8

HYDRAULIC STRUCTURES

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Hydraulic structures are used to control water flow characteristics such as velocity, direction and depth. Structures may also be used to control the elevation and slope of the channel bed, as well as the general configuration and stability of the waterway. Hydraulic structures include channel drop structures, low flow check structures, energy dissipators, bridges, transitions, chutes, bends and many other specific drainage structures.

8.1 CHANNEL DROP STRUCTURES

The term "drop structure" is broadly defined. Included are structures built to restore previously damaged channels, structures constructed during new urban development to prevent accelerated erosion caused by increased runoff, and applications in which other specialized hydraulic conditions are created in the flow channel. The focus of the following discussion and criteria is on drop structures with design flows up to 10,000 cfs.

8.1.1 Basic Components of a Drop Structure

Figure 8.1 shows a typical channel drop structure with its various components. Once a particular structure type is selected for design, analyses are conducted to determine the optimal sizing and extent of the various components.

8.1.2 Drop Structure Types

Design guidance is presented in this section for the following drop structures:

- Baffled Chute Drops
- Vertical Hard Basin Drops
- Vertical Riprap Basin Drops
- Sloping Concrete Drops
- Low Flow Check Structures

Due to a high failure rate and excessive maintenance costs, drop structures having loose riprap on a sloping face are not permitted.

Figure 8.2 shows schematic profiles for each type of structure listed above.

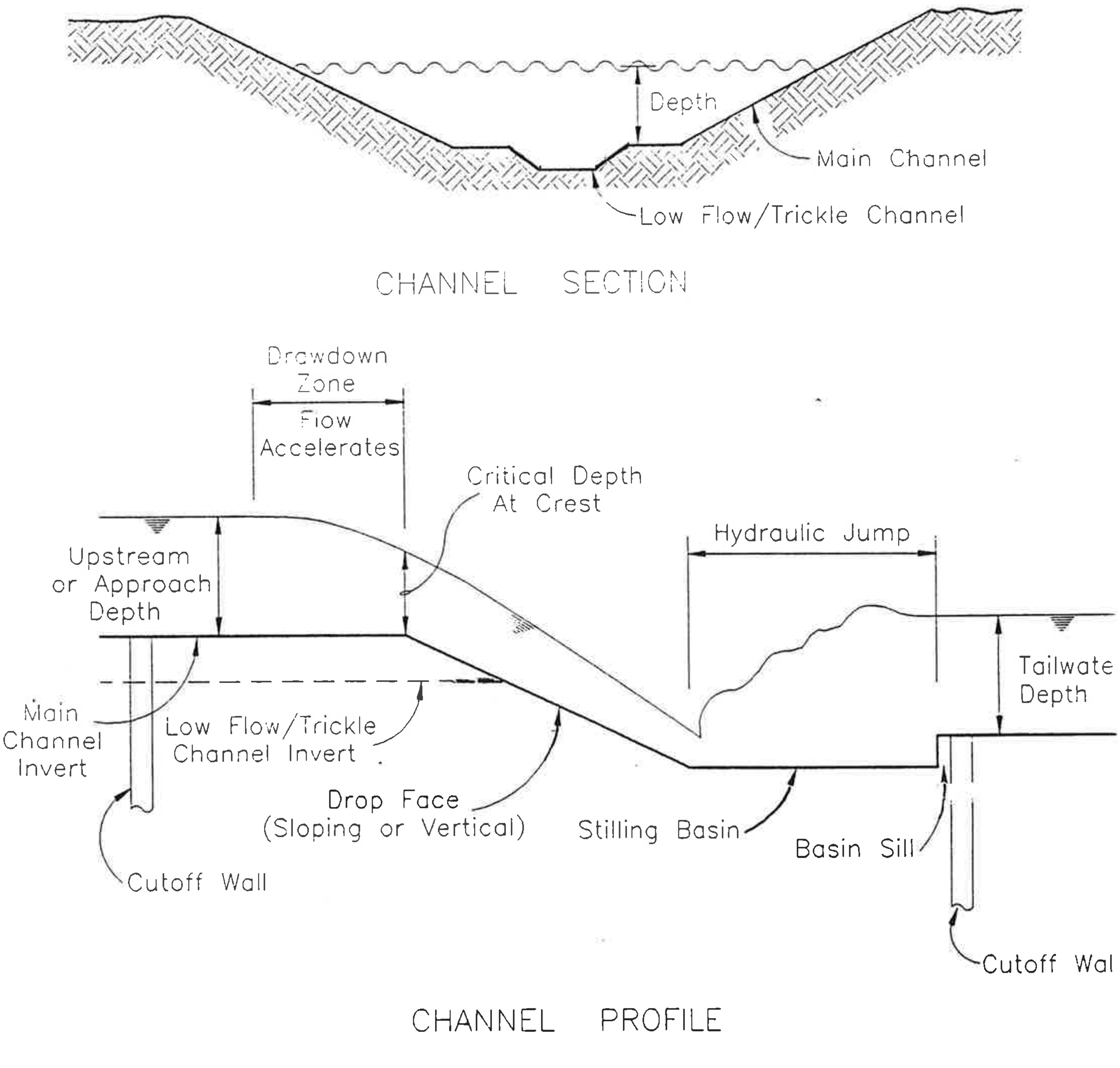


Figure 8.1
Typical Drop Structure Components

8.2 GENERAL DESIGN PROCEDURES

These design procedures are generalized. Use them to identify the most suitable approach, with the understanding that detailed analytical methods and design specifications may vary as a function of site conditions and hydraulic performance. A standard drop structure design approach would include *at least* the following steps:

1. Define the maximum design discharge (usually the 100-year) and other discharges appropriate for analysis (selected discharges expected to occur on a more frequent basis, which may behave differently at the drop).

2. Select possible drop structure alternatives to be considered (Section 8.2.3).

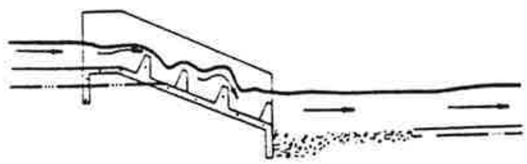
3. Determine the required longitudinal channel slope and the total drop height required to produce the desired hydraulic conditions.

4. Establish the channel hydraulic parameters, reviewing drop structure and channel combinations that may be most effective.

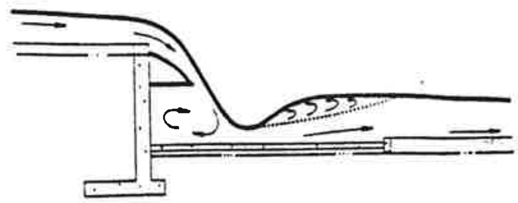
5. Conduct hydraulic analyses for the structure. Where appropriate, apply separate hydraulic analyses to the main channel and the low flow zones of the drop to determine the extent of protection required, as well as the potential problems/solutions for each.

6. Perform soils and seepage analyses to obtain foundation and structural design information. Combine seepage and hydraulic analysis data to determine forces on the structure. Evaluate uplift, overturning and sliding.

7. Use specific design criteria to determine the drop structure dimensions, material requirements and construction methods necessary to complete the design for the selected structures.

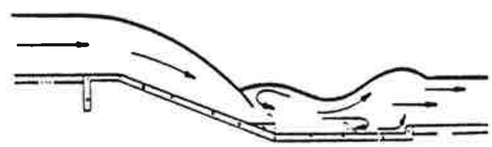


1. BAFFLE CHUTE

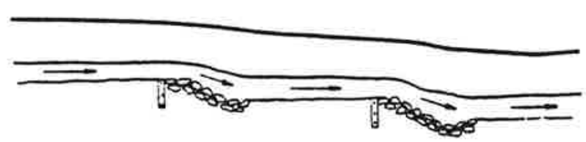


2. VERTICAL HARD BASIN

3. VERTICAL RIPRAP BASIN



4. SLOPING CONCRETE



5. LOW FLOW CHECK STRUCTURES

Figure 8.2
Drop Structure Types

8.2.1 Crest and Upstream Hydraulics

Usually, the starting point of drop analysis and design occurs on the crest section at the top of the drop. As flow passes through critical depth near the crest, upstream hydraulics are separated from downstream. The Critical flow state must be calculated and compared with the downstream tailwater effect which may submerge the crest and effectively control the hydraulics at the crest.

With control at the drop crest, upstream water surface profile computations are used to estimate the distance that protection should be maintained upstream, that is, the distance to where localized velocities are reduced to acceptable values. The water surface profile computations should include a transition/expansion head loss, which should typically range from 0.3 (modest transitions) to 0.5 (more abrupt transitions) times the change in velocity head. For a given discharge, there is a balance between the crest base width, upstream and downstream flow velocities, the Froude number in the drop basin and the location of the hydraulic jump. These parameters must be selected for each specific application.

Vertical or Near Vertical Abutments at Drop Crest: Figure 8.3 presents alternative drop crests at a vertical drop structure. In general, the objectives of upstream hydraulics and crest design are:

1. To maintain freeboard in the approach channel,

2. To optimize crest and basin dimensions to achieve the most cost-effective structure, and

3. To prevent erosion in the transition zone, where flow accelerates approaching the crest.

Sloping Abutments at Drop Crest: Figures 8.3 and 8.4 show a schematic layout for the drop crest and upstream channel at a sloping drop structure. The design objectives discussed previously also apply here.

8.2.2 Water Surface Profile Analysis

Backwater computations should be completed for the channel reaches upstream and downstream of the proposed drop structure to establish approach flow conditions and tailwater conditions for the range of design flows. Next determine the location of the hydraulic jump so that the stilling basin can be sized to adequately contained the zone of turbulence.

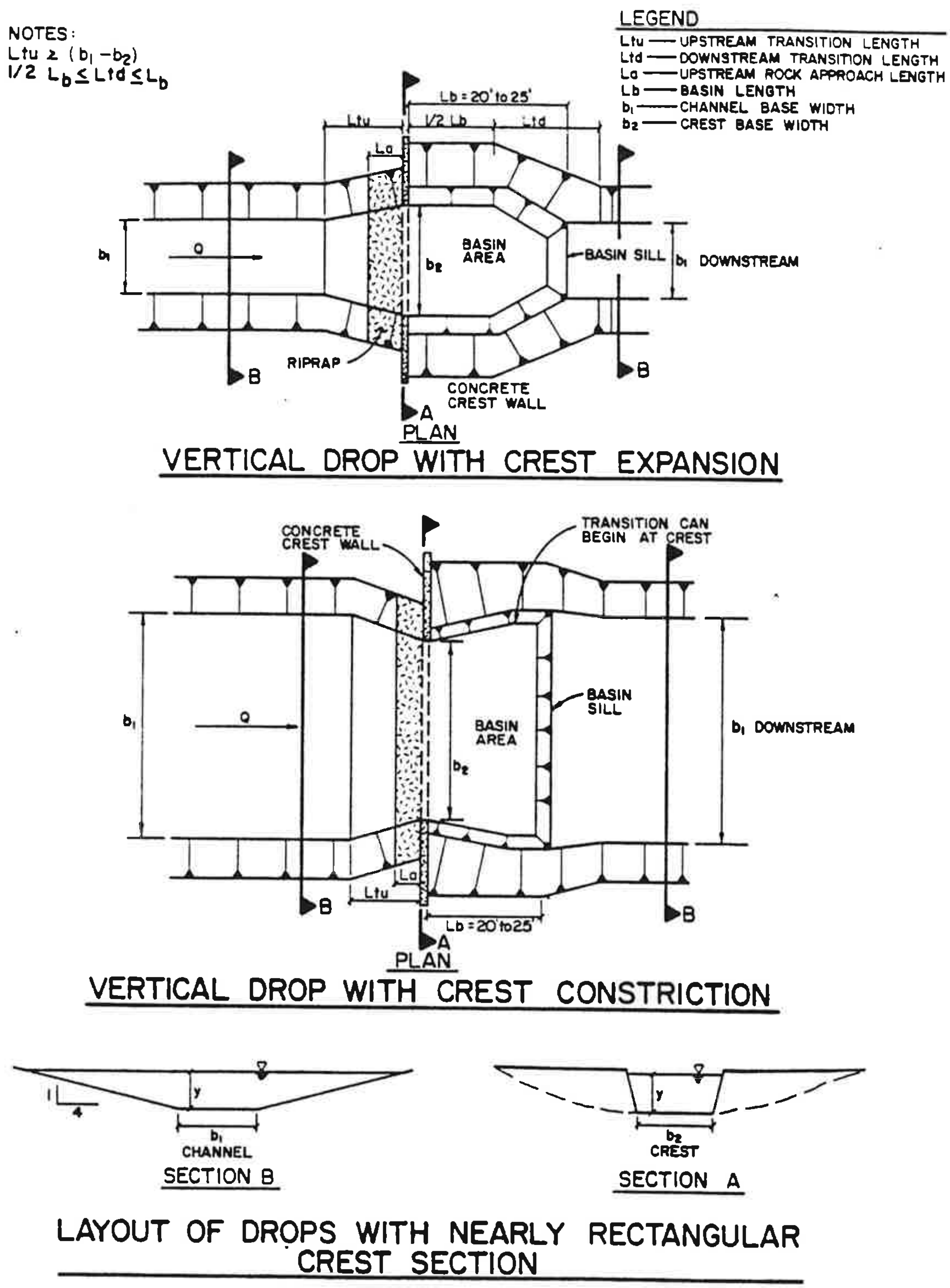


Figure 8.3
Typical Vertical Drop Crest

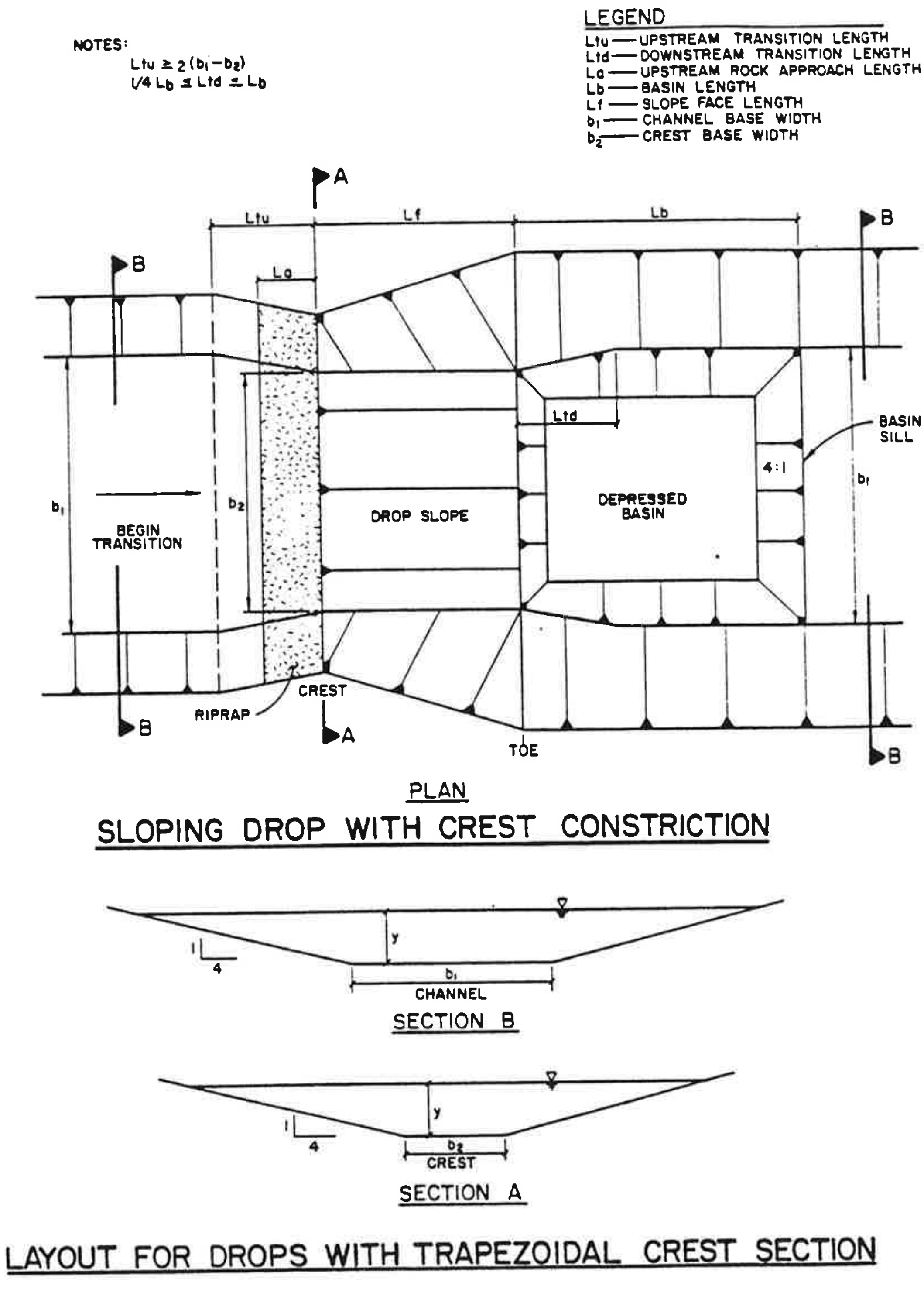


Figure 8.4
Typical Sloping Drop Crest Configuration

Soil Conditions	Drop Height, feet			
	2	4	8	12
Sand and gravel over bedrock with sufficient depth of material to provide support—groundwater prevalent	S*	S*	S/SwB*	S/SwB*
	CTc	CTc/ST	ST	ST
	CTf	CTf/CTl		
Sand and gravel with shallow depth to bedrock—groundwater prevalent	CTc	CTc/ST	ST	ST
	CW	CW	CW	CW
	S**	S**	S**	SwB**
Sand and gravel, great depths to bedrock—groundwater prevalent	S	S	S	S/SwB
	CTc	CTc/ST	ST	ST
Sand and gravel, no groundwater, or water table normally below requirement (for variation caused by depth to bedrock see first case)	S	S	S	S/SwB
	CTf/CTl	CTl	CTl	CTl
	CW	CW		
Clay (and silt)—medium to hard	CTc	CTc	CTc	CTc
	CW	reduce length for difficult backfill conditions		
	CTf/CTl	only for local seepage zones/silts		
	ST	expensive—for special problems		
Clay (and silt)—soft to medium with lenses of permeable material—groundwater present	S	S	S	S/SwB
	CTc	CTc	CTc/ST	ST
Clay (and silt)—soft to medium with lenses of permeable material—may be moist but not significant groundwater source	S	S	S	S/SwB
	CTc	CTc	CTc/ST	ST
	CTF	CTl	CTl	CTl
	CW	CW	CW	CW

* (consider scour in sheetpile support)

** (excavate into bedrock and set into concrete)

Legend

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

Table 8-1
General Seepage Cutoff Technique Suitability

For sloping drop structures, water surfaces must be determined for the supercritical profiles down the face of the drop. The location of the hydraulic jump is determined by comparison of the specific force, F_s, above and below the toe of the drop, using the following equation:

$$F_s = \frac{q^2}{gy} + \frac{y^2}{2} \qquad \textbf{(8.1)}$$

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

Where a major channel incorporates a low flow channel, separate analyses should be completed for the low flow zone and the major channel overbank zone. This is because the deeper flow profile in the low flow channel has a higher energy grade line profile. Specific force analysis in this zone shows that the hydraulic jump will not occur in the same location as the rest of the flow over the drop, and in most cases the jump will occur further downstream. To avoid this condition, the low flow channel should daylight at the downstream face of the structure.

An additional discussion of the hydraulic jump phenomenon is presented in Section 8.6.

8.2.3 Drop Selection

There are four major considerations for the selection of the type of drop structure for a particular application: 1) surface flow hydraulic performance; 2) foundation and seepage control; 3) economic considerations; and 4) construction considerations. Other factors which can affect selection are land uses, aesthetics, safety, maintenance and anticipated downstream channel degradation.

Surface Flow Hydraulic System: The primary consideration for the selection of a drop structure should be functional hydraulic performance. The surface flow hydraulic system combines channel approach and crest hydraulics, sloping or vertical drop hydraulics and downstream tailwater conditions.

Foundation and Seepage Control Systems: Table 8.1 presents some typical foundation conditions and control systems typically used for various drop heights. Table 8.1 is presented only as a guide. The hydraulic engineer must calculate hydraulic loadings which can occur for a variety of conditions such as interim construction conditions, low flow and flood flow.

8.3 DESIGN GUIDELINES FOR DROP STRUCTURES

8.3.1 Baffle Chute Drops

The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of the drop, which are commonly referred to as baffled apron or baffle chute drops. There are excellent references, such as *Design of Small Canal Structures* (USBR 1974), that should be used for the design of these structures.

The optimal performance occurs for a unit flow (q) at the chute width of 35 to 60 cfs/ft. The USBR states that the recommended design flow of 60 cfs/ft for baffle chute drops has been exceeded at several locations without causing significant problems.

Typical designs consist of upstream transition walls, a rectangular approach chute, a sloping apron of 2:1, or flatter, slope with multiple rows of baffle piers (see Figure 8.5). The toe of the chute extends below grade and is backfilled with loose rock to prevent undermining the structure by eddy currents or minor degradation of the downstream channel. The potential for debris flow must be considered. Use caution when conditions include streams with heavy debris flow, because the baffles can become clogged between the interstices, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet on the downstream channel.

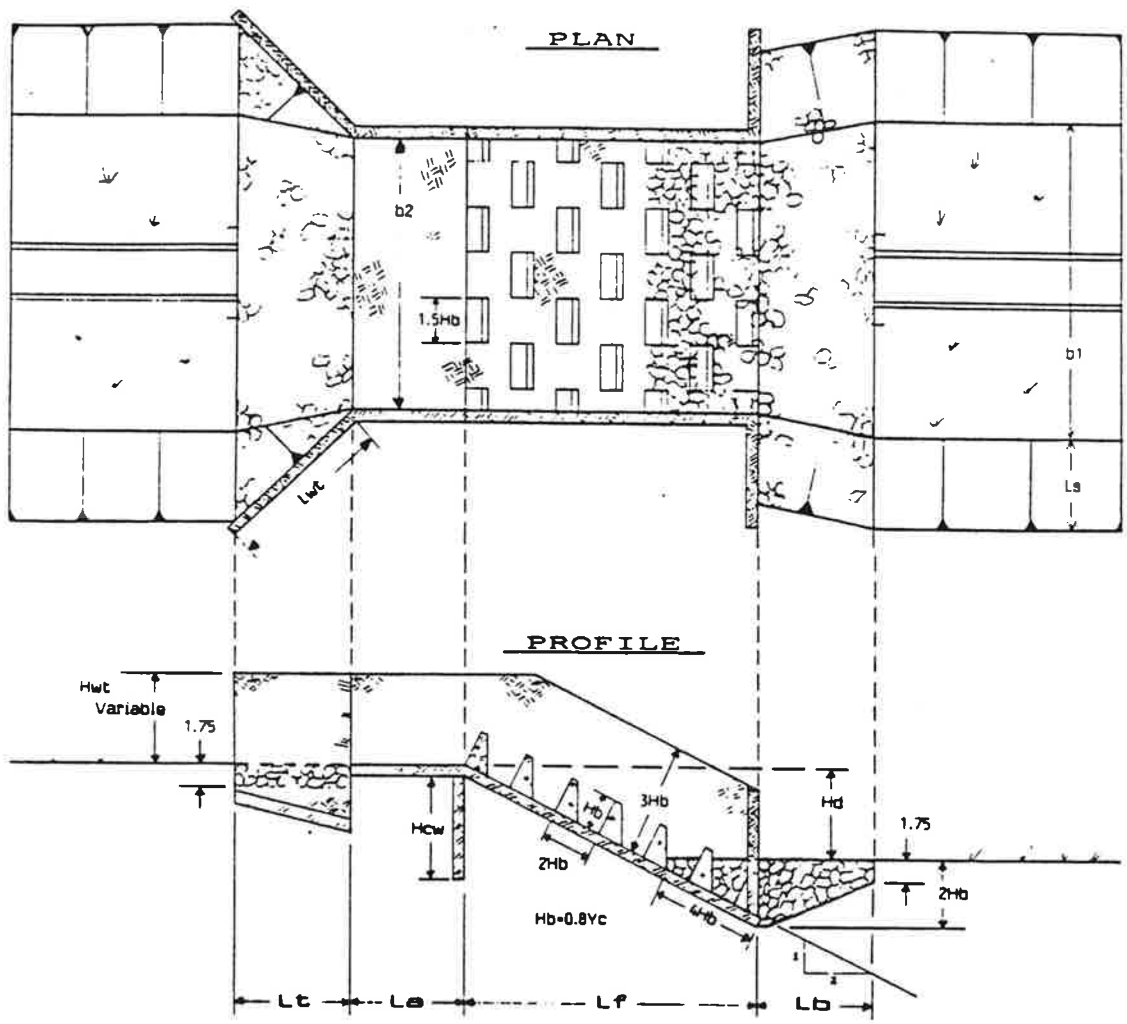
Basic design criteria and modification details are given in Figures 8.6 and 8.7. Remaining structural design parameters must be determined for specific site conditions. Recommended design procedures are discussed below.

General Hydraulic Design Procedure:

1. Determine the maximum inflow rate and the design discharge per unit width:

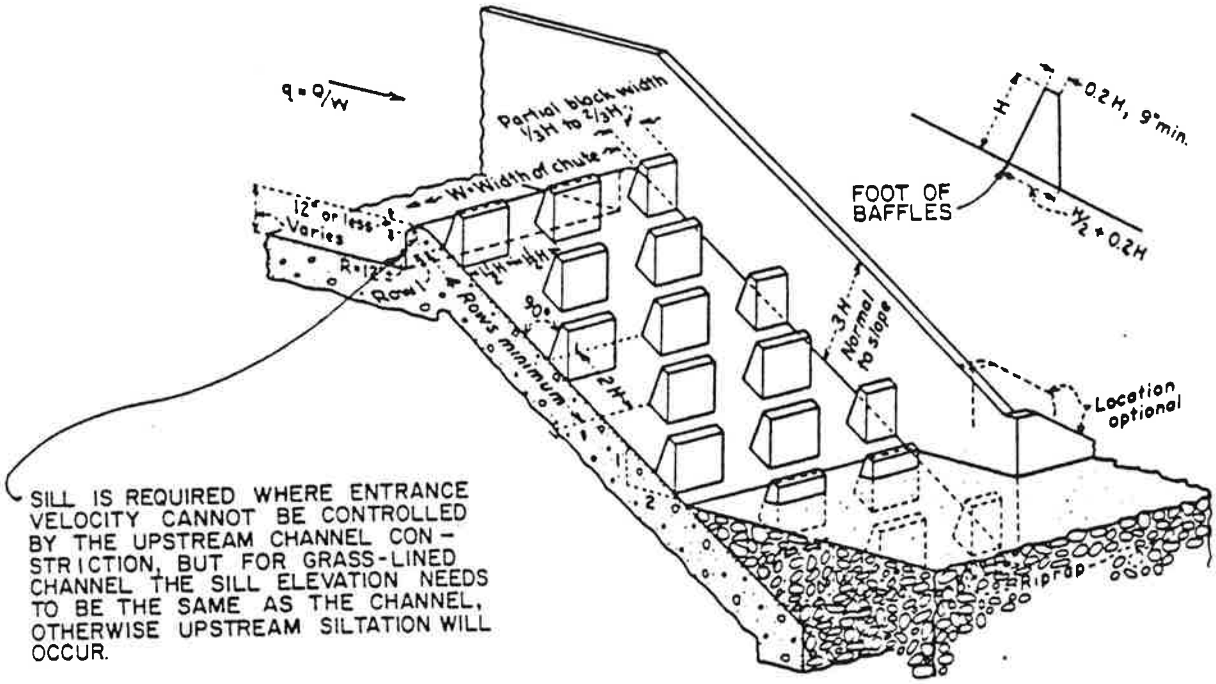
$$q_d = \frac{Q}{W} \qquad (8.2)$$

The chute width, W, may depend on the upstream or downstream channel width, the upstream hydraulic control or local site topography. Generally, a unit discharge between 35 to 60 cfs/ft is most economical.

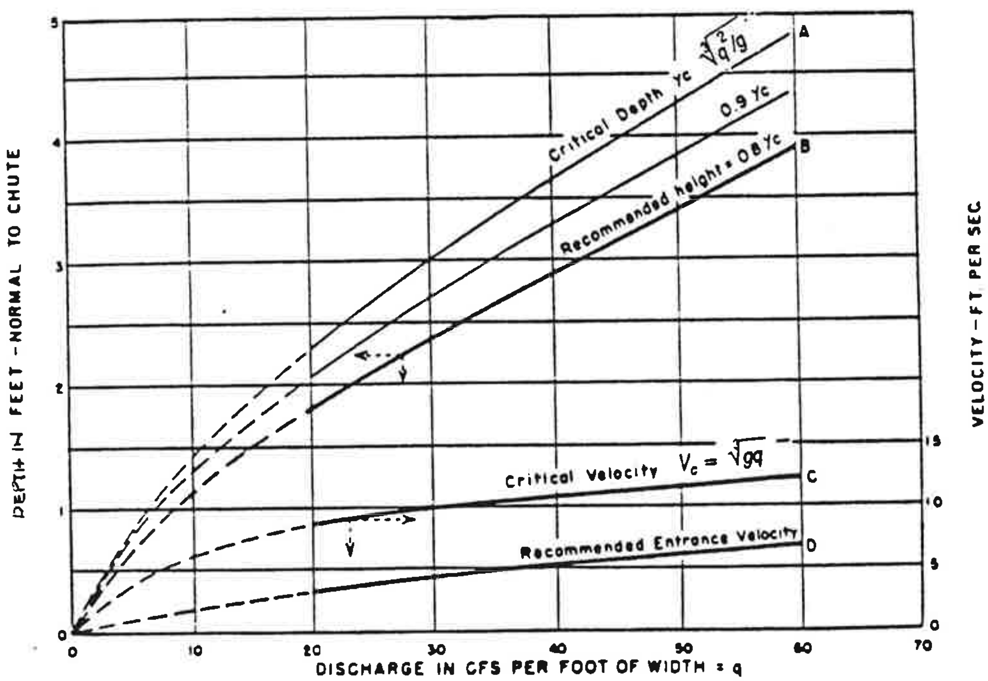


Lt - Transition Length
La - Apron Length
Lf - Face Length
Lb - Basin Rock Length
Lwt - Wing Wall Length
Hwt - Wing Wall Height
Hcw - Cutoff Wall Height
Hd - Height of Drop
Hb - Baffle Height - 0.8 x Critical Depth

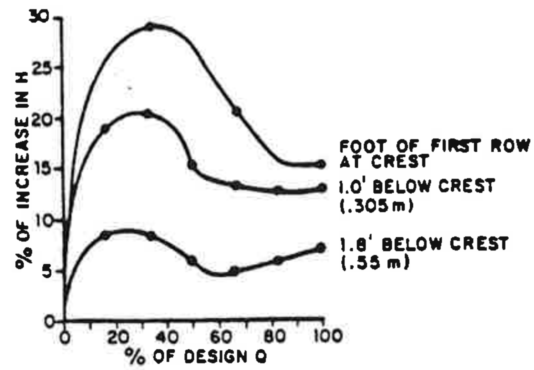
Figure 8.5
Typical Baffled Chute Drop Structure



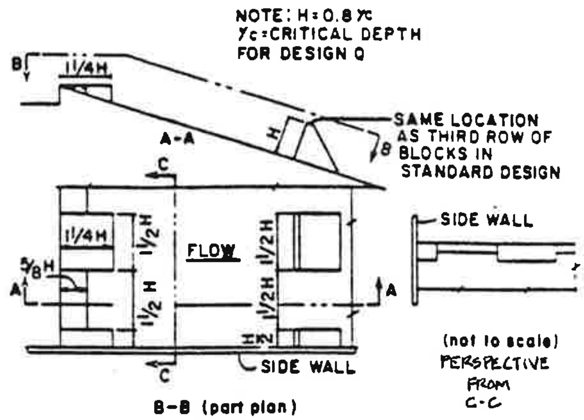
(A) USBR ISOMETRIC



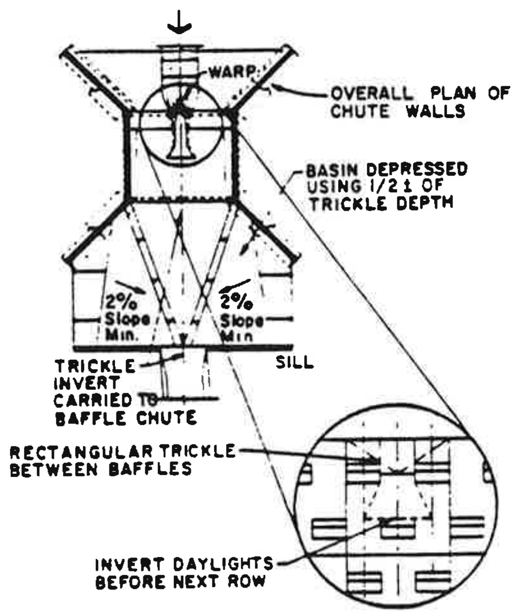
(B) DESIGN CRITERIA



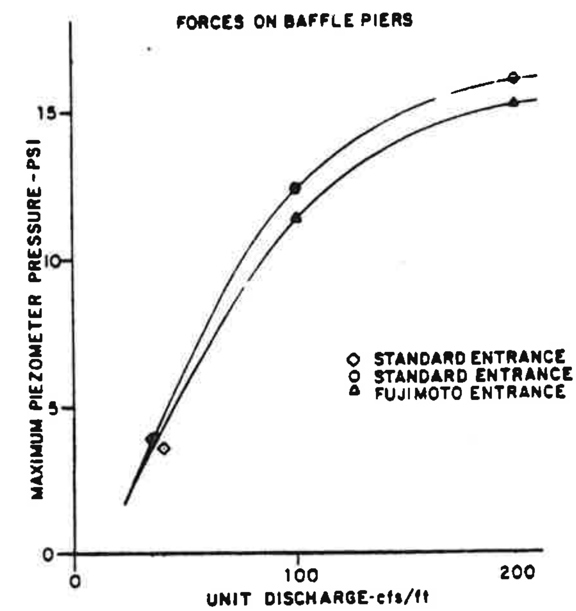
(a) EFFECT OF BLOCK LOCATION ON HEADWATER ELEVATION



(b) FUJIMOTO ENTRANCE MODIFICATION



(c) DETAILS FOR TRICKLE CHANNEL AT CREST AND BASIN MODIFICATIONS



(d) FORCES ON BAFFLES

2. An upstream channel transition section with vertical wing walls, constructed 45 degrees to the flow direction, causes flow approaching the rectangular chute section to constrict. It is also feasible to use walls constructed at 90 degrees to the flow direction. In either configuration, it is important to analyze the approach hydraulics and water surface profile. Often, the effective flow width at the critical cross section is narrower than the width of the chute opening due to flow separation at the corners of the abutment. To compensate for flow separation, it is recommended that the actual width constructed be one foot wider than the design analysis width if the constricted crest width is less than 90 percent of the upstream channel flow width. In any case, the design should carefully consider the approach hydraulics and contraction/separation effects. Depth and approach velocities should be evaluated through the transition to determine freeboard, scour and sedimentation zones.

3. The entrance transition is followed by a rectangular flow alignment apron, typically 5 feet in length. The upstream approach channel velocity, V, should be as low as practical and less than critical velocity at the control section of the crest. Figure 8.6b gives the USBR recommended entrance velocity. In a typical channel, the entrance transition to the rectangular chute section will produce the desired upstream channel velocity reduction. The elevated chute crest above the channel elevation, as shown in Figure 8.6a, should only be used when approach velocities cannot be controlled by the transition. Special measures to prevent aggradation upstream would be necessary with the raised crest configuration.

Entrance Modification:

1. The trickle flow channel should be maintained through the apron, approach and crest sections. It may be routed between the first row of baffle piers. The trickle channel should start again at the basin rock zone which should be slightly depressed and then graded up to transition to the downstream channel. Figure 8.7c illustrates one method of designing the low flow channel through the crest.

2. The conventional design shown in Figure 8.6b results in the top of the baffles being higher than the crest, which causes a higher backwater effect upstream. Figure 8.7a may be used to estimate extent the of the effect and to determine corrective measures, such as increasing the upstream freeboard or widening the chute.

Another means of alleviating these problems is the Fujimoto entrance, developed by the USBR and illustrated in Figure 8.7b. The upper rows of baffles are moved one row increment downstream. The important advantage of this entrance is that there is no backwater effect of the baffles. The serrated treatment of the modified crest begins disrupting flow entering the chute without increasing the headwater. More importantly, this configuration provides a level crest control. Therefore, the invert of the upstream low flow channel may be adjusted to the crest elevation, widening the low flow channel as it approaches the crest, or the low flow channel may be lower and brought through the serrated crest as described above.

Structural Design Dimensions:

1. Assume critical flow at the crest and determine critical depth for both the peak flow and for 2/3 of peak flow. For unit discharges exceeding 60 cfs/ft, Figure 8.7b may be extrapolated. The unit critical depth associated with a discharge in a rectangular chute may be determined by Equation 8.3:

$$y_c = \left(\frac{q^2}{g}\right)^{0.33} \tag{8.3}$$

2. The chute section (baffled Apron) is concrete with baffles of height, H_b, equal to 0.8 times the critical depth. The chute face slope is 2:1 for most cases, but may be reduced for low drops or where a flatter-slope is desirable. For unit discharges greater than 60 cfs/ft, the baffle height may be based on 2/3 of the peak flow; however, the chute side walls should be designed for peak flow (see 4 below).

   Baffle pier widths and spaces should be equal, preferably about $1.5H_b$ but not less than H_b. The spacing between rows of baffle blocks should be H_b times the slope. For example, a 2:1 slope makes the row spacing equal to $2H_b$ parallel to the chute floor. The baffle piers are usually constructed with the upstream face normal to the chute floor.

3. Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. At least one row of baffles are buried in riprap where the chute extends below the downstream channel grade. Riprap protection continues from the chute outlet to a distance of approximately $4H_b$, or as necessary to prevent eddy currents from undermining the walls. Additional rows of baffles may be buried below channel grade to allow for downstream channel degradation.

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

5. Determine upstream transition and apron side wall height as required by backwater analysis. Lower basin wing walls are generally constructed normal to the chute side walls at the chute outlet to prevent eddy current erosion at the drop toe. These transition walls are of a height equal to the channel normal depth plus 1 foot, and length sufficient to inhibit eddy current erosion.

6. All concrete walls and footing dimensions are determined by conventional structural methods.

7. The most troublesome aspect of the design is the determination of the hydraulic impact forces in the baffles to allow the structural engineer to size adequate steel reinforcing. Figure 8.7d may be used as a guideline. The structural engineer should apply a conservative factor of safety, as the curve is based on relatively sparse information.

8.3.2 Vertical Hard Basin Drops

The vertical hard basin is a generalized category which can include a wide variety of structure design modifications and adaptions. A variety of components can be used for both the hard basin and the wall, various contraction effects can be implemented to reduce approach velocities and different trickle channel options can be selected.

The maximum vertical drop height from crest to basin for a vertical hard basin drop is limited to 3 feet.

With sufficient tailwater, a hydraulic jump is initiated. Otherwise, the flow continues horizontally in the supercritical regime until the specific force of the tailwater is sufficient to force the jump. Energy is dissipated in the turbulence through the hydraulic jump; therefore the basin is sized to contain the supercritical flow and the erosive turbulent zone.

Generally, a rough basin is advantageous since increased roughness will result in a shorter basin. Figure 8.8 shows a vertical drop with a grouted boulder basin, and illustrates several more important design considerations.

General Hydraulic Design Procedure:

1. The design approach uses the unit discharge in the main channel and the trickle channel to determine the separate water surface profiles and jump locations in these zones. The basin is sized to adequately contain the hydraulic jump and associated turbulent flows.

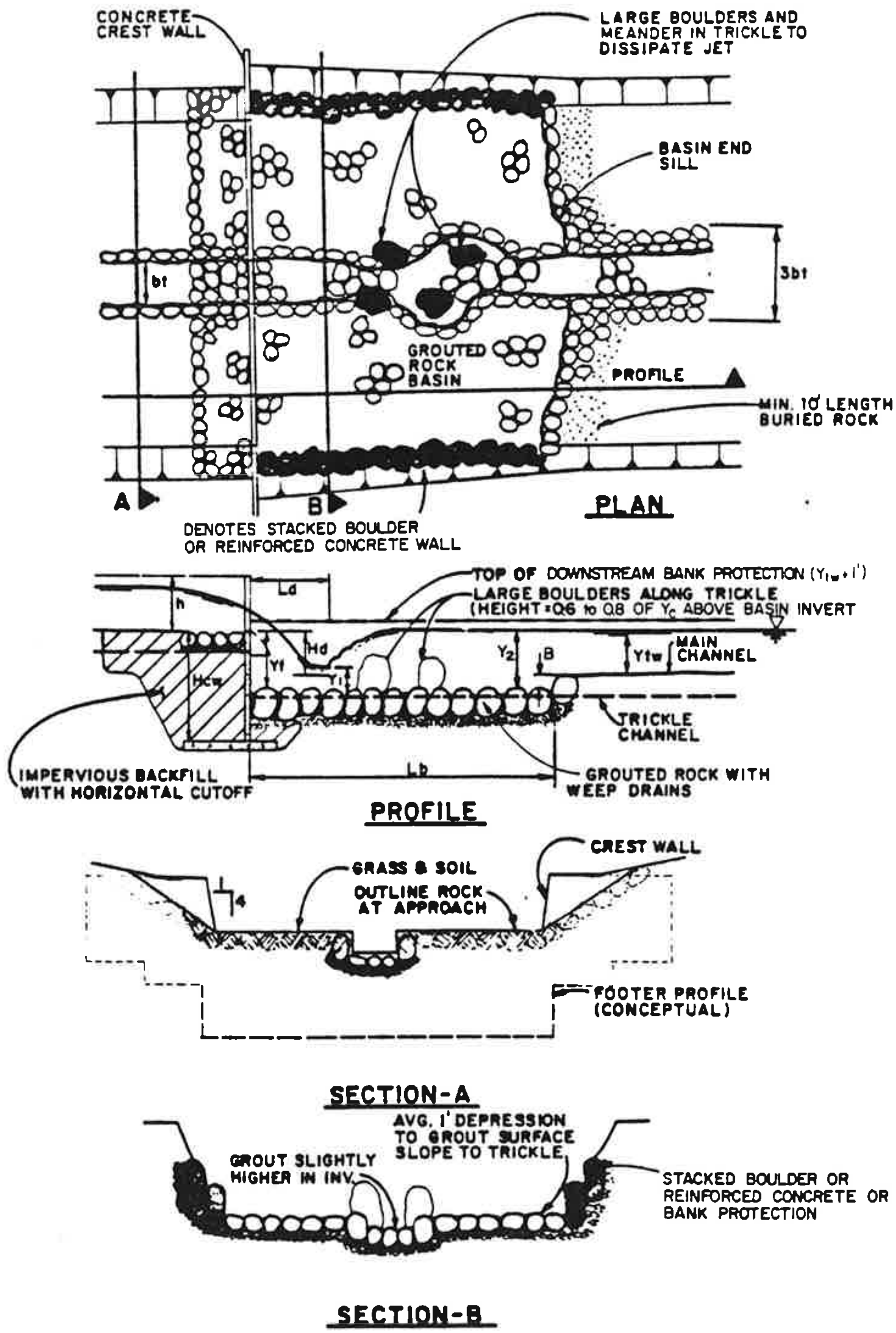


Figure 8.9
Vertical Hard Basin Drop

2. The rock lined approach length ends abruptly at the structural retaining crestwall which has a nearly rectangular cross-section and trickle channel section.

3. Crest wall and footing dimensions are determined by conventional structural methods.

4. *Open Channel Hydraulics* (Chow 1959), makes a brief presentation for the "Straight Drop Spillway", which applies here. Separate analysis would need to be undertaken for the trickle channel area and the main channel area as previously discussed. Add subscript *t* for the trickle channel area, and subscript *m* for the main channel area in the following equations.

   Refer to Figure 8.9 to identify the following parameters. L_b is the design length which includes L_d and the distance to the jump, D_j, which is measured from the downstream end of L_d. The jump length, L_j, is approximated as six times the sequent depth, Y_2. As a factor of safety, to assure a sufficient length for L_b, $0.6L_j$ is added in the design of L_b, such that

$$L_b \geq L_d + D_j + 9.6 Y_2 \qquad \textbf{(8.4)}$$

   When a hydraulic jump occurs immediately where the napp hits the basin floor, the following variables are defined:

$$\frac{L_d}{Y_f} = 4.3 D_n^{\ 0.27} \qquad \textbf{(8.5a)}$$

   Where

$$D_n = \frac{9_c^2}{(g Y_f^{\ 3})} \qquad \textbf{(8.5b)}$$

   and

$$\frac{Y_p}{Y_f} = 1.0 D_n^{\ 0.22} \qquad \textbf{(8.6)}$$

$$\frac{Y_1}{Y_f} = 0.54 D_n^{\ 0.425} \qquad \textbf{(8.7)}$$

$$\frac{Y_2}{Y_f} = 1.66 D_n^{0.27} \tag{8.8}$$

5. In the case where the tailwater does not provide a depth equivalent to or greater than Y_2, the jet will wash downstream as supercritical flow until its specific force is sufficiently reduced to allow the jump to occur. Determination of the distance to the hydraulic jump, D_j, requires a separate water surface profile analysis for the main and low flow zones. Any change in tailwater affects the stability of the jump in both locations.

6. The basin floor elevation is depressed at depth B, variable with drop height and practical for trickle flow drainage. Note the basin depth adds to the effective tailwater depth. The basin is constructed of concrete or grouted rock.

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

8.3.3 Vertical Riprap Basin Drops

As shown in Figure 8.11, this structure is essentially a plunge pool drop that incorporates a reinforced concrete crest wall with a riprap lined dissipation pool below. A nearly rectangular crest section is recommended to reduce the width of the plunge pool.

Maximum drop depth is limited to three (3) feet due to safety considerations and the practicality of obtaining large basin riprap for higher drops.

In this structure, flow passing over the vertical crest wall plunges into a riprap basin area. Energy is dissipated by turbulence in the plunge pool. Submergence by high tailwater can limit the ability of the structure to dissipate energy.

Loose riprap is placed according to the initial design specifications. The rock is successively rearranged by inflows until a more stabilized plunge pool is formed. The depth of the scour hole, d_s, and the nominal rock size are inversely related.

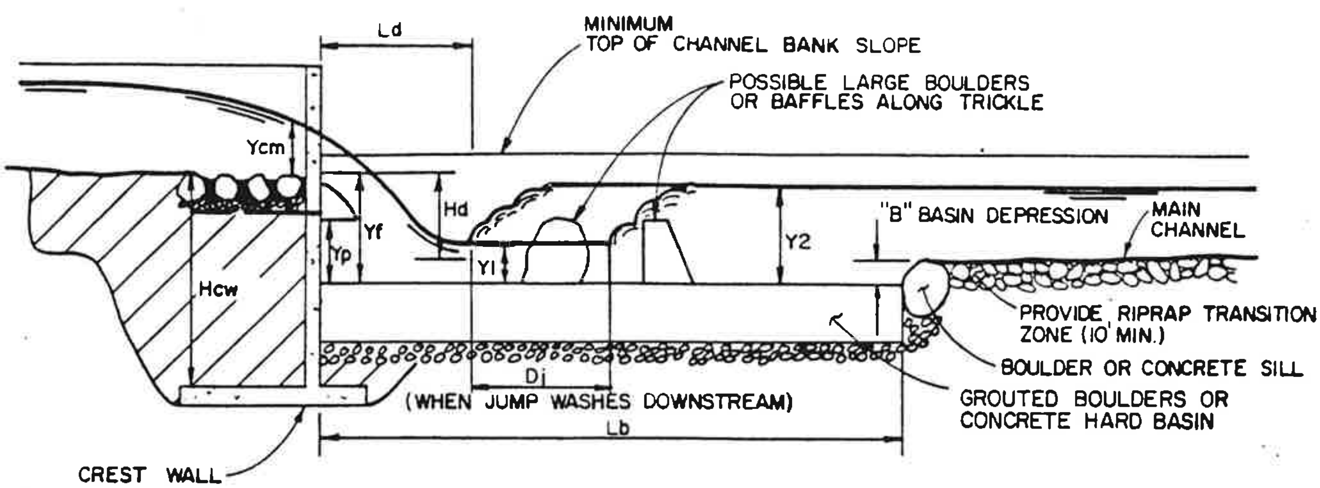
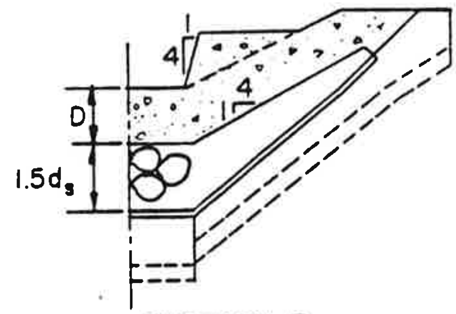
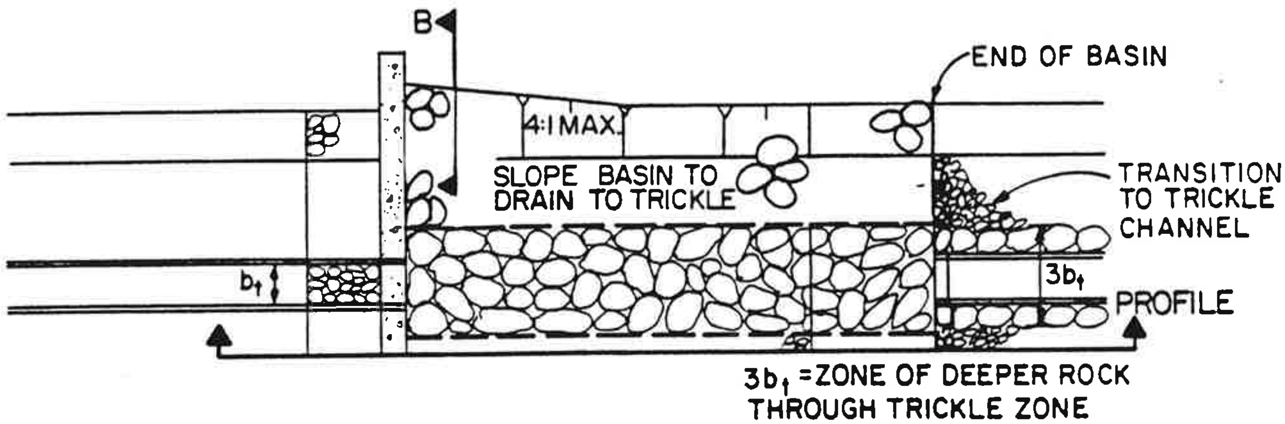


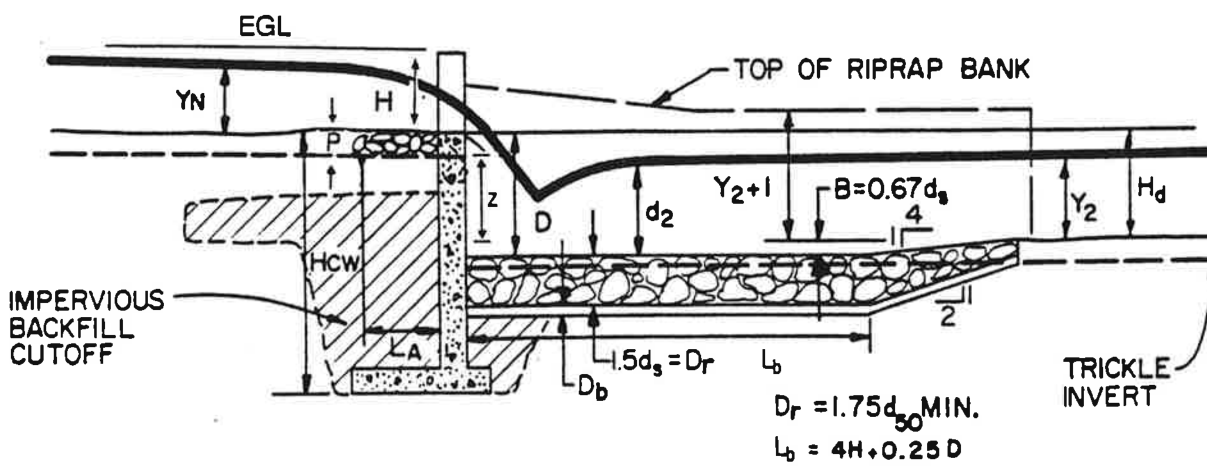
Figure 8.9
Vertical Drop Hydraulic System



SECTION B



PARTIAL PLAN



PROFILE

Figure 8.10
Vertical Riprap Basin Drop

General Hydraulic Design Procedure: The hydraulic analysis of this type of drop is generally similar to that presented in Section 8.3.2 for crest hydraulics. The design of the flexible plunge pool is described below.

The desired drop across the structure is the difference in the bed elevations of the approach channel at the weir and the downstream channel at the end of the structure. Let this difference be H_d. It follows from Figure 8.10 that:

$$H_d = D - 0.67 d_s \qquad \textbf{(8.9)}$$

The designer must find the combination of rock size and plunge height D that gives a depth of scour which balances Equation 8.9. The relationship between rock size d_{50}, plunge height, H, (H=1.5Yc) and depth of scour d_s is given by Figure 8.11.

To obtain an adequate cutoff, the depth of the vertical wall that forms the weir crest must be extended below the bottom of the excavation for the riprap. Thus, it usually becomes uneconomical to design a scour depth d_s, any greater than 0.3D. To meet this limitation in the field it is necessary to: increase the rock size d_{50}; decrease the plunge height D (by using more drops); decrease H (by using a wider structure); or, to use another type of drop structure.

The side slopes in the basin must be riprapped also as there are strong back currents in the basin. Granular filter material is required under the riprap. The side slopes in the basin should be the same slope as for the downstream channel.

8.3.4 Sloping Concrete Drops

The hydraulic concept of these structures is to dissipate energy by formation of a conventional hydraulic jump, usually associated with a reverse current surface flow as the supercritical flow down the face converts to subcritical flow downstream.

Numerous concepts have been investigated. Among them are the Saint Anthony Falls (SAF) Stilling Basin,and the USBR Basins I, II, III, and IV. These drops and associated basins are suited for different kinds of situations. The Saint Anthony Falls Stilling Basin and the USBR Basins (except Type I) all work at techniques to shorten the basin length.

Figure 8.12 illustrates the various types of stilling basins for use with sloping concrete drops.

General Hydraulic Design Procedure: Design procedures for USBR Basins II, III, and IV and the SAF Stilling Basin are presented in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, HEC-14, 1983) and *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka, 1984).

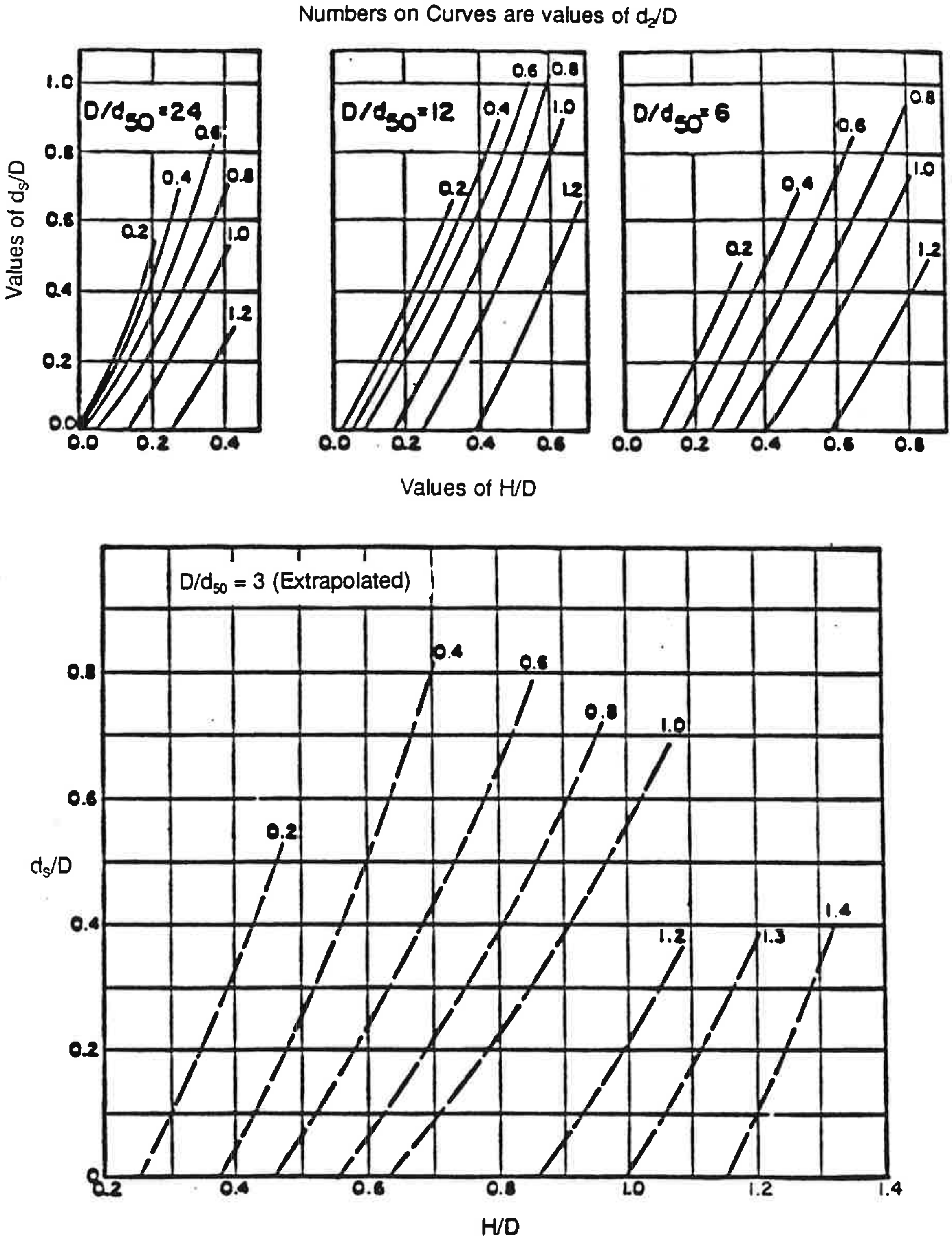
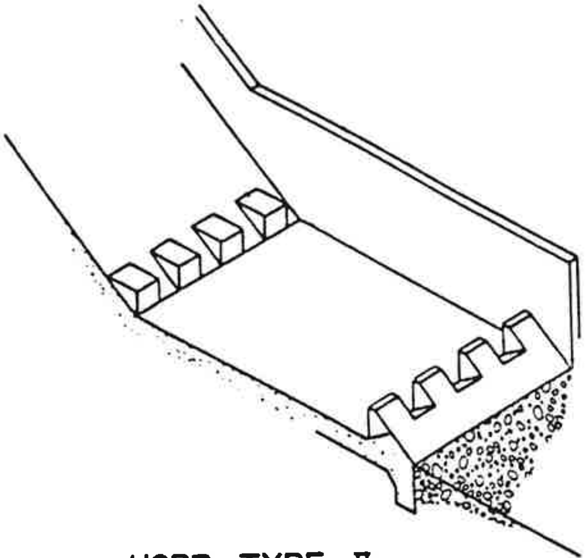
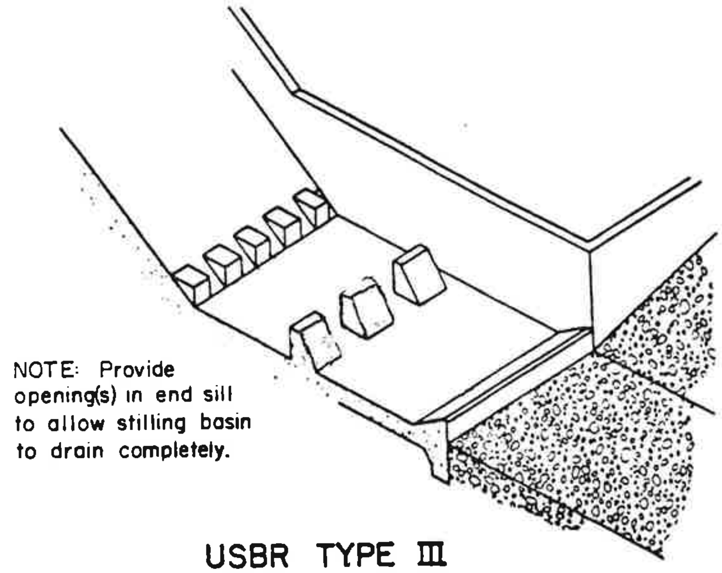


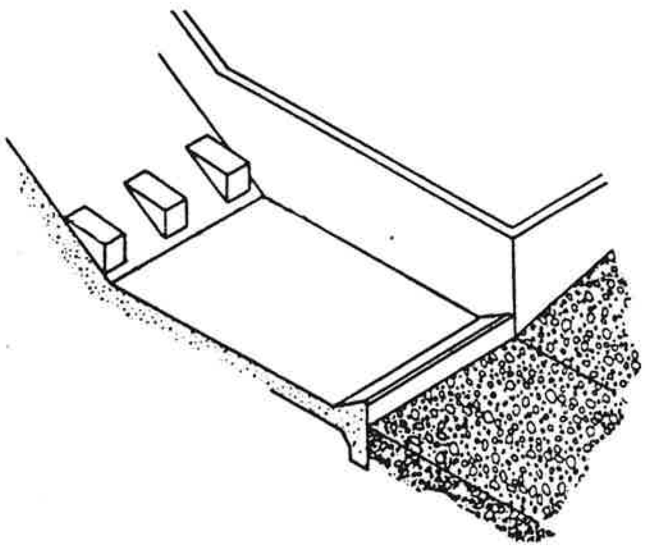
Figure 8.11
Curves for Scour Depth at Vertical Drop



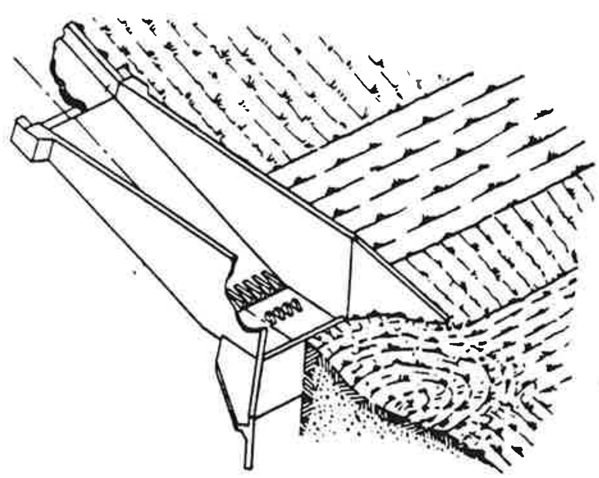
USBR TYPE II



USBR TYPE III



USBR TYPE IV



SAF STILLING BASIN

Figure 8.12
Stilling Basins for Sloping Concrete Drops

Analysis of channel approach and crest hydraulics generally follows the guidelines presented in Section 8.2.

8.3.5 Low Flow Check Structures

Low flow check structures and associated erosion control techniques can be effective in stabilizing natural channels and other unlined channels. These structures are designed to provide control points and establish/maintain stable bed slopes within the base flow channel. Check structures are frequently submerged during higher flood events. The application and sizing is complex because of the need to address a wide range of flows.

8.4 CONDUIT OUTLET STRUCTURES

Concrete energy dissipation or stilling basin structures are required to prevent scour damage caused by high exit velocities and flow expansion turbulence at conduit outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Concrete outlet structures can be designed easily and are suitable for a wide variety of site conditions. In some cases, they are more economical than large rock basins, particularly where long term costs are considered.

8.4.1 Riprap Protection at Conduit Outlets

A stilling basin constructed of loose, graded riprap can be an effective and economical energy dissipation measure for a conduit outlet. Long term analysis and observation of operating characteristics have lead to the following conclusions.

- The depth (h_s), length (l_s) and width (W_s) of the scour hole were related to the characteristic size of riprap (d_{50}), discharge (Q), brink depth (Y_o) and tailwater depth (TW).

- The dimensions of the scour hole in a basin constructed with angular rock were approximately the same as the dimensions of a scour hole in a basin constructed of rounded material when rock size and other variables were similar.

- When the ratio of tailwater depth to brink depth (TW/Y_o) was less than 0.75 and the ratio of scour depth to size of riprap (h_s/d_{50}) was greater than 2.0, the scour hole functioned very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and flow was generally well dispersed as it left the basin.

- The mound of material which formed on the bed downstream of the scour hole contributed to the dissipation of energy and reduced the size of the scour hole.

- For high tailwater basins (TW/Y_o greater than 0.75) the high velocity core of water emerging from the culvert retained its jet-like character as it passed through the basin, and diffused in a manner very similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole was much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

General details of the basin are shown in Figure 8.13. Principal features of the basin are:

- The basin is pre-shaped and lined with riprap of median size d_{50}.

- The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation, h_s, below the culvert invert. Elevation h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} constructed at the outfall of the culvert if subjected to design discharge. The ratio of h_s to d_{50} of the material should be greater than 2.0 and less than 4.0.

- The length of the energy dissipating pool, L_s, is $10h_s$ or $3W_o$ whichever is larger. The overall length of the basin, L_b, is $15h_s$ or $4W_o$ whichever is larger.

General Hydraulic Design Procedure:

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that TW/Y_o is less than or equal to 0.75 for the design discharge.

2. For subcritical flow conditions, use Figure 8.14 or 8.15 to obtain Y_o/D, then obtain V_o by dividing Q by the area of flow associated with Y_o. D is the height or diameter depending on the culverts cross-section. If the culvert is on a steep slope, V_o will be the normal velocity obtained from the of Manning's Equation for the appropriate slope, section and discharge.

3. From site inspection or field experience in the area, determine whether or not channel protection is required at the culvert outlet.

4. If the channel protection is required, compute the Froude Number for brink conditions $y_c(\frac{A}{2})^{\frac{1}{2}}$ (for non-rectangular sections). Select d_{50}/y_c appropriate for locally available riprap (usually 0.25 is less than d_{50}/y_c is less than 0.45). Obtain $h_s y_e$ from Figure 8.16, and check to see that 2.0 is less

than h_s/d_{50} is less than 4.0. This process is iterative, the computations should be adjusted until this value falls between 2.0 and 4.0.

5. Size the basin as shown in Figure 8.13.

6. Design procedures where allowable dissipator exit velocity is specified:

   - Determine the average normal flow depth in the natural channel for the design discharge.

   - Extend the length of the basin so that the width of the basin (at Section A-A, Figure 8.13), times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

7. In the exit region of the basin, the walls and apron of the basin should be warped so that the cross section of the basin at the exit conforms to the cross-section of the natural channel. Abrupt transitions should be avoided to minimize separation zones and resultant eddies.

8. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

   - Design a conventional basin for low tailwater conditions in accordance with the instructions above. Estimate centerline velocity at a series of downstream cross-sections using the information shown in Figure 8.17. Shape the downstream channel and size riprap using guidelines presented in Section 7 and the stream velocity obtained above.

8.4.2 Concrete Outlet Structures

Impact stilling basins and adaptation of a baffled apron to conduit outlets are viable alternatives for the dissipation of energy. Initial design selection should include at least the following aspects concerning conduit outlet structures.

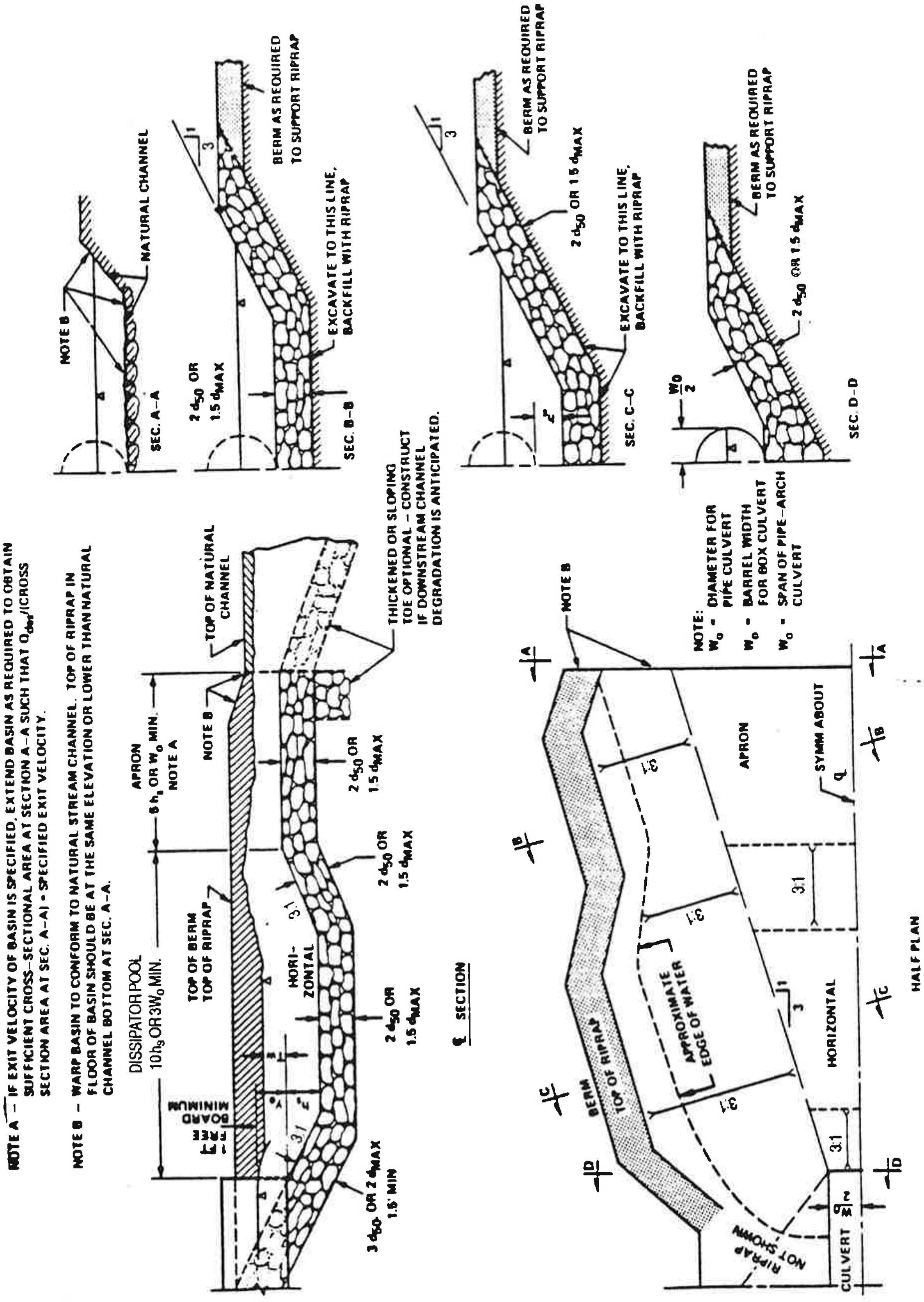
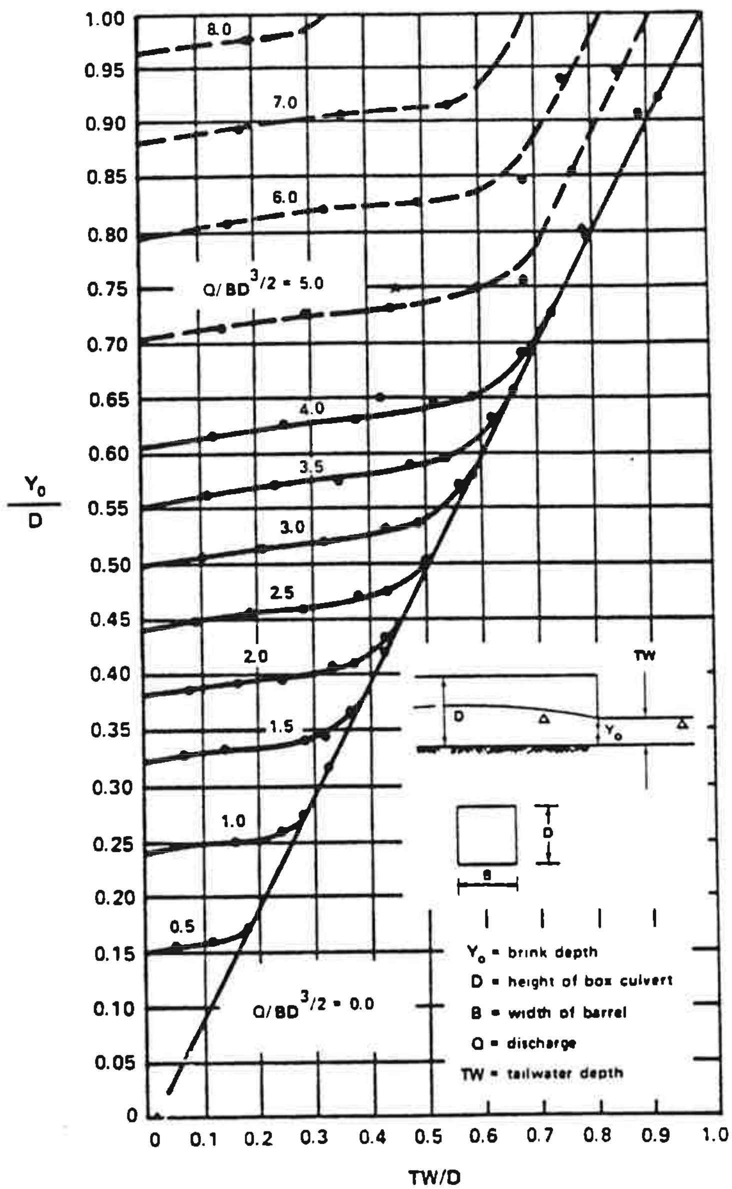


Figure 8-13
Details for Riprapped Culvert Energy Basin



Q/BD^3/2 = 5.0
Q/BD^3/2 = 0.0
8.0
7.0
6.0
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
Y_o / D
TW/D
TW
D
Y_o
B
Y_o = brink depth
D = height of box culvert
B = width of barrel
Q = discharge
TW = tailwater depth

Figure 8.14

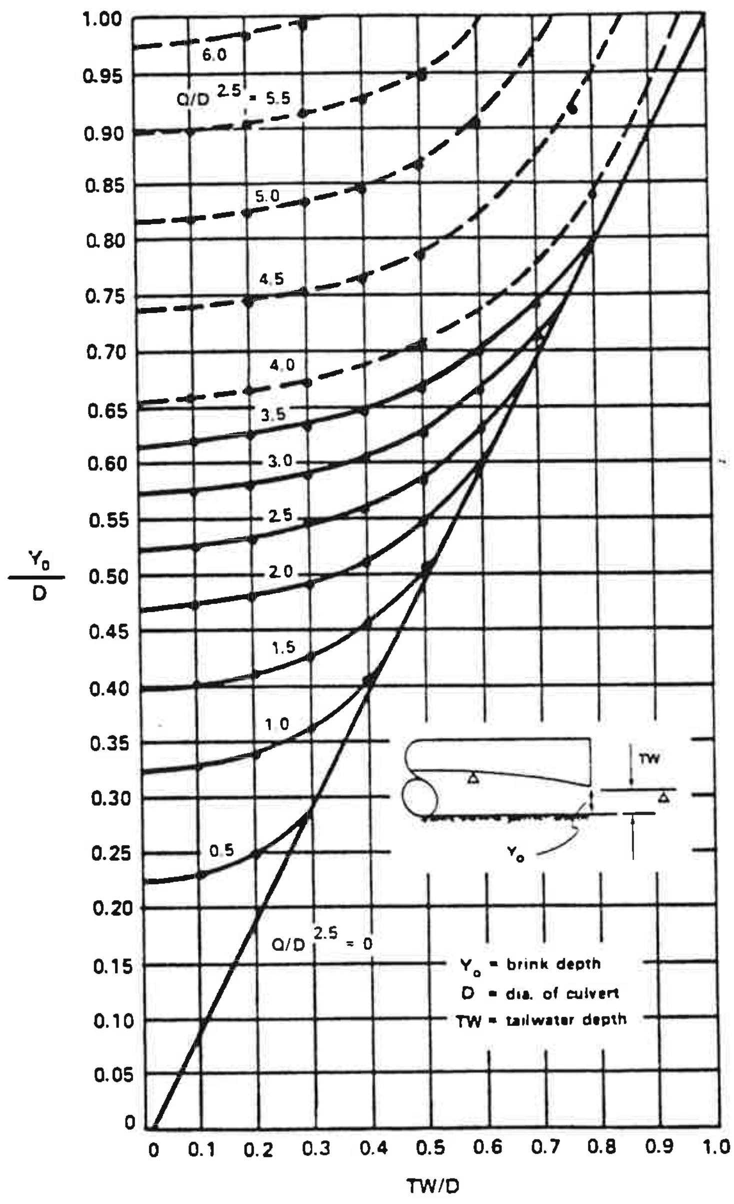


Figure 8.15
Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

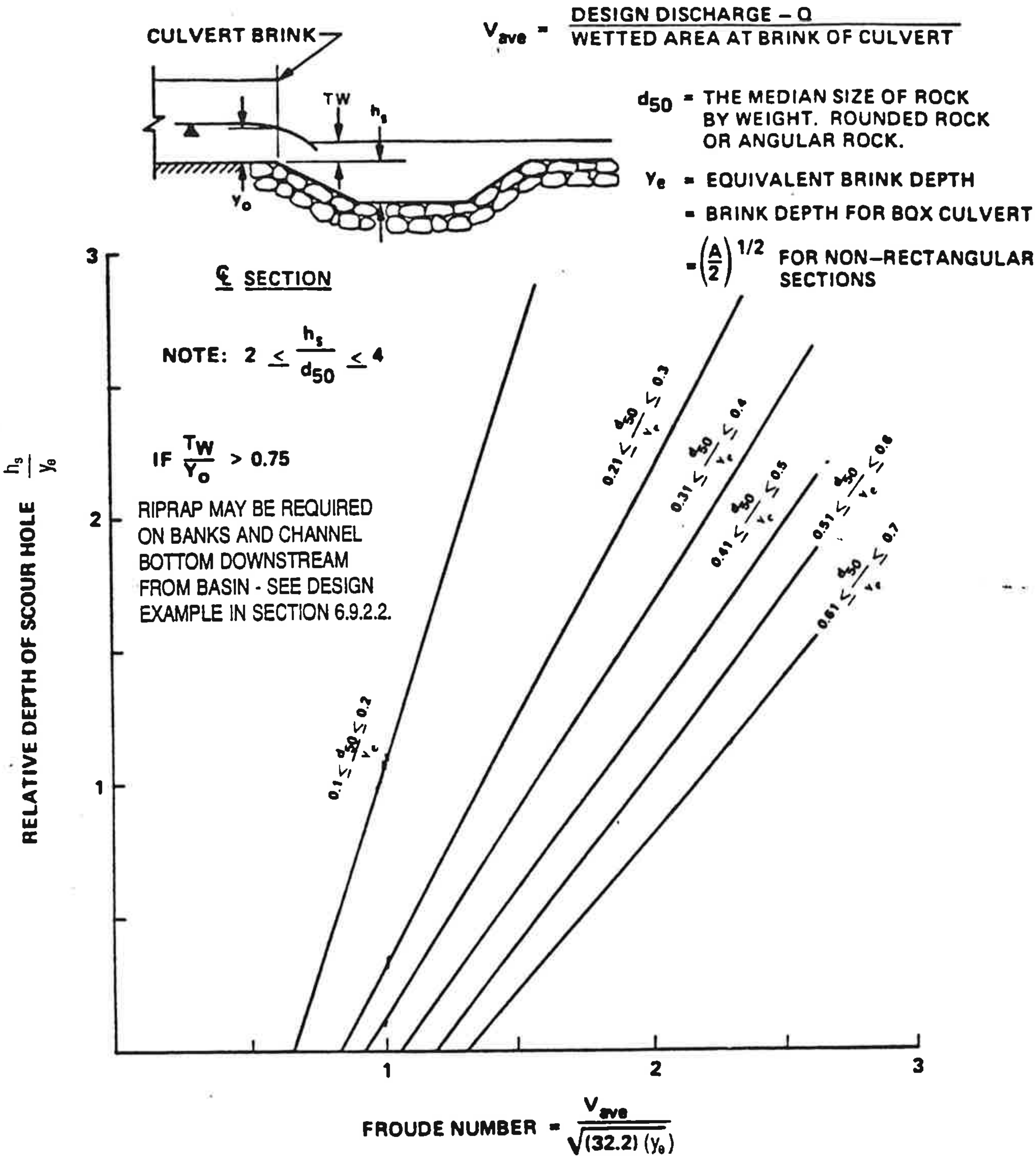


Figure 8.16
Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

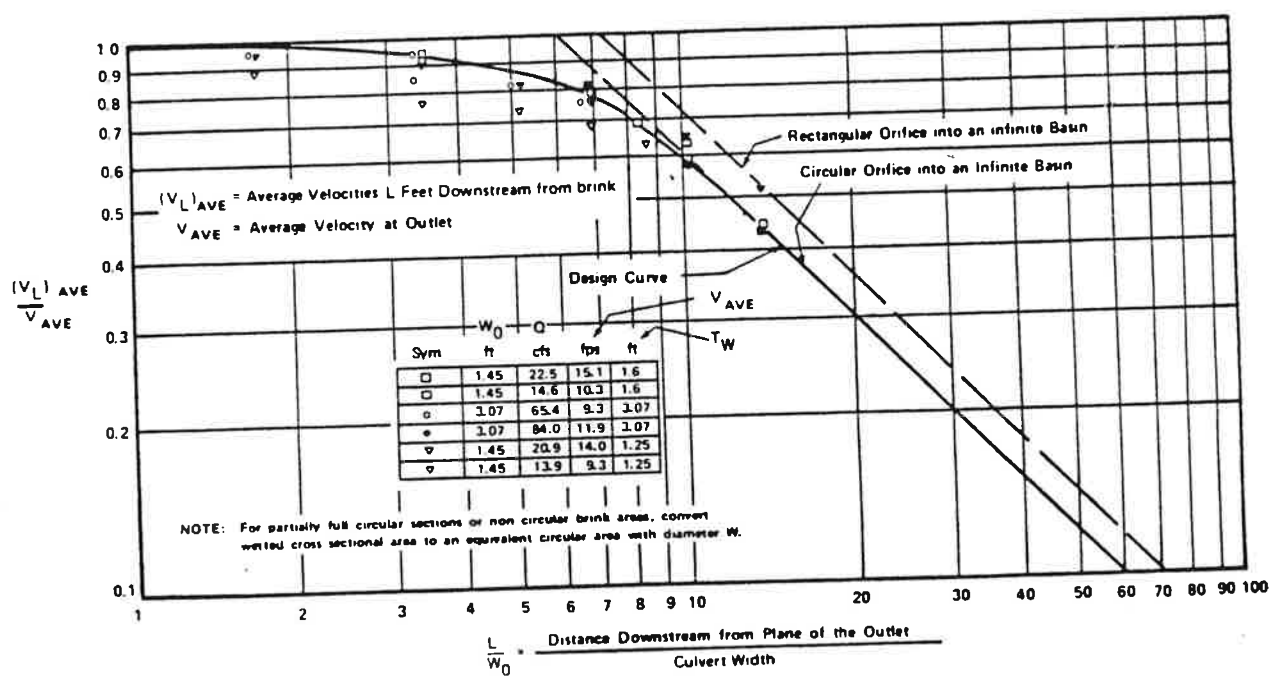


Figure 8.17
Distribution of Centerline Velocity for Flow
From Submerged Outlets
(to be used for Predicting Channel Velocities Downstream
from Culvert Outlets Where High Tailwater Prevails)

1. High energy dissipation is required - hydraulic conditions exceed the limits for alternative designs, such as riprap outlet protection.

2. Low tailwater control is anticipated. For example, at outfalls to detention/retention facilities that are empty or at low water levels.

3. Use of concrete is more economical due to structure size of local availability of materials.

4. Site conditions direct the use of an outlet structure such as public use areas where plunge pools and standing water are unacceptable or locations with severe space limitations.

8.4.3 Impact Stilling Basin

Design standards for this structure are based on the USBR Type VI Basin, commonly referred to as an impact dissipator or conduit outlet stilling basin. The Type VI Basin is relatively small structure which provides highly efficient energy dissipation characteristics without tailwater control.

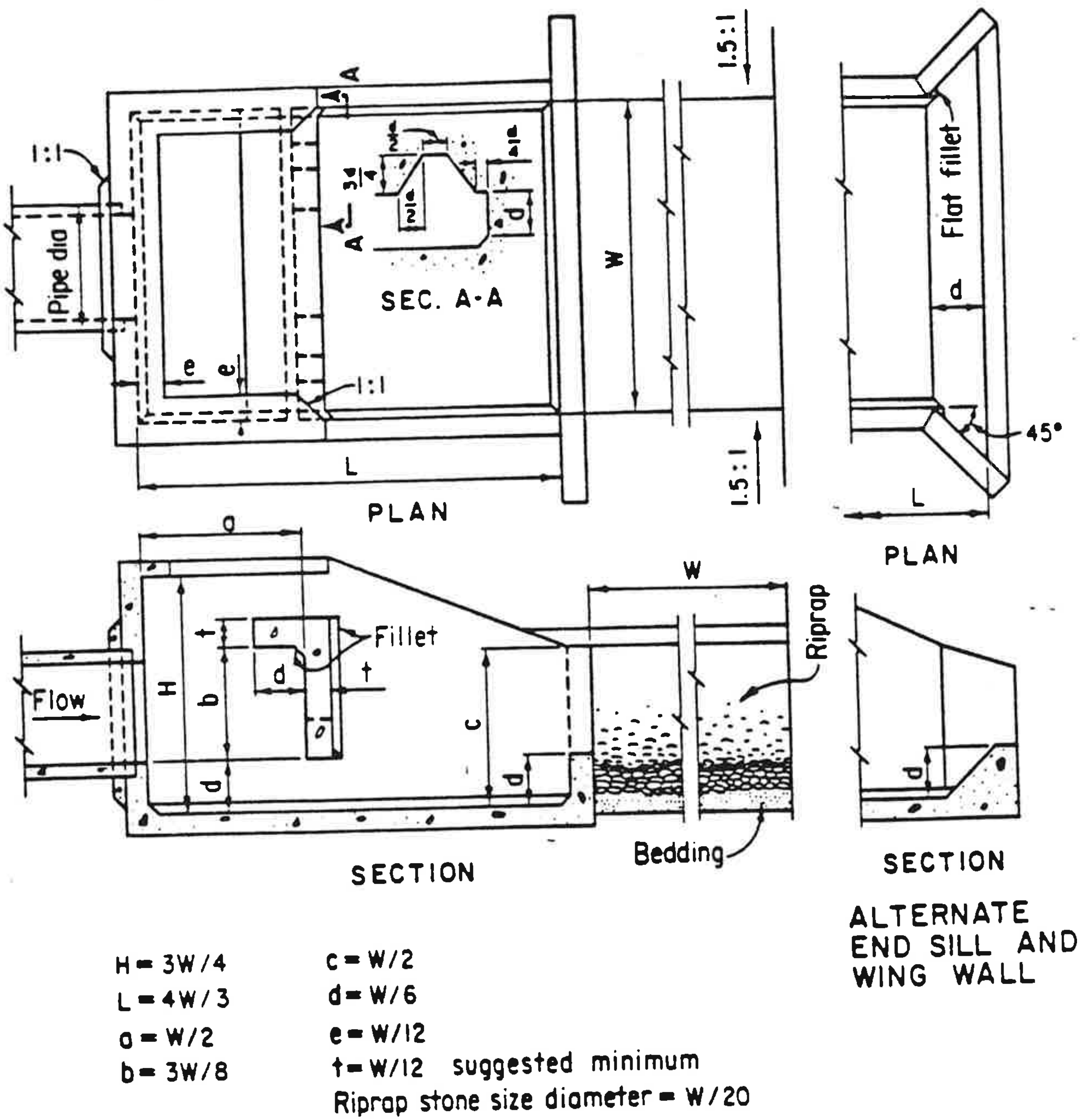
The structure is designed to operate continuously at the design flow rate. Maximum entrance conditions are up to 50 feet per second for velocity and a Froude number less than 9.0. Conditions exceeding this criteria would be extremely rare in typical urban drainage applications. As a result, the use of this outlet basin is limited only by structural and economic considerations. A generalized design configuration for this outlet basin is illustrated in Figure 8.18.

The standard USBR design has been modified for urban applications to allow drainage of the basin bottom during dry periods. The impact basin can also be adapted to multiple pipe installations. These modifications are discussed following the basic criteria. It should be noted that modifications to the design may effect the hydraulic performance of the structure.

General Hydraulic Design Procedure for Stilling Basins (see Figure 8.17):

1. Determine the design pipe flow rate, Q, and the effective flow area, A, at the outlet. For partial flow conditions, refer to the partial flow diagram in Section 8.6.6. Using the continuity equation determine the flow velocity at the pipe outlet. Assume the depth is equal to the square root of the area, A, and compute the Froude using Equation 7.3.

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



NOTE: See Figure 6.23 for W

Figure 8.18
General Design of the USBR Type VI
Impact Stilling Basin

3. Do not use this type of outlet energy dissipator when velocities exceed 50 feet per second or the Froude Number exceeds 9.0. These conditions would be extreme and must be considered a special case. Performance is achieved with a tailwater depth equal to half full flow level in the pipe outlet.

4. Determine the basin width (W) by entering the appropriate Froude Number and effective flow depth on Figure 8.19. The remaining dimensions are proportional to the basin width according to the legend on Figure 8.18. Note that the baffle thickness, t, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis.

   The basin width should not be increased since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.

5. Structure wall thickness, steel reinforcement and anchor walls should be designed using accepted structural engineering methods. Hydraulic forces on the overhanging baffle may be approximated by determination of the jet momentum force:

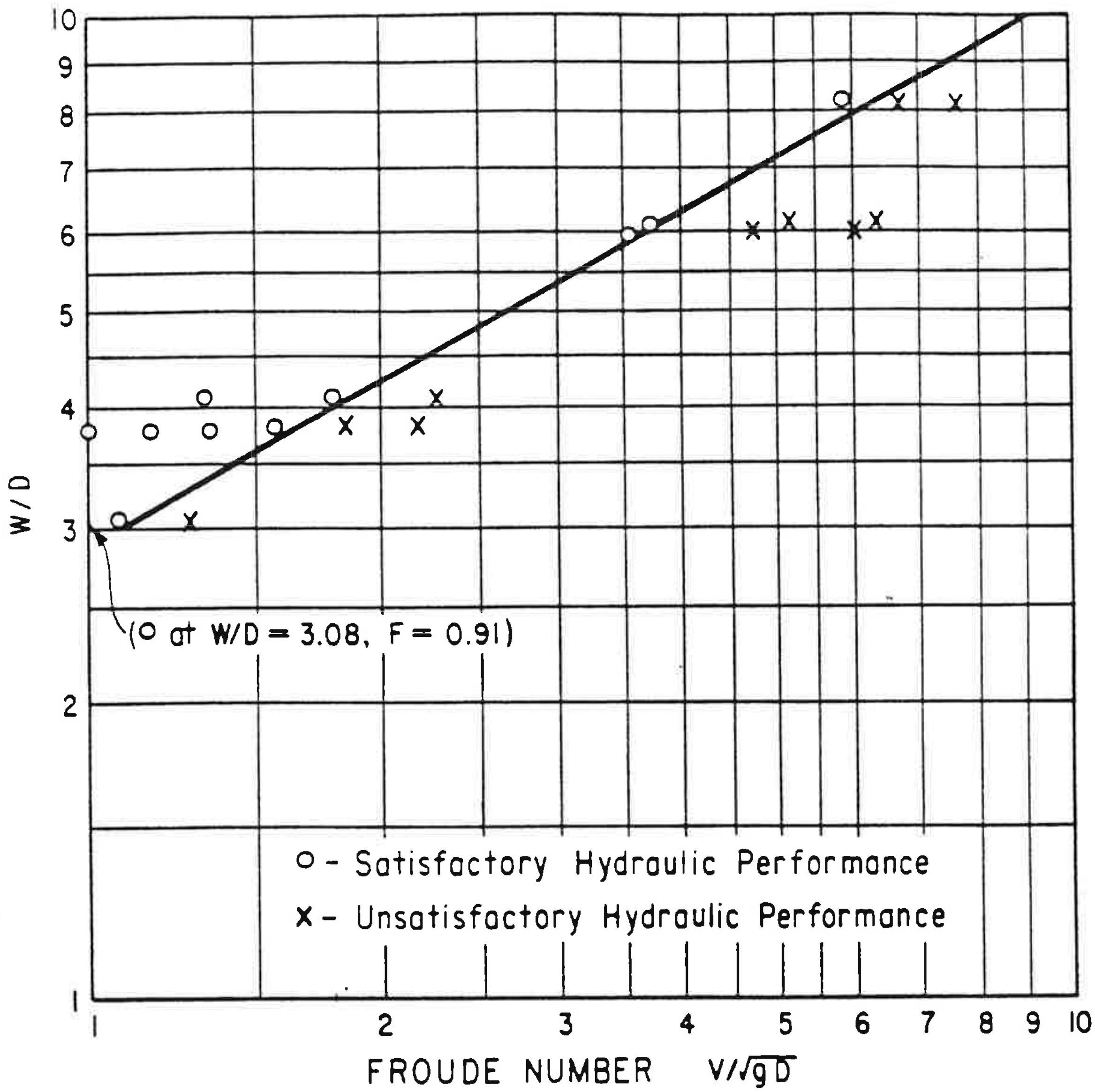
$$F_m = pVQ = 1.94\,VQ \qquad \textbf{(8.10)}$$

6. Riprap with a minimum D_{50} of 18 inches should be placed in the receiving channel from the end of the sill to a minimum distance equal to the basin width. The depth of rock should be equal to the sill height or at least 2.5 feet.

Low Flow Modifications: The standard design will retain a standing pool of water in the basin bottom which is generally undesirable from a safety and maintenance standpoint.

A low flow gap should be extended through the basin end sill wall. The gap in the sill should be as narrow as possible to minimize effects on the sill hydraulics. This implies that a narrow and deeper (1.5 to 2.0 feet) low flow channel will work better than a wider gap section. The low flow width should not exceed 60 percent of the pipe diameter to prevent the jet from short-circuiting through the cleanout notches.

The optimal configuration results in continuous drainage of the basin area and helps to reduce the amount of sediment entrapment.



"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 8.19
Design Width of the USBR Type VI Basin

1. For large basins where the sill height is greater than 2.0 feet, the depth dimension, d, (see Figure 8.18) may be reduced to avoid a secondary drop from the sill to the main channel. The low flow invert thereby matches the floor invert at the basin end and the main channel elevation is equal to the sill. Dimension d should not be reduced by more than one-third and not less than 2.0 feet. This implies that a deeper low flow channel (1.5 to 2.0 feet) will be advantageous for these installations.

   Note that dimension d is also reduced at the minimum pipe invert height and at the bottom of the baffle wall.

2. A sill section should be constructed directly in front of the low flow notch to break up bottom flow velocities. The length of this sill section should overlap the width of the low flow by about one foot. The general layout for the low flow modifications is shown in Figure 8.20.

Multiple Conduit Installations: Where more than one conduit of different sizes have outlets in close proximity, a composite structure can be constructed to take advantage of common walls. This can be somewhat awkward since each basin "cell" must be designed as an individual basin with different dimensions. Where two conduits of the same size have close outlets, the structures may be combined into a single basin as shown in Figure 8.20.

The total width of a dual inlet basin can be reduced to three-fourths of the total width for separate basins. For example, if the design width for each pipe is w, the combined basin width would be 1.5w. The remaining structure dimensions are based on the design width of a separate basin. If the two pipe have different flows, the combined structure should be based on the higher Froude Number flows.

8.4.4 Baffle Chute Energy Dissipator

The baffle chute has also been adapted for use at pipe outlets. This structure is particularly suited to situations with vary large conduit outfalls and at outfalls in which some future degradation is anticipated. Generally, this type of structure is only cost effective if a grade drop is necessary below the outfall elevation and a hydraulic backwater can be tolerated in the culvert design.

Figure 8.21 illustrates a general configuration for a baffled outlet for a double box culvert. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to achieve the flow velocity at the chute entrance as described in Section 8.3.1. The remaining hydraulic design is the same as for a standard baffle chute. 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 at the upstream row of baffles.

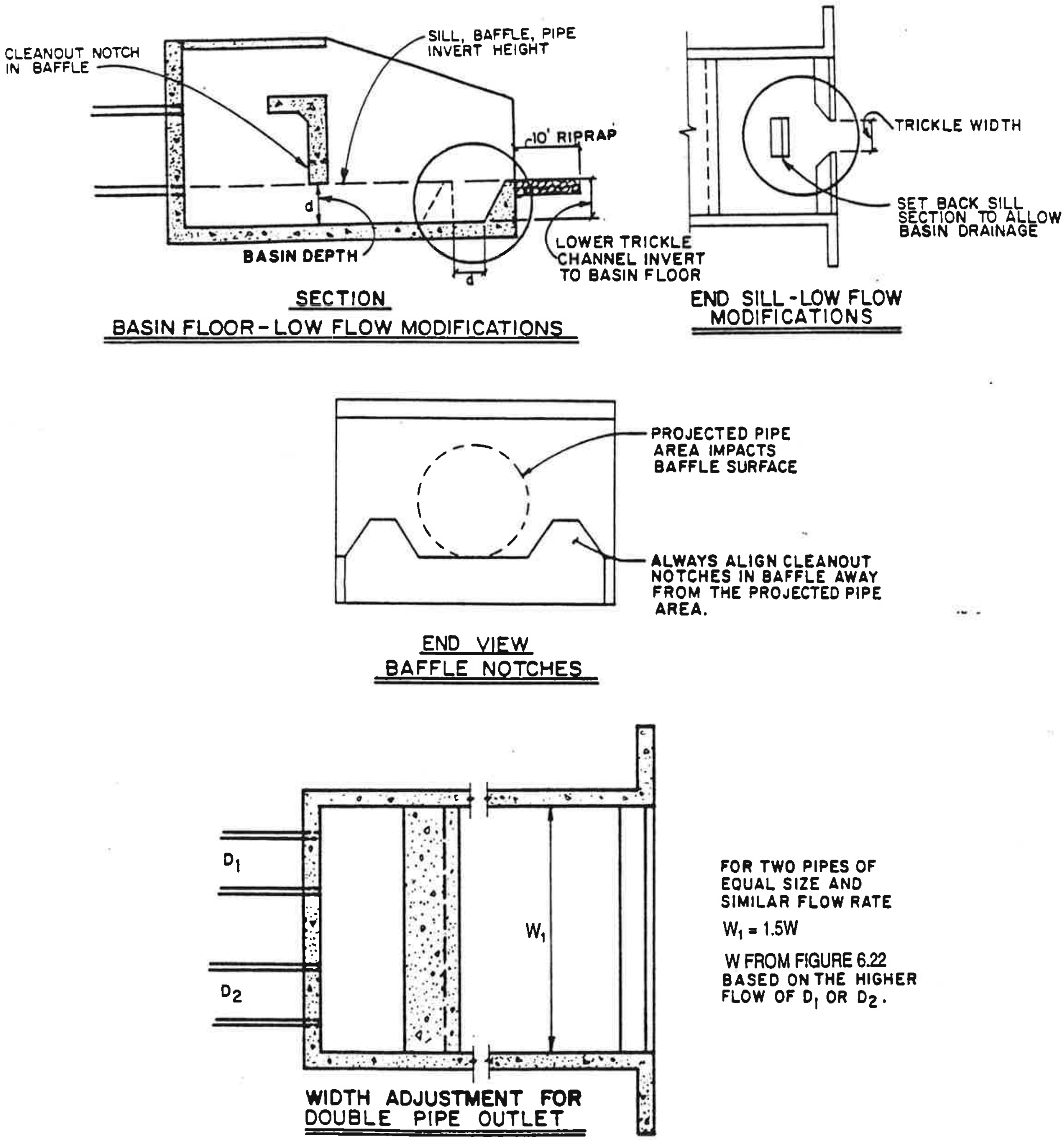


Figure 8.20
Modifications to Impact Stilling Basins
(to Allow Basin Drainage for Urban Application)

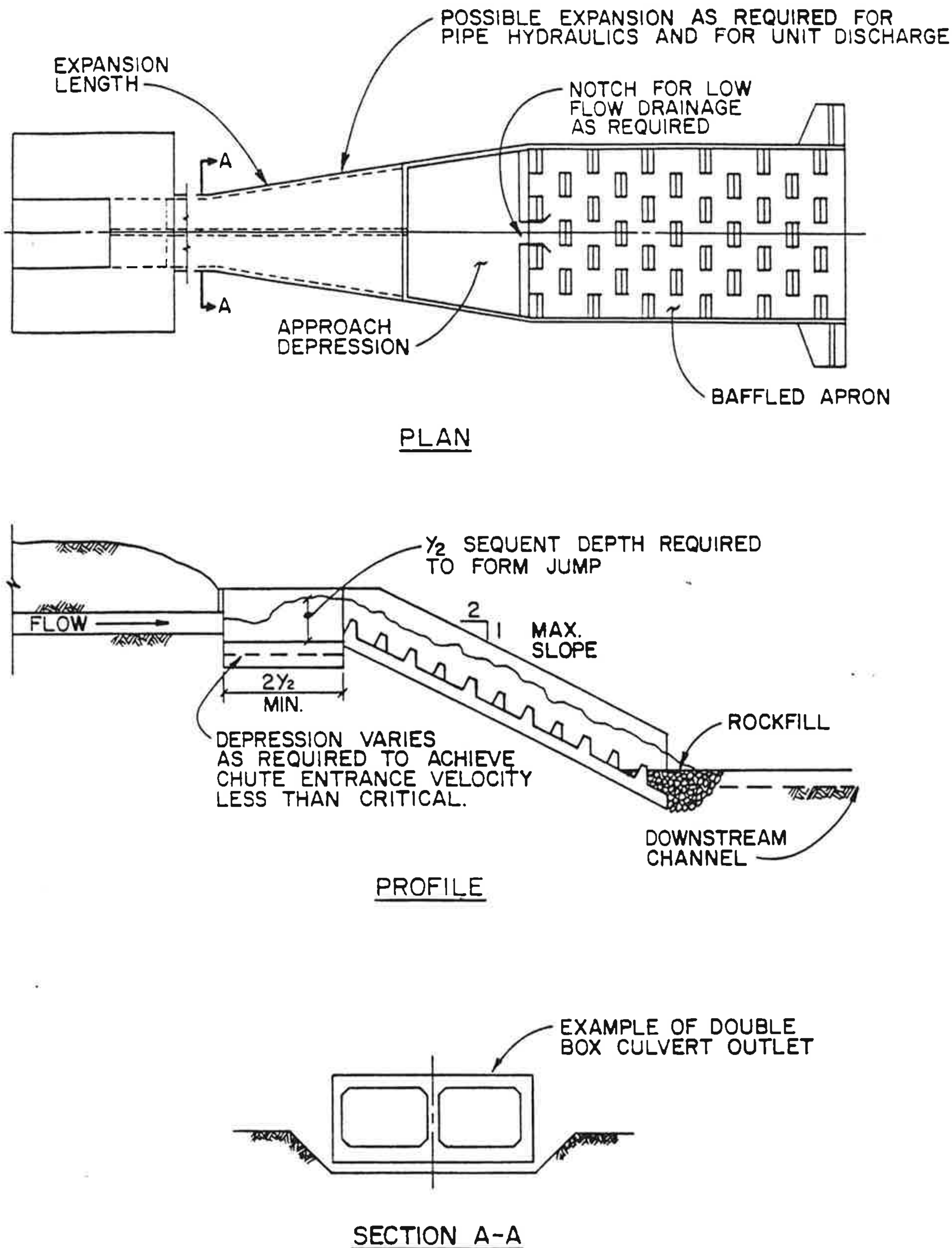


Figure 8.21
Baffle Chute at Conduit Outlet

8.5 SPECIAL CHANNEL STRUCTURES

8.5.1 Channel Transitions

A flow transition is a change of open channel flow cross section designated to be accomplished in a short distance with a minimum amount of flow disturbance. Specially designed open channel flow transitions are normally not required for highway culverts. A culvert is normally designed to operate with an upstream headwater pool which dissipates the approach velocity and, therefore, negates the need for an approach flow transitions.

Outlet transitions must be considered in the design of all culverts, channel protection and energy dissipators. Design considerations for subcritical flow channel transitions are presented in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, HEC-14, 1983).

8.5.2 Supercritical Flow Structures

Acceleration Chutes: Acceleration chutes, whether leading into box culverts, pipes or high velocity channels, are often used to reduce downstream cross sections, hence, reducing costs.

Acceleration chutes are potentially hazardous if inadequately designed (see USBR 1974). High velocity flow can wash out channels and structures downstream in short order, resulting in property damage and uncontrolled flow.

The reference cited above addresses chutes in greater detail than can be discussed in this manual. Refer to this publication for detailed analysis and design procedures.

Bends: Structures are generally unnecessary in subcritical flow channels unless the bend is of small radius. Structures for supercritical flows are complex and require careful hydraulic design to control flow. Bends are normally not used in supercritical flow channels because of the costs involved and the hazards introduced.

When a bend is necessary, and it is not practical to take the flow into the subcritical regime, the designer will generally conclude that the channel should be placed in a closed conduit for the entire reach of the bend, and downstream far enough to eliminate the main oscillations.

In bends, forces are usually larger than what is intuitively assumed. The momentum equation permits solution for the force acting upon the flow boundary at a bend.

$$F = M \Delta V \qquad \textbf{(8.11)}$$

where "delta v" represents the change in direction and/or magnitude of the velocity through the bend.

The force due to pressure on the bend should also be calculated when conduits flow under pressure.

$$\Delta P = \frac{P}{2}(\Delta (V)^2) \qquad \textbf{(8.12)}$$

where "delta P" represents the pressure change caused by the difference in the squares of the velocities through the bend. The total exerted force on the bend, the total of momentum and pressure forces, must be counteracted by external forces. Allowable soil bearing should be determined using soil tests if necessary. Forces which cannot be handle by conduit bearing in the soil must be compensated for by additional thrust blocks or other structures.

8.6 CHARACTERISTICS OF THE HYDRAULIC JUMP

With the exception of the baffle chute drop, all of the drop structures described in the previous sections use the formation of a hydraulic jump to dissipate energy.

A hydraulic jump occurs when flow changes rapidly from low stage supercritical to high stage subcritical. Hydraulic jumps can occur: 1) when the slope of a channel changes from steep to mild; 2) at sudden expansions or contractions in the channel section; 3) at locations where a barrier, such as a culvert or bridge, occurs in a channel of steep slope; 4) at the downstream side of dip crossings or culverts; and 5) where a channel of steep slope discharges into another channel.

The type of hydraulic jump that forms, and the amount of energy that it dissipates, is dependent on the Froude Number (F). The various type of hydraulic jumps that can occur are listed in Table 8.2

Table 8.2
Types of Hydraulic Jumps

Upstream Froude Number	Type of Jump	Energy Loss, %
$1 < F \leq 1.7$	Undular Jump	0.5
$1.7 < F \leq 2.5$	Weak Jump	5 to 18
$2.5 < F \leq 4.5$	Oscillating Jump	18 to 44
$4.5 < F \leq 9$	Steady Jump	44 to 70
$9 < F$	Strong Jump	70 to 85

8.6.1 Jump Height

The depth of flow immediately downstream of a hydraulic jump is referred to as the sequent depth. The sequent depth in rectangular channels can be computed using the following equation:

$$Y_{2D} = \frac{1}{2} Y_{1u} [(1 + 8F_1 2)^{1/2} - 1] \qquad \textbf{(8.13)}$$

The solution for the sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation 8.14, which is derived from momentum equations. It is also acceptable, for design purposes, to determine the sequent depth in trapezoidal channels from Equation 8.13. Equation 8.13 is much easier to solve and produces only slightly greater values for sequent depth than does Equation 8.14.

$$\frac{2Y^3_{1u}}{3} + \frac{bY^2_{1u}}{2} + \frac{Q}{qA_1} = \frac{2Y^3_{2b}}{3} + \frac{2Y^2_{2D}}{2} + \frac{Q}{gA} \qquad \textbf{(8.14)}$$

Figures 8.22 and 8.23 provide graphs of hydraulic jumps for a horizontal rectangular channel and a horizontal trapezoidal channel, respectively.

8.6.2 Jump Length

The length of a hydraulic jump is defined as the distance from the front face of the jump to a point immediately downstream of the roller. Jump length can be determined from Figures 8.24 and 8.25.

8.6.3 Surface Profile

The surface profiles of a hydraulic jump may be needed to design the profile of extra bank protection, or training walls for containment of the jump. The surface profile can be determined from Figure 8.26.

8.6.4 Jump Location

In most cases, a hydraulic jump will occur at the location in a channel where the initial and sequent depths and initial Froude Number satisfy Equation 8.13. This location can be found by performing direct step calculations in either direction toward the suspected jump location, until the terms of the are satisfied. Specific force analysis can then be used by employing Equation 8.1 to establish where a jump will occur. The hydraulic jump will begin to form where the unit specific force of the downstream tailwater is greater than the force of the supercritical approach flow.

8.6.5 Undular Jump

An undular hydraulic jump is the type of jump which occurs where the upstream Froude Number in between 1.0 and 1.7. This type of jump is characterized by a series of undular waves which form on the downstream side of the jump. Experiments have shown that the first wave of an undular jump is higher than the height given by Equation 8.13. Therefore, the height of this wave should be determined as follows:

$$(Y_2D - Y_1u) \ / \ Y_1u = F^2{}_1 - 1 \qquad \textbf{(8.15)}$$

where all terms are previously defined.

8.6.6 Design Aids

In addition to the Tables and Figures previously referenced, Table 6.3 has been included as an additional aid to the user of this manual.

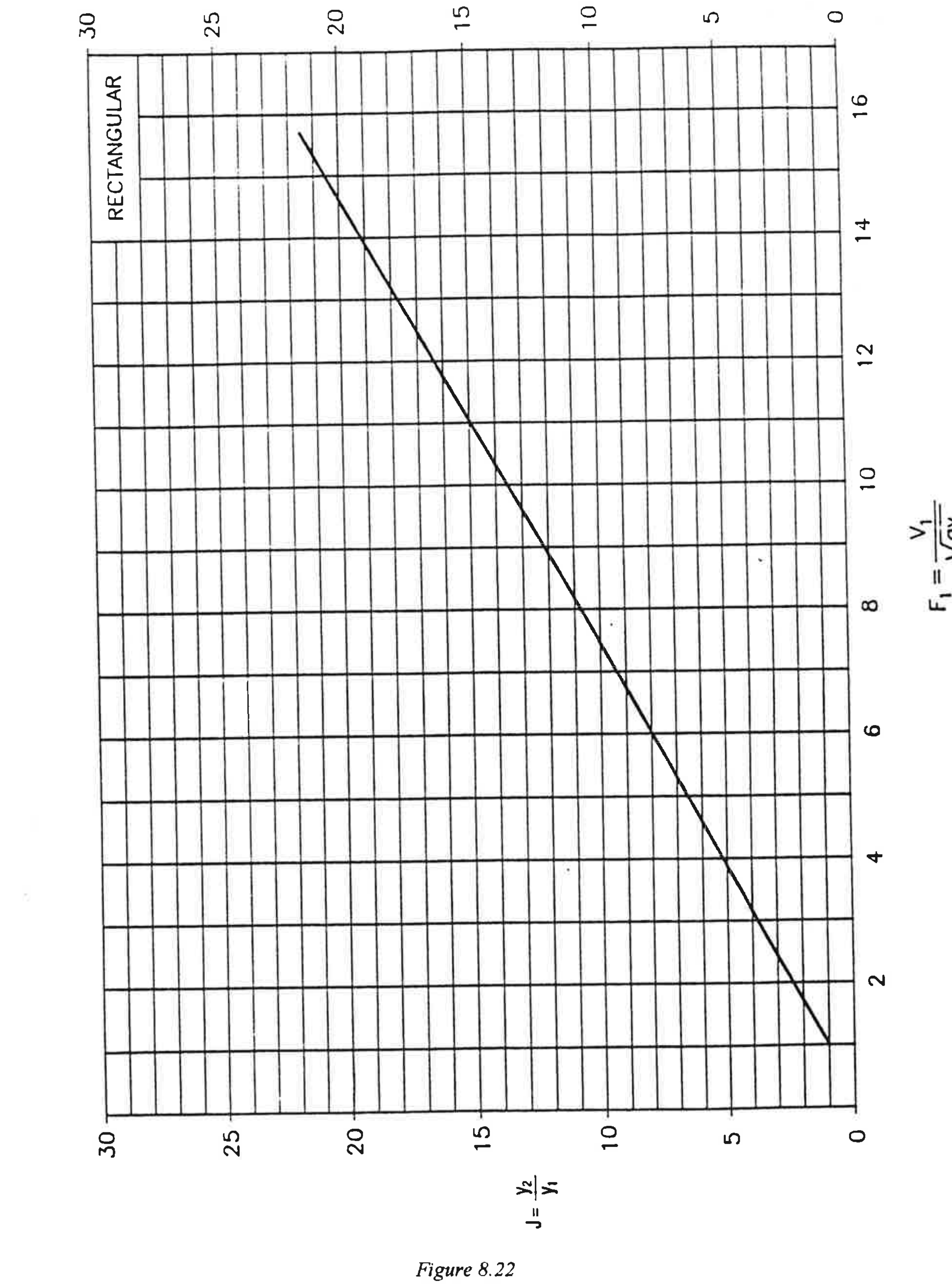


Figure 8.22
Height of a Hydraulic Jump for a
Horizontal Retangular Channel

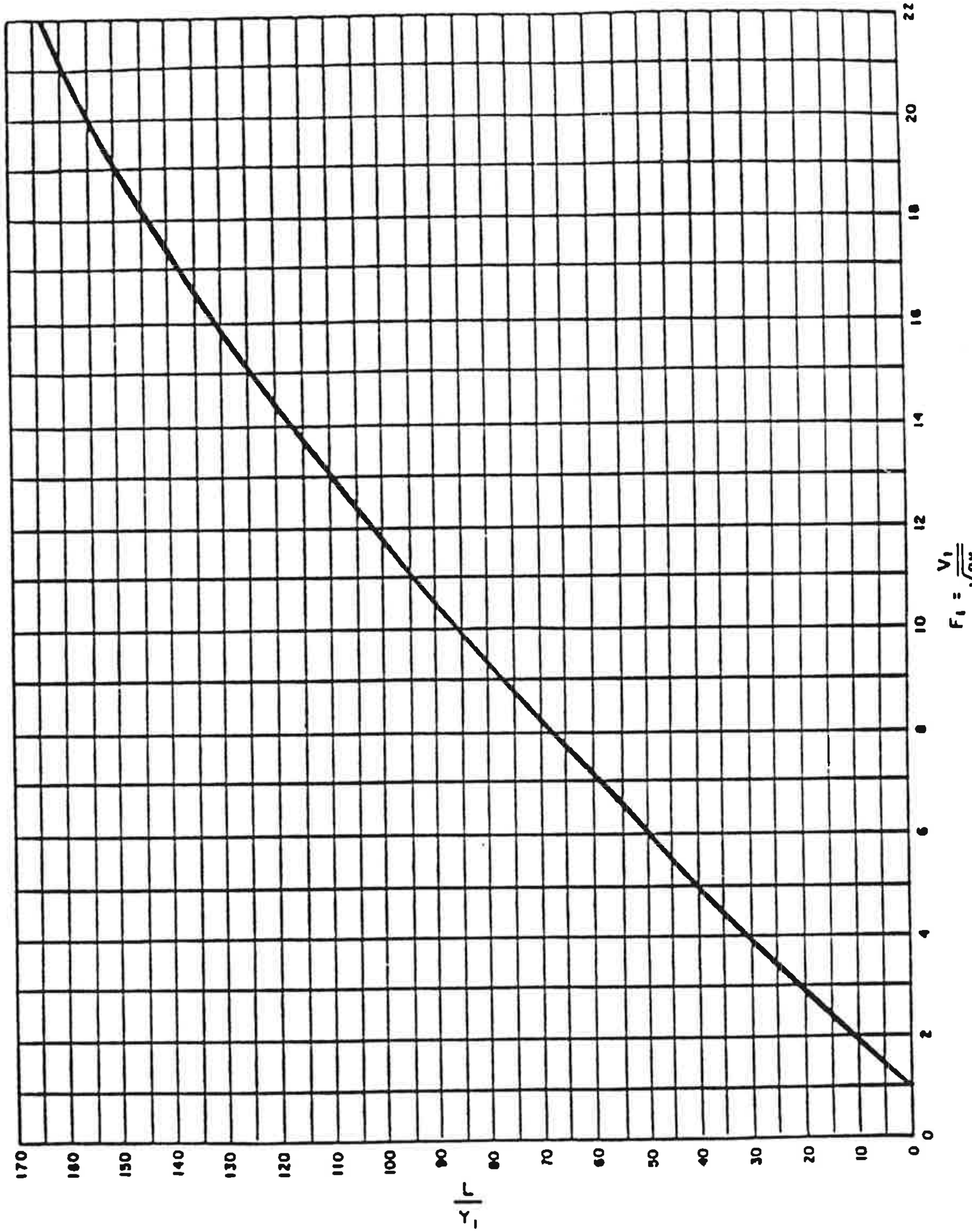


Figure 8.24
Length of a Hydraulic Jump for
Rectangular Channels

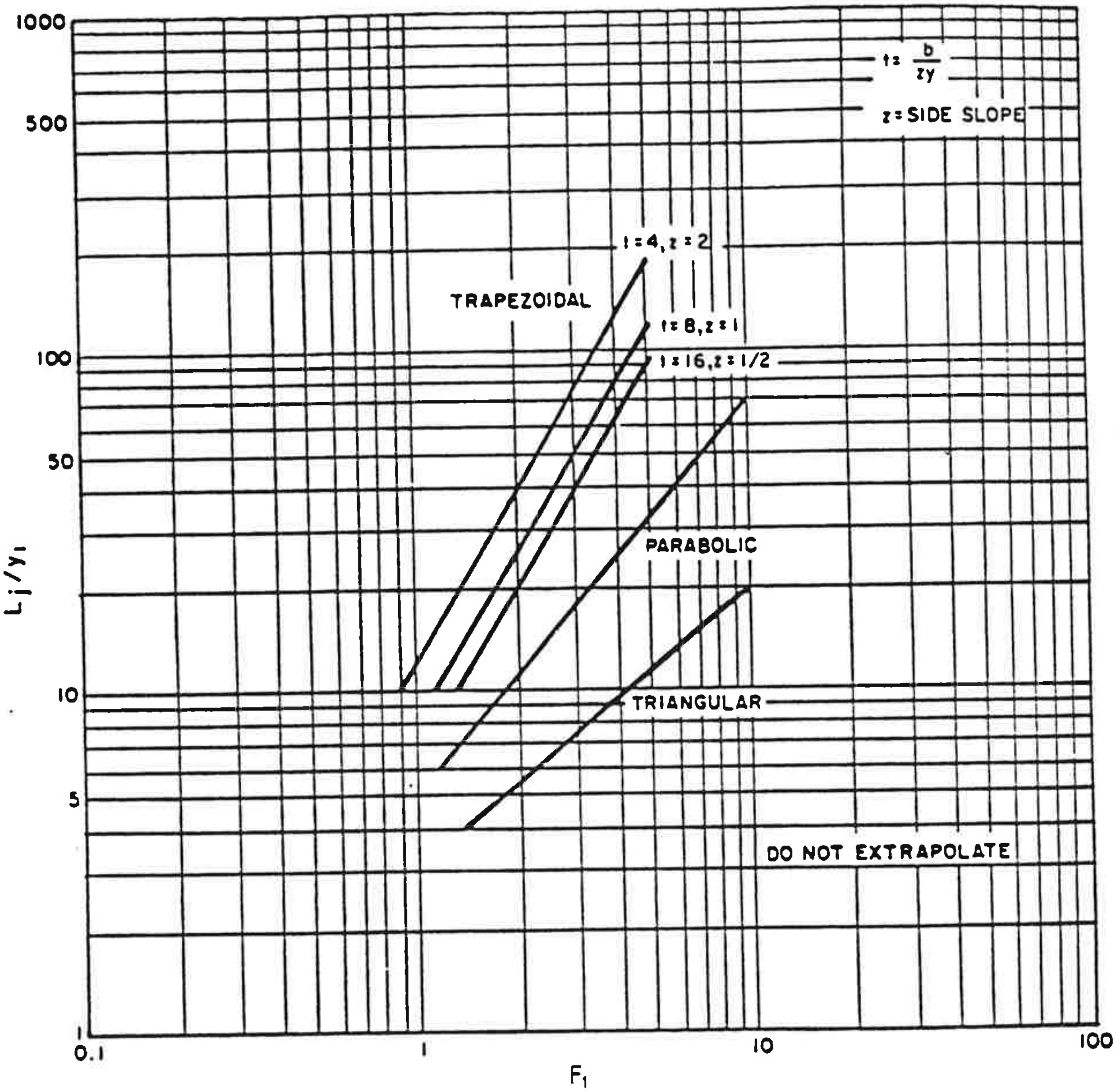


Figure 8.25
Length of a Hydraulic Jump for Non-rectangular Channels

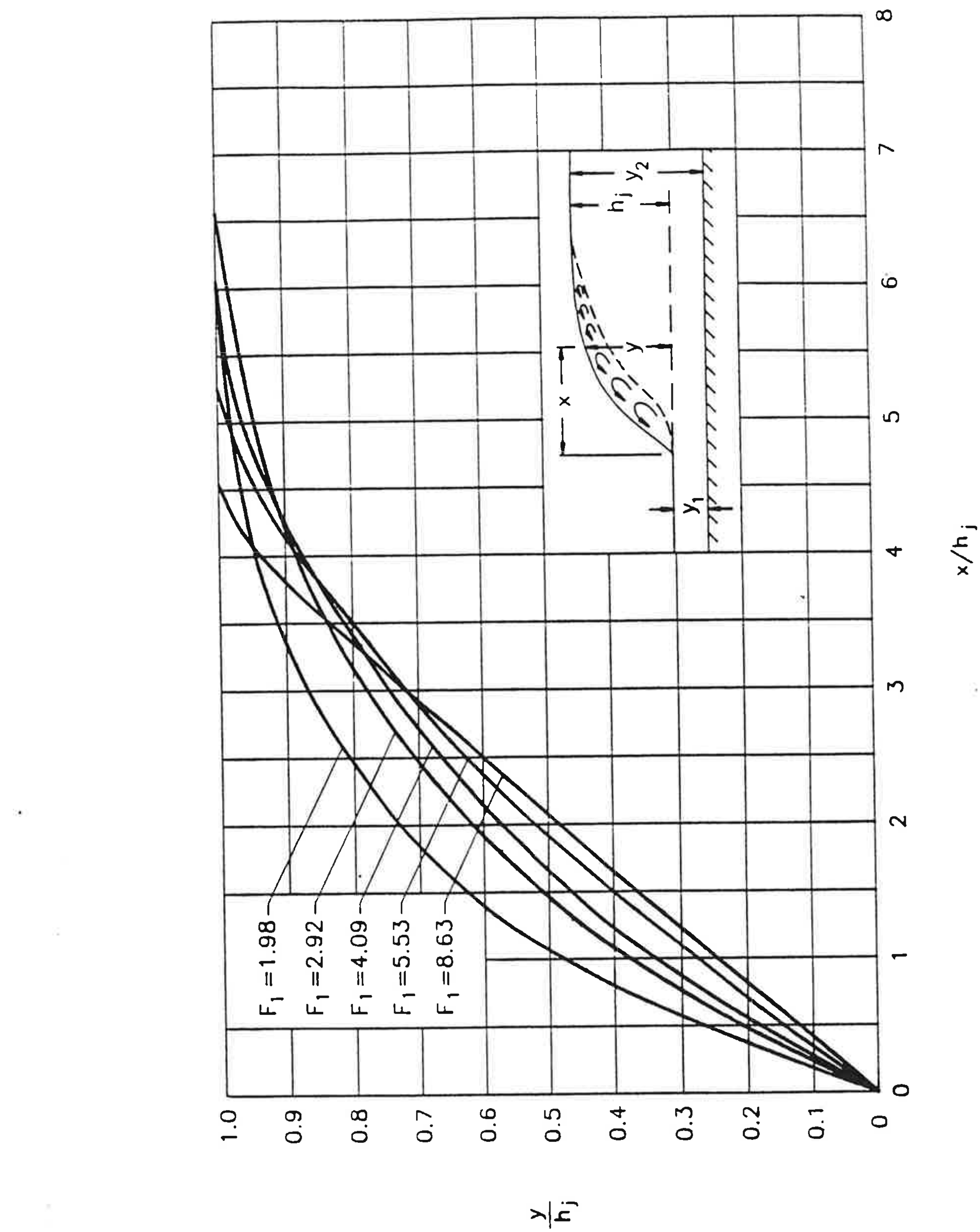


Figure 8.26
Surface Profile for a Hydraulic Jump

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

Table 8.3

Uniform Flow in Circular Sections Flowing Partly Full

8.7 DESIGN EXAMPLES

8.7.1 Open Channel Drop Structures

Design Example for Vertical Drop with Riprap Basin (Stevens 1981) A rock riprap vertical drop structure is to be used to drop the flow in a drainage channel a vertical distance of 4 feet. See Figure 8.10.

Approach Conditions: The approach channel is trapezoidal in cross-section with:

- » bed with is $B_o = 10$ ft
- » side slopes are 2 horizontal to 1 vertical
- » design discharge is Q = 360 cfs
- » the depth of flow is $y_o = 4.00$ ft
- » the average velocity is $V_o = 5.00$ fps
- » the specific energy in the approach channel is

$$E_o = y_o + V_o 2/2g = 4.00 + 0.39 = 4.39 ft \qquad \textbf{(8.16)}$$

Weir Crest: Make the length of the weir B = 10 feet.

The head on the weir required to pass the design flow is as follows:

$$\begin{aligned} H &= [Q/(5.67B)]^{0.67} \\ H &= [360/(5.67x10)]^{0.67} = 3.43ft \end{aligned} \qquad \textbf{(8.17)}$$

The height of the weir crest is:

$$\begin{aligned} P &= E_o - H \\ P &= 4.39 - 3.43 = 0.96ft \end{aligned} \qquad \textbf{(8.18)}$$

If draw down can be tolerated in the approach channel, the weir crest should be placed at a bed level of the approach channel; that is P = 0. If not, make P = 1.0 foot. Here use P = 1.0 foot.

The height of the wingwalls is:

$$h = H = 3.5ft \qquad \textbf{(8.19)}$$

Basin: The problem is to find the drop height D and rock size d_m such that z is 4.0 feet. The depth of flow leaving the basin is:

$$D/d_{50} = 6.0 \tag{8.21}$$

Start by using the largest rock size that has been tested in the model. That is:First, try D = 6.0 feet, so that d_{50} = 1.0 foot, and H/D = 0.57. Now a trial and error solution is required to find the depth d_2 that satisfies both the curves in Figure 8.12 (page 8-41) and the equation:

$$d_2 = Y_2 + 0.67 d_s \tag{8.22}$$

Here d_s is the depth of scour, in feet. The calculations are summarized in the Table 8.4.

The procedure is to select a trial value of d_2 compute d_2/D, obtain d_s/D from Figure 8.11, and then compute d_2 from the above equation. When the trial value and the computed value of d_2 agree, that is one solution. For the first, try:

D	=	6.0 feet
d_2	=	5.5 feet and
z	=	6.0 - 1.0 - 1.5 = 3.5 feet, which is less than the required drop (4 feet).

Therefore, a larger D is needed. In this case, a value of 8 feet is assumed while d_s = 1.3 feet to maintain the D/d_s ratio of 6. This results in d_2 = 5.5 feet and z = 5.5 feet, which is too large.

These two trial results for D and z are graphically plotted in Figure 8.28 so that a new (and hopefully) final value of D can be selected to yield the desired drop, z = 4.0 feet. The interpretation is that D = 6.5 feet which, as shown in Table 8.4, turns out to be the required result. The design values are:

D	=	6.5 feet
P	=	1.0 feet
d_{50}	=	1.1 feet
d_s	=	2.2 feet

From Figure 8.10, the length of the basin is calculated as follows:

$$L_b = 4H + 0.25D$$

$$L_b = 4 \times 3.43 + 0.25 \times 6.5$$

$$= 15.5 \text{ feet}$$

The minimum depth of rock in the plunge pool is 1.5 d_s = 3.3 feet (Figure 8.10), and the minimum thickness of the riprap on the side slopes is t = 2.0 d_{50} = 2.20 feet (Figure 8.14).

Table 8.4
Calculations of Basin Dimensions

D,ft	**d_{50},ft**	**H/D**	**Trial d_2, ft**	**d_2/D**	**d_s/D**	**d_s,ft**	**0.67 d_s,ft**	**Resulting d_2, ft**
6.0	1.0	0.57	5.0	0.83	0.46	2.76	1.85	5.85
			6.0	1.00	0.30	1.80	1.21	5.21
			5.5	0.92	0.38	2.28	1.53	5.53 ok
z = 6.0 — 1.0 — 1.5 = 3.5 ft. ≠ 4.0 ft.								
8.0	1.3	0.43	7.0	0.88	0.18	1.44	0.96	4.96
			6.0	0.75	0.25	2.00	1.33	5.33
			5.5	0.69	0.28	2.24	1.49	5.49 ok
z = 8.0 — 1.0 — 1.5 5.5 ft ≠ 4.0 ft.								
6.5	1.1	0.53	6.0	0.92	0.28	1.82	1.21	5.21
			5.5	0.85	0.33	2.15	1.44	5.44 ok
z = 6.5 — 1.0 — 1.4 = 4.1 ≈ 4.0 ft.								

Figure 8.28
Solution for Vertical Drop with Riprap Basin Design Example

Design Example 2

Given: 8 foot by 6 foot box culvert
Q = 800 cfs
supercritical flow in culvert
normal flow depth = brink depth
y_o = 4 feet
Tailwater depth (TW) = 4.2 feet
Downstream channel can tolerate 7 fps for design discharge

Find: Riprap basin dimensions for these conditions.

Solutions:

Note: High tailwater depth, $TW/y_o = 1.05 > 0.75$.

1. Design riprap basin using steps 1 through 6 of Design Example 1.

   d_{50} = 1.8 feet; h_s = 6.4 feet; L_s = 64 feet; L_B = 96 feet

2. Design riprap for downstream channel Use Figure 8.17 to estimate the average velocity along the channel. Compute the equivalent circular diameter, D_e, for the brink area, A, from:

$A = 3.14\ D_e 2/4 = 2y_o 2$

$W_o = (4)\ (8) = 32$ feet^2

$D_e = 25$ fps (Design Example 1)

L/D_e	L (compute)	V_L/V_o (Figure 6.21)	V_L
10	64	0.59	14.7
15	96	0.36	9.0
20	128	0.30	7.5
21	135	0.28	7.0

The channel should be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream from the culvert brink.

Design Example 3

Given: 6 feet diameter cmp

Q = 135 cfs
S_o = 0.004
Mannings n = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 feet
Normal velocity is 5.9 fps
Flow is subcritical
Tailwater depth (TW) is 2.0 feet

Find: Riprap basin dimensions for these conditions.

Solution:

1. Determine y_o and V_o:

$Q/D^{2.5} = 135/(6)^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

From Figure 6.19, $y_o/D = 0.45$

$y_o = (0.45)(6) = 2.7$ feet

$TW/y_o = 2.0/2.70 = 0.74$, $TW/y_o < 0.75$ OK

Brink Area (A) for $y_o/D = 0.45$ is:

$A = (0.343)(36) = 12.3\ ft_2$ [0.343 is from Table 8.3]

$V_o = Q/A = 135/12.3 = 11.0$ fps

2. $y_2 = (A/2)^{0.5} = (12.3/2)^{0.5} = 2.48$ feet

3. $F = V_o/[(32.2)(y_e)]^{0.5} = 11/[32.2)(2.48)]^{0.5} = 1.23$

4. Try $d_{50}/y_e = 0.25$, $d_{50} = (0.25)(2.48) = 0.623$ feet

   From Figure 6.20, $h_s/y_e = 0.75$, $h_s = (0.75)(2.48) = 1.86$ feet

   check: $h_s/d_{50} = 1.86/0.62 = 3$, $2 < h_s/d_{50} < 4$ OK

5. $L_s = (10)(h_s) = (10)(1.86) = 18.6$ feet

   or

   $L_s = (3)(W_o) = (3)(6) = 18$ feet, Use $L_s = 18.6$ feet

   $L_B = (15)(h_s) = (15)(1.86) = 27.9$ feet

   or

   $L_B = (4)(W_o) = (4)(6) = 24$ feet, Use $L_B = 27.9$ feet

   $d_{50} = 0.62$ feet, Use $d_{50} = 8$ inches

Other basin dimensions designed in accordance with details are shown on Figure 8.14 (page 8-43).

8.8 LIST OF VARIABLES

D	=	Plunge height
ds	=	Scour depth
F	=	Forces in pounds per square inch
F_s	=	Specific force
P	=	Pressure in pounds per square inch

g = Acceleration of gravity, 32.2 FT/S^2

q = Discharge per unit width in cubic feet per second per foot.

q_D = Design discharge per unit width

V = Velocity in feet per second

y = Depth of flow

y_c = Critical depth

Y_f = Vertical fall at a drop in feet

8.9 REFERENCES

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