CHAPTER 20 HYDRAULIC STRUCTURES

20.1 INTRODUCTION

Hydraulic structures are used to positively control water flow velocities, directions and depths as well as to sustain the stream bed elevation and slope and the general configuration of a waterway characteristics.

Many of these structures are special and costly, where their selection requires careful and thorough hydraulic engineering judgement. Proper application of hydraulic structures can reduce capital and maintenance costs by changing the characteristics of the flow and by reducing the size of related facilities to fit the needs of a particular project.

The shape, size, and other features of a hydraulic structure can vary widely for different projects, depending upon the functions to be accomplished. Hydraulic design procedures govern the final geometry/shape of all structures. This may include model testing when a proposed design requires a configuration that differs significantly from documented practices.

20.2 EROSION AND SCOUR PROTECTION

20.2.1 Description

When the flow velocity at a conduit outlet (Figure 20.1) exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required at the outlet and, possibly the inlet. This protection usually consists of an erosion resistant reach, such as riprap, located between the outlet and the downstream channel. The design of such protection is normally based on design ARIs from minor and major system application. Lower tailwater conditions during smaller events can create more stressful conditions at the outlet and need to be checked for. When protection is needed at the outlet, one option is to provide a horizontal (zero slope) apron.

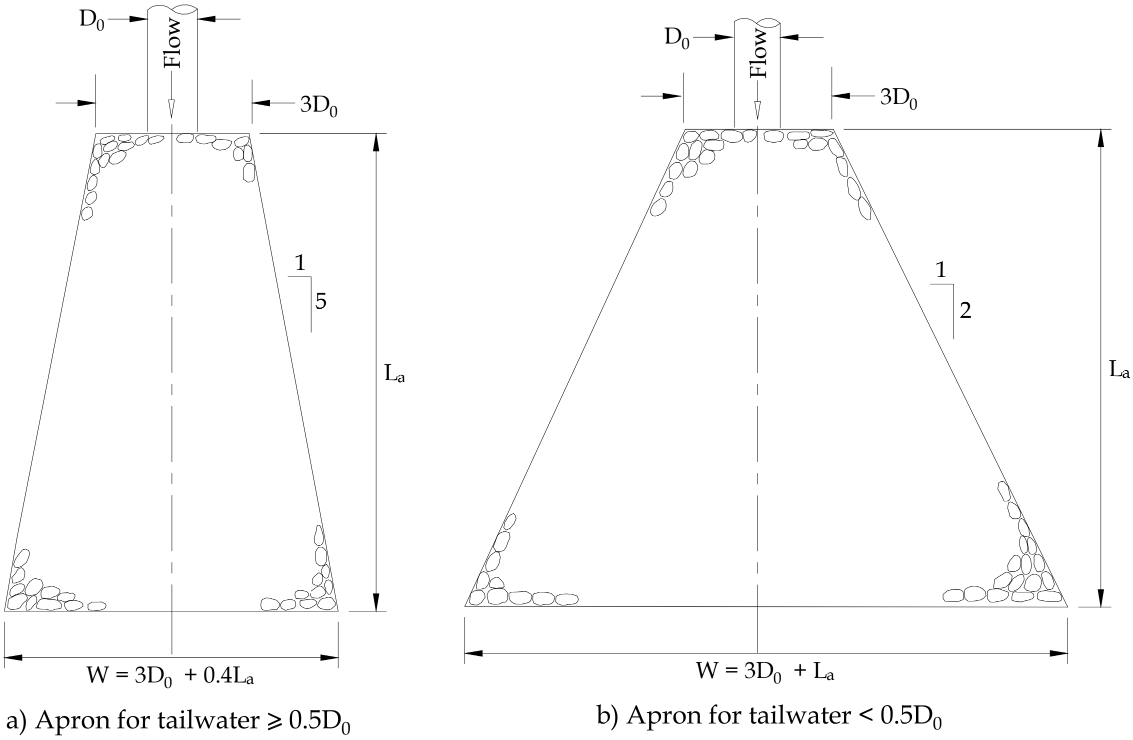


Figure 20.1: Configuration of Conduit Outlet Protection (U.S. EPA, 1976)

20.2.2 Design Consideration

The length of an apron (L_a) is determined using the following empirical relationships (USEPA, 1976):-

$$L_a = \frac{3.26\ Q}{D_o^{3/2}} + 7D_o \qquad \text{for } TW < D_o/2 \tag{20.1}$$

and

$$L_a = \frac{5.44\ Q}{D_o^{3/2}} + 7D_o \qquad \text{for } TW > D_o/2 \tag{20.2}$$

where,

TW	=	Tailwater depth (m);
D_o	=	Maximum inside culvert diameter (m); and
Q	=	Pipe discharge (m^3/s);

Where there is no well defined channel downstream of the apron, the width, W, of the outlet and of the apron (as shown in Figure 20.1) should be as follows:-

$$W = 3D_o + 0.4L_a \qquad \text{for } TW \geq D_o/2 \tag{20.3}$$

and

$$W = 3D_o + L_a \qquad \text{for } TW < D_o/2 \tag{20.4}$$

The following criteria apply in apron design:-

- The width of the apron at the culvert outlet should be at least 3 times the culvert width.
- Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least 300 mm above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert;
- The side slopes should be 2:1 or flatter;
- The bottom slope should be level; and
- There should be an overfall at the end of the apron or culvert.

The apron material of median stone, diameter d_{50}, is determined from the following equation:

$$d_{50} = \frac{0.066(Q)^{4/3}}{TW(D_o)} \tag{20.5}$$

Existing or pre-shaped scour holes may be used where flat aprons are impractical. Figure 20.2 shows the general design of such a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.041(Q)^{4/3}}{TW(D_o)} \qquad \text{for } y = D_o/2 \tag{20.6}$$

also

$$d_{50} = \frac{0.027(Q)^{4/3}}{TW(D_o)} \qquad \text{for } y = D_o \tag{20.7}$$

where,

y = Depth of scour hole below culvert invert (m).

Aprons constructed of man-made materials are often a viable alternative. Designer should refer to the man-made materials for design consideration.

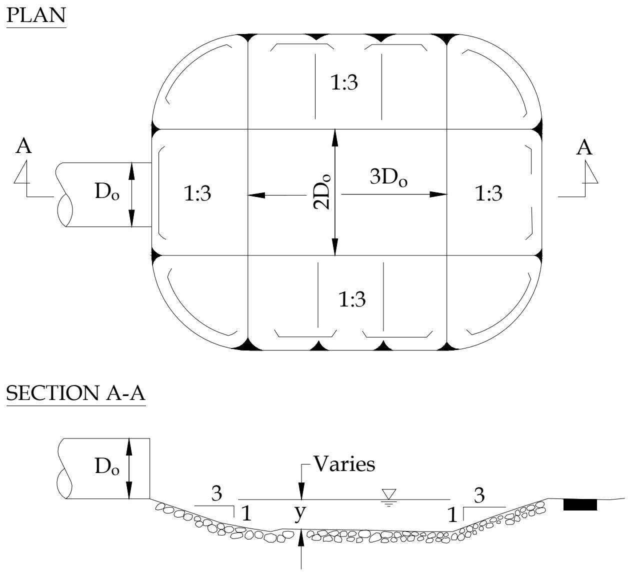


Figure 20.2: Preformed Scour Hole (ASCE, 1975)

20.3 ENERGY DISSIPATORS

20.3.1 Description

Energy dissipators are required in the immediate vicinity of hydraulic structures where high impact loads, erosive forces, and severe scour are expected. In other words, they are usually required where the flow regime changes from supercritical to subcritical, or where the flow is supercritical and the tractive forces or flow velocities are higher than the maximum allowable values. The basic hydraulic parameter that identifies the flow regime, and is used in connection with energy dissipators in general, and with hydraulic jump dissipators in particular, is the Froude number. The Froude number is a ratio of the flow velocity and wave celerity.

Energy dissipation structures act as transitions, which reduce high flow velocities that may exist under a range of flows. Energy dissipators localise hydraulic jumps and act as stilling basins. The use of energy dissipators is very common downstream of hydraulic structures where common channel protection cannot be used alone because of potential damage. If riprap or other protection is used for energy dissipation, it should be confined in a basin and secured in place with grout or mesh.

20.3.2 Riprap Basins

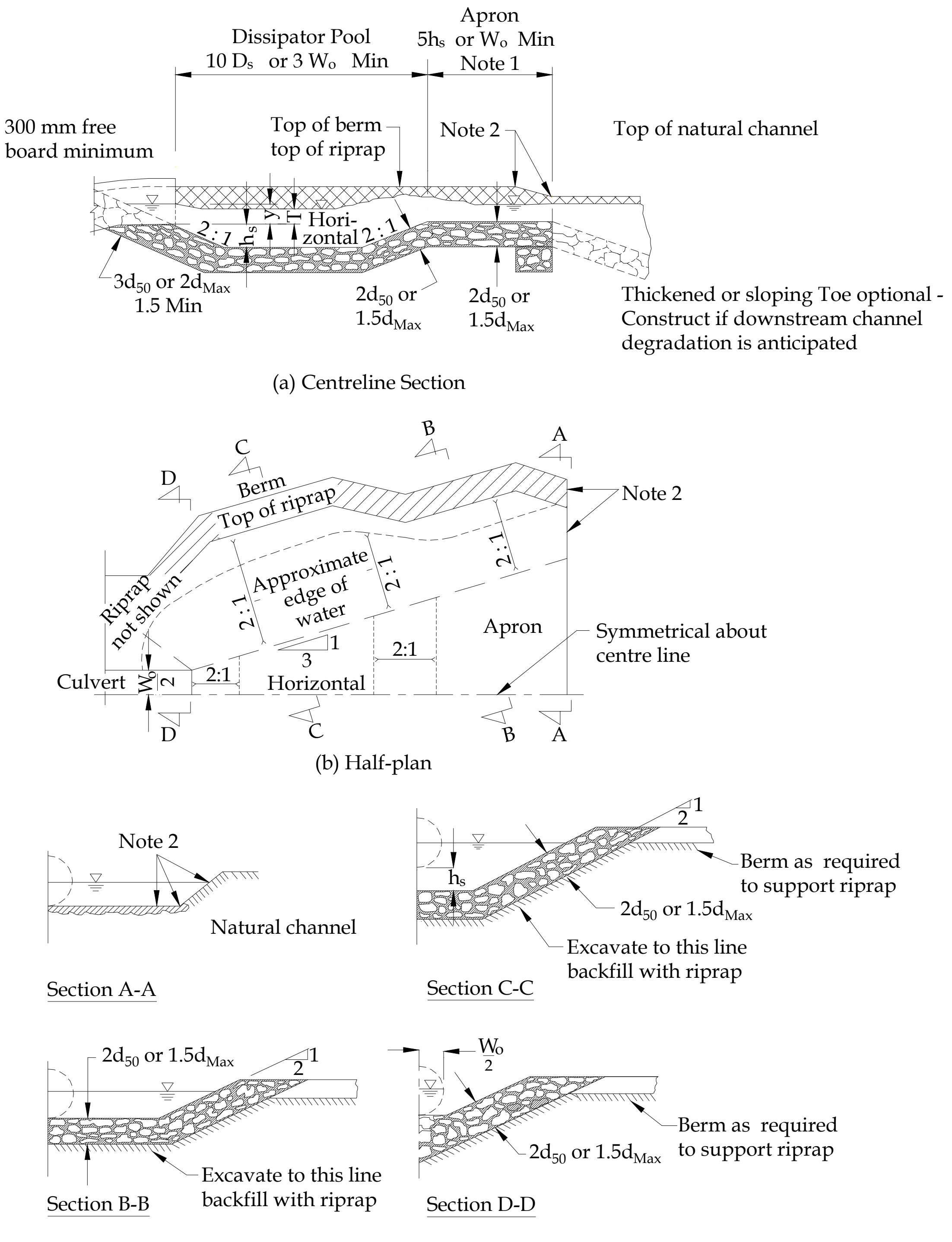
The most commonly used energy dissipators is riprap basins (Figure 20.3). The riprap placed in the basin must be inspected and repaired, if necessary, after major storms. The median stone diameter can be estimated based on the exit velocity of the pipe or culvert. The length of the basin is estimated based on the width or diameter of the conduit. The depth of the basin is based on the median stone diameter.

20.3.3 Stilling Basins

If a hydraulic jump is used for energy dissipation, it should be confined to a heavily-armoured channel reach, the bottom of which is protected by a solid surface such as concrete to resist scouring.

20.3.3.1 Design Considerations

There are several considerations that should be included in designing hydraulic jumps and stilling basins (Chow, 1959; US DOT, 1983):



Notes:

1. If a maximum allowable exit velocity, V_e ,from the basin is specified, extend the basin as required to obtain sufficient cross-sectional area at Section A-A (i.e. $A_{A\text{-}A} = Q/V_e$) for the specified velocity;
2. Warp the basin to conform to the natural stream channel. The top of the riprap in the basin floor should be at the same elevation or lower than the natural channel bottom at Section A-A.

Figure 20.3: Typical Riprap Basin: (a) Centreline Section (b) Half-plan and (c) Sections (US FHWA, 1983).

- Jump Position: there are three positions or alternative patterns that allow a hydraulic jump to form downstream of the transition in the channel. These positions are controlled by tailwater.
- Tailwater Conditions: tailwater fluctuations due to changes in discharge complicate the design procedure. They should be taken into account by classification of tailwater conditions using tailwater and hydraulic jump rating curves; and
- Jump Types: various types of hydraulic jumps that may occur are summarised in Figure 20.4. Oscillating jumps in a Froude number range of 2.5 to 4.5 are best avoided unless specially designed wave suppressers are used to reduce wave impact.

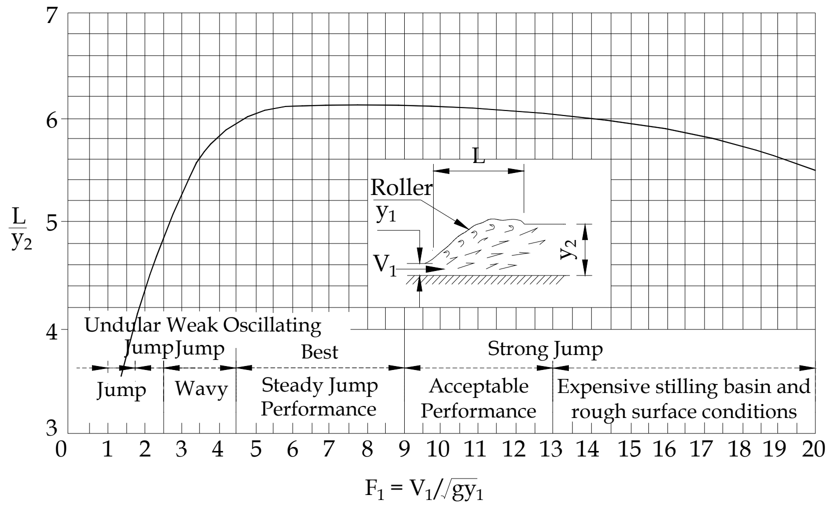


Figure 20.4: Lengths and Types of Hydraulic Jumps in Horizontal Channels (Bradley and Peterka, 1957; Chow, 1959)

As Froude number increases, the tailwater effect becomes more significant. Therefore, for a Froude number as low as 8, the tailwater depth should be greater than the sequent depth downstream of the jump so that the jump will stay on the apron. When the Froude number is greater than 10, the common stilling basin dissipator may not be as cost-effective as a special bucket type dissipator will be required (Peterka, 1958).

20.3.3.2 Controls of Jumps

Jumps can be controlled by several types of appurtenances such as sills, chute blocks and baffle piers. The purpose of a sill located at the end of a stilling basin is to induce jump formation and to control its position under most probable operating conditions. Sharp crested or broad crested weirs can be used to stabilise and control the jump. Chute blocks are used at the entrance to the stilling basin. Baffle piers are blocks placed in intermediate positions across the basin floor for dissipating energy mostly by direct impact action.

20.3.3.3 Stilling Basin Categories

The following three major categories of basins are used for a range of hydraulic conditions. Design details can be found in the AASHTO Drainage Handbook (1987), Chow (1959), and US DOT (1983).

- The SAF ("St. Anthony Falls" Stilling Basin): This basin, shown in Figure 20.5, is recommended for use on small structures such as spillways and outlet works where the Froude number varies between 1.7 and 17. The appurtenances used for this dissipator can reduce the length of the basin by approximately 80%. This design has great potential in urban stormwater systems because of its applicability to small structures. Stilling Basin III developed by the US Bureau of Reclamation (UBSR) is similar to the SAF basin, but it has a higher factor of safety.
- The UBSR Stilling Basin II: This basin, shown in Figure 20.6, is recommended for controlling jumps with Froude numbers greater than 4.5 at large spillways and channels. This basin may reduce the length of the jump by a third and is used for high-dam and earth-dam spillways. Appurtenances used in this basin

include chute blocks at the upstream end of the basin and a dentated sill at the downstream end. No baffle piers are used in this basin because of the cavitation potential.

- The UBSR Stilling Basin IV: This basin, shown in Figure 20.7 is used where jumps are imperfect or where oscillating waves occur with Froude numbers between 2.5 to 4.5. This design reduces excessive waves by eliminating the wave at its source through deflection of directional jets using chute blocks. When a horizontal stilling basin is constructed without appurtenances, the length of the basin is made equal to the length of the jump.

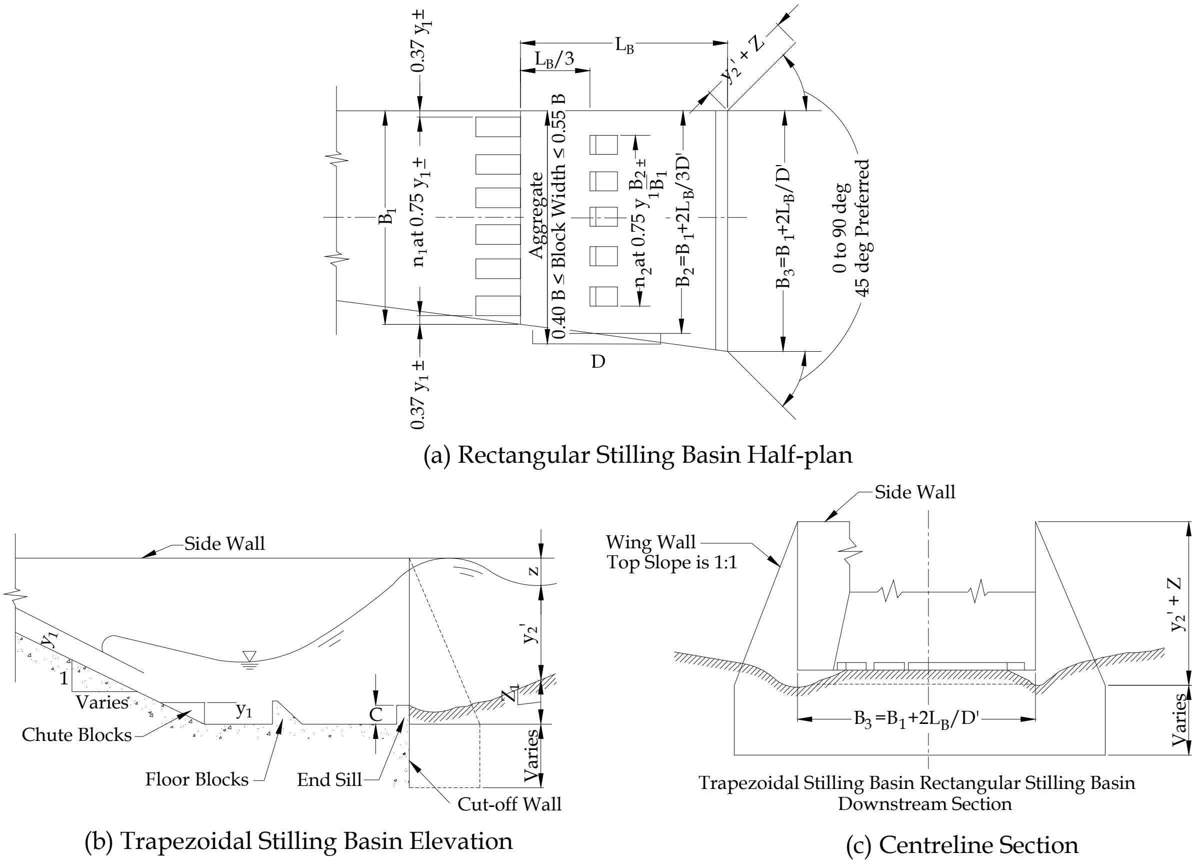


Figure 20.5: Proportions of the SAF Basin (Chow, 1959)

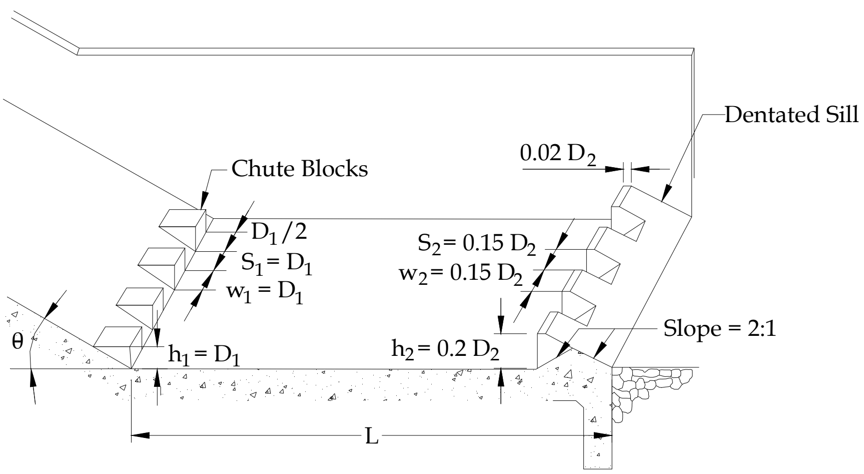


Figure 20.6: Proportions of the USBR Basin II (Chow, 1959)

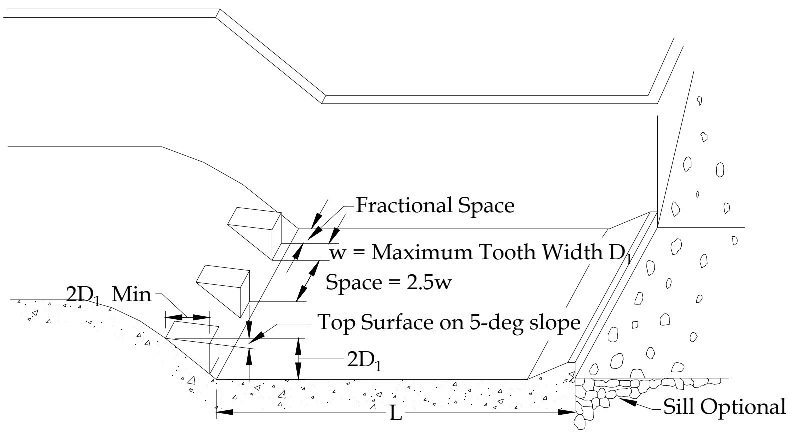


Figure 20.7: Proportions of the USBR Basin IV (Chow, 1959)

20.3.4 Energy-Dissipating Headwalls

Another simple type of energy dissipators that can be used at culvert outlets is an energy dissipating headwall. Three typical headwalls are shown in Figures 20.8 to 20.10.

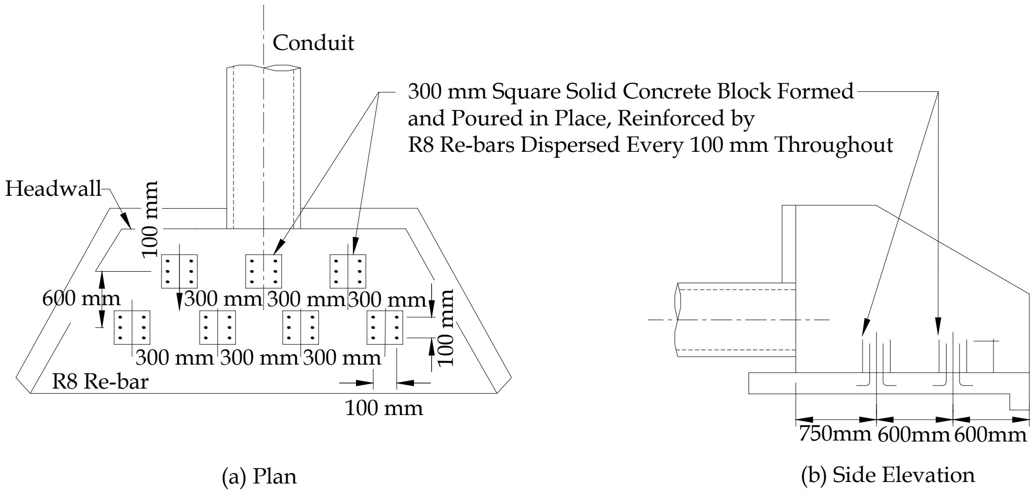


Figure 20.8: Standard Energy Dissipating Headwall, Type I (Chow, 1959)

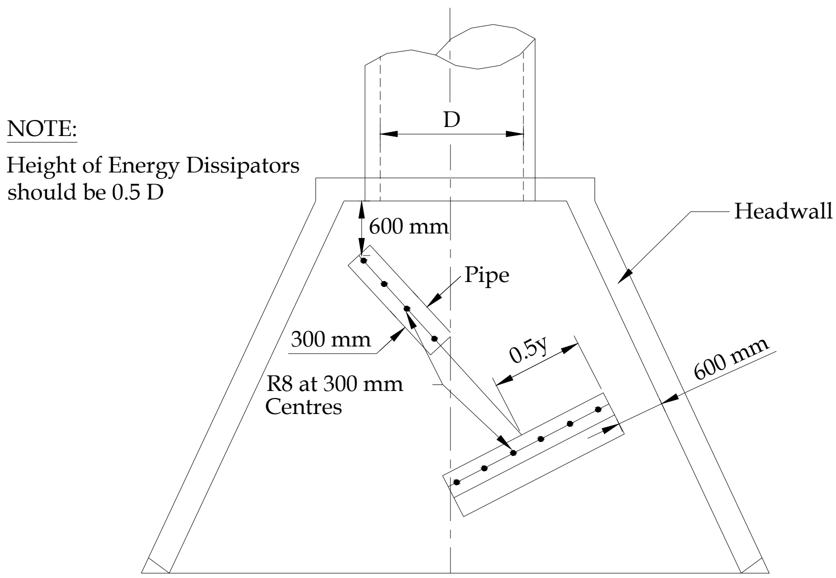


Figure 20.9: Standard Energy Dissipating Headwall, Type II (ASCE, 1992)

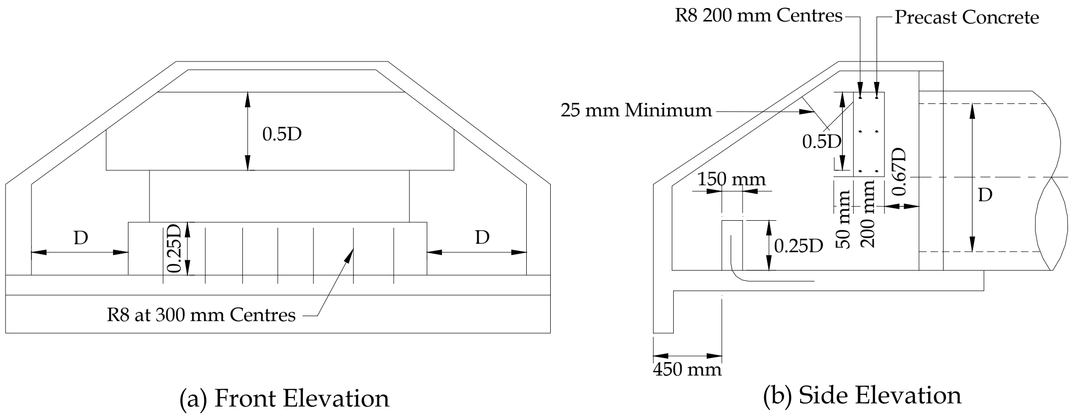


Figure 20.10: Standard Energy Dissipating Headwall, Type III (ASCE, 1992)

20.3.5 Design Criteria

The design criteria for energy dissipators are as follows:

- Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs;
- Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system;
- Energy dissipator designs will vary based on discharge characteristics and tailwater conditions. Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence;
- Energy dissipators should be designed to return flows to non-erosive velocities to protect downstream channels; and
- Care must be taken during construction that design criteria are followed exactly. Each part of the criteria is important to the proper function.

Table 20.1 provides a summary of selected parameters, and may be used for preliminary identification of alternative types of energy dissipators.

Table 20.1: Dissipator Criteria (U.S. Department of Transportation, 1983)

Dissipator Type	Froude Number, Fr	Allowable Debris			Tailwater, TW	Special Considerations
		Silt and Stand	Boulders	Floating		
Free Hydraulic Jump	> 1	H	H	H	Required	
CSU Rigid Boundary	< 3	M	L	M	–	
Tumbling Flow	> 1	M	L	L	–	4% < S_o < 25%
Increased Resistance	–	M	L	L	–	Check Outlet Control HW
USBR Type II	4 to 14	M	L	M	Required	
USBR Type III	4.5 to 17	M	L	M	Required	
UBSR Type IV	2.5 to 4.5	M	L	M	Required	
SAF	1.7 to 17	M	L	M	Required	
Contra Cost	< 3	H	M	M	< 0.5D	
Hook	1.8 to 3	H	M	M	–	
USBR Type VI	–	M	L	L	Desirable	Q < 11 m^3/s, V < 15 m/s
Forest Service	–	M	L	L	Desirable	y < 900 mm
Drop Structure	< 1	H	L	M	Required	Drop < 5 m
Manifold	–	M	N	N	Desirable	
Corps Stilling Well	–	M	L	N	Desirable	
Riprap	< 3	H	H	H	–	

Note: N = None; L = Low; M = Moderate; H = Heavy

20.4 DROP STRUCTURES

20.4.1 Description

Vertical drop structures are controlled transitions for energy dissipation in steep channels where riprap or other energy dissipation structures are not as cost effective. Drop structures used for stormwater drainage can be categorised primarily as either open channel transitions (drop spillways) or transitions between open channels and closed conduits (drop shafts).

Drop structures should be constructed from concrete because of the forces involved; however, riprap or gabion stilling basins may be used where physical, economic, and other constrains arise.

Drop structures in open channels change the channel slope from steep to mild by combining a series of gentle slopes and vertical drops. Flow velocities are reduced to non-erosive velocities, while the kinetic energy or flow velocity gained by the water as it drops over the crest of each spillway is dissipated by an apron or stilling basin.

Open channel drop structures generally requires aerated nappe and subcritical flow conditions at both the upstream and downstream sections of the drop. The stilling basin can vary from a simple concrete apron to baffle blocks or sills. Figure 20.11 shows the flow geometry and important variables at a vertical (straight) drop structure. The flow geometry at such drops can be described by the drop number, D_N, which is defined (Chow, 1959) as:

$$D_N = \frac{q^2}{gh^3} \tag{20.8}$$

where,

q = Discharge per unit width of crest overfall ($m^3/s/m$);
g = Acceleration due to gravity (9.81 m/s^2); and
h = Height of drop (m);

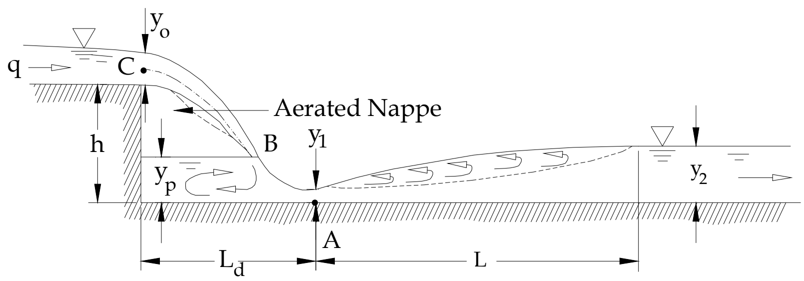


Figure 20.11: Flow Geometry of a Straight Drop Spillway (Chow, 1959)

The drop functions are:

$$\frac{L_d}{h} = 4.30{D_N}^{0.27} \tag{20.9a}$$

$$\frac{y_p}{h} = 1.0{D_N}^{0.22} \tag{20.9b}$$

$$\frac{y_1}{h} = 0.54{D_N}^{0.425} \tag{20.9c}$$

$$\frac{y_2}{h} = 1.66 D_N^{\ 0.425} \tag{20.9d}$$

where,

L_d = drop length (m);
y_1 = the depth of the toe of nappe (m);
y_p = pool depth under the nappe (m); and
y_2 = tailwater depth sequent to y_1 (m).

For a given drop height, h, discharge, q, drop length, L_d, sequent depth, y_2 and can be estimated by Equation 20.9(a) and 20.9(d), respectively. The length of the jump can be estimated by techniques discussed in Section 20.3. If the tailwater is less than y_2, the hydraulic jump will recede downstream. Conversely, if the tailwater is greater than y_2, the jump will be submerged. If the tailwater is equal to y_2, no supercritical flow exists on the apron and the distance L_d is minimum.

When the tailwater depth is less than y_2, it is necessary (according to the US Department of Transportation, 1983) to provide either;

- An apron at the bed level and a sill or baffles; and
- An apron below the downstream bed level and an end sill.

The choice of drop structure type and dimensions depends on the unit discharge, q, drop height, h, and tailwater depth, TW. The design should take into consideration the geometry of the undisturbed flow. If the spillway (overflow crest) length is less than the width of the approach channel, the approach channel must be designed properly to reduce the effect of the end contractions to avoid scour. The two most common vertical open channel drops are the straight drop structure and the box inlet drop structure.

(a) Straight Drop Structure

Figure 20.12 shows the layout of a typical straight drop structure and hydraulic design criteria developed by US Soil Conservation Service. McLaughlin Water Engineers (1983) provides specific criteria and reviews design considerations related to the hydraulic, geotechnical, and structural design of drop structures.

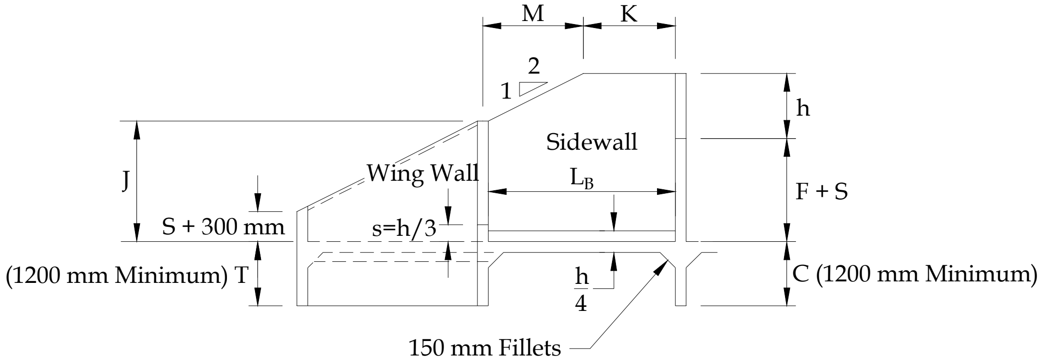
(b) Box Inlet Drop Structure

The box inlet drop structure is a rectangular box with openings at the top and downstream end as shown in Figure 20.13. Water is directed to the crest of the box inlet by earth dikes and a headwall. Flow enters over the upstream end and two sides. The long crest of the box inlet permits large flows to pass at relatively low heads. The width of the structure should not be greater than the downstream channel. Box inlet drop structures are applicable to drops from 0.6 to 3.6 m.

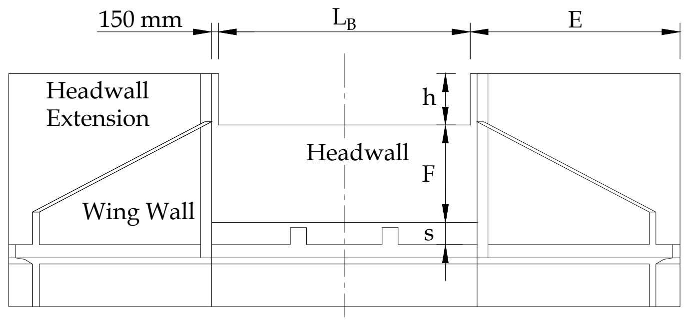
20.4.2 Design Criteria

Design criteria for these structures, based on US Soil Conservation Services and St. Anthony Falls Hydraulic Laboratory, are available in US Department of Transportation (1983) and Blaisdell and Donnely (1956). The parameters to be considered for the hydraulic design of the drops are,

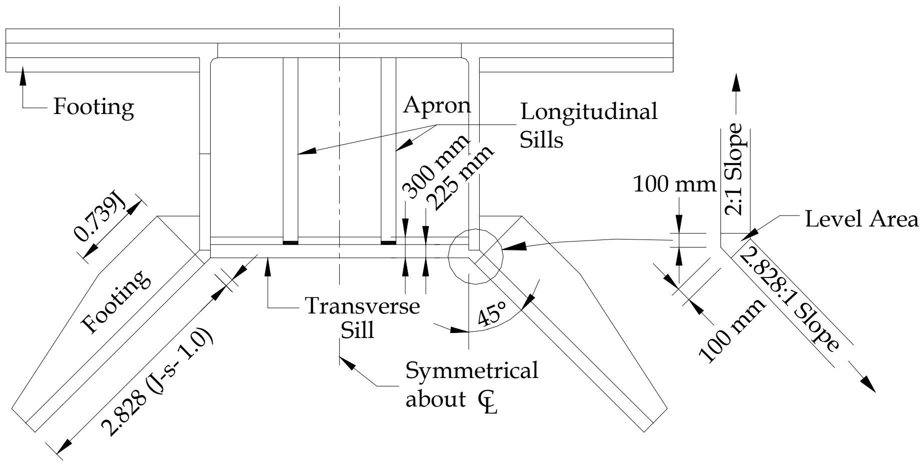
- Section (length) of the crest of the box inlet;
- Opening of the headwalls;
- Discharge, discharge coefficients, and flow regime changes;
- Box inlet length and depth; and
- Minimum length and width of stilling basin.



(a) Section on Centreline



(b) Downstream Elevation



(c) Plan

Where :

E = Minimum length of headwall extension = [3h+0.61] or [1.5F] whichever is greater

J = Height of wing wall and sidewall at junction = [2h] or $\left[F+h+s-\left(\frac{L_B+0.13}{2}\right)\right]$ or [t+1] whichever is greater

L_B= Length of basin = $\left[F\left(2.28\ \frac{h}{F}+0.52\right)\right]$

M= [2 (F+ 1.3 h-J)]

K = [(L_B + 0.13)- M]

Figure 20.12: Typical Drop Spillway and Some Hydraulic Design Criteria (US SCS, 1954)

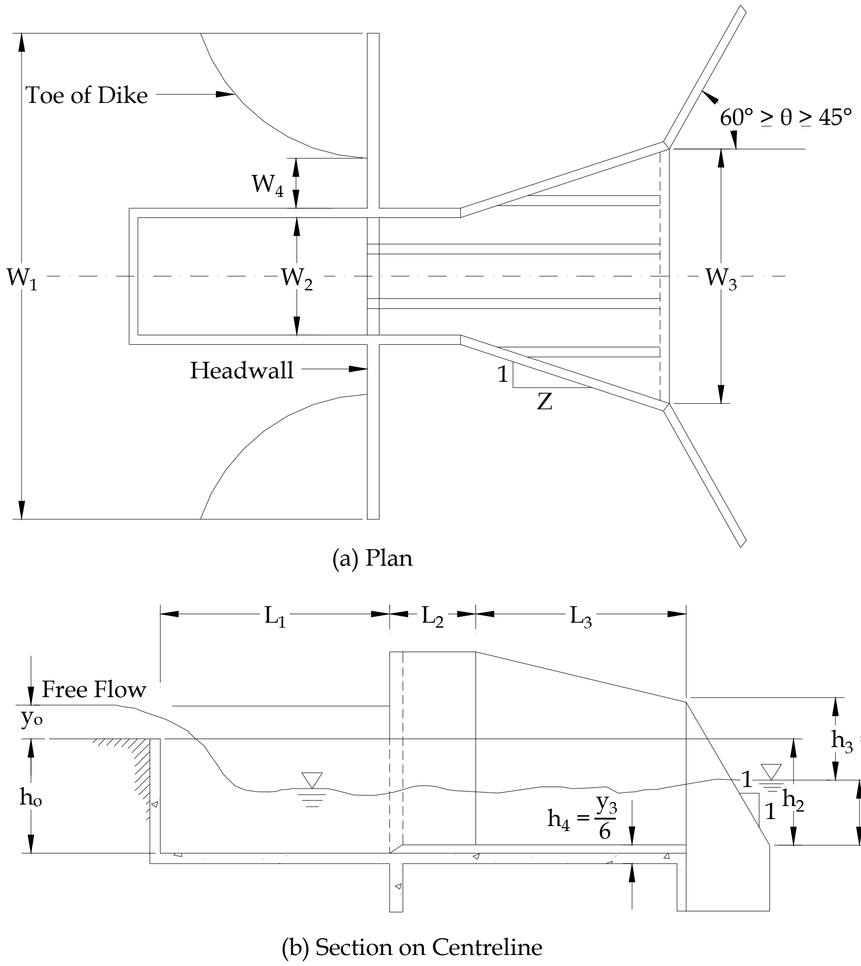


Figure 20.13: Box Inlet Drop Structure (US Dept. of Transportation, 1983)

20.5 OUTFALLS

20.5.1 Description

All stormwater drains of a locality have an outlet where flow from the local drainage system is discharged. The discharge point, or outfall, can be either a natural river or stream, or a stormwater drain or channel. The procedure for calculating the hydraulic grade line through a storm drainage system begins at the outfall. Therefore, consideration of the outfall conditions is an important part of stormwater drainage design.

20.5.2 Design Criteria

Most of the design criteria for stormwater drain outfalls are included:

- The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall;
- Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation;
- If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time;
- Energy dissipation is always needed to protect the storm drain outlet and the receiving natural or man-made channel. Riprap aprons or energy dissipators should be provided if high velocities are expected; and
- The outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction.

20.6 TRANSITION AND CONSTRICTIONS

20.6.1 Description

Channel transitions (Figure 20.14) 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 specialised high velocity conveyance. Constrictions can appreciably restrict and reduce the conveyance capacity in a manner which is either detrimental or beneficial. The purpose of this section is to briefly outline typical design procedures for transition and constriction structures that may be required for stormwater systems.

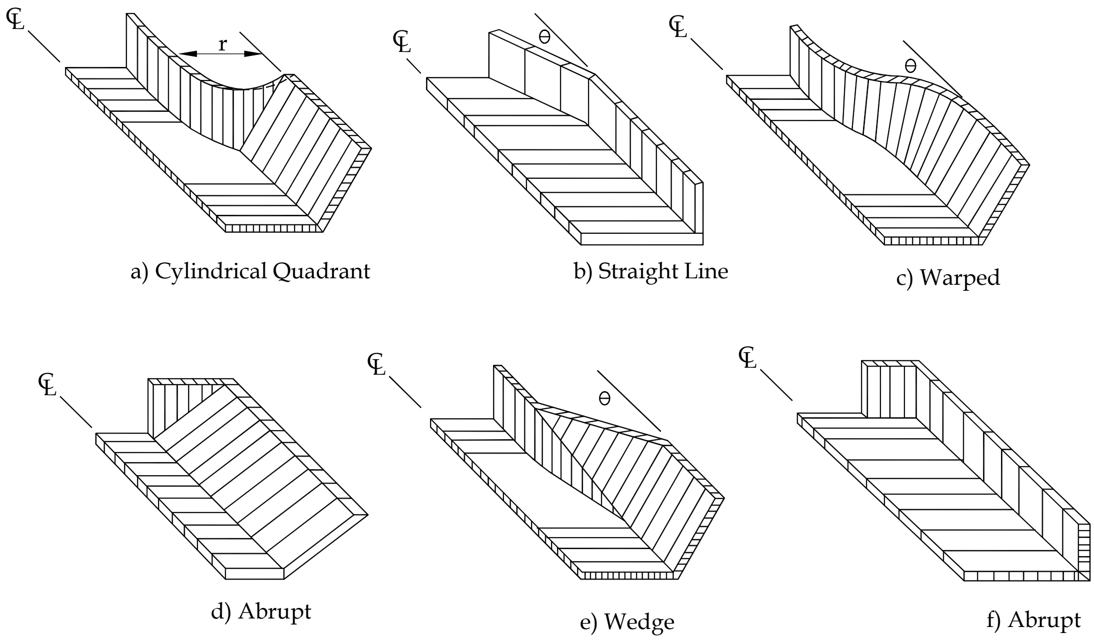


Figure 20.14: Transition Types

20.6.2 Transitions Analysis

(a) Subcritical Transitions

Transitions for subcritical flow frequently involve localised or bank lining configurations which 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 minimise friction losses and/or erosion tendencies.

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 20.14 illustrate some of these transitions.

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

(b) Supercritical Transition

The configuration of a supercritical transition is entirely different from subcritical transitions. Improperly designed and configured supercritical transitions can produce shock waves which result in channel overtopping and other hydraulic and structural problems.

20.6.3 Constrictions Analysis

(a) Constriction with Upstream Subcritical Flow

There are a variety of structures that are considered as 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.

The hydraulic distinction of constrictions is that they can cause rapidly varied flow. 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. Chow (1959) presents guidelines developed by the USGS for constrictions where the Froude number in the contracted section should not exceed 0.8. These cases are generally mild constrictions.

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 ensuring a high degree of accuracy is limited. It is advisable to use models tested or proven prototype layouts.

(b) Constriction with Upstream Supercritical Flow

Possible shock waves or choked flows causing high upstream backwater or a hydraulic jump are major concerns. The situation is to be avoided in urban drainage because of inherent instabilities.

20.7 BEND AND CONFLUENCES

20.7.1 Description

Channel confluences are commonly encountered in design. Flow rates can vary disproportionately with time so that high flows from upstream channel can discharge into 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.

20.7.2 Bends

(a) Subcritical Bends

Chow (1959), Rouse (1949) and others illustrate flow patterns, superelevation, and backwater or flow resistance characteristics of bends in detail. Superelevation refers to the rise in the water surface on the outer side of the bend. Effectively, the bend can behave like a contraction, causing backwater upstream and in 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 :

$$\Delta y = C\left[\frac{V^2 T}{gr}\right] \qquad 20.10$$

where,

C = Coefficient, generally 0.5 for subcritical flow;
V = Mean channel velocity (m/s);
T = Width of water surface in channel (m) ;
g = Acceleration of gravity (9.81 m/s); and
r = Channel centreline radius (m).

(b) Supercritical Bends

Supercritical channels are generally not desirable in urban drainage. However, special situations may occur where supercritical flows enters a curved channel, for example:

- At confluences where one channel is largely empty, and the entering flow expands and becomes supercritical;
- At a sharp bend in a conduit where slope inherently leads to supercritical conditions; and
- At a channel drop that unavoidably ends up on a curve.

The key 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, resulting in a rise in the 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 of an angular bend, a separation will occur, resulting in a negative shock wave or drop in the water surface. 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 allow the positive shock wave to intersect the negative wave the point where the later is propagated. A bend usually requires two deflections on the outside and one bend on the inside of the bend. A beneficial aspect of the shock wave is that it turns the flow in a predictable pattern, and thus the channel walls have no more force imposed on them other than that caused by the increased (or decreased) depths.

Other control techniques include very gradual bends, super elevated floors, and control sills, but these methods are generally less efficient.

20.7.3 Confluences

One of the most difficult problems to deal with is confluences where the difference in flow characteristics may be great. When entering the combined channel, the flow can diverge and drop in level if the flow capacity is suddenly increased. This can result in high velocities or unstable supercritical flow conditions with high erosion potentials. When significant sediment flows exist, aggradation can occur at the confluence, resulting in the loss of capacity in one or both upstream channels.

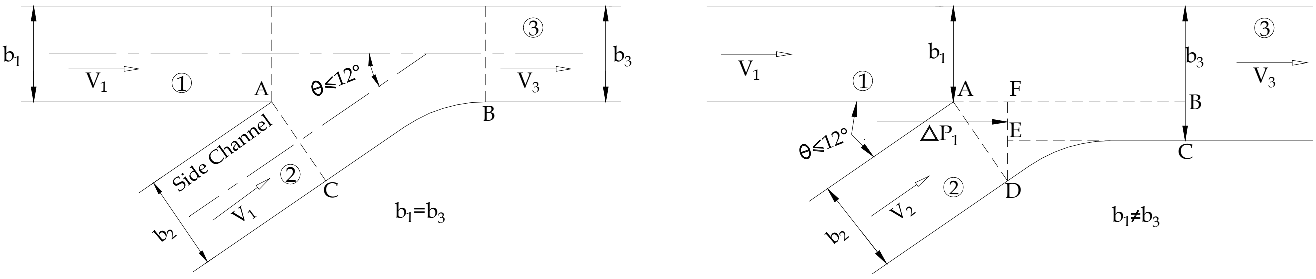
(a) Subcritical Flow Confluence Design

The design of channel junctions is complicated by many 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.

Figure 20.15 illustrates two types of junctions. The following assumptions are made for combining subcritical flows:

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

The assumption 'flows are parallel to the channel walls immediately above and below the junction' implies that hydrostatic pressure distributions can be predicted, and assumption 'the velocity is uniform over the cross-sections immediately above and below the junction' suggests that the momentum correction factors are equal at the reference sections.



(a) Plan - Constant Width (b) Plan - Unequal Width

Figure 20.15: Channel Junction Definition Sketches

(b) Supercritical Flow in Confluences

In contrast with subcritical flows at junctions, supercritical flows with changes in boundary alignments is generally complicated (Ippen, 1951, Rouse, 1949). In subcritical flow, backwater effects are propagated upstream, thereby tending to equalise the flow depths in the main and side channels. However, backwater cannot be propagated upstream in supercritical flow and flow depths in the main and side channels cannot generally be expected to be equal. Junctions for rapid flows and very small junction angles are designed assuming equal water surface elevations in the side and main channels.

Standing waves (Ippen, 1951) in supercritical flow at open channel junctions complicate flow conditions. These waves may necessitate increased wall heights in the vicinity of the junction. Wave conditions that may be produced by rapid flow at the downstream of a typical junction are shown in Figure 20.16. One area of maximum wave height can occur on the side channel wall opposite the junction point and another on the main channel right wall downstream from the junction. Supercritical flow may unavoidably occur in certain confluences.

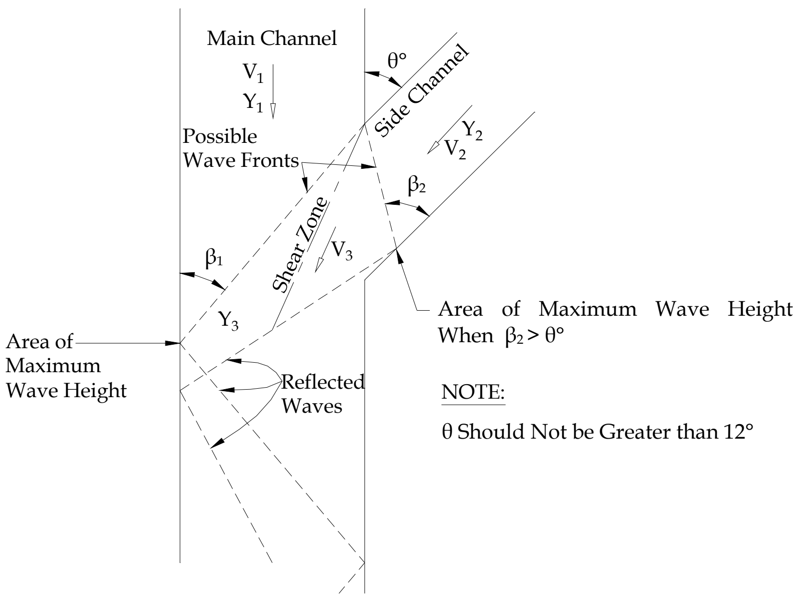


Figure 20.16: Open Channel Confluence, Standing Waves - Supercritical Flow

20.8 SIDE-OVERFLOW WEIRS

20.8.1 Description

Side-overflow weirs facilitate overflow and diversion of stormwater by directing the discharge away from the original channel. Such structures are commonly used to direct channel discharges above predetermined levels into off-line stormwater detention facilities. Flow diversions occur only during storms.

20.8.2 Design Considerations

The design of side-overflow weirs is based on empirical equations which quantify the relationship between the discharge over the weir and geometric parameters at the weir, including the length of the weir and head (Hager, 1987). Figure 20.17 (Metcalf and Eddy, 1972) shows three head or water surface profile conditions that can prevail at a side-overflow weir:

- Condition 1 - the channel bed slopes steeply, producing supercritical flow. Under this condition, the weir has no effect upstream and along the weir there is a gradual reduction in depth. The flow depth in the original channel increases at the downstream of the weir before tending asymptotically to the normal depth corresponding to the remaining discharge;
- Condition 2 - The channel bed slopes mildly. Under this condition, subcritical flow prevails and the weir impact is noticed upstream of the weir only. The water surface profile downstream of the weir corresponds to the normal depth of the remaining discharge. Along the weir there is a gradual increase in depth and upstream of the weir the flow depth tends asymptotically to the normal depth for the initial discharge; and
- Condition 3 - The channel bed slopes mildly, but the weir crest is below the critical depth corresponding to the initial flow, and the flow at the weir is supercritical. Frazer (1957) indicates that conditions 1 and 3 may result in the development of a hydraulic jump at the weir.

The most common condition that a designer will encounter is Condition 3, where the weir elevation is below the critical depth. When only a relatively small amount of the flow is diverted, a rising water surface profile occurs. According to Metcalf and Eddy Inc. (1972), a falling water profile will occur if the ratio of the height of the weir, c, to the channel specific energy, E_w referenced to the top of the weir, is less than 0.6.

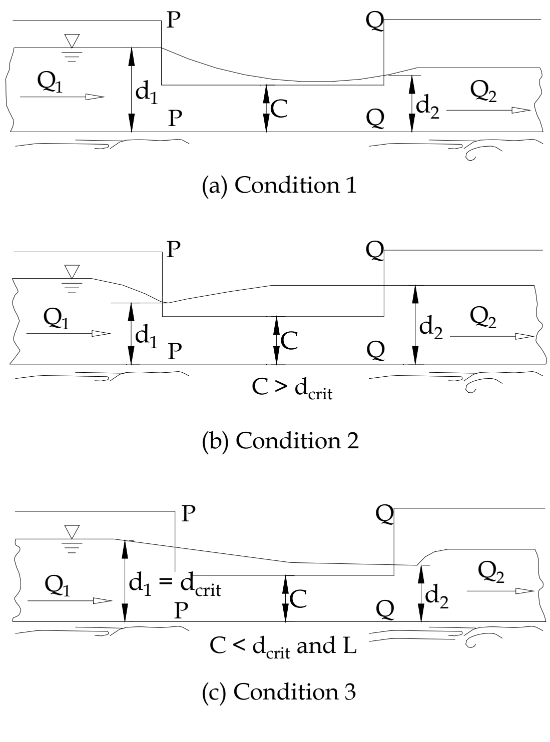


Figure 20.17: Possible Types of Water Surface Profiles at a Side-overflow Weir (Metcalf & Eddy, 1972)

20.8.3 Design Criteria

The design criteria refers to Figure 20.18.

(a) Falling Water Surface

The equations for computing weir length for the falling water surface profile combine Bernoulli's theorem with a weir discharge formula.

$$L=2.03\,B\,\left(5.28\text{-}2.63\frac{C}{E_w}\right) \tag{20.11}$$

where,

L = Length of weir (m);
C = Height of weir (m);
B = Channel width (m); and
E_w = Channel specific energy (m).

and,

$$E_w = \alpha\frac{V^2}{2g} + \alpha'(y_n - C) \tag{20.12}$$

where,

α = Velocity coefficient;
V = Normal velocity in the approach channel (m/s);
α' = Pressure-head correction;
C = Height of the weir above the channel bottom (m);
g = Acceleration due to gravity (m/s²); and
y_n = Normal depth of flow in approach channel (m).

Values for α and α' of 1.2 and 1.0 respectively can be used in the approach channel, while at the lower end of the weir values of 1.4 and 0.95 can be used for α and α' respectively.

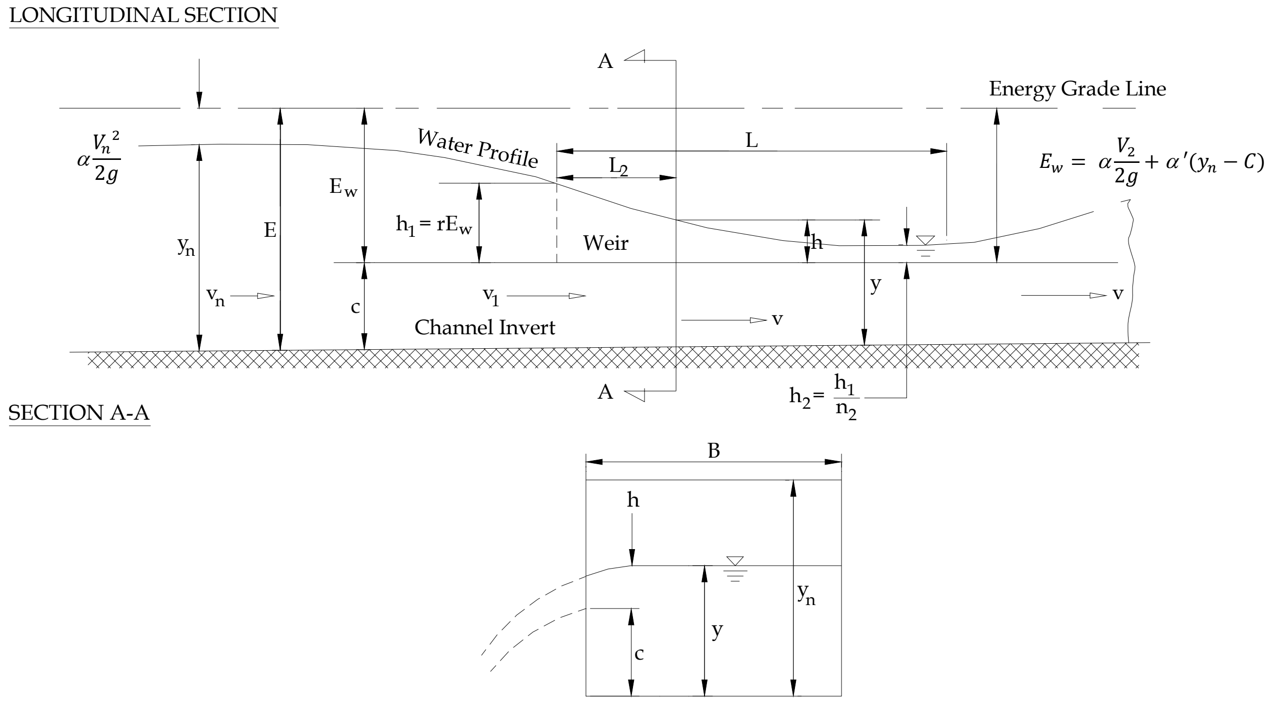


Figure 20.18: Typical Cross-sectional Hydraulics at a Side-Overflow Weir (Metcalf & Eddy, 1972)

(b) Rising Water Surface

The analysis for estimating the weir length for the rising water surface profile is based on Equations 20.13 below:

$$L=\frac{B}{C}\left[\emptyset\left(\frac{y_2}{E}\right)-\emptyset\left(\frac{y_1}{E}\right)\right] \tag{20.13}$$

where,

L	=	Length of weir (m);
C	=	Constant (0.35 for a free nappe);
B	=	Channel width (m);
E	=	Specific energy (m);
y_1, y_2	=	Depth in channel (m); and
$\left[\emptyset\left(\frac{y}{E}\right)\right]$	=	Varied flow function (Collinge, 1957).

Equation 20.13 is recommended for use only in the case of a rising water surface profile. Metcalf and Eddy Inc. (1972) indicates that this equation works best when the Froude number is between 0.3 and 0.92.

20.9 FLOW SPLITTER

20.9.1 Description

A flow splitter is a special structure designed to divide a single flow and divert the parts into two or more downstream channels. A flow splitter can serve three functions:

- Reduction in Water Surface Elevation - by dividing the flow from a large pipe into multiple conduits, the height of flow measured from the flow line to the water surface (or for pipes flowing full, the inside diameter can be reduced. This may be necessary to route flows under immovable obstructions.
- Dividing Flows **-** examples of this include division of existing large special-design conduits, such as arches or horseshoes, into less expensive multiple-pipe continuations and division of flow between low and high-flow conduits at the intake of an inverted siphon.
- Restriction of Flows to Water Quality Treatment Facilities - to restrict flows to water quality treatment facilities and bypass the remaining higher flows around the facilities (off-line). This can be accomplished by splitting flows in excess of the water quality design flow upstream of the facility and diverting higher flows to a bypass pipe or channel. The bypass typically enters a detention pond or the downstream receiving drainage system. A crucial factor in designing flow splitters is to ensure that low flow is delivered to the treatment facility up to the water quality design flow rate. Above this rate, additional flows are diverted to the bypass system with minimal increase in head at the flow splitting structure to avoid surcharging the water quality facility under high flow conditions.

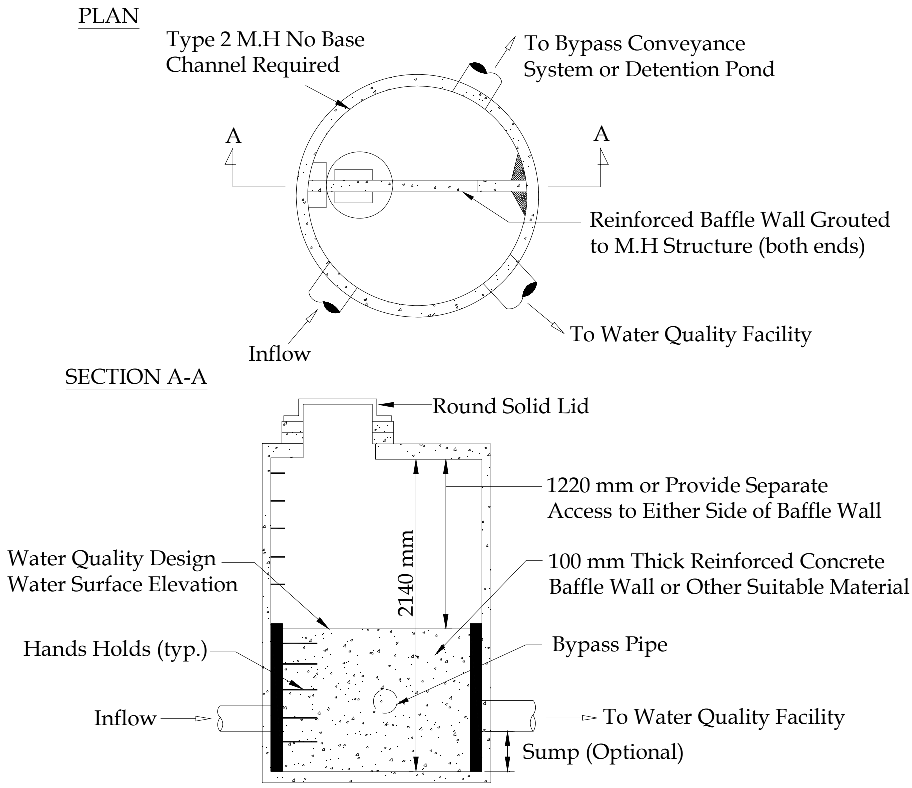
Figure 20.19 shows a typical flow splitter made of manholes with concrete baffles. Figure 20.20 shows a typical diversion/isolation structure.

20.9.2 Design Considerations

Two major considerations for the design of flow-splitting devices are as follows:

(a) Head Loss - Hydraulic disturbances at the point of flow division result in unavoidable head losses. These losses, however, may be reduced by the inclusion of proper flow deflectors in the design of the structure. Deflectors minimise flow separation by providing a gradual transition for the flow, rather than by forcing abrupt changes in flow direction.

(b) Debris - In all transitions from larger to smaller pipes, debris accumulation is a potential problem. Tree limbs and other debris that flow freely in the larger pipe may not fit in the smaller pipe(s) and may restrict flow. In addition, flow splitters cause major flow disturbances resulting in a region of decreased velocity.



NOTE:
The water quality discharge Pipe may require an orifice plate be installed on the Outlet to control the height of the design water surface (weir height). The design water surface should be set to provide a minimum headwater/diameter ratio of 2.0 on the outlet Pipe.

Figure 20.19: Typical Flow Splitter Devices

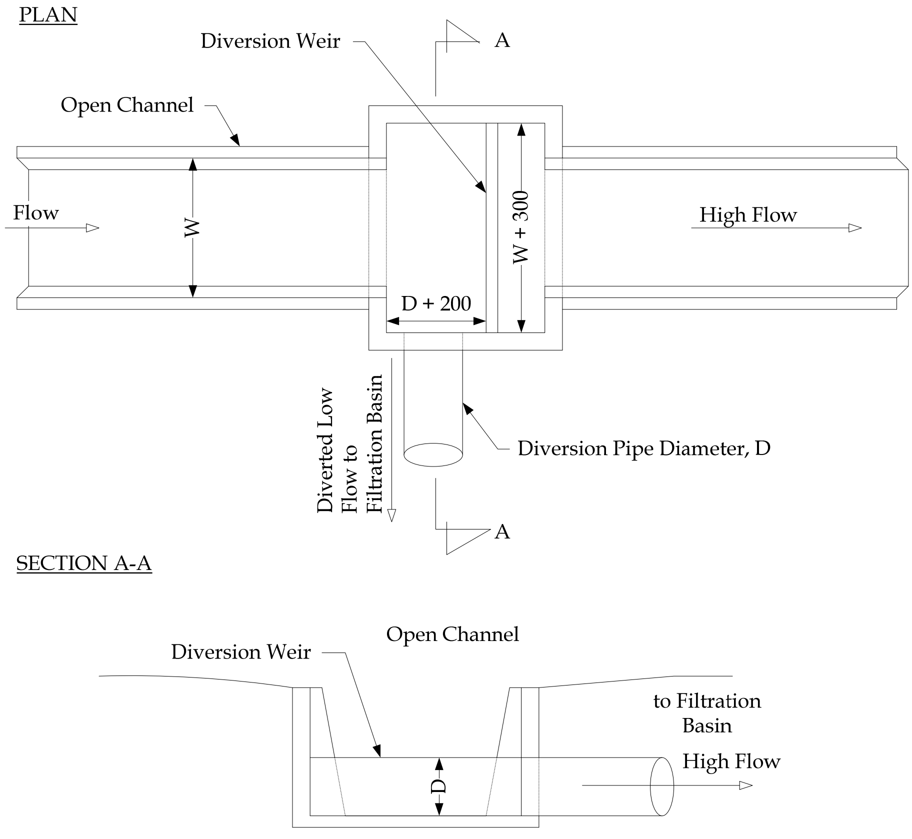


Figure 20.20: Typical Isolation/Diversion Structure

This reduction causes suspended materials in stormwater flow to settle in the splitter box. Although the deflector design should minimise velocity reduction as much as possible, total elimination of the problem is unlikely. Therefore, positive maintenance access must be provided. Because flow splitting devices are maintenance-intensive, their use should be judiciously controlled by the engineer.

20.9.3 Design Criteria

The design criteria for flow splitter are as follows (MassHighway, 2004):

- The top of the weir shall be located at the water surface for the 40mm rainfall depth for water quality design storm;
- The maximum head over the weir shall be minimised for flow in excess of the water quality design flows;
- Outlets must discharge to stable areas;
- Splitter structures must be designed to sustain anticipated dead and live loads;
- Construct splitters in accessible locations; and

20.10 FLOW SPREADER

20.10.1 Description

Flow spreaders are used to uniformly spread flows across the inflow portion of water quality facility (e.g. sand filter, biofiltration swale, or filter strip). Options A through C can be used for spreading flows that have already concentrated. Option D is only for flows that are already unconcentrated and enter a filter strip or biofiltration swale.

20.10.2 General Design Criteria

- Where flow enters the spreader through a pipe, it is recommended that the pipe be submerged to practically dissipate energy; and
- Rock protection is required at outfalls.

20.10.3 Design Criteria for Flow Spreading Options

The following presents the design criteria for each of the flow spreading options:

(a) Anchored Plate (Option A)

Figure 20.21 shows the details of the spreader.

- The spreader shall be preceded by a sump having a minimum depth of 200 mm and minimum width of 600 mm. The sump area shall be lined with steps to reduce erosion and to provide energy dissipation;
- The top of the flow spreader plate shall be level, projecting a minimum of 50 mm above the final grade of the invert of the water quality facility;
- The plate shall extend horizontally beyond the bottom width of the facility to prevent water from eroding the side slope;
- The plate shall be securely fixed in place; and
- The flow spreader plate may be either wood, metal, fibreglass reinforced plastic, or other durable material.

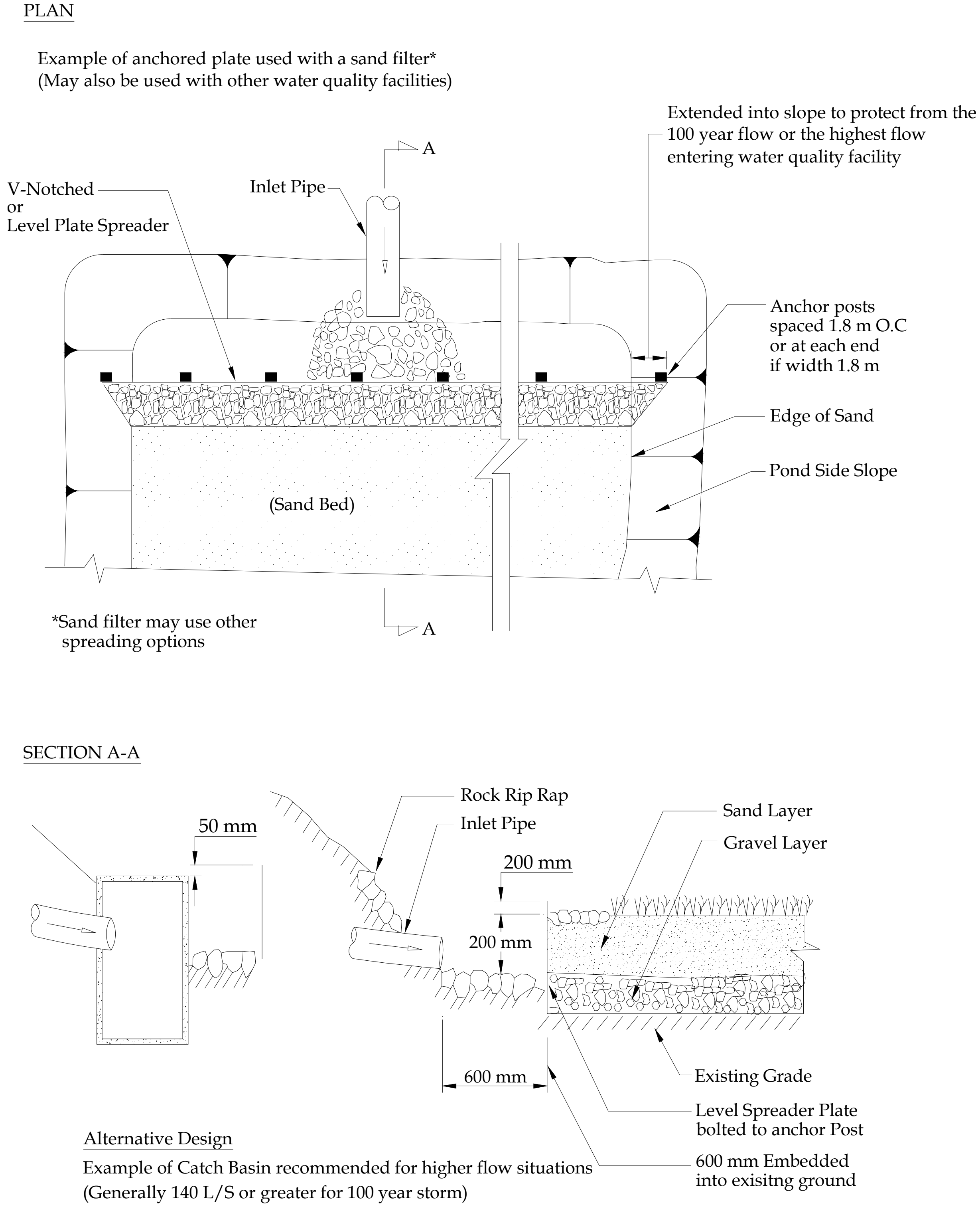


Figure 20.21: Flow Spreader (Option A)

(b) Concrete Sump Box (Option B)

This alternative uses a rectangular concrete sump (see Figure 20.22 for details).

- The wall of the downstream side of the concrete sump shall extend a minimum 50 mm above the invert of the treatment bed; and
- The downstream wall of the box shall have "returns" at both ends. Side walls and returns shall be slightly higher than the weir so that erosion of the side slope is minimised.

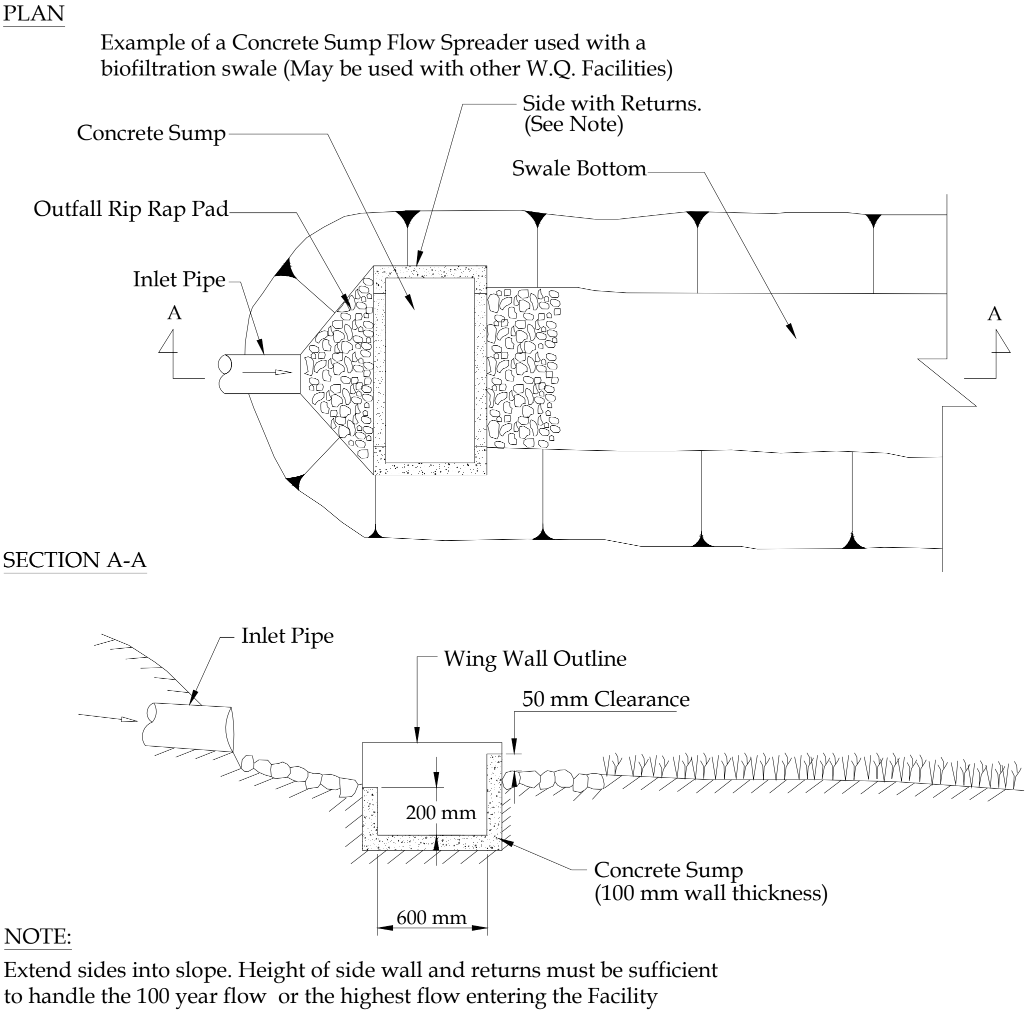


Figure 20.22: Flow Spreader (Option B)

(c) Flat-topped Notched Curb Spreader (Option C)

An example of flat-topped notched curb spreader used with a grassed swale is shown in Figure 20.23. The spreader sections are made of extruded concrete laid side by side and level. Typically four “teeth” per 1.25 m section provide good spacing. The space between adjacent “teeth” forms a v-notch.

(d) Through-Curb Ports (Option D)

Details of the spreader are shown in Figure 20.24. Unconcentrated flows from paved areas entering filter strips or continuous flow biofiltration swales can use curb ports to allow flows to enter the strip or swale. Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to convey water to the water quality facility. Openings in the curbing shall be at regular intervals of 2 m (minimum). The width of each curb port opening shall be 275 mm minimum. Approximately 15 percent or more of the curb section length shall be in open ports, and no port should discharge more than about 10 percent of flow.

20.11 GATES

20.11.1 Description

The main operational requirements for gates are the control of floods, watertightness, minimum hoist capacity, convenience of installation and maintenance and above all failure free performance and avoidance of safety hazards to the operating staff and the public. Despite robust design and precautions, faults can occur and the works must be capable of tolerating these faults without unacceptable consequences (Novak et al., 2007). Table 20.2 shows the summary of types of gates.

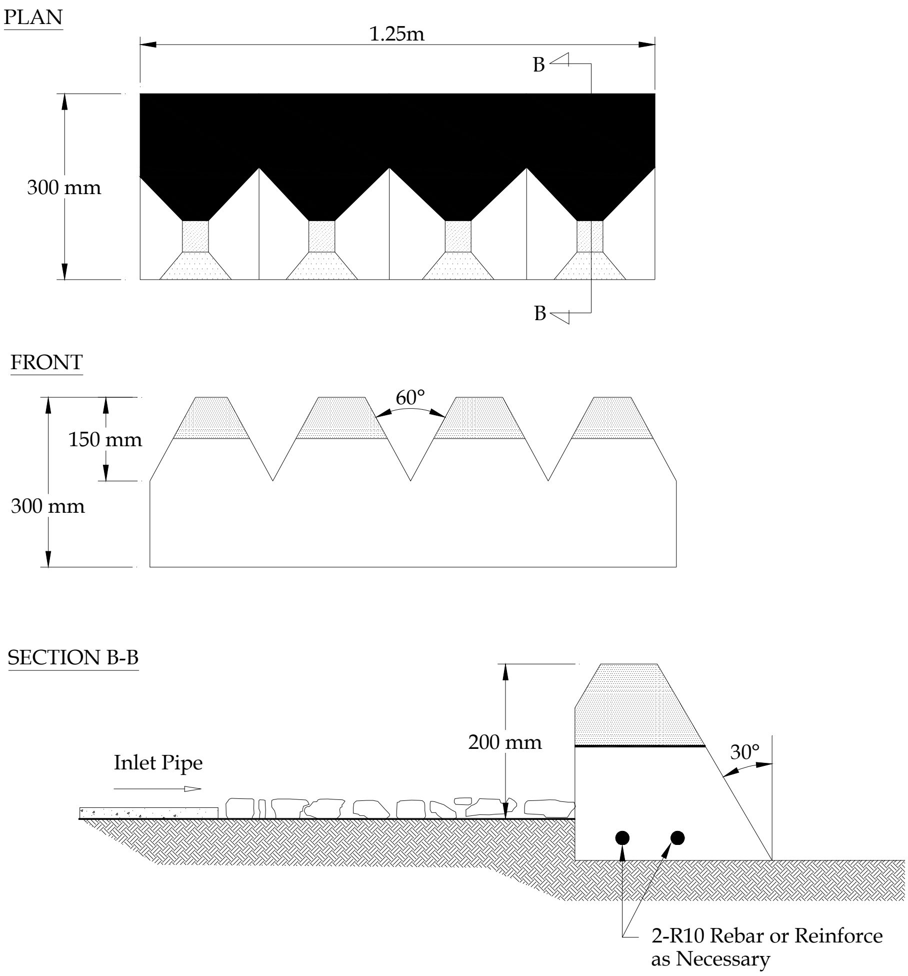


Figure 20.23: V-notch Flow Spreader (Option C)

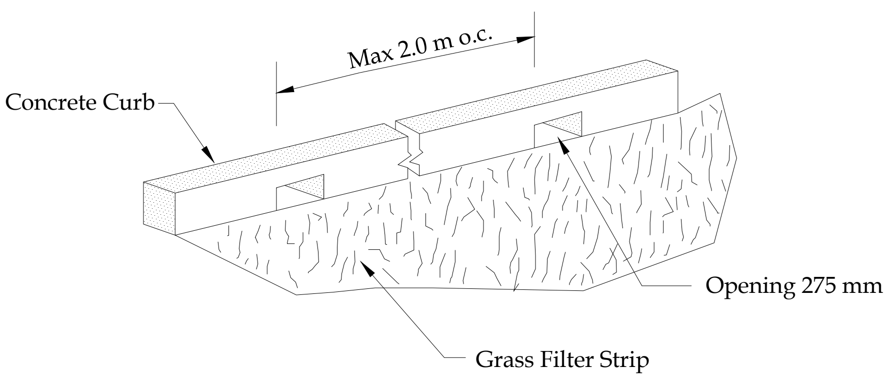


Figure 20.24: Through-curb Port (Option D)

20.11.2 Type of Gates

(a) Radial Gates

Radial gates are usually constructed as portals with cross-bars and arms (straight, radial, or inclined), but could also be cantilevered over the arms. Their support hinges are usually downstream but (for low heads) could also be upstream, resulting in shorter piers. The gate is usually hoisted by cables fixed to each end to prevent it from twisting and jamming. If the cables are connected to the bottom of the gate its top can be raised above the level of the hoist itself, if the layout of the machinery allows it (Novak et al., 2007).

(b) Drum and Sector Gates

Drum and sector gates are circular sectors in cross-section. Drum gates are hinged upstream and sector gates downstream. Gates on dam crests are usually of the upstream hinge type. Drum gates float on the lower face of the drum, whereas sector gates are usually enclosed only on the upstream and downstream surfaces (Novak et al., 2007).

(c) Flap Gates

Flap gates are one of the simplest and most frequently used types of regulating gates used mainly on weirs and barrages (rarely on dam crests), either on their own or in conjunction with plain lift gates. They were developed as a replacement for wooden flash-boards, originally as a steel-edged girder flap, which was later replaced by a torsion-rigid pipe; further development was achieved by placing the pipe along the axis of the flap bearing, with the skin plate transmitting the water pressure to cantilevered ribs fixed to the pipe. Next in use were torsion-rigid gate bodies with curved downstream sides (fish-belly gates), with torsion-rigid structures using prism-shaped sections being the latest development in flap gates (Brouwer, 1988).

(d) Top Hinged Flap Gates

Top-hinged flap gates are used in tidal structures to prevent flooding of an inland region by sea waters during rising tides or flood surges and to permit inland waters to drain off into the sea during ebb tide. They are also used in culverts and pumped drainage outfalls to rivers. They do not require an outside source of power and operate automatically. The construction of the gates is simple and little maintenance is required. The gates will not entirely exclude ingress of saline water if the downstream water level rises above the sill during discharge under the gate, when a lens of saline water can penetrate upstream against the flow.

They control water in one direction only and perform like a non-return valve. They cannot control upstream level. In stormwater discharge this facility is not required. Top-hinged flap gates can be operated under clear discharge conditions or drowned. When designing gates of this type a gravity bias is required in the closed position so that the gates close immediately before reversal of flow occurs (Lewin, 2001).

(e) Side Hinged Flap Gates

Side hinged flap gates are used for the conversion of wetland environments into agricultural land. These are very efficient in draining upstream lands, preventing the intrusion of saline waters and back flooding during high tides and some minor floods. These lead to the development of freshwater systems, where they were previously saline / brackish. Only suitable for tidal gating of channels set back from main rivers and when complete tidal isolation is required. Side hinged flap gate only allow flow in one direction (downstream) for drainage purposes (NSW Government of Industry and Investment, 2009).

(f) Roller Gates

A roller gate consists of a hollow steel cylinder, usually of a diameter somewhat smaller than the damming height; the difference is covered by a steel attachment, most frequently located at the bottom of the cylinder (in the closed position). The gate is operated by rolling it on an inclined track. Because of the great stiffness of the gate, large spans may be used, but roller gates require substantial piers with large recesses. New roller gate installations are not being used nowadays partly because large units of this type are very vulnerable to single point failure (Novak et al., 2007).

(g) Fabric Gates

Inflatable rubber or fabric gates can be pressurized by air, water or both. They usually have an inner shell and an outer casing, and can be used to close very large spans (Novak et al., 2007).

(h) Vertical Lift Gates

Vertical lift gates designed as a lattice, box girder, a grid of horizontal and vertical beams and stiffeners, or a single slab steel plate, may consist of single or double section (or even more parts can be involved in the closure of very high openings); in the case of flow over the top of the gate it may be provided with an additional flap gate. The gates can have slide or wheeled support. In the latter case fixed wheels (most frequent type), caterpillar or a roller train (Stoney gate) may be used; for fixed wheels their spacing is reduced near the bottom. The gate seals are of specially formed rubber (Novak et al., 2007).

Table 20.2: Summary of Type of Gates

Types	Advantages	Pictures
Radial Gates	• Smaller hoist; • Higher stiffness; • Lower (but longer) piers; • Absence of gate slots; and • Easier automation;	
Drum and Sector Gates	• Ease of automation and absence of lifting gear; • Fast movement; • Accuracy of regulation; and • Low piers.	
Top Hinged Flap Gates	• Provide fine level regulation; • Ease flushing of debris; • Cost effective and often environmentally more acceptable; • Naturally automated; • Simplistic construction and installation; • Requires little maintenance; • Long lifespan; and • Gates manufactured from non-metal materials have no scrap value and will have no attraction to scrap metal thieves.	

Table 20.2: Summary of Type of Gates (Continued)

Types	Advantages	Pictures
Side Hinged Flap Gates	• Naturally automated; • Requires small amount of energy (water pressure) to open gates. As a result, more robust materials can be used for the gate without compromising the efficiency of the structure; • Once open, the gate will remain open until the tide recedes; • Gates open wider than top hinged flap gates and for a greater period; • Full depth of the water column is available for fish passage; • Simplistic and more difficult to vandalise; • Requires little maintenance; and • Long lifespan.	
Roller Gates	• Very reliable; • Capable of operating at differential heads; and • Can be manufactured very wide.	
Fabric Gates	• Low cost; • Low weight; • Absence of lifting mechanism; • Little need for maintenance; • Acceptance of side slope (at river banks); and • Ease of installation.	
Vertical Lift Gates	• Can be fitted with overflow sections; • Short piers; • Wide span gates can be engineered to provide good navigation openings; and • Up and over gates can reduce height of supporting structure.	

20.12 VALVES

20.12.1 Description

Many types of valves have been invented by man to control the flow of fluids. Of those which have survived the test of time, each has at least several features which are unique or important. One offers tight shut-off, another low cost, others effective control of the fluid flow, still others, perhaps combinations of these, and on and on. To date, however, no valve inventor has discovered the ideal valve which combines all these features into one package and experience teaches us that it is unlikely anyone will. Thus, valve designers have created the globe, plug, ball, gate and numerous other valve types, all of which are in extensive use throughout the world's vast and complex industrial processes. The gate valve is among the most common because it offers several advantages of function and cost effectiveness over other types (Norden, 1975). Table 20.3 presented the summary of type of valves.

20.12.2 Type of Valves

(a) Check Valve

The check valve can be used to provide flow in one direction only through a culvert for floodplain drainage and flood and saline intrusion mitigation purposes. The check valve is constructed from an elastomer with a vertical slot that is flexible yet quite stiff and closed in its relaxed position. This elastomer material is resistant to the corrosive effects of marine and highly acidic waters, a significant issue associated with most metallic structures. The check valve can be mounted flush on a flat or curved headwall, or be clamped to culverts of varying shape, size or material. During backflow, the check valve seals shut (NSW Government of Industry and Investment, 2009).

(b) Cone Dispersion Valve

The cone dispersion valve is probably the most frequently used type of regulating valve installed at the end of outlets discharging into the atmosphere. It consists of a fixed 90° cone disperser, upstream of which is the opening covered by a sliding cylindrical sleeve The fine spray associated with the operation of the valve may be undesirable, particularly in cold weather; sometimes, therefore, a fixed large hollow cylinder is placed at the end of the valve downstream of the cone, resulting in a ring jet valve (Novak et al., 2007).

(c) Needle Valve

The needle valve, (and its variation the tube valve), has a bulb-shaped fixed steel jacket, with the valve closing against the casing in the downstream direction. When open, the valves produce solid circular jets and can also be used in submerged conditions. The valves may suffer from cavitation damage and produce unstable jets at small openings, and are expensive as they have to withstand full reservoir pressures (Novak et al., 2007).

(d) Hollow-Jet Valve

Most of the disadvantages of valves are overcome in the hollow-jet valve, which closes in the upstream direction (when closed the valve body is at atmospheric pressure); because of this the valve is, of course, not suitable for use in submerged conditions (Novak et al., 2007).

Table 20.3: Summary of Type of Valve

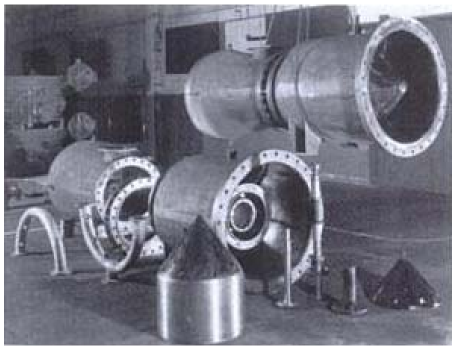
Types	Advantages	Pictures
Check Valve	• Self automated; • Can be mounted to a variety of culvert configurations - bolted to headwalls or clamped to pipes; • Can be installed at any angle including vertical; • Smaller valves can usually be installed by one person with simple hand tools; • Corrosion resistant and outer wrapping resistant to ozone; • Not as susceptible to jamming and will seal around entrapped floating debris during backflow. Can still operate when partially buried in sediment; • Flexible nature reduces risk of damage from floating debris; • No clearance is required for operation; and • Has no moving mechanical parts subject to wear and can last up to 50 years.	
Cone Dispersion Valve	• Very efficient energy dissipation valve; • Simple construction; • Relatively low cost; • Can be operated electro-mechanically or by oil hydraulics; • Good discharge coefficient; • Available in large sizes; and • Least flow obstruction of any terminal discharge valve.	
Needle Valve	• Dissipates energy; • Can be used as an in-line pressure-reducing valve; and • Best valve for pressure reduction when cavitation of downstream conduit is of concern.	

Table 20.3: Summary of Type of Valve (Continued)

Types	Advantages	Pictures
Hollow-jet Valve	• Dissipates energy; and • Can be arranged to discharge into a stilling basin at an angle.	

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