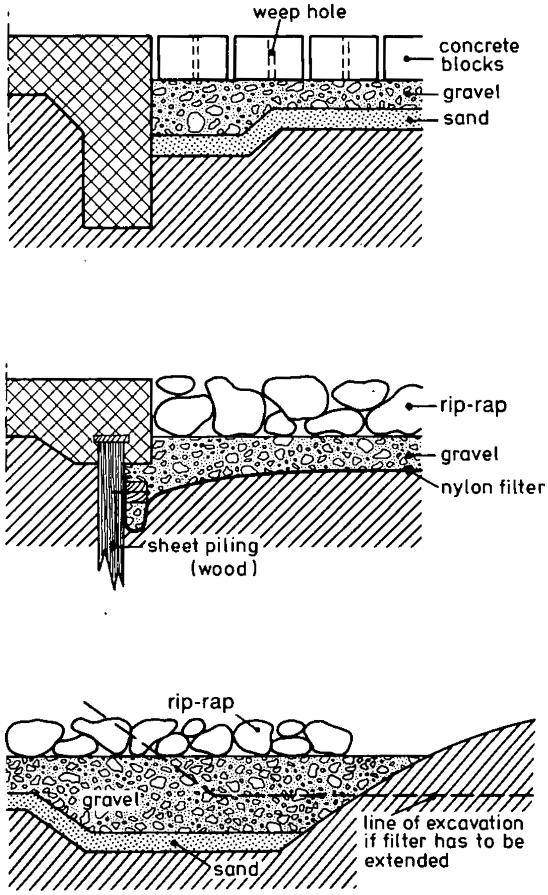


Figure 19.33 Filter construction details (after Van Bendegom 1969)

that the thickness of the filter construction be increased at these places. Some examples of common constructional details are shown in Figure 19.33.

19.7 Energy Dissipators

19.7.1 Introduction

As we saw in Section 19.6.1, channels with a hydraulic gradient flatter than the land slope require structures that dissipate surplus energy. Such a structure can be divided into four parts:

1) The part upstream of the (control) section, where flow is accelerated to critical flow;
2) The part in which water is conveyed to the anticipated lower elevation;
3) The part immediately downstream of section U (Figure 19.34), where energy is dissipated;
4) The channel reach that requires a construction to protect it against erosion.

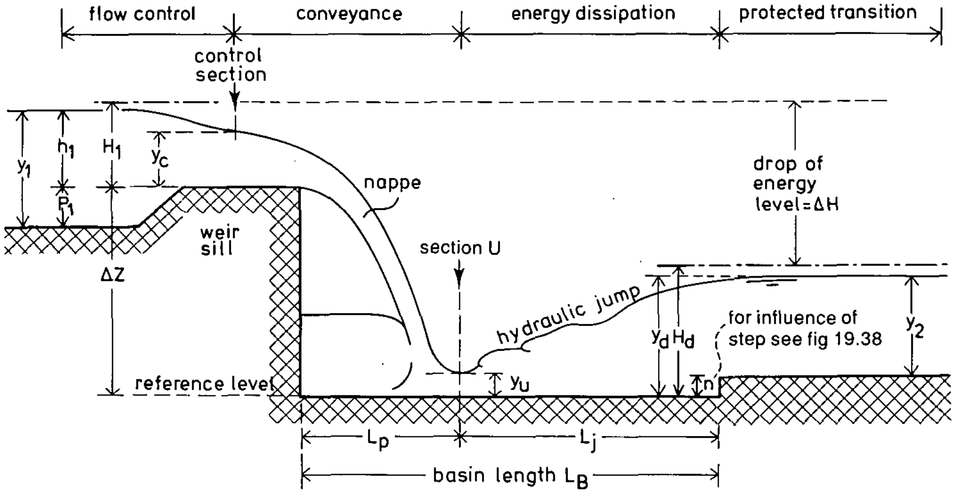


Figure 19.34 Illustration of terminology for a straight drop structure

In the upstream part of the structure, the flow over the sill is controlled. The head, h_1, versus discharge, Q, relationship of this control is a function of the sill height, p_1, the longitudinal profile of the weir crest, the shape of the control section perpendicular to the flow, and the width of this control section, b. Each combination of these four properties yields one out of an infinite number of combinations of h_1 and Q (Bos 1989).

Further, the channel upstream of the structure has a discharge capacity that can be characterized by the water depth, y_1, versus discharge, Q, relationship, written as

$$Q = K_1 y_1^u \tag{19.48}$$

where

K_1 = a factor which varies with the shape and hydraulic properties of the channel

u = the exponent to y_1, varying between 1.7 for trapezoidal channel with wide bottom, to 2.3 for a narrow-bottomed channel

To avoid sedimentation upstream of the structure, the control should be dimensioned so that the head-discharge curve of the structure coincides with the y_1 versus Q curve of the channel throughout the flow range with sediment transport (see Figure 19.35).

For a broad-crested weir sill with a rectangular control section perpendicular to the flow (Figure 19.34), the head-discharge relationship reads

$$Q = C_d C_v \frac{2}{3}\sqrt{\frac{2}{3}g}\, b_c h_1^{1.5}$$

where

C_d = discharge coefficient (–)
C_v = approach velocity coefficient (–)
b_c = width of control section (m)

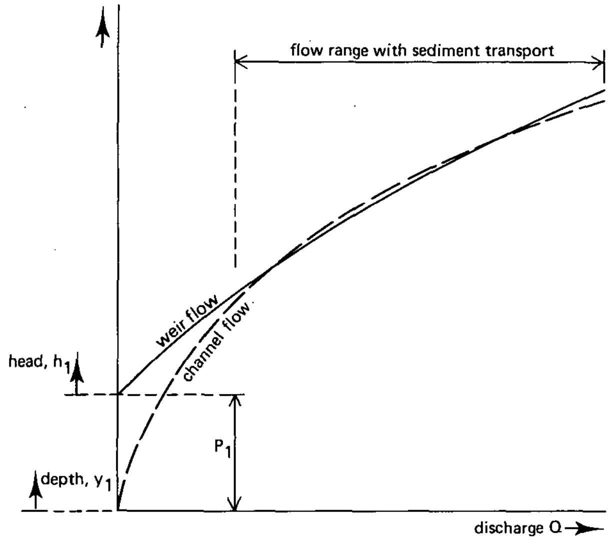


Figure 19.35 Matching of Q-y_1 and Q-h_1 curves for a structure with sediment transport

The product of the discharge- and the approach velocity coefficients may, for general design purposes, be taken as $C_dC_v \simeq 1.0$.

For detailed information on the head-discharge relationship of control structures, consult Bos 1989; and Bos, Replogle, and Clemmens 1984.

We can dimension the conveyance and energy dissipation parts of the structure in relation to the following variables (Figure 19.34):

H_1 = upstream sill-referenced energy head (m)
ΔH = change in energy head across structure (m)
H_d = downstream energy head (m)
q = discharge per unit width of sill (m^2/s)
g = acceleration due to gravity, being 9.81 m/s^2
n = step height (m)
y_u = flow depth at section U (m)
y_d = downstream flow depth relative to basin floor (m)
y_2 = flow depth in downstream channel (m)

These variables can be combined to calculate H_1 and H_d, after which we can make a first estimate of the drop height

$$\Delta Z = (\Delta H + H_d) - H_1 \tag{19.50}$$

Subsequently, we can estimate the flow velocity and depth at section U with

$$v_u = \sqrt{2g\Delta Z} \tag{19.51}$$

and with the continuity equation, we calculate

$$y_u = \frac{q}{v_u} \tag{19.52}$$

The flow at section U can best be characterized by the dimensionless Froude number

$$Fr_u = \frac{v_u}{\sqrt{gy_u}} \tag{19.53}$$

This Froude number classifies the flow phenomena at the downstream side of the structure and enables the selection of a satisfactory alternative of this part of the structure. From a practical viewpoint, we can state:

1) If $Fr_u \leq 2.5$, no baffles or special devices are required, but the downstream channel should be sufficiently protected against scouring over a length as specified in Section 19.6.2 (Gebler 1991);
2) If Fr_u ranges between 2.5 and 4.5, the hydraulic jump is not well stabilized. The entering jet oscillates from bottom to surface and creates waves with irregular period in the downstream channel. It is therefore advisable to dissipate energy through increased turbulence and not to rely on the jump;
3) If $Fr_u \geq 4.5$, a stable jump will be formed, which can dissipate energy effectively.

For a known discharge per unit width, q, and an estimated drop height, ΔZ, Figure 19.36 gives an indication of which structure is appropriate. A more detailed hydraulic design will give a better ΔZ value, which may lead to another structure.

Construction of a complex energy dissipator for a low discharge and drop but high

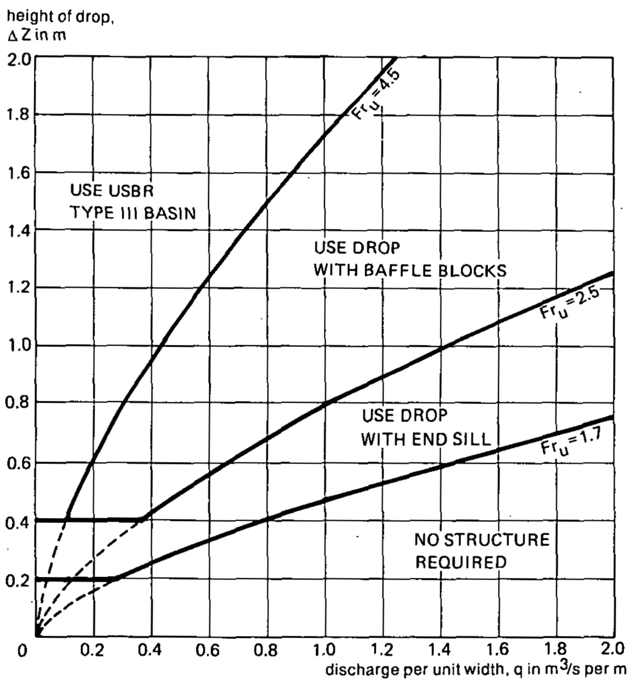


Figure 19.36 Diagram for estimating the type of structure to be used prior to a detailed design, (Bos, Replogle, and Clemmens 1984)

Froude number is impractical, because the energy to be dissipated is low. Thus, we have placed limits on the minimum drop height for these structures at 0.2 and 0.4 m, as shown in Figure 19.36. Moreover, large straight drops often require massive structures that may be overly expensive and hydraulically unreliable. So, it is better not to use straight drops of more than 1.5 m, except under special circumstances. These limits on drop height, energy drop, and Froude number, are not absolute, but give the designer practical limits for quick decision-making.

The energy dissipators described in this Section may not be suitable for every project, and they certainly do not exhaust the possibilities open to the designer. For further information on straight drops, end sills, baffle blocks, and tapered sidewalls, to name but a few, see Peterka (1964), Aisenbrey et al. (1974), Brakensiek et al. (1979) and U.S. Bureau of Reclamation (1973).

19.7.2 Straight Drop

The free-falling nappe strikes the basin floor and turns downstream at section U (Figure 19.34). Because of the impact of the nappe and the turbulent circulation in the pool beneath it, some energy is dissipated. Further energy is dissipated in the hydraulic jump downstream of section U. The remaining downstream energy head, H_d, does not vary greatly with the ratio $\Delta Z/H_1$, and is equal to about $1.67H_1$ (adapted from Henderson 1966). This value provides a satisfactory estimate for the level of the basin floor below the energy level of the downstream canal. The hydraulic dimensions of a straight drop can be related to the Froude number at section U. The Froude number can be related directly to the straight drop geometry with the length ratios $y_d/\Delta Z$ and $L_p/\Delta Z$ (Figure 19.37).

We can calculate the length of the undisturbed hydraulic jump, L_j, downstream of Section U with the following (Henderson 1966)

$$L_j = 6.9\,(y_d - y_u) \tag{19.54}$$

It is important to realize that the downstream water depths, y_d and y_2, are caused not by the drop structure, but by the flow characteristics of the downstream canal. If these characteristics produce the required depth, y_d, a jump will form; otherwise it will not form, and not enough energy will be dissipated within the basin. Additional steps, such as lowering the basin floor and adding an end sill, must be taken to assure adequate energy dissipation.

Because of seasonal changes in the hydraulic resistance of the downstream canal, however, the flow velocity as calculated by Manning's equation changes with the water depth, y_d. The jump thus tends to drift up and down the canal. This unstable behaviour is often undesirable, and can be suppressed by increasing the flow resistance with an abrupt step at the end of the basin. Usually, this step is constructed at a distance of

$$L_n = 5\,(n + y_2) \tag{19.57}$$

downstream of section U. For design purposes, Figure 19.38 can be used to determine the largest required value of n, if $Fr_u = v_u/\sqrt{gy_u}$, y_u, and y_2 are known.

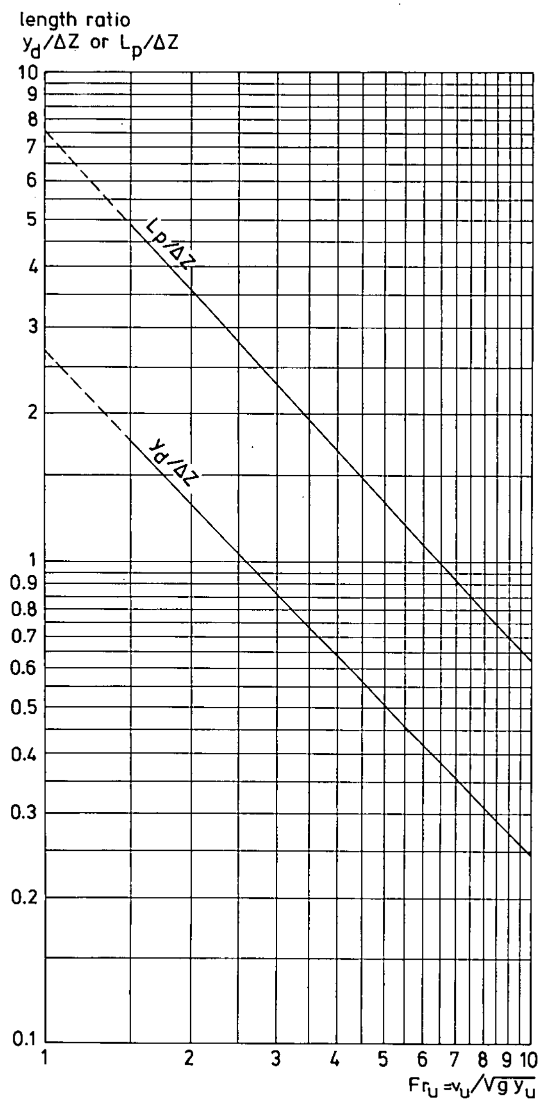


Figure 19.37 Dimensionless plot of straight drop geometry (from Bos, Replogle, and Clemmens 1984)

If the above structure discharges into a relatively wide canal or if the downstream water depth, y_2, is not determined by the flow over the structure but by a downstream control, the step height, n, must also be determined for lower flow rates and the expected value of y_2. The largest n value must be used for the design.

The total basin length, L_B, of the structure is greatly influenced by the length, L_n. As we have seen, the hydraulic jump can be stabilized and shortened by increasing the flow resistance downstream of section U. To shorten the basin downstream of section U, the hydraulic resistance can be further increased by placing baffle blocks on the basin floor.

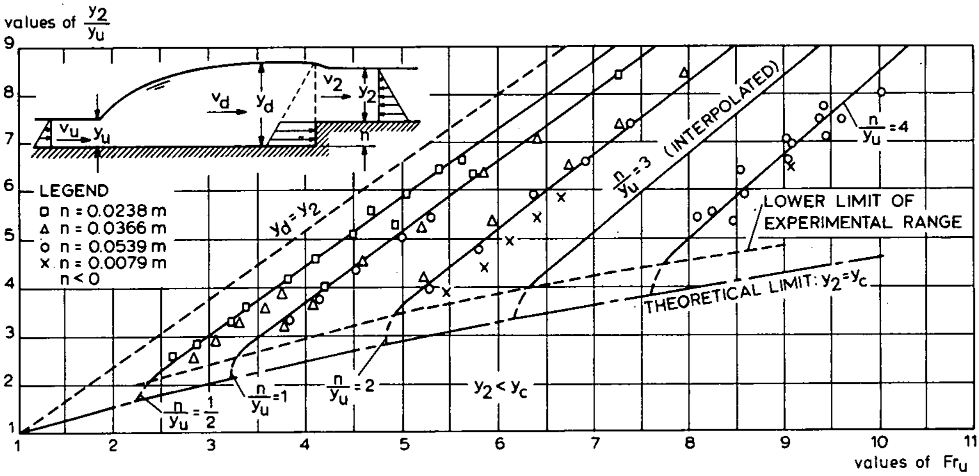


Figure 19.38 Experimental relations between Fr_u, y_2/y_u, and n/y_u for an abrupt step (Forster and Skrinde 1950)

19.7.3 Baffle Block Type Basin

As mentioned above, the basin length of the energy dissipator can be shortened by adding baffle blocks to the basin floor. Although this is a significant advantage, the baffle blocks have one major drawback: they collect all types of floating and suspended debris, which may lead to overtopping of the basin and damaging of the baffle blocks. To function properly therefore these basins require regular cleaning.

The baffle block type basin was developed for low drops in the energy level, and it dissipates energy reasonably well for a wide range of downstream water depths. Energy is dissipated principally by turbulence induced by the impingement of the incoming flow upon the baffle blocks. The required downstream water depth therefore can be slightly less than for a straight drop, but can vary independently of the drop height, ΔZ. To function properly, the downstream water depth, y_d must not be less than $1.45H_1$ while at Q_{max} the Froude number, Fr_u, should not exceed 4.5.

Upstream of section U, we can determine the length, L_p, with Figure 19.37. The linear proportions of the basin downstream from section U as a function of H_1 are shown in Figure 19.39.

19.7.4 Inclined Drop

Downstream of the control of a drop structure, a sloping face, guiding the overfalling nappe, is a common design feature, especially if the energy drop exceeds 1.5 m. In drop structures, the slope of the downstream face is often as steep as possible. If a sharp-edged, broken plane transition is used between the control and the downstream face, it is advisable to use a slope no steeper than 2 to 1 (see Figure 19.40). This will prevent flow separation at the sharp edge. If a steeper slope (1 to 1) is required, the

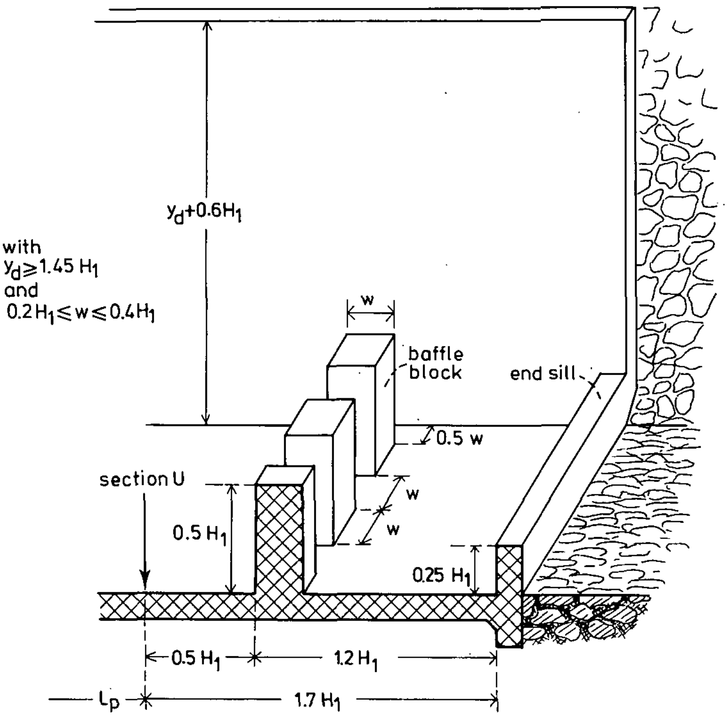


Figure 19.39 Section over center line of the baffle block type basin downstream from section U (Donelly and Blaisdell 1954)

sharp edge should be replaced by a transitional curve with a radius of r $\simeq$ 0.5 H_1 (Figure 19.40).

Values of y_u and H_d that are suitable for the basin downstream of Section U can be determined from Table 19.9. In this context, note that the energy level, H_u, of the nappe entering the basin at Section U has a much higher value if there is a sloping downstream face than if the nappe were free-falling, with the straight drop. This is because with a straight drop, energy is dissipated by the impact of the nappe on the basin floor and the turbulent circulation of water in the pool beneath the nappe. With the inclined drop there is only some dissipation due to friction and turbulent flow over the sloping face.

19.7.5 USBR Type III Basin

In selecting a basin lay-out, note that the basin with baffle blocks in Figure 19.39 was designed to dissipate energy by turbulence. Such a basin is satisfactory if the Froude number at maximum anticipated flow, Fr_u, does not exceed 4.5 (see Figure 19.36). For higher Froude numbers, use the USBR Type III basin shown in Figure 19.41.

Table 19.9 Dimensionless ratios for hydraulic jumps (from Bos, Replogle and Clemmens 1984)

$\Delta H/H_1$	y_d/y_u	y_u/H_1	$v_u^2/2gH_1$	H_u/H_1	y_d/H_1	$v_d^2/2gH_1$	H_d/H_1
0.2446	3.00	0.3669	1.1006	1.4675	1.1006	0.1223	1.2229
0.2688	3.10	0.3599	1.1436	1.5035	1.1157	0.1190	1.2347
0.2939	3.20	0.3533	1.1870	1.5403	1.1305	0.1159	1.2464
0.3198	3.30	0.3469	1.2308	1.5777	1.1449	0.1130	1.2579
0.3465	3.40	0.3409	1.2749	1.6158	1.1590	0.1103	1.2693
0.3740	3.50	0.3351	1.3194	1.6545	1.1728	0.1077	1.2805
0.4022	3.60	0.3295	1.3643	1.6938	1.1863	0.1053	1.2916
0.4312	3.70	0.3242	1.4095	1.7337	1.1995	0.1030	1.3025
0.4609	3.80	0.3191	1.4551	1.7742	1.2125	0.1008	1.3133
0.4912	3.90	0.3142	1.5009	1.8151	1.2253	0.0987	1.3239
0.5222	4.00	0.3094	1.5472	1.8566	1.2378	0.0967	1.3345
0.5861	4.20	0.3005	1.6407	1.9412	1.2621	0.0930	1.3551
0.6525	4.40	0.2922	1.7355	2.0276	1.2855	0.0896	1.3752
0.7211	4.60	0.2844	1.8315	2.1159	1.3083	0.0866	1.3948
0.7920	4.80	0.2771	1.9289	2.2060	1.3303	0.0837	1.4140
0.8651	5.00	0.2703	2.0274	2.2977	1.3516	0.0811	1.4327
0.9400	5.20	0.2639	2.1271	2.3910	1.3723	0.0787	1.4510
1.0169	5.40	0.2579	2.2279	2.4858	1.3925	0.0764	1.4689
1.0957	5.60	0.2521	2.3299	2.5821	1.4121	0.0743	1.4864
1.1763	5.80	0.2467	2.4331	2.6798	1.4312	0.0723	1.5035
1.2585	6.00	0.2417	2.5372	2.7789	1.4499	0.0705	1.5203
1.3429	6.20	0.2367	2.6429	2.8796	1.4679	0.0687	1.5367
1.4280	6.40	0.2321	2.7488	2.9809	1.4858	0.0671	1.5529
1.5150	6.60	0.2277	2.8560	3.0837	1.5032	0.0655	1.5687
1.6035	6.80	0.2235	2.9643	3.1878	1.5202	0.0641	1.5843
1.6937	7.00	0.2195	3.0737	3.2932	1.5368	0.0627	1.5995
1.7851	7.20	0.2157	3.1839	3.3996	1.5531	0.0614	1.6145
1.8778	7.40	0.2121	3.2950	3.5071	1.5691	0.0602	1.6293
1.9720	7.60	0.2085	3.4072	3.6157	1.5847	0.0590	1.6437
2.0674	7.80	0.2051	3.4723	3.7254	1.6001	0.0579	1.6580
2.1641	8.00	3.6343	3.6343	3.8361	1.6152	0.0568	1.6720
2.2620	8.20	3.7490	3.7490	3.9478	1.6301	0.0557	1.6858
2.3613	8.40	3.8649	3.8649	4.0607	1.6446	0.0548	1.6994
2.4615	8.60	3.9814	3.9814	4.1743	1.6589	0.0538	1.7127
2.5630	8.80	4.0988	4.0988	4.2889	1.6730	0.0529	1.7259
2.6656	9.00	4.2171	4.2171	4.4045	1.6869	0.0521	1.7389
2.7694	9.20	4.3363	4.3363	4.5211	1.7005	0.0512	1.7517
2.8741	9.40	4.4561	4.4561	4.6385	1.7139	0.0504	1.7643
2.9801	9.60	4.5770	4.5770	4.7569	1.7271	0.0497	1.7768
3.0869	9.80	4.6985	4.6985	4.8760	1.7402	0.0489	1.7891
3.1949	10.00	4.8208	4.8208	4.9961	1.7530	0.0482	1.8012
3.4691	10.50	5.1300	5.1300	5.2999	1.7843	0.0465	1.8309
3.7491	11.00	5.4437	5.4437	5.6087	1.8146	0.0450	1.8594
4.0351	11.50	5.7623	5.7623	5.9227	1.8439	0.0436	1.8875
4.3267	12.00	6.0853	6.0853	6.2413	1.8723	0.0423	1.9146
4.6233	12.50	6.4124	6.4124	6.5644	1.9000	0.0411	1.9411
4.9252	13.00	6.7437	6.7437	6.8919	1.9268	0.0399	1.9667
5.2323	13.50	7.0794	7.0794	7.2241	1.9529	0.0389	1.9917
5.5424	14.00	7.4189	7.4189	7.5602	1.9799	0.0379	2.0178
5.8605	14.50	7.7625	7.7625	7.9006	1.0032	0.0369	2.0401
6.1813	15.00	8.1096	8.1096	8.2447	2.0274	0.0361	2.0635
6.5066	15.50	8.4605	8.4605	8.5929	2.0511	0.0352	2.0863
6.8363	16.00	8.8153	8.8153	8.9450	2.0742	0.0345	2.1087
7.1702	16.50	9.1736	9.1736	9.3007	2.0968	0.0337	2.1305
7.5081	17.00	9.5354	9.5354	9.6601	2.1190	0.0330	2.1520
7.8498	17.50	9.9005	9.9005	10.0229	2.1407	0.0323	2.1731
8.1958	18.00	10.2693	10.2693	10.3894	2.1619	0.0317	2.1936
8.5438	18.50	10.6395	10.6395	10.7575	2.1830	0.0311	2.2141
8.8985	19.00	11.0164	11.0164	11.1290	2.2033	0.0305	2.2339
9.2557	19.50	11.3951	11.3951	11.5091	2.2234	0.0300	2.2534
9.6160	20.00	11.7765	11.7765	11.8887	2.2432	0.0295	2.2727

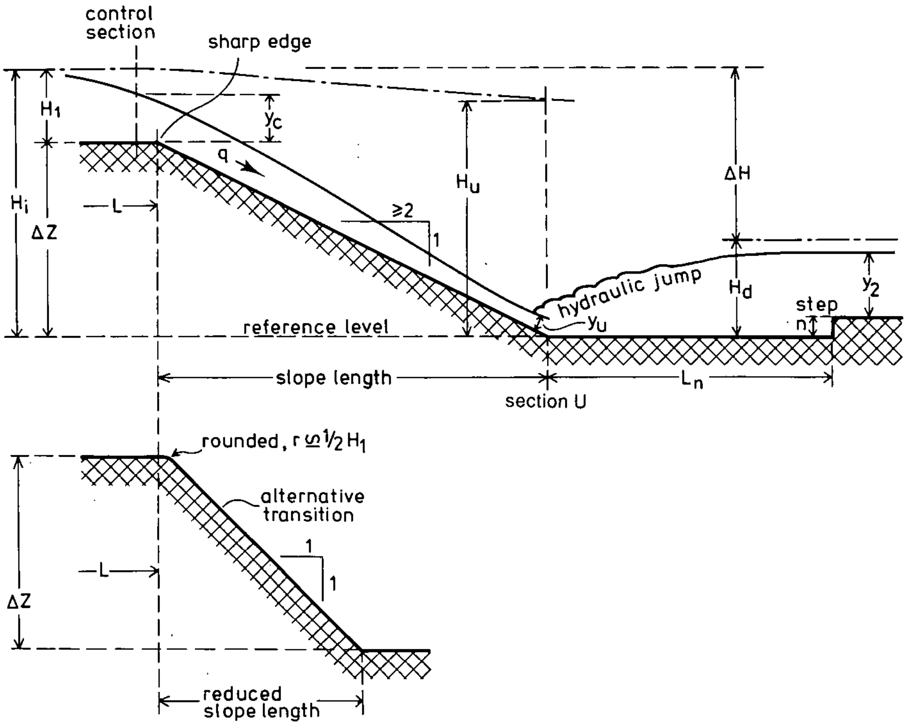


Figure 19.40 Definition sketch to Table 19.9 (from Bos, Replogle and Clemmens 1984)

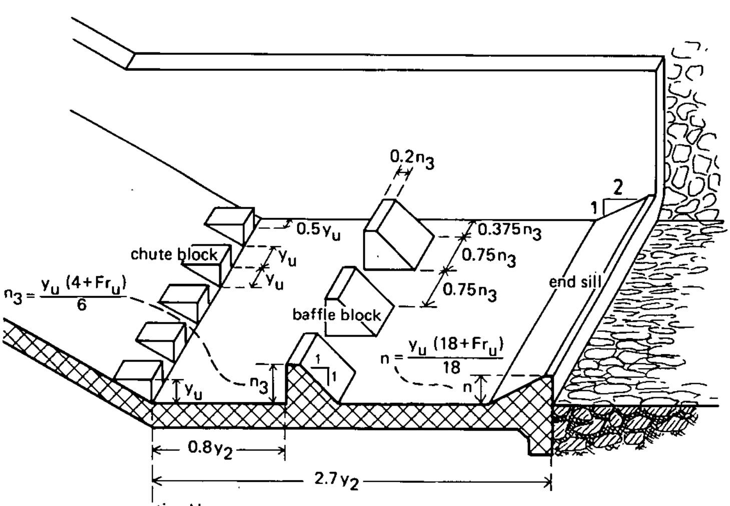


Figure 19.41 Stilling basin characteristics for use with Froude numbers above 4.5; USBR Type III basin (Bradley and Peterka 1957)

19.8 Culverts

19.8.1 General

A culvert can be defined as a reduction in the channel's wetted area, and as a change in its slope, or in the direction of flow. In culverts, flow usually remains subcritical. Nearly all culverts of interest to the drainage engineer are structures that are short in comparison to the remaining channel. Yet the culverts affect the flow far upstream if their discharge capacity is too low in comparison with that of the open drain.

The possibility of this occurring is very real, as the discharge characteristics of a culvert change in a transitional zone from 'free water surface flow' to 'pipe flow' if the upstream drain water level exceeds the elevation of the internal culvert crest (see Figure 19.42). If the water level in the culvert is below the crest, this free flow can be expressed in an equation similar to Equation 19.48

$$Q = K\,y^{u} \tag{19.58}$$

where

K = a factor dependent on the shape, size, and hydraulic properties of the culvert

u = an exponent to the water depth in the culvert, y, which varies between 1.6 and 2.0 for commonly used culverts where there is a free water surface. If, however, the culvert pipe flows full, u approaches 0.5 (see Figure 19.42)

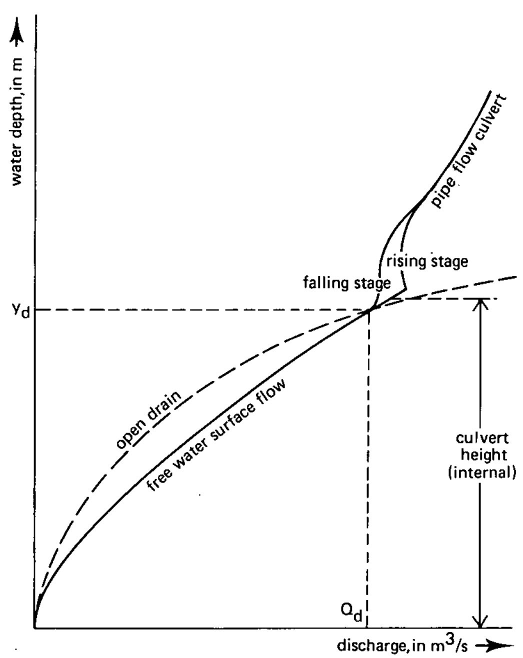


Figure 19.42 Discharge characteristics of a culvert versus those of an open drain

A comparison of the discharge characteristics of a culvert and a (trapezoidal) channel shows a great difference between their discharge capacities as soon as the culvert pipe flows full. Especially in drainage channels, where the design discharge, Q_d, is statistically determined, a culvert with full pipe flow will seriously obstruct discharges greater than Q_d. To avoid such an obstruction, it is recommended to over-dimension drainage culverts to the capacity of the bank-full-flow of the upstream channel ($y = D$), exept when that channel must function as storage for discharges greater than Q_d.

A culvert whose pipe commonly flows full is called a 'siphon'. Because of their hydraulic characteristics, they are more popular for irrigation channels than drainage channels.

19.8.2 Energy Losses

The energy losses over a culvert are due to:
- The conversion of potential energy into kinetic energy at the entrance transition, and vice versa at the downstream transition;
- Turbulence and flow separation in bends or elbows;
- Friction losses over the length of the entrance transition, the downstream transition, and the culvert pipe.

The total loss of energy over the structure has to be reconciled with the available fall. Consequently, culverts have been designed with either discontinuous boundaries and sharp breaks in the wall alignment, resulting in extensive separation zones and local eddying whenever economy of construction was more important than the loss of energy head, or with careful streamlining with gradual transitions when the fall over a structure was limited by the available head.

Transition Losses

For transitions in open channels where the Froude number of the accelerated flow does not exceed 0.5, it is common to express the loss of energy over the inlet and outlet of the culvert, ΔH_{in} or ΔH_{out}, by a simple equation, valid for closed conduits

$$\Delta H_{in} = \xi_{in} \frac{(v_a - v)^2}{2g} \tag{19.57}$$

or

$$\Delta H_{out} = \xi_{out} \frac{(v_a - v)^2}{2g} \tag{19.58}$$

where

$\xi_{in, out}$ = an energy loss factor dependent on the hydraulic shape of the transition and on whether it is an inlet or outlet transition (–)
v_a = accelerated average flow velocity in the culvert pipe (m/s)
v = average flow velocity in either the upstream or downstream channel (m/s)

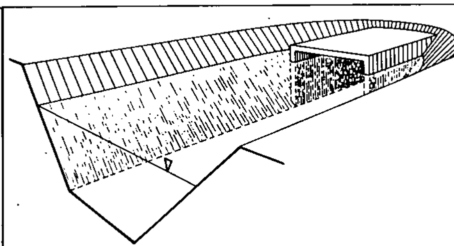
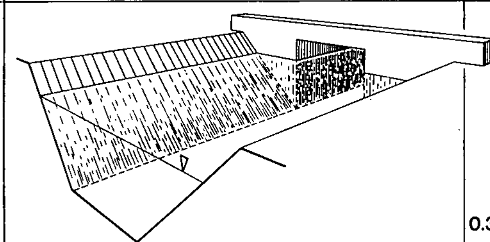
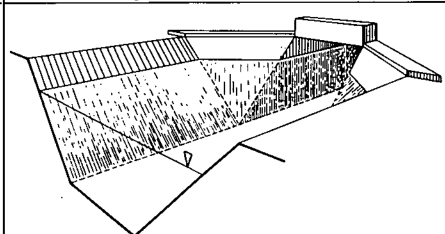
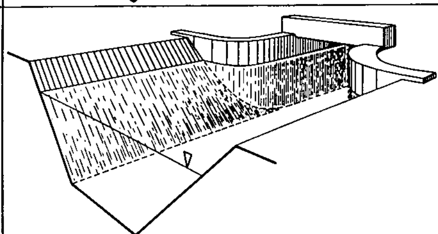
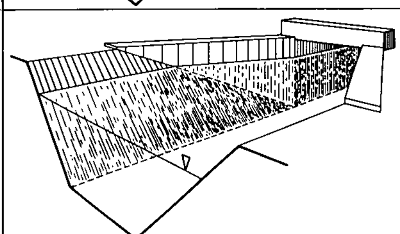
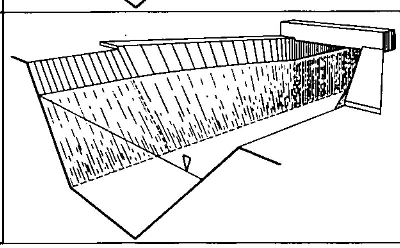
		equation	
		19.59	19.60
		ξ_{in}	ξ_{out}
(A) culvert pipe terminates in channel side slope transition		0.50	1.00
(B) culvert pipe terminates in headwall across channel		0.30	1.10
(C) broken-back transition with flare angle of 1:1 or 1:2		0.10	0.80
(D) headwall with rounded transition of which the radius exceeds 0.1y		0.05	1.00
(E) broken back transition with flare angle of about 1:5		0.05	0.30
(F) gradual transition between trapezoidal and rectangular cross sections		0.05	0.20

Figure 19.43 Head loss coefficients for trapezoidal to rectangular transitions with a free water surface and vise versa (after Bos and Reinink 1981, and Idel'cik 1969)

If v is small with respect to v_a, we can reduce the above equations to

$$\Delta H_{in} = \xi_{in} \frac{v_a^2}{2g} \tag{19.59}$$

and

$$\Delta H_{out} = \xi_{out} \frac{v_a^2}{2g} \tag{19.60}$$

These equations have the same structure as the head-loss equations for bends, elbows, trash-racks, valves, and so forth.

Figure 19.43 illustrates some designs for transitions for culverts with a free water surface, and Figure 19.44 illustrates some designs for culverts with a full flowing pipe.

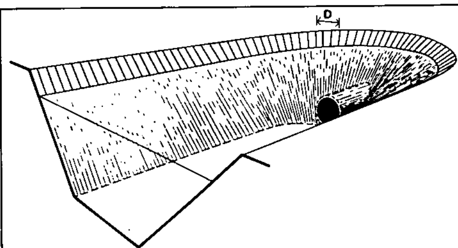
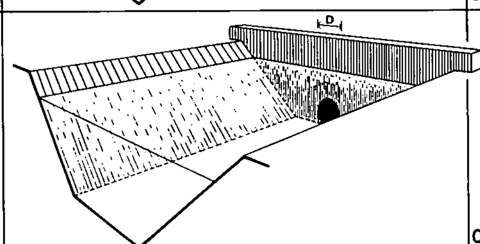
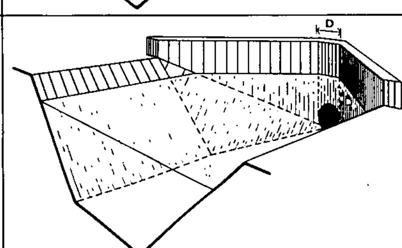
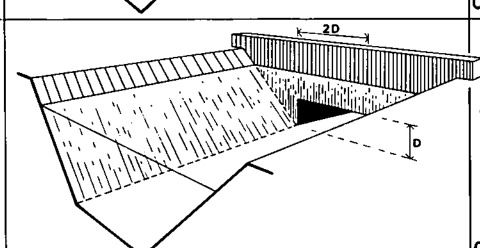
		equation	
		19.59	19.60
		ξ_{in}	ξ_{out}
(A) pipeline terminates in channel side slope transition		0.65	1.00
(B) barrel of pipeline connects directly to headwall across channel		0.55	1.10
(C) barrel of pipeline connected to conventional broken-back transition with 1:4 flare angle		0.50	0.65
(D) 6D-long pipe transition connects pipeline to headwall across channel (round to rectangular)		0.40	0.10

Figure 19.44 Head loss coefficients for transitions from trapezoidal channel to pipe and vice versa (after Simmons 1964, and Idel'cik 1969)

The figures yield the following general conclusions:

- If energy losses need to be reduced, it is better to invest in an outlet transition than in an inlet transition;
- The construction of a head wall in which the pipe terminates directly is expensive in relation to the reduction in head loss;
- Rounded or very gradual transitions (Figure 19.43D and E) are costly and relatively ineffective.

Energy Losses in Elbows and Bends

Elbows and bends in pipes cause a change in the direction of flow and consequently, a change in the general velocity distribution. Owing to this change in velocity distribution, there is an increase of piezometric pressure at the outside of the bend and a decrease at the inside of the bend. This decrease in pressure may be so high that the flow separates from the solid boundary, and thus causes additional energy losses due to turbulence. Losses in elbows and bends in excess of those due to friction have been studied by various investigators, and may be expressed by an equation similar to Equation 19.56

$$\Delta H_b = \xi_b \frac{v_a^2}{2g} \tag{19.61}$$

The approximate value of the head loss coefficient, ξ_b, for an elbow is given in Table 19.10. ξ_b values for the square profile are somewhat higher than for the circular profile due to the less favourable velocity distribution and some consequent additional turbulence (Figure 19.45).

Table 19.10 ξ_b values for elbows

δ	5°	10°	15°	22.5°	30°	45°	60°	75°	90°
ξ_b value									
(O-profile)	0.02	0.03	0.04	0.05	0.11	0.24	0.47	0.80	1.1
ξ_b profile									
(□-profile)	0.02	0.04	0.05	0.06	0.14	0.3	0.6	1.0	1.4

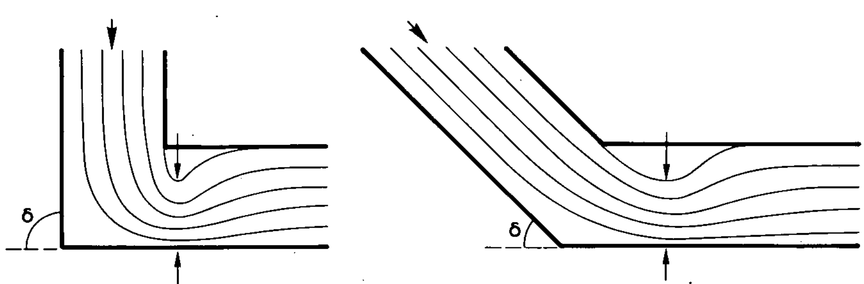


Figure 19.45 Flow separation in an elbow

Bend losses in closed conduits, in addition to those losses due to friction, can be expressed as a function of the ratio R_b/D, where R_b is the radius of the conduit centre line and D is the diameter of a circular conduit or the height of the conduit cross-section in the plane of the bend for rectangular conduits.

Figure 19.46 shows a curve giving suitable ξ_b values for large-diameter conduits as a function of R_b/D. As we can see, a ratio of R_b/D greater than 4 does not make savings in energy head commensurate with the extra expense, so this value is advisable as a maximum for conduits with subcritical flow. For bends of other than 90°, a correction factor should be applied to the values given in Figure 19.46. Values of this factor as a function of the angle, δ, of the bend are given in Figure 19.47.

Friction Losses
In culvert pipes with a free water surface, and in inlet and outlet transitions, the energy losses due to friction can be calculated with Manning's equation (Equation 19.12). With this equation, we can also calculate the slope, s, of the energy gradient, if we know the average velocity and the hydraulic radius. For this purpose we may use the average velocity, $(v_a + v)/2$, and the related hydraulic radius over an inlet or outlet transition.

We can find the summed head loss due to friction with

$$\Delta H_f = \Delta H_{f,in} + \Delta H_{f,pipe} + \Delta H_{f,out} \tag{19.62}$$

where each ΔH is the product of the slope of the hydraulic gradient and the length of the considered channel reach, s × L.

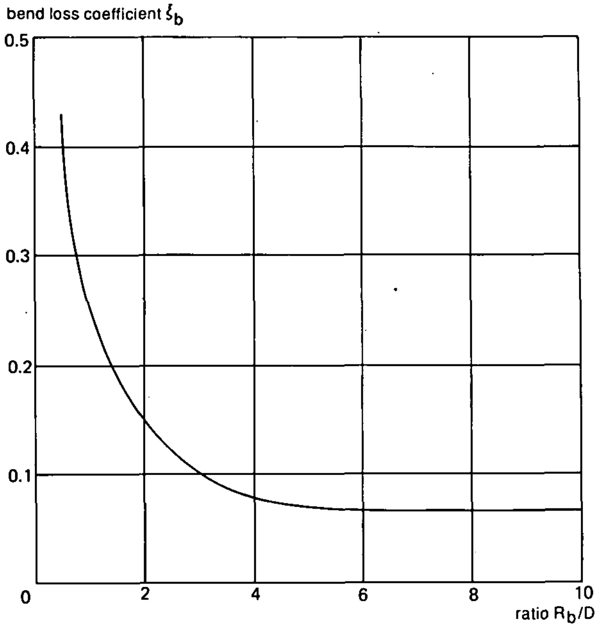


Figure 19.46 Suitable ξ_b values for large-diameter conduits (after U.S. Bureau of Reclamation 1967)

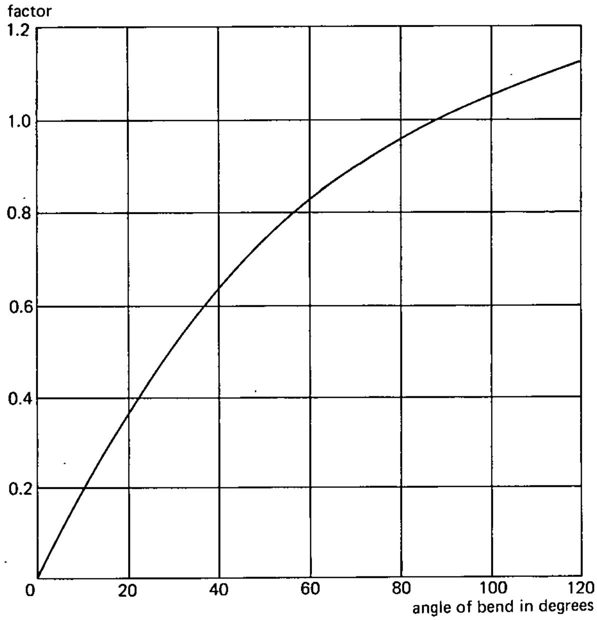


Figure 19.47 Reduction factor for bend loss coefficients in conduits

For full flowing pipes, use the Weisbach-Darcy equation to calculate the energy loss due to friction

$$\Delta H_{f,pipe} = n_t \frac{L}{D} \times \frac{v_a^2}{2g} \tag{19.63}$$

where

n_t = component resistance coefficient. For conservative culvert designs, use $n_t = 0.020$ (–)
D = average pipe diameter (m)

Use Equations 19.57 through 19.63 to calculate the total energy loss over a culvert or siphon. As this loss is linearly proportional to $v_a^2/2g$, we can calculate ΔH_{total} if we know the size and shape of the structure

$$\Delta H_{total} = \Delta H_{in} + \Delta H_f + \Delta H_b + \Delta H_{out} = \Sigma\,\text{coeff.}\frac{v_a^2}{2g} \tag{19.64}$$

If the total head loss is limited by an available value, use Equation 19.64 to calculate the velocity, v_a, and then select the shape and size of the component parts of the culvert to enable Q_d.

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