10.8 Outlet Hydraulics

10.8.1 General

A stage-discharge (performance) curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility will have both a principal and an emergency outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. The structure for the principal outlet will typically consist of a pipe culvert, weir, orifice, or other appropriate hydraulic control device. Multiple outlet control devices are often used to provide discharge controls for multiple frequency storms.

Development of a composite stage-discharge curve requires consideration of the discharge rating relationships for each component of the outlet structure. The following sections present design relationships for typical outlet controls.

10.8.2 Orifices

For a single orifice as illustrated in Figure 10-11 (a), orifice flow can be determined using equation 10.17.

$$Q = C_o A_o (2 g H_o)^{0.5} \tag{10.17}$$

where:
- Q = the orifice flow rate, m^3/s (ft^3/s)
- C_o = discharge coefficient (0.40 - 0.60)
- A_o = area of orifice, m^2 (ft^2)
- H_o = effective head on the orifice measured from the centroid of the opening, m (ft)
- g = gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

If the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces. This latter condition of a submerged discharge is shown in Figure 10-11(b).

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used.

For circular orifices with C_o set equal to 0.6, the following equation results:

$$Q = K_{or} D^2 H_o^{0.50} \tag{10.18}$$

where:
- K_{or} = 2.09 in S.I. units (3.78 in English units)
- D = orifice diameter, m (ft)

Pipes smaller than 0.3 m (1 ft) in diameter may be analyzed as a submerged orifice as long as H_o/D is greater than 1.5. Pipes greater than 0.3 m (1 ft) in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.

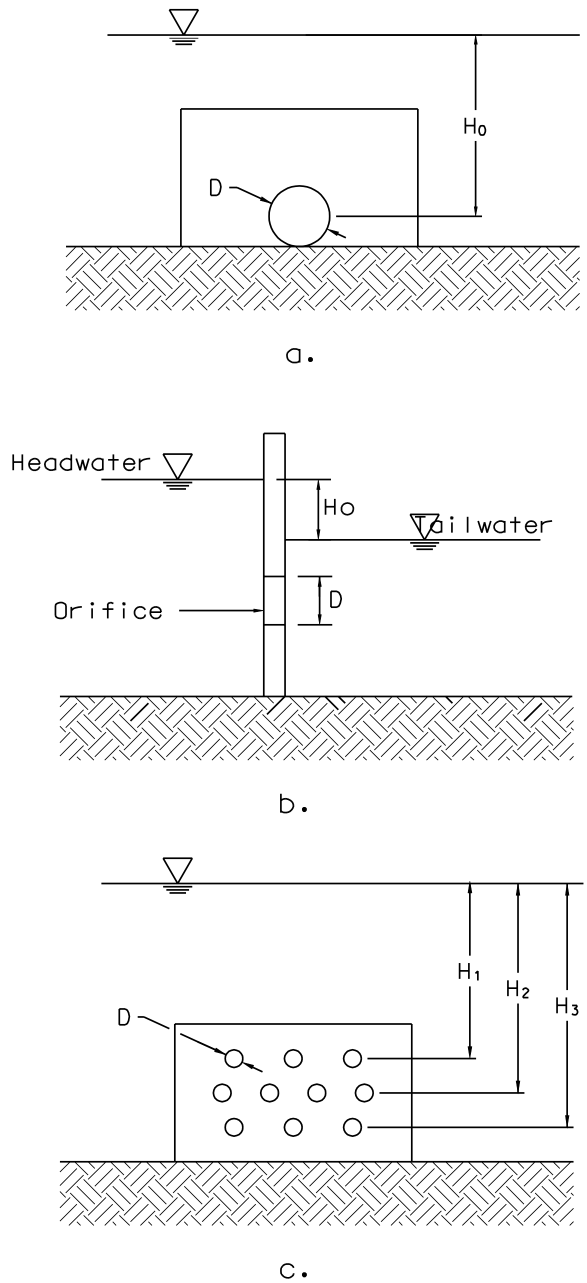

Figure 10-11 Definition Sketch for Orifice Flow

Flow through multiple orifices (see Figure 10-11(c)) can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings. The procedure is demonstrated in the following example:

Example 10-4

Given: *Given the orifice plate in Figure 10-11 (c) with a free discharge and:*

orifice diameter	=	*25 mm (1.0 in)*
H_1	=	*1.1 m (3.6 ft)*
H_2	=	*1.2 m (3.9 ft)*
H_3	=	*1.3 m (4.3 ft)*

Find: *Total discharge through the orifice plate.*

Solution: *Using a modification of equation 10.18 for multiple orifices,*

$$Q_i = K D^2 (H_i)^{0.5} N_i$$
$$Q_i = (2.09)(0.025)^2 (H_i)^{0.5} N_i = 0.0013 H_i^{0.5} N_i$$

$$Q_1 = 0.0013 (1.1)^{0.5} (3) = 0.0040$$
$$Q_2 = 0.0013 (1.2)^{0.5} (4) = 0.0058$$
$$Q_2 = 0.0013 (1.3)^{0.5} (3) = 0.0045$$

$$Q_{total} = Q_1 + Q_2 + Q_3 = 0.0143 \text{ m}^3/\text{s} \ (0.50 \text{ ft}^3/\text{s})$$

Example 10-5

Given: *Given the circular orifice in Figure 10-11(a) with:*

orifice diameter = 0.15 m (0.5 ft)
orifice invert = 10.0 m (32.8 ft)
discharge coeff. = 0.60

Find: *The stage - discharge rating between 10 m (32.8 ft) and 12.0 m (39.4 ft).*

Solution: *Using equation 10.17 with D = 0.15 m yields the following relationship between the effective head on the orifice (H_o) and the resulting discharge:*

$$Q = 0.047\, H_o^{0.5}$$
$$H_o = Depth - D/_2$$

The solution of this equation in table form is as follows:

Stage Discharge Tabulation

DEPTH		*STAGE*		*DISCHARGE*	
(meters)	*(feet)*	*(meters)*	*(feet)*	*(m^3/s)*	*(ft^3/s)*
0.00	*0.0*	*10.0*	*32.8*	*0.000*	*0.00*
0.20	*0.7*	*10.2*	*33.5*	*0.011*	*0.37*
0.40	*1.3*	*10.4*	*34.1*	*0.024*	*0.83*
0.60	*2.0*	*10.6*	*34.8*	*0.032*	*1.11*
0.80	*2.6*	*10.8*	*35.4*	*0.038*	*1.34*
1.00	*3.3*	*11.0*	*36.1*	*0.043*	*1.53*
1.20	*3.9*	*11.2*	*36.7*	*0.048*	*1.70*
1.40	*4.6*	*11.4*	*37.4*	*0.053*	*1.85*
1.60	*5.2*	*11.6*	*38.0*	*0.057*	*2.00*
1.80	*5.9*	*11.8*	*38.7*	*0.061*	*2.13*
2.00	*6.6*	*12.0*	*39.4*	*0.064*	*2.26*

10.8.3 Weirs

Relationships for sharp-crested, broad-crested, V-notch, and proportional weirs are provided in the following sections:

Sharp Crested Weirs

Typical sharp crested weirs are illustrated in Figure 10-12. Equation 10.19 provides the discharge relationship for **sharp crested weirs** with no end contractions (illustrated in Figure 12a).

$$Q = C_{scw}\ L\,H^{1.5} \qquad \textbf{(10.19)}$$

where: Q = discharge, m^3/s (ft^3/s)
L = horizontal weir length, m (ft)
H = head above weir crest excluding velocity head, m (ft)
C_{SCW} = $1.81 + 0.22\,(H/H_c)$ [$3.27 + 0.4\,(H/H_c)$ in English units]

As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c (see Figure 10-12c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{SCW} of 1.84 (3.33 in English units) is often used.

Equation 10.20 provides the discharge equation for **sharp-crested weirs with end contractions** (illustrated in Figure 10-12(b)). As indicated above, the value of the coefficient C_{scw} is known to vary with the ratio H/H_c (see Figure 10-13c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{scw} of 1.84 (3.33 in English units) is often used.

$$Q = C_{SCW}\,(L - 0.2\,H)\,H^{1.5} \qquad \textbf{(10.20)}$$

Sharp-crested weirs will be effected by submergence when the tailwater rises above the weir crest elevation, as shown in Figure 10-12(d). The result will be that the discharge over the weir will be reduced.

The discharge equation for a **submerged sharp-crested weir** is:

$$Q_s = Q_r\,(1 - (H_2 / H_1)^{1.5})^{0.385} \qquad \textbf{(10.21)}$$

where: Q_s = submerged flow, m^3/s (ft^3/s)
Q_r = unsubmerged weir flow from equation 10.19 or 10.20, m^3/s (ft^3/s)
H_1 = upstream head above crest, m (ft)
H_2 = downstream head above crest, m (ft)

Flow over the top edge of a riser pipe is typically treated as flow over a sharp crested weir with no end constrictions. Equation 10.19 should be used for this case.

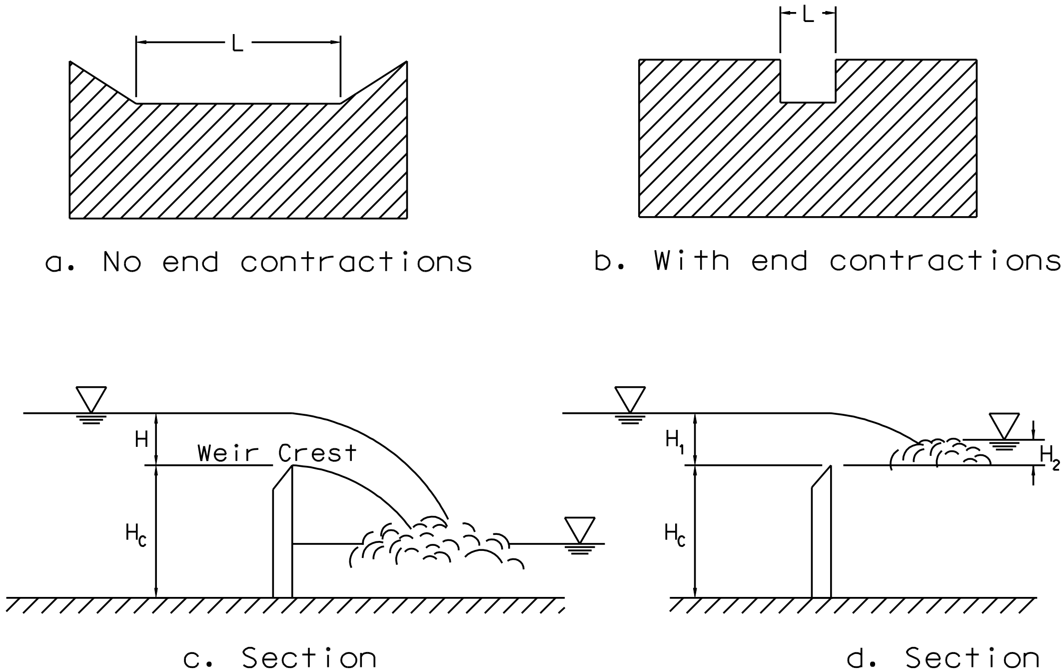

Figure 10-12 Sharp Crested Weirs

Example 10-6

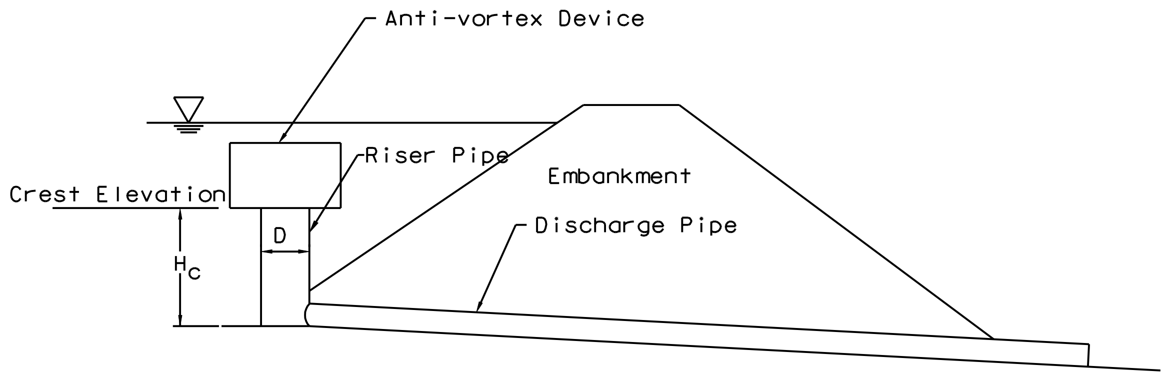

Figure 10-13 Riser Pipe

Given: *A riser pipe as shown in Figure 10-13 with the following characteristics:*

diameter (D) = 0.53 m (1.75 ft)
crest elevation = 10.8 m (35.4 ft)
weir height (H_c) = 0.8 m (2.6 ft)

Find: *The stage - discharge rating for the riser pipe between 10 m (32.8 ft) and 12.0 m (39.4 ft).*

Solution: *Since the riser pipe functions as both a weir and an orifice (depending on stage), the rating is developed by comparing the stage - discharge produced by both weir and orifice flow as follows:*

Using equation 10.18 for orifices with D = 0.53 m (1.75 ft) yields the following relationship between the effective head on the orifice (H_o) and the resulting discharge:

$$Q = K_{or} D^2 H_o^{0.50}$$
$$Q = (2.09)(0.53)^2 H_o^{0.50}$$
$$Q = 0.587 H_o^{0.50}$$

Using equation 10.19 for sharp crested weirs with C_{SCW} = 1.84 (H/H_c assumed less than 0.3), and L = pipe circumference = 1.67 m (5.5 ft) yields the following relationship between the effective head on the riser (H) and the resulting discharge:

$$Q = C_{SCW} L H^{1.5}$$
$$Q = (1.84)(1.67) H^{1.5}$$
$$Q = 3.073 H^{1.5}$$

The resulting stage - discharge relationship is summarized in the following table:

STAGE		*EFFECTIVE HEAD*		*ORIFICE FLOW*		*WEIR FLOW*	
(m)	*(ft)*	*(m)*	*(ft)*	*(m^3/s)*	*(ft^3/s)*	*(m^3/s)*	*(ft^3/s)*
10.0	*32.8*	*0.0*	*0.0*	*0.00*	*0.0*	*0.00*	*0.0*
10.8	*35.4*	*0.0*	*0.0*	*0.00*	*0.0*	*0.00*	*0.0*
10.9	*35.7*	*0.1*	*0.3*	*0.19*	*6.6*	*0.10*	*3.4*
11.0	*36.1*	*0.2*	*0.7*	*0.26*	*9.2*	*0.27*	*9.5*
11.2	*36.7*	*0.4*	*1.3*	*0.37*	*13.1*	*0.78*	*27.5*
11.4	*37.4*	*0.6*	*2.0*	*0.45*	*15.9*	*1.43*	*50.5*
11.6	*38.1*	*0.8*	*2.6*	*0.53*	*18.7*	*2.20*	*77.7*
11.8	*38.7*	*1.0*	*3.3*	*0.59*	*20.8*	*3.07*	*108.4*
12.0	*39.4*	*1.2*	*3.9*	*0.64*	*22.6*	*4.04*	*142.7*

Designates controlling flow.

The flow condition, orifice or weir, producing the lowest discharge for a given stage defines the controlling relationship. As illustrated in the above table, at a stage of 10.9 m (35.7 ft) weir flow controls the discharge through the riser. However, at and above a stage of 11.0 m (36.1 ft), orifice flow controls the discharge through the riser.

Broad-Crested Weir

The equation typically used for a broad-crested weir is:

$$Q = C_{BCW} L H^{1.5} \tag{10.22}$$

where:
- Q = discharge, m^3/s (ft^3/s)
- C_{BCW} = broad-crested weir coefficient, 1.44 - 1.70 (2.61 to 3.08)
- L = broad-crested weir length, m (ft)
- H = head above weir crest, m (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 1.70. For sharp corners on the broad crested weir, a minimum value of 1.44 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 10-2.

Table 10-2 Broad-crested weir coefficient C values as a function of weir crest breadth and head (coefficient has units of $m^{0.5}$/sec) (metric only)

Head[(1)] (m)	BREADTH OF CREST OF WEIR (m)														
	0.15	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.25	1.50	2.00	3.00	4.00
0.10	1.59	1.56	1.50	1.47	1.45	1.43	1.42	1.41	1.40	1.39	1.37	1.35	1.36	1.40	1.45
0.15	1.65	1.60	1.51	1.48	1.45	1.44	1.44	1.44	1.45	1.45	1.44	1.43	1.44	1.45	1.47
0.20	1.73	1.66	1.54	1.49	1.46	1.44	1.44	1.45	1.47	1.48	1.48	1.49	1.49	1.49	1.48
0.30	1.83	1.77	1.64	1.56	1.50	1.47	1.46	1.46	1.46	1.47	1.47	1.48	1.48	1.48	1.46
0.40	1.83	1.80	1.74	1.65	1.57	1.52	1.49	1.47	1.46	1.46	1.47	1.47	1.47	1.48	1.47
0.50	1.83	1.82	1.81	1.74	1.67	1.60	1.55	1.51	1.48	1.48	1.47	1.46	1.46	1.46	1.45
0.60	1.83	1.83	1.82	1.73	1.65	1.58	1.54	1.46	1.31	1.34	1.48	1.46	1.46	1.46	1.45
0.70	1.83	1.83	1.83	1.78	1.72	1.65	1.60	1.53	1.44	1.45	1.49	1.47	1.47	1.46	1.45
0.80	1.83	1.83	1.83	1.82	1.79	1.72	1.66	1.60	1.57	1.55	1.50	1.47	1.47	1.46	1.45
0.90	1.83	1.83	1.83	1.83	1.81	1.76	1.71	1.66	1.61	1.58	1.50	1.47	1.47	1.46	1.45
1.00	1.83	1.83	1.83	1.83	1.82	1.81	1.76	1.70	1.64	1.60	1.51	1.48	1.47	1.46	1.45
1.10	1.83	1.83	1.83	1.83	1.83	1.83	1.80	1.75	1.66	1.62	1.52	1.49	1.47	1.46	1.45
1.20	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.70	1.65	1.53	1.49	1.48	1.46	1.45
1.30	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.82	1.77	1.71	1.56	1.51	1.49	1.46	1.45
1.40	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.77	1.60	1.52	1.50	1.46	1.45
1.50	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.66	1.55	1.51	1.46	1.45
1.60	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.81	1.74	1.58	1.53	1.46	1.45

(1) Measured at least 2.5 H_c upstream of the weir

V-Notch Weir

The discharge through a v-notch weir is shown in Figure 10-14 and can be calculated from the following equation:

$$Q = 1.38 \tan(\theta / 2)\, H^{2.5} \qquad \textbf{(10.23)}$$

where: Q = discharge, m^3/s (ft^3/s)
θ = angle of v-notch, degrees
H = head on apex of v-notch, m (ft)

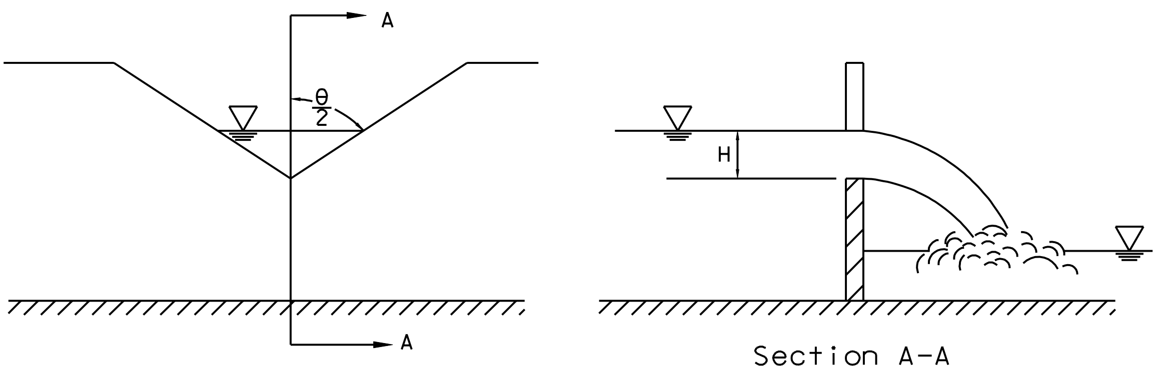

Figure 10-14 V-notch Weir

Proportional Weir

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. This relationship is achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are as follows:

$$Q = 2.74\, a^{0.5}\, b\,(H - a/3) \qquad \textbf{(10.24)}$$

$$x/b = 1 - (0.315)\,[\arctan(y/a)^{0.5}] \qquad \textbf{(10.25)}$$

where: Q = discharge, m^3/s (ft^3/s)
H = head above horizontal sill, m (ft)
Dimensions a, b, x, and y are as shown in Figure 10-15.

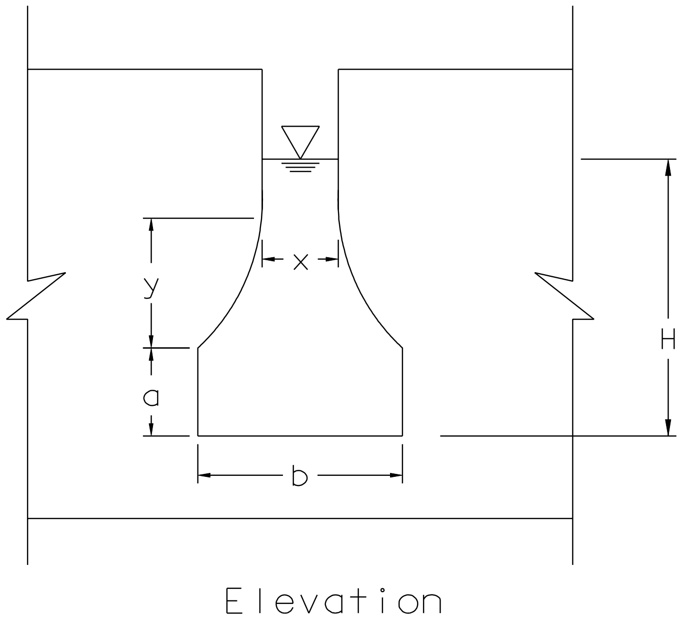

Figure 10-15 Proportional Weir Dimensions

10.8.4 Discharge Pipes

Discharge pipes are often used as outlet structures for detention facilities. The design of these pipes can be for either single or multistage discharges. A single stage discharge system would consist of a single culvert entrance system and would not be designed to carry emergency flows. A multistage inlet would involve the placement of a control structure at the inlet end of the pipe. The inlet structure would be designed in such a way that the design discharge would pass through a weir or orifice in the lower levels of the structure and the emergency flows would pass over the top of the structure. The pipe would need to be designed to carry the full range of flows from a drainage area including the emergency flows.

For single stage systems, the facility would be designed as if it were a simple culvert. Appropriate design procedures are outlined in Chapter 8, Culverts. For multistage control structures, the inlet control structure would be designed considering both the design flow and the emergency flows. A stage-discharge curve would be developed for the full range of flows that the structure would experience. The design flows will typically be orifice flow through whatever shape the designer has chosen while the higher flows will typically be weir flow over the top of the control structure. Orifices can be designed using the equations in section 10.8.2 and weirs can be designed using the equations in section 10.8.3. The pipe must be designed to carry all flows considered in the design of the control structure.

In designing a multistage structure, the designer would first develop peak discharges that must be passed through the facility. The second step would be to select a pipe that will pass the peak flow within the allowable headwater and develop a performance curve for the pipe. Thirdly, the designer would develop a stage-discharge curve for the inlet control structure, recognizing that the headwater for the discharge pipe will be the tailwater that needs to be considered in designing the inlet structure. And lastly, the designer would use the stage-discharge curve in the basin routing procedure.

Example 10-7

Given: *A corrugated steel discharge pipe as shown in Figure 10-13 with the following characteristics:*

maximum head on pipe	=	*0.75 m (2.3 ft) (conservative value of 0.05 m less than the riser height*
inlet invert	=	*10.0 m (32.8 ft)*
length (L)	=	*50 m (164 ft)*
slope	=	*0.04 m/m (ft/ft)*
roughness	=	*0.024*
square edge entrance (K_e	=	*0.5)*

discharge pipe outfall is free (not submerged)
Runoff characteristics as given

Find: *The size pipe needed to carry the maximum allowable flow rate from the detention basin.*

Solution: *The maximum predeveloped discharge from the watershed is 0.55 m^3/s (19.4 ft^3/s). Since the discharge pipe can function under inlet or outlet control, the pipe size will be evaluated for both conditions. The larger pipe size will be selected for the final design.*

Using chart 2 from HDS-5 yields the relationship between head on the pipe and the resulting discharge for inlet control. From the chart, the pipe diameter necessary to carry the flow is 750 mm (30 in).

Using chart 6 from HDS-5 yields the relationship between head on the pipe and discharge for barrel control. From the chart, the pipe diameter necessary to carry the flow is 675 mm (30 in).

10.8.5 Emergency Spillway

The purpose of an emergency spillway is to provide a controlled overflow relief for storm flows in excess of the design discharge for the storage facility. An inlet control structure is commonly used to release emergency flows. Another suitable emergency spillway for detention storage facilities for highway applications is a broad-crested overflow weir cut through the original ground next to the embankment. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated in Figure 10-16. The invert of the spillway at the outfall should be at an elevation 0.3 m (1 ft) to 0.6 m (2 ft) above the maximum design storage elevation. It is preferable to have a freeboard of 0.6 m (2 ft) minimum. However, for very small impoundments (less than 0.4 to 0.8 hectare surface area) an absolute minimum of 0.3 meter of freeboard may be acceptable.

Equation 10.26 presents a relationship for computing the flow through a broad-crested emergency spillway. The dimensional terms in the equation are illustrated in Figure 10-16.

$$Q = C_{SP}\, b\, H_p^{1.5} \qquad \textbf{(10.26)}$$

where: Q = emergency spillway discharge, m^3/s (ft^3/s)
C_{SP} = discharge coefficient
b = width of the emergency spillway, m (ft)
H_p = effective head on the emergency spillway, m (ft)

The discharge coefficient, C_{SP}, in equation 10.26 varies as a function of spillway bottom width and effective head. Figure 10-17 illustrates this relationship. Table 10-3 provides a tabulation of emergency spillway design parameters.

The critical slopes of Table 10-3 are based upon an assumed n = 0.040 for turf cover of the spillway. For a paved spillway, the n should be assumed as 0.015. Equations 10.27 and 10.28 can be used to compute the critical velocity and slope for spillway materials having other roughness values.

$$V_c = K_{SP} \left(\frac{Q}{b} \right)^{0.33} \qquad \textbf{(10.27)}$$

where: V_c = critical velocity at emergency spillway control section, m/s (ft/s)
Q = emergency spillway discharge, m^3/s (ft^3/s)
b = width of the emergency spillway, m (ft)
K_{SP} = 2.14 (3.18 in English units)

$$S_c = K_{SP}' \, n^2 \left(\frac{V_c \, b}{Q} \right)^{0.33} \qquad \textbf{(10.28)}$$

where: S_c = critical slope, m/m (ft/ft)
n = Manning's coefficient
V_c = critical velocity at emergency spillway control section, m/s (ft/s)
Q = emergency spillway discharge, m^3/s (ft^3/s)
b = width of the emergency spillway, m (ft)
K_{SP}' = 9.84 (14.6 in English units)

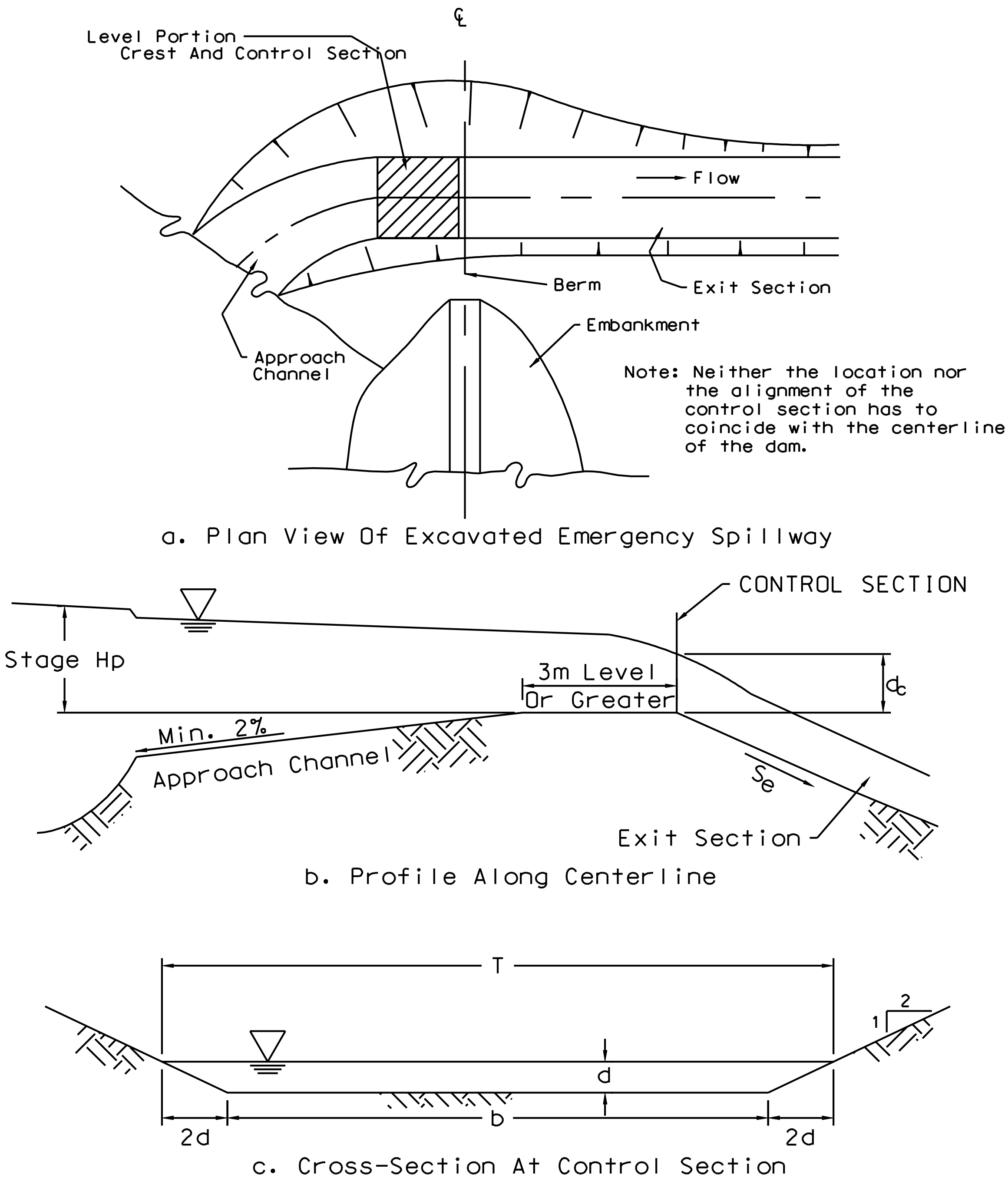

Figure 10-16 Emergency Spillway Design Schematic

Table 10-3 Emergency spillway design parameters (metric units)

H_p (m)		Spillway Bottom Width, b, meters														
		2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
	Q	0.19	0.26	0.35	0.40	0.44	0.50	0.58	0.65	0.69	0.73	-	-	-	-	-
0.20	V_c	0.98	1.02	1.06	1.05	1.03	1.04	1.05	1.06	1.05	1.04	-	-	-	-	-
	S_c	3.4%	3.3%	3.2%	3.2%	3.3%	3.3%	3.2%	3.2%	3.2%	3.2%	-	-	-	-	-
	Q	0.34	0.43	0.52	0.60	0.67	0.75	0.85	0.94	1.02	1.09	-	-	-	-	-
0.25	V_c	1.19	1.20	1.20	1.19	1.19	1.19	1.19	1.20	1.19	1.19	-	-	-	-	-
	S_c	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	-	-	-	-	-
	Q	0.53	0.63	0.72	0.83	0.95	1.06	1.18	1.29	1.41	1.52	1.64	1.75	1.87	1.96	2.07
0.30	V_c	1.38	1.36	1.34	1.33	1.33	1.33	1.33	1.33	1.33	1.33	1.33	1.32	1.32	1.32	1.32
	S_c	2.7%	2.7%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%
	Q	0.68	0.82	0.95	1.10	1.24	1.37	1.51	1.66	1.81	1.94	2.08	2.21	2.34	2.49	2.62
0.35	V_c	1.50	1.48	1.46	1.46	1.45	1.45	1.44	1.44	1.44	1.44	1.43	1.43	1.43	1.43	1.42
	S_c	2.5%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%
	Q	0.86	1.04	1.20	1.38	1.55	1.72	1.89	2.07	2.25	2.42	2.58	2.74	2.90	3.09	3.27
0.40	V_c	1.62	1.60	1.58	1.57	1.57	1.56	1.55	1.55	1.55	1.55	1.54	1.53	1.53	1.53	1.53
	S_c	2.4%	2.4%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%
	Q	1.05	1.27	1.48	1.69	1.90	2.11	2.32	2.53	2.74	2.95	3.15	3.35	3.56	3.78	4.00
0.45	V_c	1.73	1.71	1.70	1.68	1.67	1.67	1.66	1.66	1.65	1.65	1.64	1.64	1.64	1.64	1.64
	S_c	2.3%	2.3%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%
	Q	1.27	1.55	1.81	2.05	2.30	2.54	2.79	3.05	3.30	3.54	3.79	4.05	4.31	4.55	4.79
0.50	V_c	1.84	1.83	1.81	1.79	1.78	1.77	1.77	1.76	1.75	1.75	1.75	1.75	1.74	1.74	1.74
	S_c	2.2%	2.2%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%
	Q	1.54	1.85	2.13	2.43	2.73	3.03	3.33	3.60	3.91	4.19	4.47	4.75	5.02	5.30	5.58
0.55	V_c	1.96	1.94	1.91	1.90	1.89	1.88	1.87	1.86	1.86	1.85	1.85	1.84	1.84	1.83	1.83
	S_c	2.1%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%
	Q	1.84	2.18	2.48	2.81	3.17	3.52	3.86	4.19	4.53	4.85	5.18	5.52	5.85	6.17	6.50
0.60	V_c	2.08	2.05	2.01	1.99	1.98	1.97	1.96	1.96	1.95	1.94	1.94	1.93	1.93	1.93	1.92
	S_c	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%

NOTE: 1. For a given H_p, decreasing exit slope from S_c decreases spillway discharge, but increasing exit slope from S_c does not increase discharge.

2. If a slope S_e steeper than S_c is used, velocity V_e in the exit channel will increase according to the following relationship:

$$V_e = V_c\,(S_e/S_c)^{0.3}$$

3. One meter is 3.28 feet; one cubic meter is 35.28 cubic feet

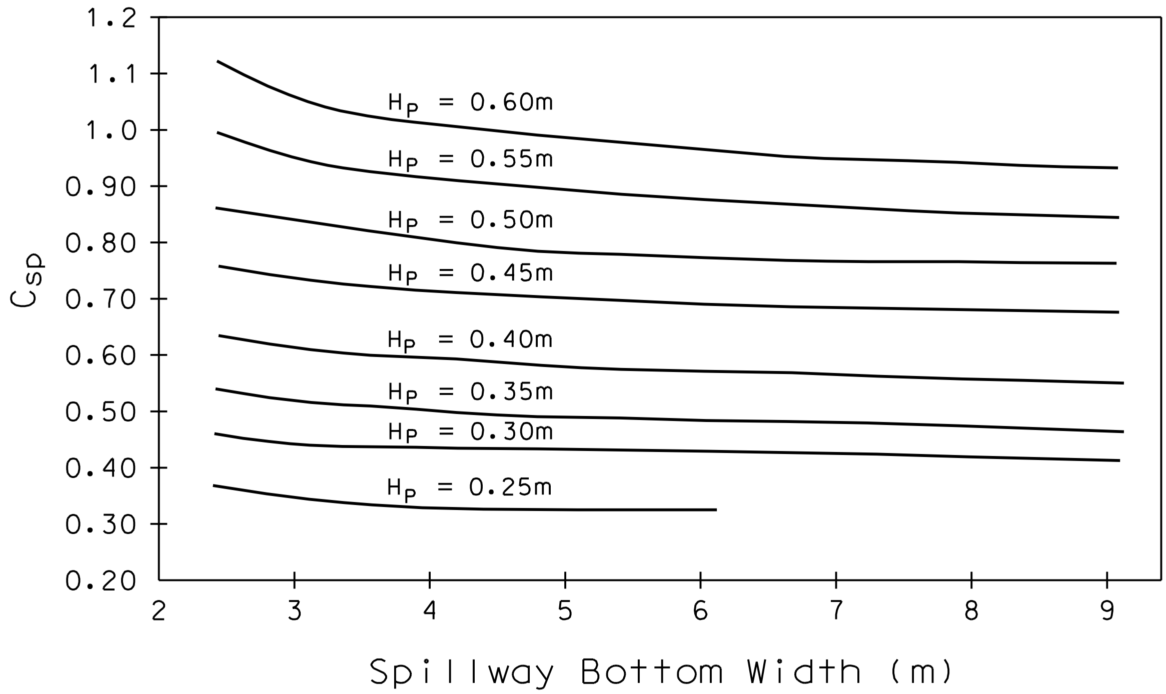

Figure 10-17 Discharge Coefficients for Emergency Spillways

Example 10-8

Given: *An emergency spillway with the following characteristics:*

invert elev. = 11.6 m (38.0 ft)
width (b) = 5 m (16.4 ft)
discharge coeff. (C_{SP}) = 1.5 (2.7) (assumed constant)

Find: *The stage - discharge rating for the spillway up to an elevation of 12.0 m (39.4 ft).*

Solution: *Using equation 10.26 with the given parameters yields:*

$$Q = C_{SP}\, b\, Hp^{1.5}$$
$$Q = (1.5)(5)Hp^{1.5} = 7.5\,(H_p)^{1.5}$$

STAGE		*EFFECTIVE HEAD ON SPILLWAY*		*SPILLWAY DISCHARGE*	
(m)	*(ft)*	*(m)*	*(ft)*	*(m^3/s)*	*(ft^3/s)*
11.6	*38.0*	*0.00*	*0.00*	*0.00*	*0.0*
11.7	*38.4*	*0.10*	*0.33*	*0.24*	*8.5*
11.8	*38.7*	*0.20*	*0.66*	*0.67*	*23.6*
11.9	*39.0*	*0.30*	*0.98*	*1.23*	*43.4*
12.0	*39.4*	*0.40*	*1.31*	*1.90*	*67.0*

10.8.6 Composite Stage Discharge Curves

As indicated by the discussions in the preceding sections, development of a stage - discharge curve for a particular outlet control structure will depend on the interaction of the individual ratings for each component of the control structure. Figure 10-18 illustrates the construction of a stage - discharge curve for an outlet control device consisting of a low flow orifice and a riser pipe connected to an outflow pipe. The structure also includes an emergency spillway. These individual components are as described in examples 10-5, 10-6, and 10-8.

The impact of each element in the control structure can be seen in Figure 10-18. Initially, the low flow orifice controls the discharge. At an elevation of 10.8 m (35.4 ft) the water surface in the storage facility reaches the top of the riser pipe and begins to flow into the riser. The flow at this point is a combination of the flows through the orifice and the riser. As indicated in example 10-6, orifice flow through the riser controls the riser discharge above a stage of 11.0 m (36.1 ft). At an elevation of 11.6 meters (38.0 ft), flow begins to pass over the emergency spillway. Beyond this point, the total discharge from the facility is a summation of the flows through the low flow orifice, the riser pipe, and the emergency spillway. The data used to construct the curves in Figure 10-18 are tabulated in Table 10-4. Additionally, the designer needs to ensure that the outlet pipe from the detention basin is large enough to carry the total flows from the low orifice and the riser section. This ensures that the outlet pipe is not controlling the flow from the basin.

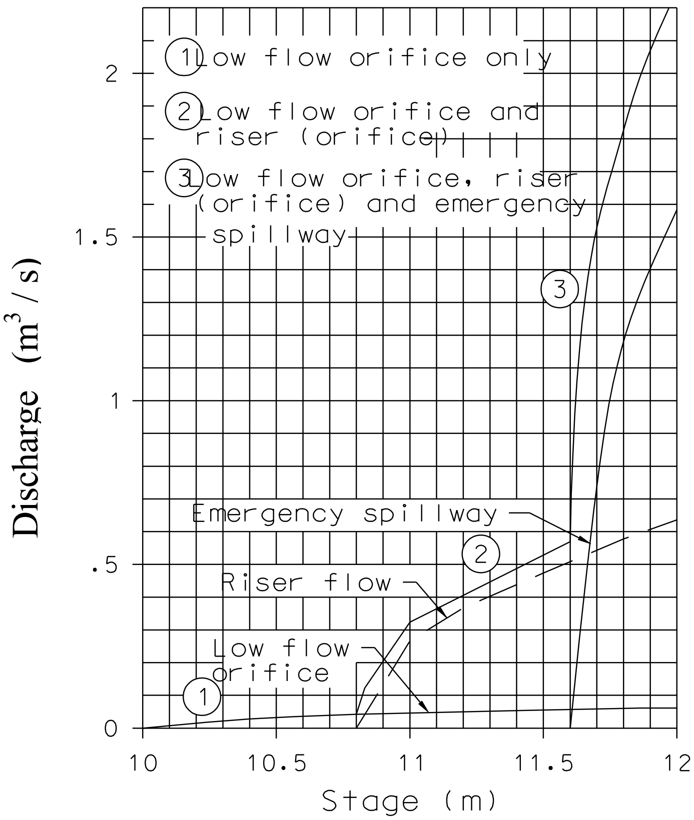

Figure 10-18 Typical Combined Stage-Discharge Relationship

Table 10-4 Stage - discharge tabulation.

STAGE		LOW FLOW ORIFICE	RISER ORIFICE FLOW	EMERGENCY SPILLWAY	TOTAL DISCHARGE	
(m)	(ft)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(ft^3/s)
10.0	32.8	0.000	0.00	0.00	0.00	0.0
10.2	33.5	0.011	0.00	0.00	0.01	0.4
10.4	34.1	0.024	0.00	0.00	0.02	0.8
10.6	34.8	0.032	0.00	0.00	0.03	1.1
10.8	35.4	0.038	0.00	0.00	0.04	1.3
11.0	36.1	0.043	0.26	0.00	0.31	10.7
11.2	36.7	0.048	0.37	0.00	0.42	14.8
11.4	37.4	0.053	0.45	0.00	0.50	17.7
11.6	38.1	0.057	0.53	0.00	0.59	20.7
11.8	38.7	0.061	0.59	1.12	1.77	62.5
12	39.4	0.064	0.64	1.58	2.28	80.6