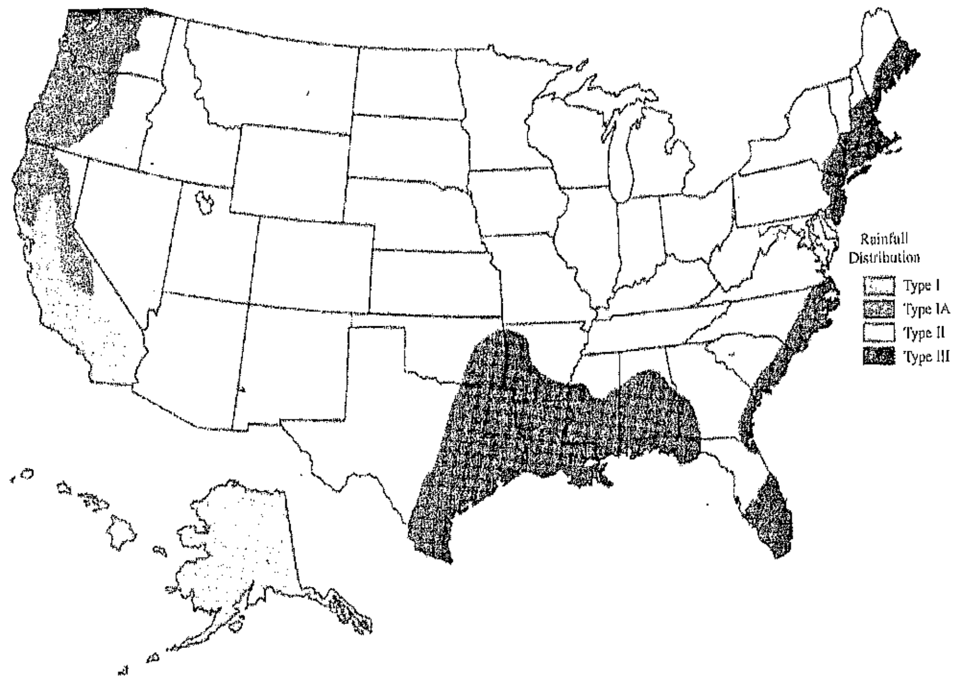

FIGURE 4.9 Regions of applicability of SCS 24-hr rainfall distributions.

TABLE 4.11 SCS Cumulative Rainfall Distributions

24-hour storm						6-hour storm		
		P_t/P_{24}						
Hour t	$t/24$	Type I	Type IA	Type II	Type III	Hour t	$t/6$	P_t/P_6
0	0	0	0	0	0	0	0	0
2.0	0.083	0.035	0.050	0.022	0.020	0.60	0.10	0.04
4.0	0.167	0.076	0.116	0.048	0.043	1.20	0.20	0.10
6.0	0.250	0.125	0.206	0.080	0.072	1.50	0.25	0.14
7.0	0.292	0.156	0.268	0.098	0.089	1.80	0.30	0.19
8.0	0.333	0.194	0.425	0.120	0.115	2.10	0.35	0.31
8.5	0.354	0.219	0.480	0.133	0.130	2.28	0.38	0.44
9.0	0.375	0.254	0.520	0.147	0.148	2.40	0.40	0.53
9.5	0.396	0.303	0.550	0.163	0.167	2.52	0.42	0.60
9.75	0.406	0.362	0.564	0.172	0.178	2.64	0.44	0.63
10.0	0.417	0.515	0.577	0.181	0.189	2.76	0.46	0.66
10.5	0.438	0.583	0.601	0.204	0.216	3.00	0.50	0.70
11.0	0.459	0.624	0.624	0.235	0.250	3.30	0.55	0.75
11.5	0.479	0.654	0.645	0.283	0.298	3.60	0.60	0.79
11.75	0.489	0.669	0.655	0.357	0.339	3.90	0.65	0.83
12.0	0.500	0.682	0.664	0.663	0.500	4.20	0.70	0.86
12.5	0.521	0.706	0.683	0.735	0.702	4.50	0.75	0.89
13.0	0.542	0.727	0.701	0.772	0.751	4.80	0.80	0.91
13.5	0.563	0.748	0.719	0.799	0.785	5.40	0.90	0.96
14.0	0.583	0.767	0.736	0.820	0.811	6.00	1.00	1.00
16.0	0.667	0.830	0.800	0.880	0.886			
20.0	0.833	0.926	0.906	0.952	0.957			
24.0	1.000	1.000	1.000	1.000	1.000			

Source: U.S. Dept. of Agriculture, Soil Conservation Service, 1973, 1986.

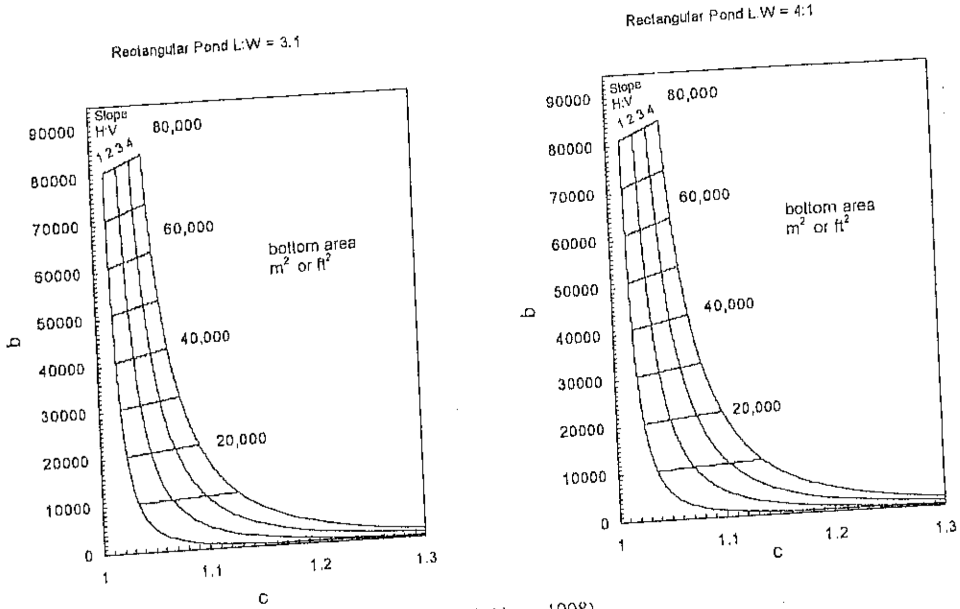

FIGURE 7.4 Values of b and c. (After Currey and Akan, 1998)

$$S = bh^c$$

where S = storage above the outlet
h = stage above the outlet
b, c = constant parameters

The constant c is dimensionless, and the constant b has the dimension of $[\text{length}]^{3-c}$. These constant parameters depend on the size and the shape of the pond. For instance, for a pond that has vertical sidewalls, $c = 1$ and b = horizontal area. Figure 7.4 displays approximate relationships between the parameter b and c, the base area, length to width ratio, and the side slope for trapezoidal ponds that has a rectangular base area and a constant side slope of z.

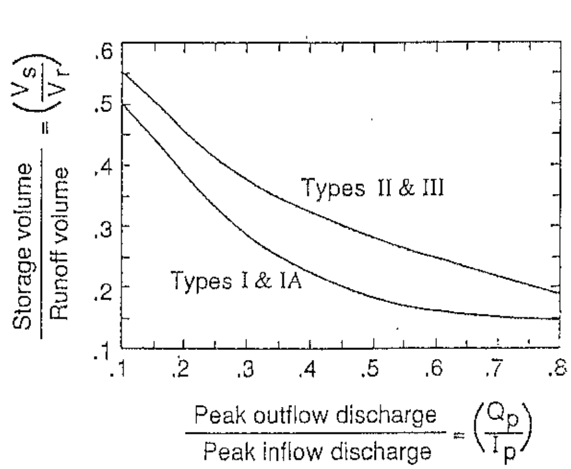

FIGURE 7.10 SCS chart for detention basin sizing. (After SCS, 1986)

Example 7.4

A detention basin will be built at the outlet of a 75-acre watershed. The stage-storage relationship for the planned basin is shown in Fig. 7.11. A weir-type outlet that has a discharge coefficient of $k_w = 0.40$ will be used. The depth of runoff resulting from a 25-year type-II design storm is 3.4 inches, and the peak runoff rate is 360 cfs. This peak should be reduced to 180 cfs at the detention basin. We are to determine the required storage volume and size of the weir if the weir crest is at elevation 100 ft.

From the problem statement $I_p = 360$ cfs, $Q_p = 180$ cfs, $Q_p/I_p = 0.50$, and $V_r = (3.4)(75)/12 = 21.2$ acre-ft. Using Fig. 7.10, for a type-II storm, $V_s/V_r = 0.28$. The required storage volume becomes, $V_s = (0.28)(21.2) = 5.94$ acre-ft.

The maximum storage volume of 5.94 acre-ft represents a water stage of 105.7 ft as we can see from Fig. 7.11. Since the crest elevation is 100 ft, the maximum head above the crest becomes, $h_p = 105.7 - 100 = 5.7$ ft. Finally, from Eq. (7.10), we can find the corresponding crest length as

$$L = 180/[(0.40)\sqrt{2(32.2)}(5.7)^{1.5}] = 4.1 \text{ ft}$$

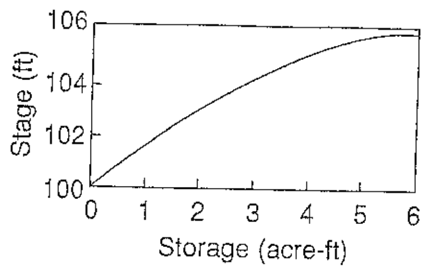

FIGURE 7.11 Stage-storage relationship for Examples 7.4 and 7.5.

Example 7.5

The watershed and the detention basin of Example 7.4 is considered again. In addition to the requirements of Example 7.4, it is decided to limit the 2-year outflow rate to 50 cfs. A 2-year type-II design storm produces a runoff depth of 1.5 in and a peak runoff rate of 91 cfs. The outlet structure will be a two-stage weir. The lower crest elevation is 100 ft. We are to determine the crest length for the lower and upper stages.

First, we consider the 2-year storm. The lower crest of the weir will be designed to control the runoff from this storm. From the problem statement, for the 2-year type-II storm, $I_p = 91$ cfs, $Q_p = 50$ cfs, $Q_p/I_p = 0.55$, and $V_r = (1.5)(75)/12 = 9.4$ ac-ft. From Fig. 7.10, $V_s/V_r = 0.26$, and therefore $V_s = (0.26)(9.4) = 2.4$ ac-ft. This represents a stage of 103.6 as obtained from Fig. 7.11. Thus, the maximum head over the lower crest is $h_p = 103.6 - 100 = 3.6$ ft, and from Eq. (7.10), the corresponding crest length will be

$$L = 50/[(0.40)\sqrt{(2)(32.2)}(3.6)^{1.5}] = 2.3 \text{ ft}$$

Now, we will consider the second stage. The upper weir crest is meant to operate for the 25-year storm but not for the 2-year storm. Therefore, the crest elevation for the second stage is chosen as $100 + 3.6 = 103.6$ ft. From Example 7.4, the maximum water stage resulting from a 25-year storm is 105.7 ft. The head over the lower crest at this stage is $105.7 - 100 = 5.7$ ft. From Eq. (7.10), the peak discharge over the lower crest, Q_{p1}, will be

$$Q_{p1} = 0.40\sqrt{2(32.2)}(2.3)(5.7)^{1.5} = 100 \text{ cfs}$$

Then, the peak discharge to be accommodated over the upper crest is $Q_{p2} = Q_p - Q_{p1} = 180 - 100 = 80$ ft. This discharge will occur under a head of $105.7 - 103.6 = 2.1$ ft. Therefore, from Eq. (7.13)

$$L = 80/[0.40\sqrt{2(32.2)}(2.1)^{1/5}] = 8.2 \text{ ft}$$

for the upper stage.

Example 7.7

An extended detention basin is to be designed for a 150-acre watershed in Norfolk, Virginia that is 40% impervious. Determine the required size if the detained runoff is to be released over 36 hours.

From Fig. 7.13, P_6 = 0.67 in. for Norfolk, Virginia. Because the watershed is 40% impervious, I = 0.40. Also, interpolating the a_r values between 24 and 48 hours, a_r = 1.422 for 36 hours. Then, from Eq. (7.20), we obtain P_o = 0.27 in. Therefore, the volume of runoff to be detained is (150)(0.27/12) = 3.38 acre ft = 147,233 ft^3. It is advisable to increase this volume by about 20% for sedimentation.

Example 7.8

An extended detention basin has a bottom length of 80 ft, a width of 20 ft, and side slopes of 3:1(H:V). The outlet is to be sized so that it will release a water quality volume of 10,200 ft^3 over a period of 40 hours.

To determine the depth of water corresponding to the water quality volume, Eq. (7.3) is written as

$$10200 = (80)(20)d + (80 + 20)3d^2 + (4/3)3^2\, d^3$$

By trial and error, d = 3.6 ft. Let the outlet structure be composed of ½-in circular ragged edge orifice holes cut around a riser pipe. Let the average elevation of the holes be 1 ft above the pond bottom. Therefore, the average head over the orifice holes is (3.6 − 1.0)/2 = 1.3 ft. To empty 10200 ft^3 over 40 hours = 144,000 seconds, the average release rate is 10200/144000 = 0.0708 cfs. Noting that the orifice area of a ½-in hole is 0.00136 ft^2, and k_o = 0.40 for ragged edge orifices, we can write Eq. (7.7) as

$$0.0708 = N(0.40)(0.00136)\sqrt{2(32.2)}\sqrt{1.3}$$

where N = number of orifice holes

Solving for N we obtain N = 14.22. Therefore, we use 14 holes evenly distributed.

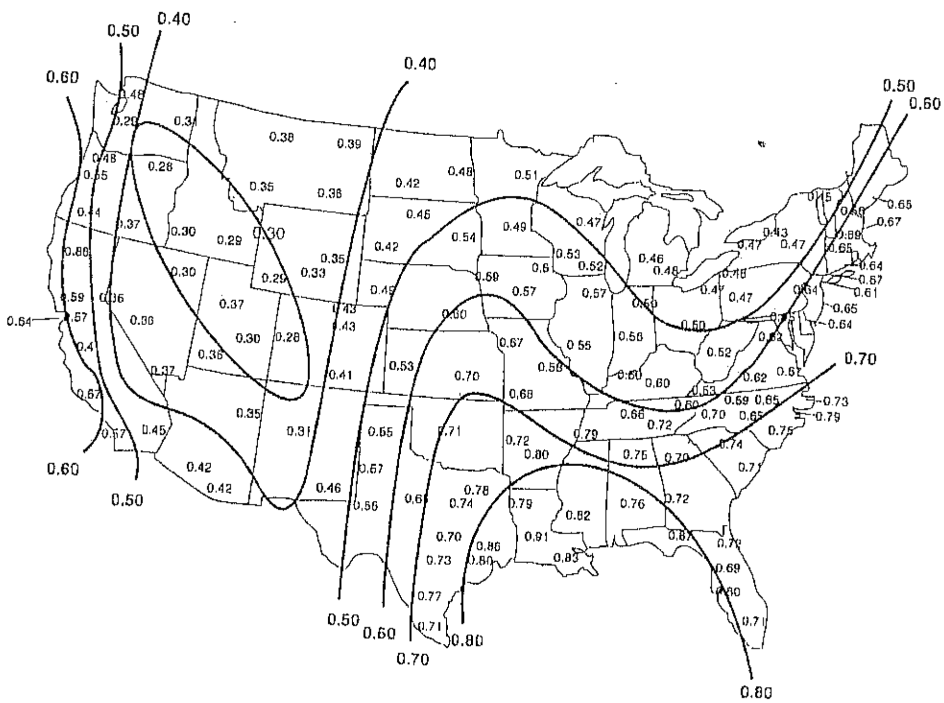

FIGURE 7.13 Mean-storm precipitation depth in inches. (After ASCE, 1998)

TABLE 7.5 Example for Modified Rational Method

(1) t_d (min)	(2) i (in/hr)	(3) t_d (sec)	(4) i (ft/sec)	(5) I_p (cfs)	(6) S_d (ft^3)
30	4.0	1,800	9.25×10^{-5}	56.41	65,538
40	3.5	2,400	8.10×10^{-5}	49.40	76,560
60	2.7	3,600	6.25×10^{-5}	38.11	83,196
90	2.0	5,400	4.62×10^{-5}	28.17	80,188

Example 7.6

An urban watershed has a drainage area of $A = 20$ acres $= 871{,}200$ ft^2, runoff coefficient of $C = 0.70$, and time of concentration of $T_c = 30$ minutes $= 1{,}800$ seconds. A detention basin is needed to reduce the peak discharge for a design return period of 10 years to $Q_p = 20$ cfs. The rainfall duration and intensity for this design return period is tabulated in columns 1 and 2 of Table 7.5. Determine the size of the detention basin needed at this site.

The calculations for this example are presented in Table 7.5. First, the given storm duration and rainfall intensities are converted to seconds and feet per hour and tabulated in columns 3 and 4 to allow the use of consistent units in the problem. Then the peak runoff rate is calculated using Eq. (7.18) for each storm duration and intensity pair included in the table and entered in column 5. For example for $t_d = 1{,}800$ sec and $i = 9.25 \times 10^{-5}$ ft/sec, $I_p = (0.7)(9.25 \times 10^{-5})(871200) = 56.41$ cfs. Finally, the detention volume is calculated using Eq. (7.19) and tabulated in column 6. For instance, for $t_d = 1{,}800$ sec and $i = 9.25 \times 10^{-5}$ ft/second

$$S_d = (56.41)(1800) - (20)\left(\frac{1800 + 1800}{2}\right) = 65{,}538 \text{ ft}^3$$

The maximum detention volume included in column 6 is 83,196 ft^3, and this value should be chosen as the size of the detention basin required. Also, it can be seen from the table that design storm for this detention basin has a duration of 60 minutes and an intensity of 2.7 in/hr.

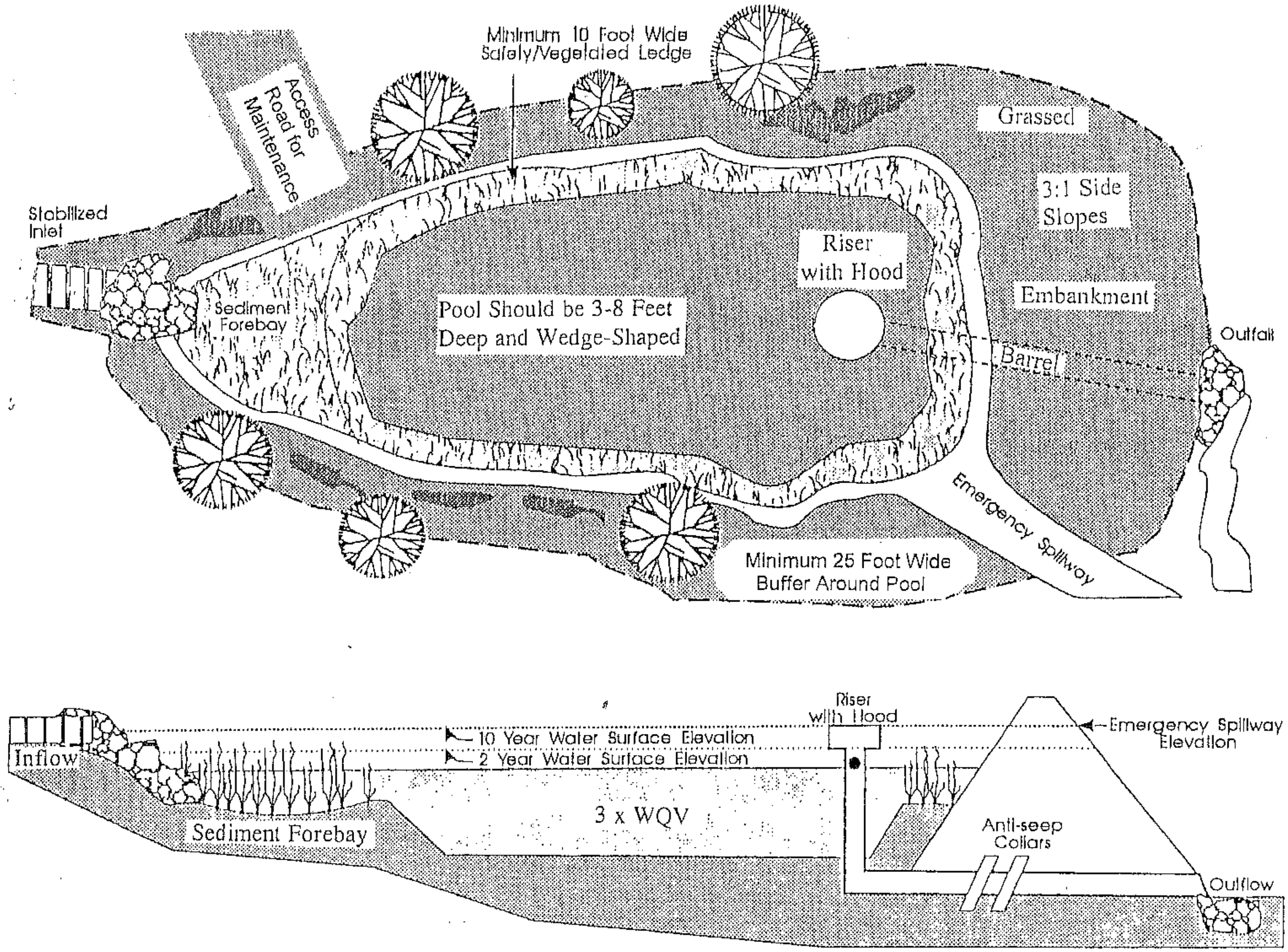

FIGURE 7.14 Schematic of a retention basin. (After Schueler, 1987)

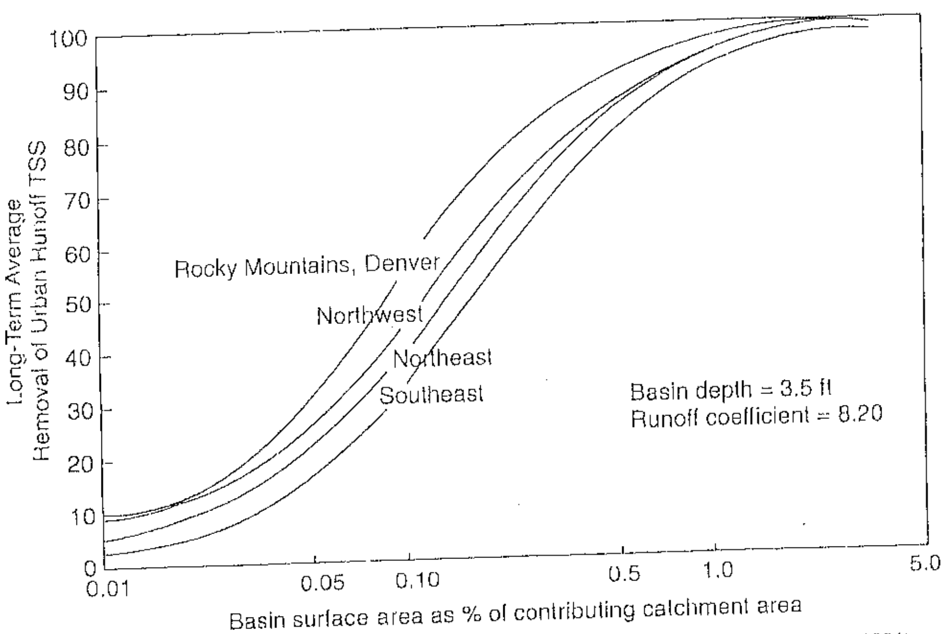

FIGURE 7.15 Retention basin solids settling for single family residential areas. (After USEPA, 1986)

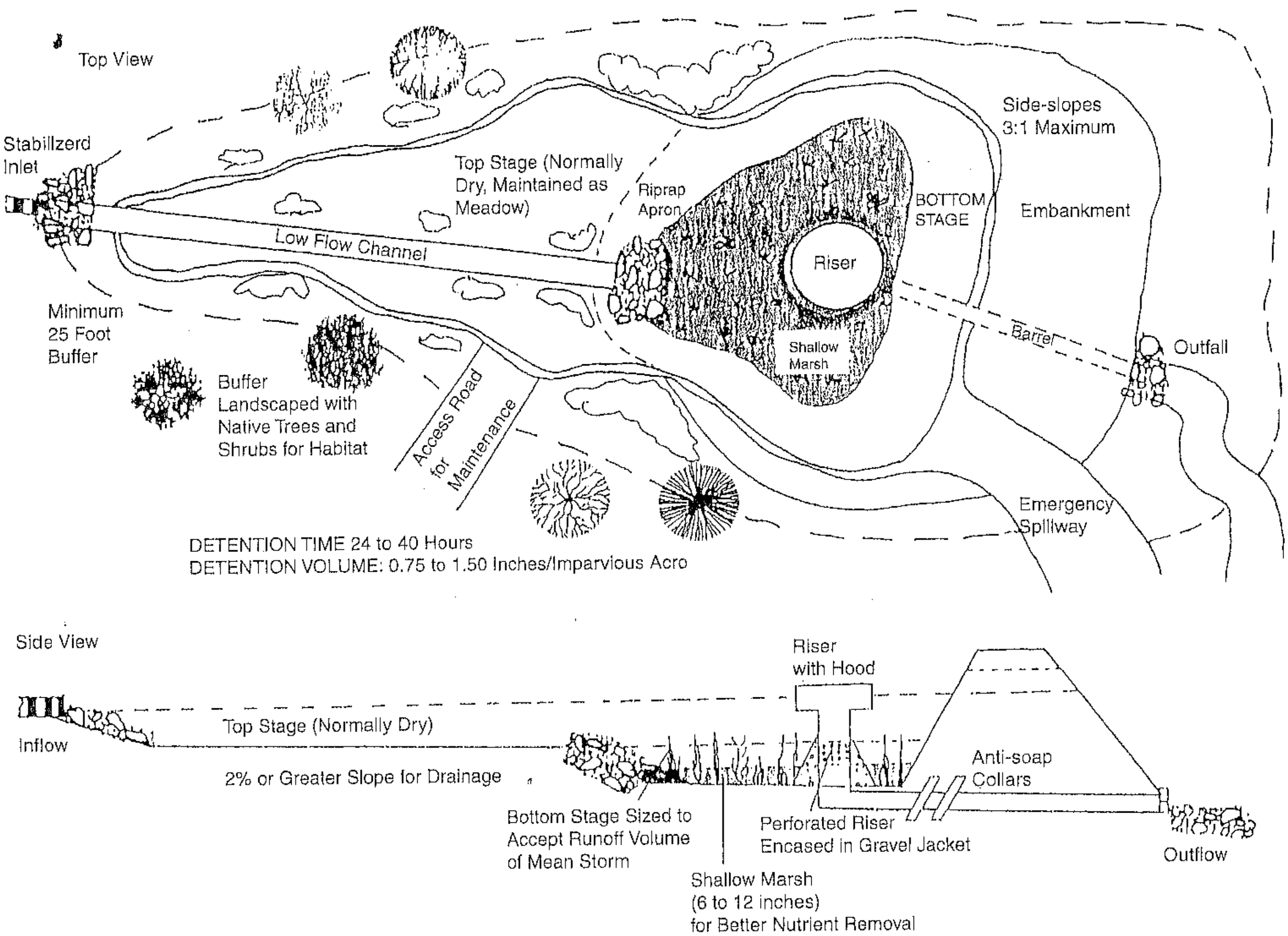

FIGURE 7.12 Schematic of an extended detention basin. (After Schueler, 1987)