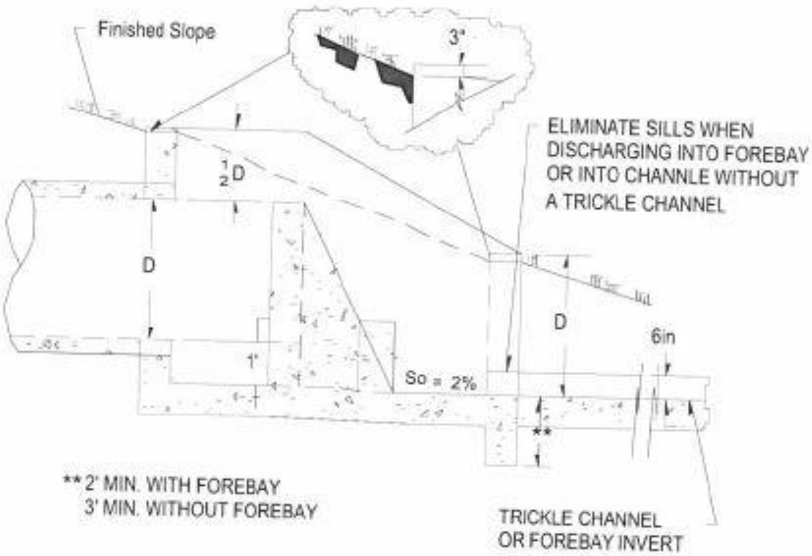


Notes: 1) Design of reinforcing steel is the responsibility of the desing engineer. Reinforce to withstand water earh pressueres.

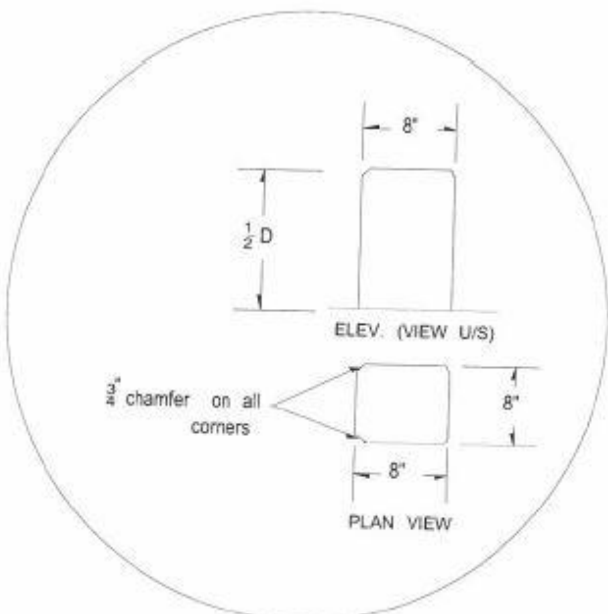
2) When dischargning into channel and not forebay install Type M buried soil riprap for a distance of 3*D downstream of structure.

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

(Sheet 1 of 2)

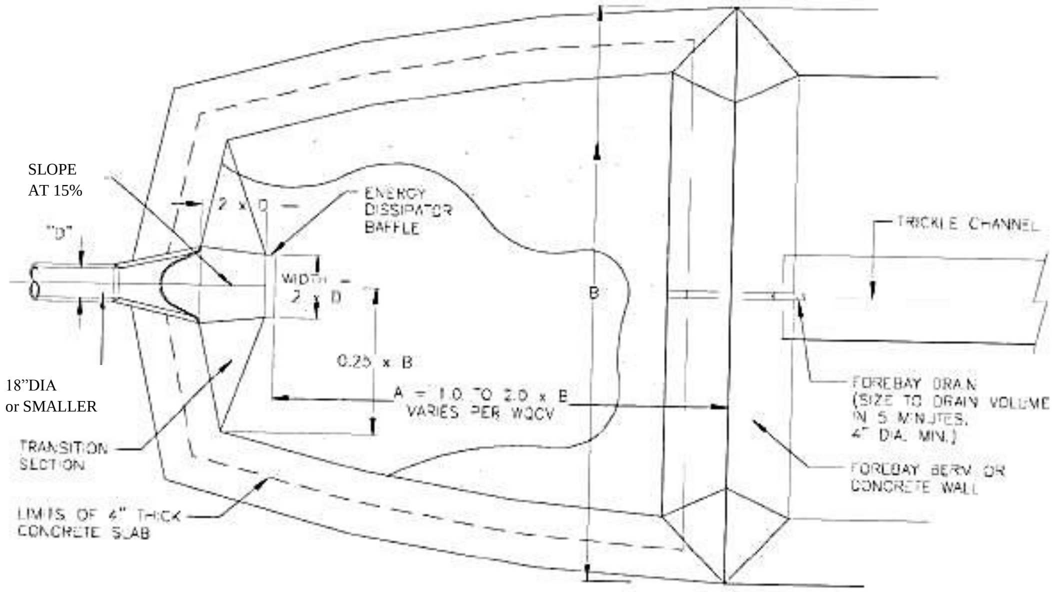


PROFILE

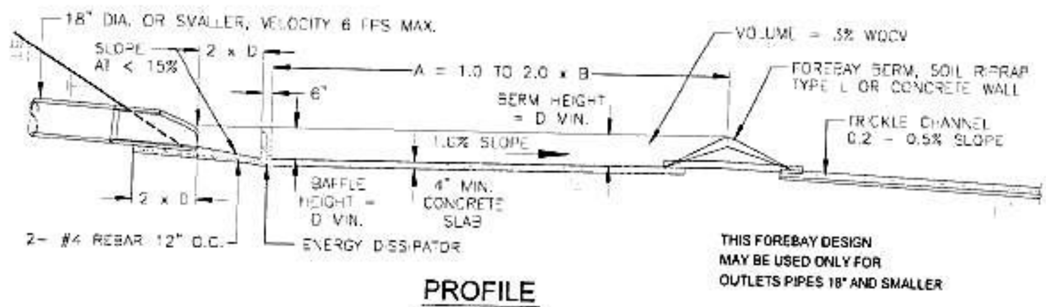


BAFFLE BLOCK GEOMETRY

Figure HS-16a. Modified Impact Stilling Basin for Conduits 18" to 48" in Diameter (Sheet 2 of 2)



PLAN
NOT TO SCALE



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)

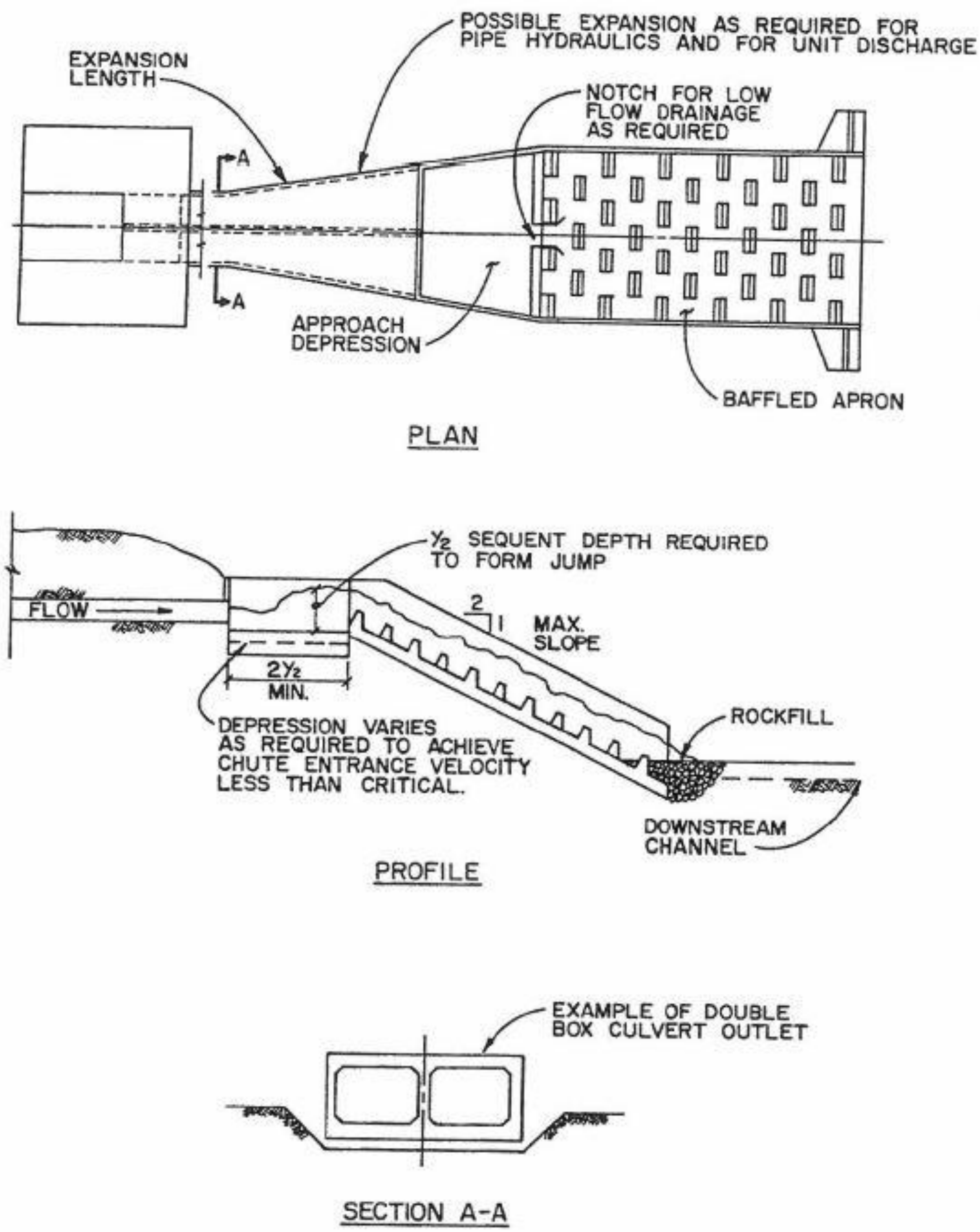
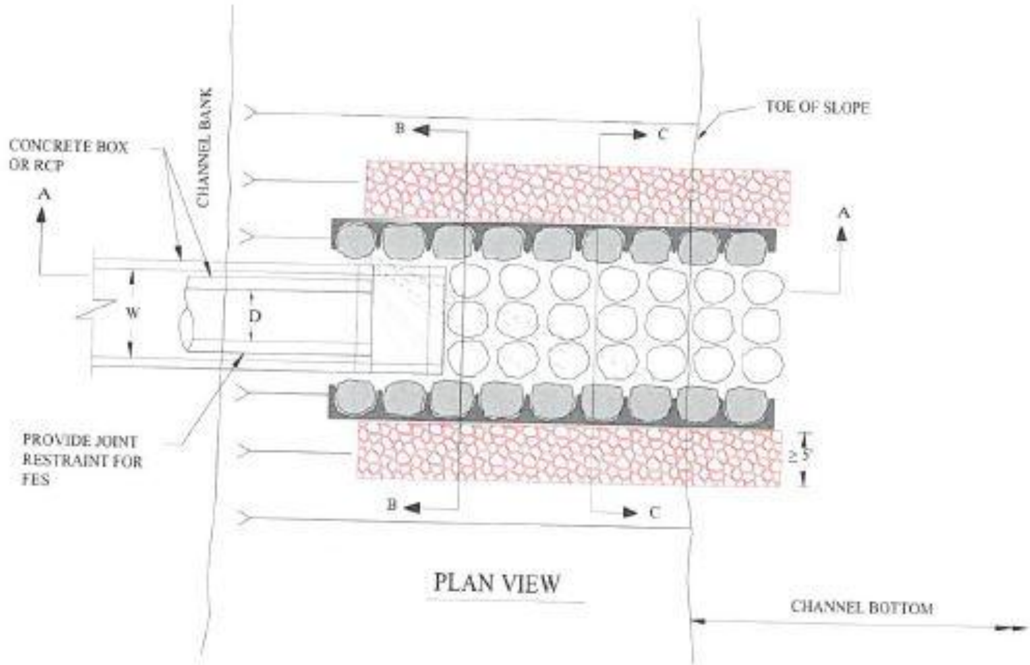
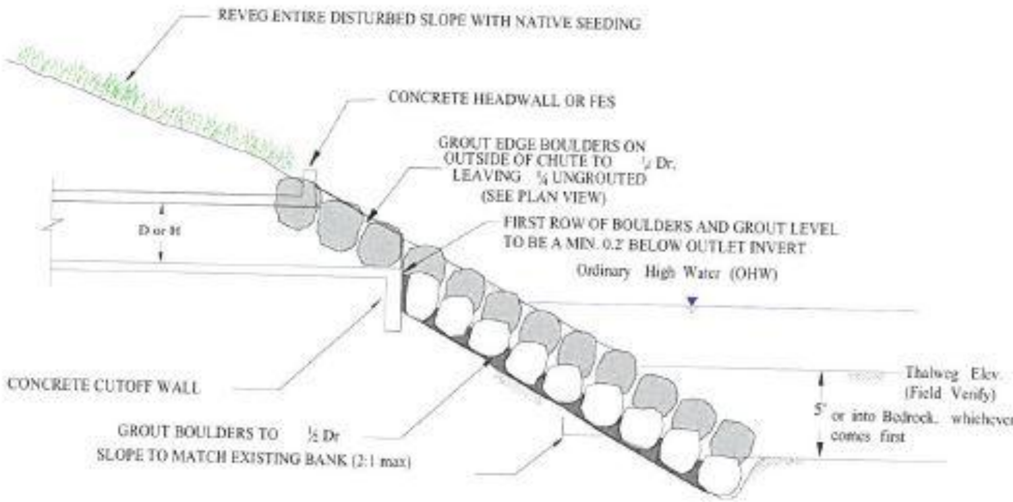


Figure HS-17—Baffle Chute Pipe Outlet



PLAN VIEW

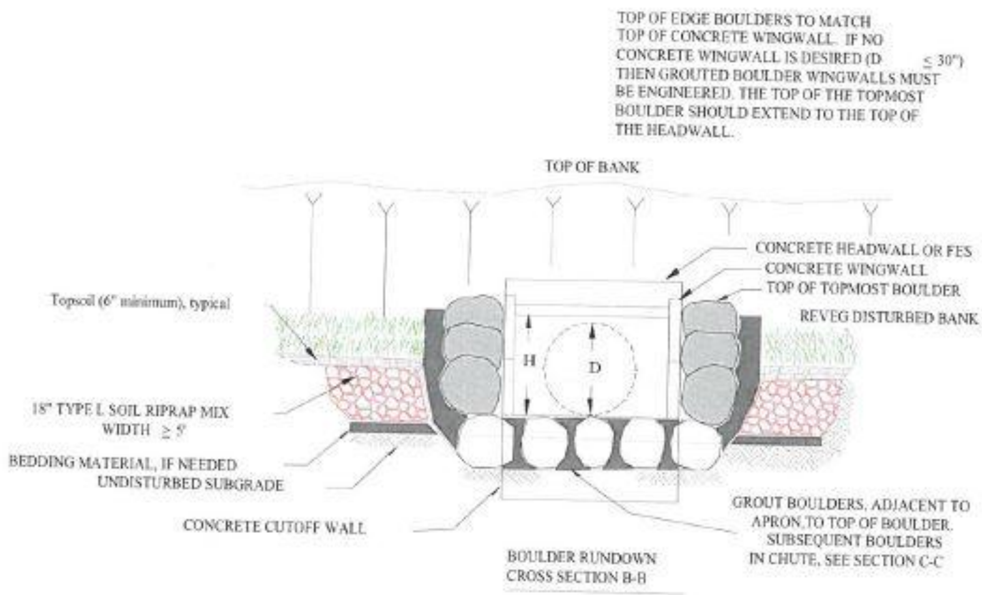


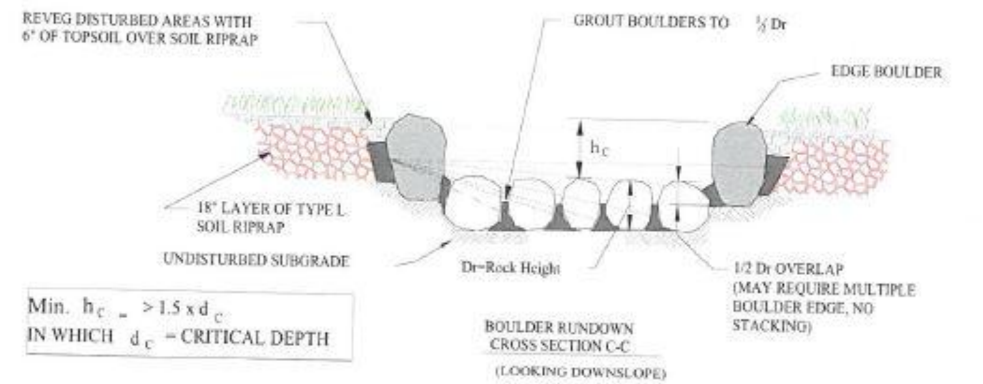
CROSS SECTION A-A

Sheet 1 of 2

Figure HS-18—Grouted Boulder Rundown

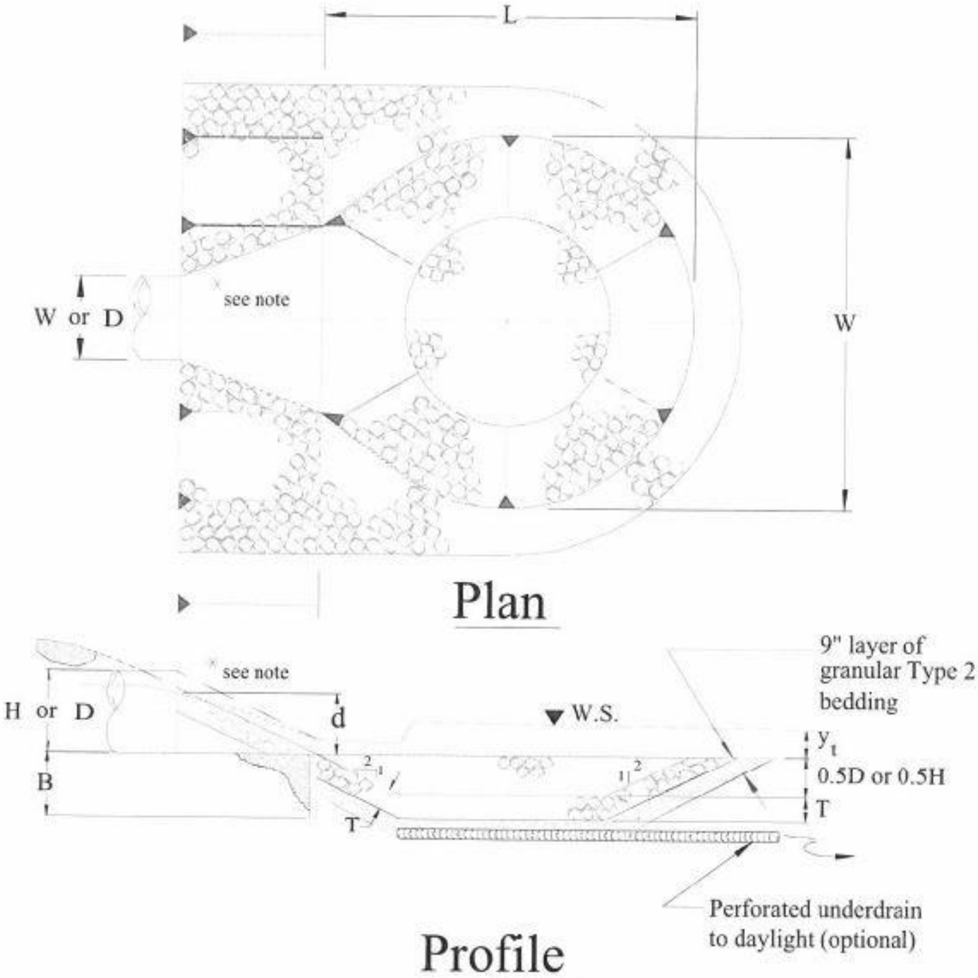
(Sheet 1 of 2)





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)

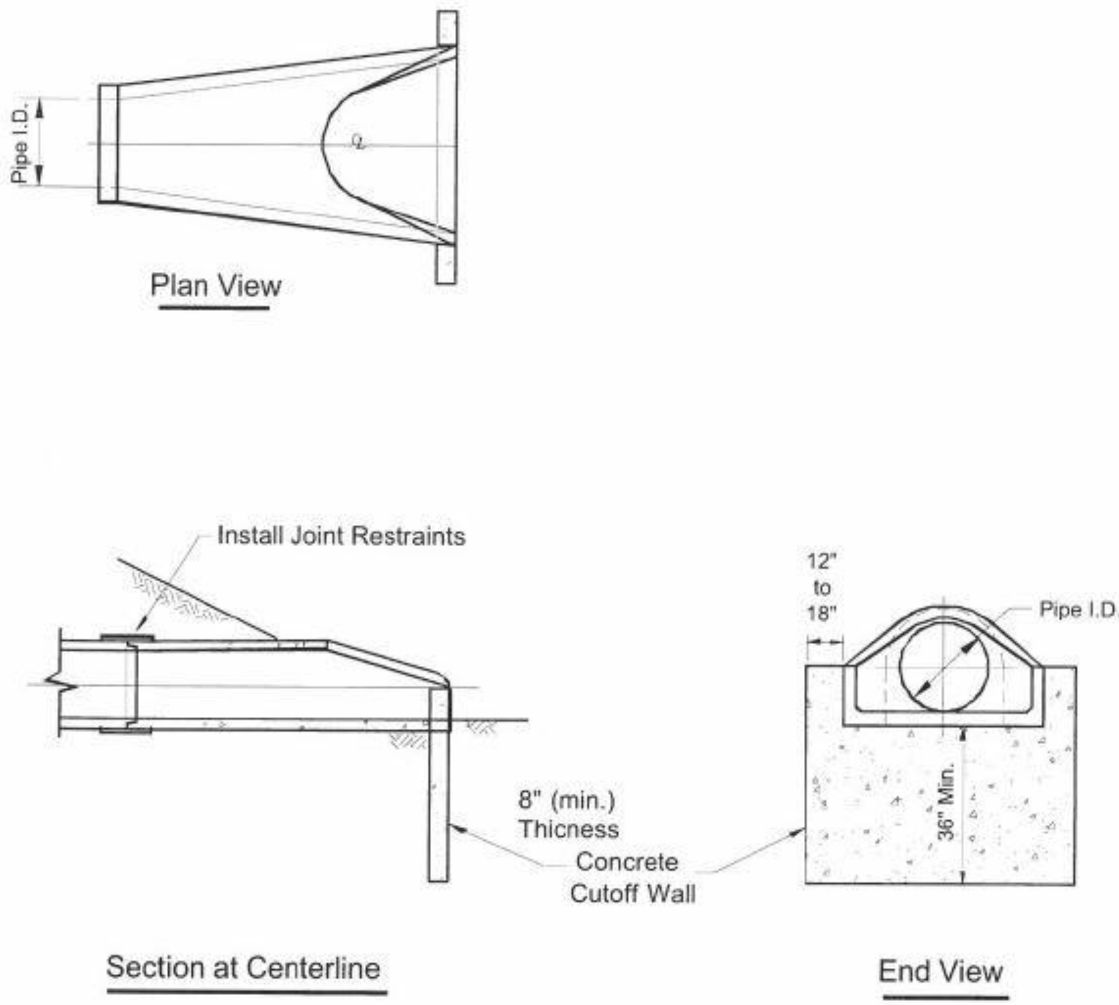


Figure HS-19a—Concrete Flared End Section with Cutoff Wall for all Pipe Outlets

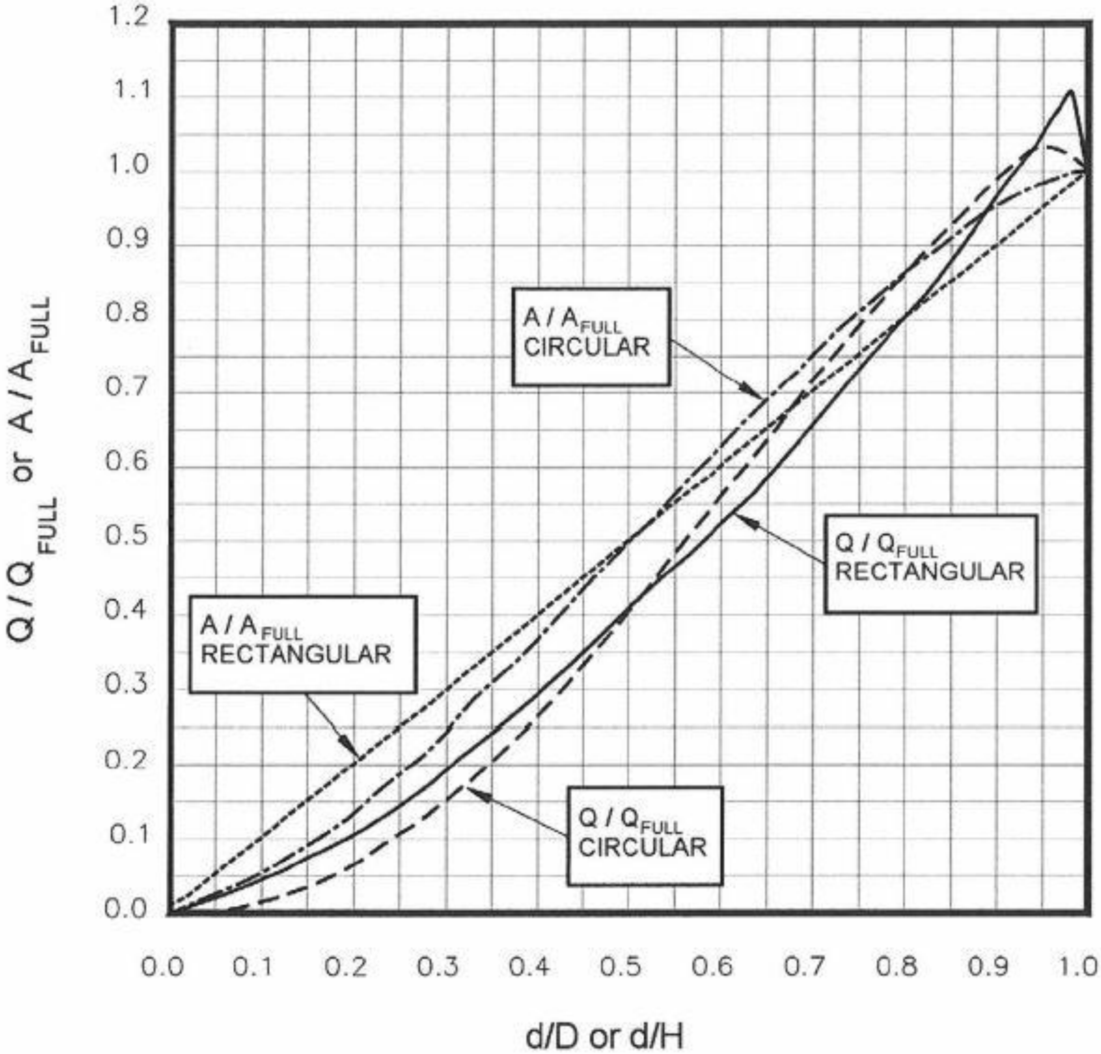


Figure HS-20a—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—Discharge and Flow Area Relationships for Circular and Rectangular Pipes
(Ratios for Flow Based on Manning's *n* Varying With Depth)
(Stevens and Urbonas 1996)

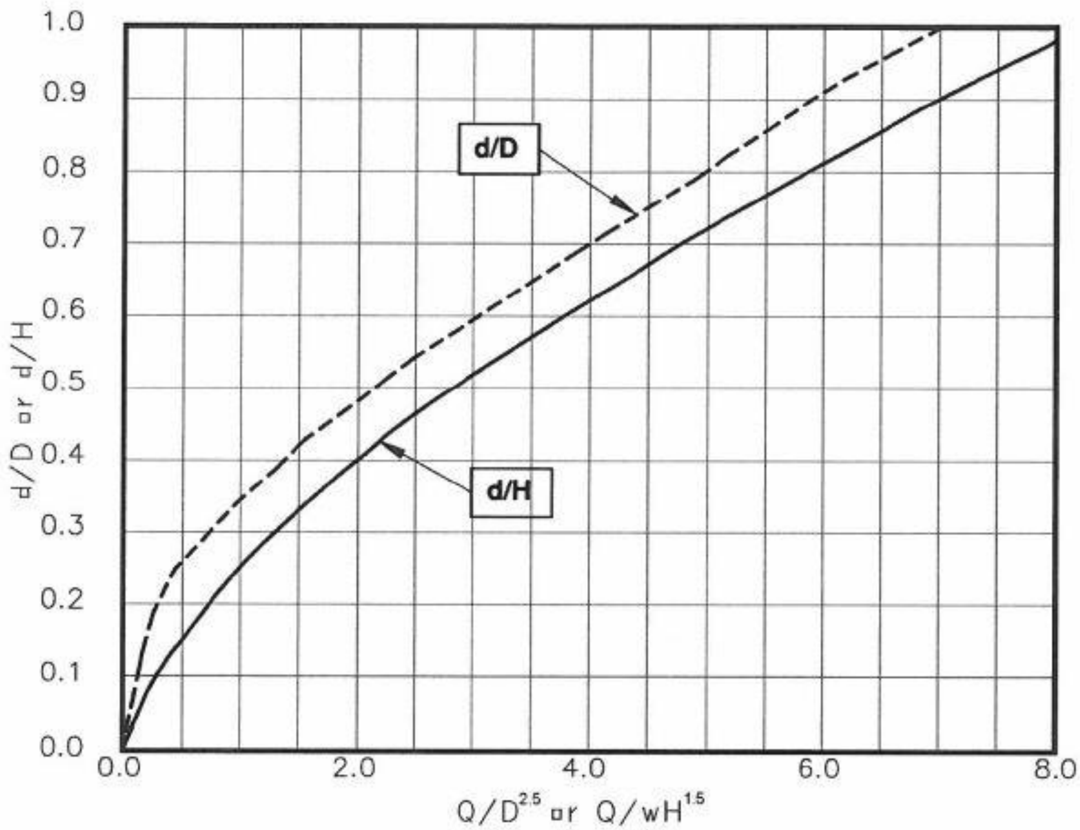


Figure HS-20b—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—
Brink Depth for Horizontal Pipe Outlets
(Stevens and Urbonas 1996)

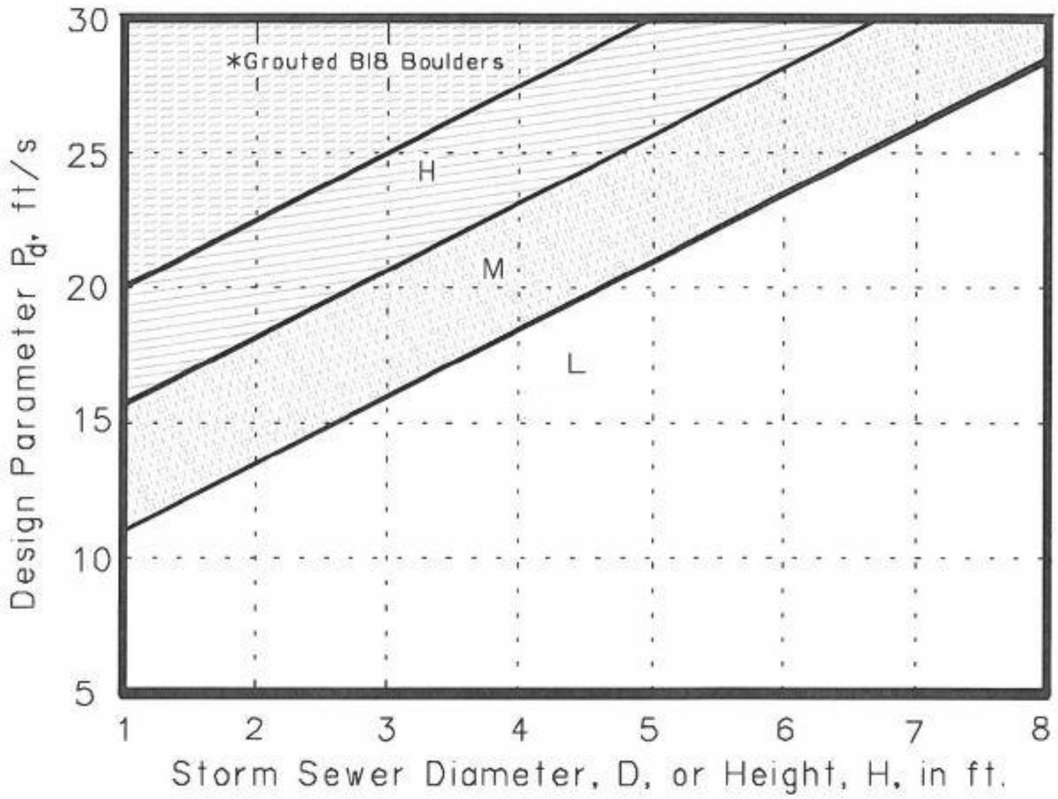


Figure HS-20c—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets—

Riprap Selection Chart for Low Tailwater Basin at Pipe Outlet

(Stevens and Urbonas 1996)

22.9.7.3 Public Facilities Additional Erosion Protection Criteria

The facility is considered Public if it maintained publicly or has a Public Drainage Easement upon it.

a. A filter fabric or gravel is to be used in all cases under the riprap.

b. The velocity leaving the energy dissipator/ erosion protection shall be 5 ft/sec or less unless justified.

22.9.7.4 Private Storm Drain Outlets to Onsite Basins or Swales

1. The following criteria is acceptable for privately maintained facilities where the storm drain is less than 18 inches in diameter. For private storm drains 18 inches and greater, refer to the sections for public facilities.
2. Criteria:
   a. Provide erosion control for velocities 5 ft/sec or greater.
   b. The pipe invert should be at or close to the invert of the receiving basin or swale.

22.9.8 Protection and Debris Barriers

a. Protection Barriers

A protection barrier is a means of preventing people from entering storm drains. Protection barriers will be provided wherever necessary to prevent unauthorized access to storm drains. In some cases the barrier may be one of the breakaway type. In other cases the barrier may be a special design. It will be the designer's responsibility to provide a protection barrier appropriate to each situation and to provide details of such on the construction drawings.

b. Debris Barriers

A debris barrier or deflector is a means of preventing large debris or trash, such as tree limbs, logs, boulders, weeds, and refuse, from entering a storm drain and possibly plugging the conduit. The debris barrier should have openings wide enough to allow as much small debris as possible to pass through and yet narrow enough to protect the smallest conduit in the system downstream of the barrier.

One type that has been used effectively in the past is the debris rack. This type of debris barrier is usually formed by a line of posts, such as steel pipe filled with concrete or steel rails, across the line of flow to the inlet. It will be the designer's responsibility to provide a debris barrier or deflector appropriate to the situation and acceptable to the City Engineer.

c. Debris Basins

Debris basins, check dams and similar structures are a means of preventing mud, boulders and debris held in suspension and carried along by storm runoff from depositing in storm drains. Debris basins constructed upstream of storm drain conduits, usually in arroyos, trap such material before it reaches the conduit. Debris basins must be cleaned out on a regular basis, however, if they are to continue to function effectively. Refer to the City Engineer and State Engineer regarding the criteria to be used in designing these structures.

22.9.9 Closed Conduit Angle of Confluence

Connector pipe may be joined to main line pipe at angles greater than 45 degrees up to a maximum of 90 degrees provided none of the above conditions exist. Under high velocity and high flow conditions it is preferable for the angle of confluence to be 45 degrees or less.

In general, the angle of confluence between main line and lateral must not exceed 45 degrees and, as an additional requirement, must not exceed 30 degrees under any of the following conditions:

(1) Where the peak flow (Q) in the proposed lateral exceeds 10 percent of the main line peak flow.

(2) Where the velocity of the peak flow in the proposed lateral is 20 f.p.s. or greater.

(3) Where the size of the proposed lateral is 60 inches or greater.

(4) Where hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence.

If, in any specific situation, one or more of the above conditions does apply, the angle of confluence for connector pipes may not exceed 30 degrees. Connections must not be made to main line pipe which may create conditions of adverse flow in the connector pipes without prior approval from the City Engineer.

The above requirements may be waived only if calculations are submitted to the City Engineer showing that the use of a confluence angle larger than 30 degrees will not unduly increase head losses in the main line.

22.9.10 Flapgates

A flapgate must be installed in all laterals outletting into a main line storm drain whenever the potential water surface level of the main line is higher than the surrounding area drained by the lateral.

The flapgate must be set back from the main line drain so that it will open freely and not interfere with the main line flow. A junction structure will be constructed for this purpose in accordance with City Engineer standards.

22.9.11 Rubber-Gasketed Pipe

Rubber-gasketed pipe will be used in all storm drain construction unless otherwise approved by the City Engineer.

22.9.12 Junctions into Existing Storm Drain Main Lines

Junctions will only be permitted on mains storm drain lines that are ≥42 inches. Junction locations cannot be more that 24' from the downstream manhole. The maximum lateral size is 24”. The City Engineer's approval will be required for variances.

22.9.13 Submittal Requirements

22.9.13.1 Hydraulic Model

a. If a Letter of Map Change is to be submitted to FEMA, the model is to be on the approved FEMA models list at the time of submittal. Approved models are shown on FEMA's website.

b. Electronic hydraulic models must meet the following criteria to be accepted:

1. Be able to produce an illustration of the HGL and EGL.
2. Have the ability to include major and minor losses.
3. Meet technical requirements of this chapter.
4. The engineer shall include a description of how the model meets the requirements of this chapter and should describe how losses were taken into account.

c. For the purposes of generating an infrastructure list, in lieu of submitting the results of an electronic model, the engineer may submit pipe capacity calculations based on gravity flow using Manning's equation.

d. An electronic model is required to design the storm drain for the construction plans.

22.9.13.2 Culverts

a. The City has adopted the Federal Highway Administration, Hydraulic Design Series Number 5, method for culvert design.

b. If a proprietary model is used to design a culvert, the engineer shall include a description of how the model is in compliance with the FHWA method.