CHAPTER 6

STORAGE FACILITIES

6.5 Outlet Hydraulics

6.5.1 Outlets

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifice, or combinations of these facilities. If culverts are used as outlet works, procedures presented in the Design of Culverts chapter should be used to develop stage-discharge data.

6.5.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated below. The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + 0.4(H / H_c)]\, LH^{1.5} \qquad \textbf{(EQ-6.5.2-1)}$$

Where: Q = discharge, cfs
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

Figure 6.5.2-1
Sharp-Crested Weir
No End Contractions

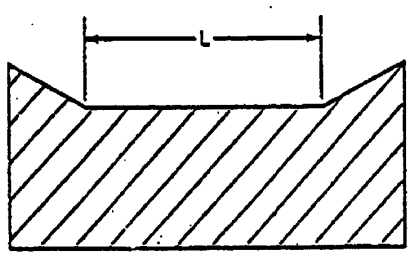
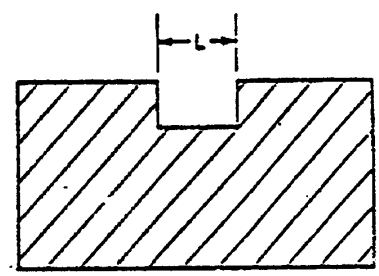


Figure 6.5.2-2
Sharp-Crested Weir
And Head

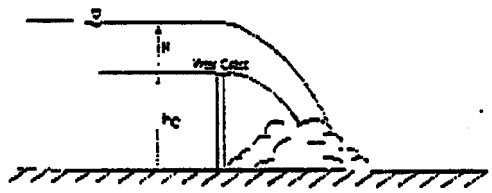


A sharp-crested weir with two end contractions is illustrated below. The discharge equation for this configuration is (Chow, 1959):

$$Q = [3.27 + .04(H / H_c)](L - 32H)\, H^{1.5} \qquad \textbf{(EQ-6.5.2-1)}$$

Where: Variables are the same as equation EQ-6.4.2-4.

Figure 6.5.2-3
Sharp-Crested Weir



A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_s = Q_f (1 - (H_2 / H_1)^{1.5})^{0.385} \qquad \textbf{(EQ-6.5.2-2)}$$

Where: Q_s = submergence flow, cfs
Q_f = free flow, cfs
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

6.5.3 Broad-Crested Weirs

The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad \textbf{(EQ-6.5.2-3)}$$

Where: Q = discharge, cfs
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft.

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C valve of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 6.5.5-1.

6.5.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5\tan(\theta/2)H^{2.5} \qquad \textbf{(EQ-6.5.4-1)}$$

Where: Q = discharge, cfs
θ = angle of v-notch, degrees
H = head on apex of notch, ft

6.5.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge areas to vary nonlinearly with head. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\,\mathrm{a}^{0.5}\,b(H - \mathrm{a}/3) \qquad \textbf{(EQ-6.5.5-1)}$$

$$x/b = 1 - (1/3.17)(\arctan(y/\mathrm{a})^{0.5}) \qquad \textbf{(EQ-6.5.5-2)}$$

Where: Q = discharge, cfs
Dimensions a, b, x, and y are shown in Figure 6.5.5-1.

Figure 6.5.5-1
Proportional Weir Dimensions

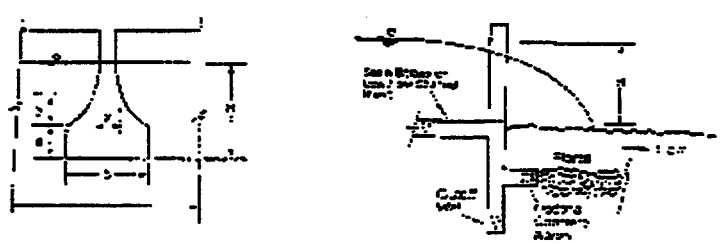
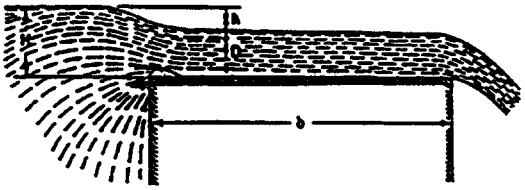


Table 6.5.5-1
Broad-Crested Weir Coefficient C Value As A Function of Weir Crest Breadth (b) And Head (H)



Measured Head, H[1]	Weir Crest Breadth (ft)										
(ft)	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	6.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Reference: Brater and King (1976).

6.5.6 Orifices

Pipes smaller than 12" may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

$$Q = 0.6A(2_g H)^{0.5} = 3.78D^2 H^{0.5} \qquad \textbf{(EQ-6.5.6-1)}$$

Where: Q = discharge, cfs
A = cross-section area of pipe, ft^2
g = acceleration due to gravity, 32.2 ft/s^2
D = diameter of pipe, ft
H = head on pipe, from the center of pipe to the water surface

6.5.7 Combination Outlets

Combinations of weirs, pipes and orifices can be put together to provide a variable control stage-discharge curve suitable for control of multiple storm flows. They are generally of two types: shared outlet control and separate outlet controls. Shared outlet control is typically a number of individual outlet openings, weirs or drops at different elevations on a riser pipe or box that all flow to a common larger conduit or pipe. Separate outlet controls are less common and normally consist of a single opening through the dam of a detention facility in combination with an overflow spillway for emergency use. For a complete discussion of outlets and combination outlets see Municipal Storm Water Management by Debo and Reese.

END OF SECTION 6.5