A. Introduction

Water quality control designs are focused more on the annual runoff volume rather than peak storm events. Typically, smaller storm events account for the majority of annual rainfall and runoff volumes. Three methods, based on varying assumptions, and subsequently varying levels of complexity, are described in this section for determining the appropriate water quality volume, water quality capture volume, and optimization of capture volume for detention basin systems. The methods use storage volume as a surrogate for water quality, which is strictly a hydraulic issue and is the limitation of these methods.

B. Water quality volume and peak flow

1. **Water quality volume calculation.** The water quality volume (WQv) is the treatment volume required to remove a significant percentage of the stormwater pollution load, defined as an 80% removal of the average annual post- development total suspended solids (TSS) load. This is achieved by intercepting and treating a portion of the runoff from all storms and all the runoff from 90% of the storms that occur on average during the course of a year. The water quality treatment volume is calculated by multiplying the 90th percentile annual rainfall event by the volumetric runoff coefficient (Rv) and the site area. Rv is defined as:

   Equation C3-S6-1

   $$Rv = 0.05 + 0.009(I)$$

   Where:
   I = percent of impervious cover (%)

   For the state of Iowa, the average 90% cumulative frequency annual rainfall event is 1.25 inches. The procedure for determining the 90% cumulative frequency rainfall depth is described in Chapter 3 – Section 2 Rainfall and Runoff Analysis.

   Therefore, WQv is calculated using the following formula:

   Equation C3-S6-2

   $$WQv = Rv \times 1.25 inches \times \frac{A}{12}$$

   Where:
   WQv = water quality volume (ac-ft)
   Rv = volumetric runoff coefficient
   A = total drainage area (ac)

   WQv can be expressed in inches simply as Q_a =1.25Rv

   Where:
   Q_a = runoff volume (in) for the water quality design rainfall event.

2. **Water quality volume peak flow calculation.** The peak rate of discharge is needed for the sizing of off-line diversion structures and to design grass channels. Conventional NRCS methods underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff bypasses the filtering treatment practice due to an inadequately- sized diversion structure, or leads to the design of undersized grass channels.

   The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the small storm hydrology method (Pitt, 1994) and uses the NRCS TR-55 peak discharge method.

a. **Step 1.** Using the WQv, a corresponding curve number (CN) is computed from the following equation:

Equation C3-S6-3

$$CN = \frac{1000}{[10 + 5P + 10Q_a] - 10({Q_a}^2 + 1.25Q_aP)^{\frac{1}{2}}}$$

Where:
P = rainfall, in inches (use 1.25 inches for the water quality storm in Iowa)
Q_a = water quality runoff volume, in watershed inches (1.25Rv)

Note: The above equation is derived from the NRCS runoff curve number method described in detail in NEH-4, Hydrology (NRCS 1985) and NRCS TR-55 Chapter 2: Estimating Runoff. The CN can also be obtained graphically using NRCS Figure D.10.1.

b. **Step 2.** Once a CN is computed, the time of concentration (T_c) is computed (based on the methods in identified in TR-55 and Chapter 3 – Section 3 Time of Concentration).
c. **Step 3.** Using the computed CN, T_c and drainage area (A), in acres, the peak discharge Q_p is determined using the procedure in TR-55. Note that the CN is computed manually outside the program and then entered manually. The T_c computation menu in TR-55 can be used. The user will need to enter the water quality design storm depth (1.25 inches) as well.

   Note: When using TR-55 software for this procedure, the user is cautioned to re-enter the original CN for the site when going on to compute the peak rate, runoff volumes, and hydrographs for the larger 1-year through 100-year recurrence intervals.

3. **Example calculation of peak discharge for water quality storm.** A 5-acre small commercial site has 1.2 acres of flat roof, 3.4 acres of parking, and 0.4 acres of open space. The weighted volumetric runoff coefficient (Rv) is:

$$I = \frac{4.6}{5.0} = 0.92\ (92\%)$$

$$Rv = 0.05 + 0.009(I)$$

$$Rv = 0.05 + 0.009(92\%)$$

$$Rv = 0.878 = 0.88$$

For the water quality rainfall depth of 1.25 inches, the runoff volume for this site is:

$$Q_a = Rv \times P = 0.88 \times 1.25 inches = 1.1 watershed\ inches$$

WQv is:

$$WQv = \frac{(1.25 inches \times 0.88 \times 5 acres)}{12} \times \frac{43{,}560 ft^2}{acres} = 19{,}995 ft^3$$

Using Q_a = 1.1 watershed inches and P = 1.25 inches compute the CN for the WQ storm.

$$CN = \frac{1000}{[10 + (5)(1.25) + (10)(1.1 inches)] - 10[(1.1 inches)^2 + 1.25(1.1)(1.25 inches)]^{0.5}} = 98.6$$

(Use 98)

Using TR-55 software:

- Manually enter the T_c = 10 minutes (0.17 hr) in the data entry menu
- Enter the manually computed CN for the WQv rainfall (CN = 98)
- Enter the watershed name and configuration when prompted
- Enter the watershed area - 5 acres
- In the rainfall data entry menu - manually enter the WQ rainfall depth of 1.25 inches
- Run the computation

The computed value for the Qwq for this problem is 6.7 cfs. The computed runoff volume in the program output is 1.018 watershed inches, which is relatively close to the runoff volume determined from the Rv method. The unit peak discharge (q_u) would be 841.6 csm/in.

As noted above, when computing the runoff volume and peak rate for the larger storms (i.e., 1- year, 2-year, 10-year, and 100-year), use the published CN's provided in TR-55 or use the custom CN menu to determine a CN that fits the impervious area configuration for the project area.

C. Method for demonstrating compliance with the Channel Protection volume (CPv) release rate criteria

It should be noted that even though the CPv criteria is termed "Channel Protection volume", compliance is shown by demonstrating that the release rate is being held below an allowable level to provide the desired level of extended detention.

1. TR-55 is used for this procedure. Begin the analysis by entering the watershed land use and site configuration information into a TR-55 software program. Use the normal TR-55 procedures for determining the composite CN for the watershed and for determining the time of concentration, T_c.
2. The channel protection volume is determined for the 1-year, 24-hour duration storm. Enter the 1-year, 24-hour storm depth in TR-55 or use the 1-year rainfall depth from the default rainfall file for the Iowa county where the project is located.
3. Run the TR-55 analysis. The software program may provide the following information:
   a. The unit peak discharge, q_u, (cfs/mi^2/in).
   b. The total runoff volume for the CPv event, in watershed inches, Q_a.

   If the software program doesn't report out this information, these values can be calculated using the procedures described on page 9 of Section 9.02.

4. Using the unit peak discharge, q_u, find the ratio of outflow to inflow (q_o/q_i) for T = 24 hours from Figure C3-S6-1.
5. The allowable release rate (q_o) that is not to be exceeded to demonstrate compliance with the Channel Protection volume (CPv) criteria, can be computed from the equation: $q_o = q_o/q_i \times q_i$, where q_o/q_i = the ratio read from the graph in Figure C3-S6-1 (from Step 4), q_i = the peak inflow rate to the practice during the CPv event (in cfs).
6. With q_o/q_i, compute the ratio of storage to runoff volume (V_s/V_r).

Equation C3-S6-4

$$\frac{V_s}{V_r} = 0.683 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3$$

7. Compute the extended detention storage volume: $V_s = (V_s/V_r) \times V_r$
   - Note that V_r is equal to Q_a and was determined from the TR-55 runoff analysis, as noted above.
   - V_s may be converted from watershed-inches to acre-feet by $V_s/12 \times A$, where V_s is the depth in inches and A is in acres.
8. Compute an estimate of the required orifice area (A_o), for extended detention design:

Equation C3-S6-5

$$A_o = \frac{q_o}{C(2gh_o)^{0.5}} = \frac{q_o}{4.81(h_o)^{0.5}}$$

Where:
q_o = Allowable release rate for the CPv event (cfs)
h_o = estimated head above orifice at projected high-water elevation from the CPv event (feet)
A_o = estimated orifice size (square feet)
C = orifice coefficient (0.60)
g = gravitational constant (32.2 ft/s^2)

9. Determine the estimated required orifice diameter (d_o): $d_o = (4A_o/\pi)^{0.5}$ (d_o is in feet)

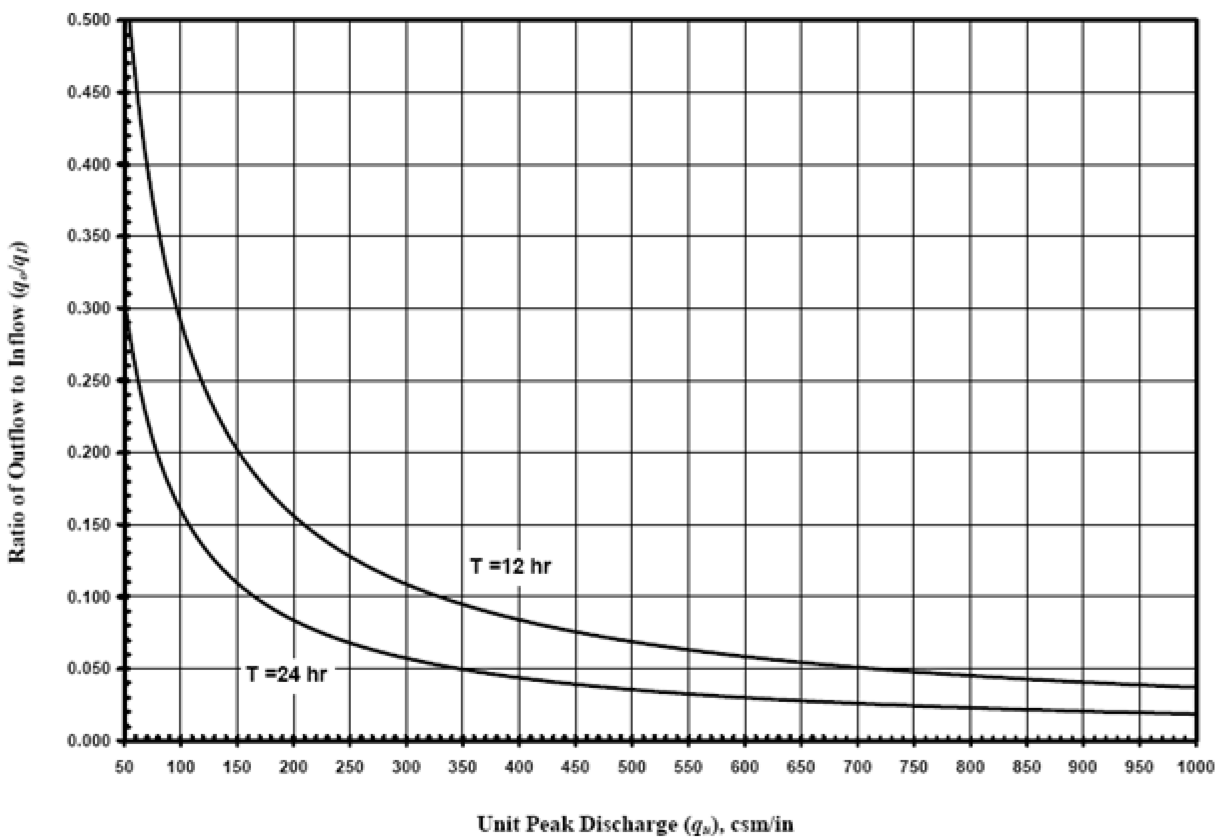

Figure C3-S6-1: Detention time vs. discharge ratios (qo/qi)

If the preceding process results in a computed orifice diameter of less than 3 inches, the final design may instead use a 3-inch orifice as the control for the CPv event. In this case, the final routing modeling of the practice may end up showing that the allowable CPv release rate is exceeded. This would most commonly occur in practices with smaller watersheds where the CPv discharge rate may end up exceeding the allowable value, typically by a fraction of a cubic foot per second (cfs).

However, the minimal increase in flow expected is acceptable when balanced against potential maintenance issues that may arise from using an orifice smaller than 3 inches in size. It should be noted that if runoff from multiple smaller sites can be treated by a best management practice that serves a larger area, the CPv requirements may be fully achieved while still using an outflow control above the minimum size of 3 inches.

If the CPv orifice size is 3 inches, exceeding the allowable CPv discharge is acceptable provided that the final design also does not exceed the recommended high-water depths for the CPv event, as described in the sections of ISWMM related to various practice types. Exceeding the recommended high-water depths would increase the head condition on the outlet, further increasing the outflow rate beyond the target maximum value.

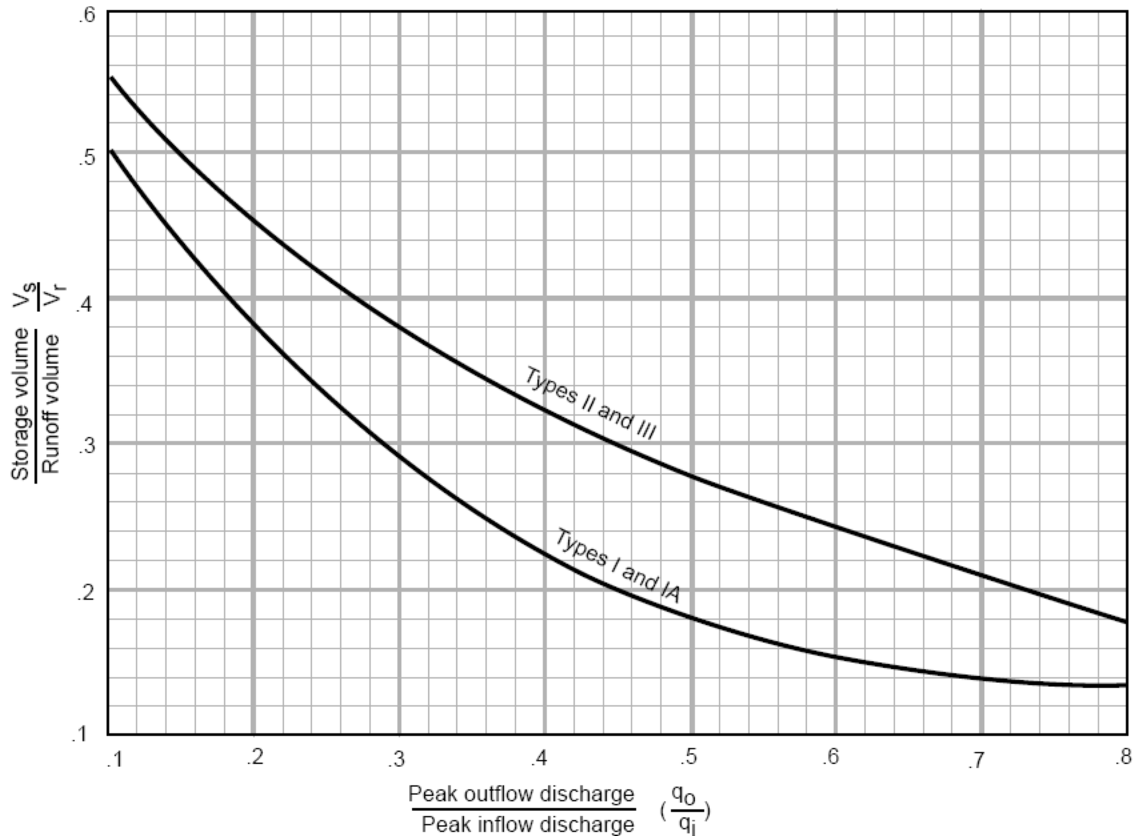

Figure C3-S6-2: Approximate detention basin routing
Source: NRCS TR-55, 1986

10. The designer should note that using the procedure above and Figure C3-S6-2 for determination of required storage volume is a first approximation to be used for initial sizing. During the final design, a full reservoir routing should be performed to confirm the performance of the storage basin outlet structure with respect to release rate and water surface elevation for the range of storm events under consideration. The final routing model should verify that the peak discharge rate expected from the final design is not expected to exceed the calculated value of the allowable release rate (q_o) (except as noted where a minimum orifice size is being employed as noted on the previous page).

D. Maximized water quality capture volume

The American Society of Civil Engineers (ASCE) and the Water and Environment Federation (WEF, 1998) have provided a regression equation to maximize the water capture volume that builds upon the earlier work by Urbonas et al. (1990). The procedure is summarized below:

1. **Long-term rainfall characteristics.** The cumulative probability distributions of daily precipitation data for 40 years in Orlando, FL and Cincinnati, OH are presented in Figure C3-S6-3. These data were screened to include only precipitation events 0.1 inch or greater in Cincinnati, and 0.06 inch or greater in Orlando. Cumulative occurrence probabilities were computed for values ranging from 0.1-2 in. The data described in Figure C3-S6-3 indicates most of the daily values to be less than 1 inch in total depth. In Cincinnati, which has 40 in/yr of precipitation, 90% of the events produce less than 0.8 in of rainfall. A similar analysis for rainfall at Ames, IA for the period of record 1960-2004 indicates a 90% cumulative rainfall depth of 1.25 inches. By contrast, the 2-year, 24-hour storm produces precipitation of 2.9 inches in Cincinnati while the 2-year, 24-hour duration storm in

central Iowa is 2.91 inches. This long-term pattern of rainfall suggests that capturing and treating runoff from smaller storms should capture a large percentage of the runoff events and runoff volume that occur in the urban landscape.

2. **Capture of stormwater runoff.** Long-term simulations of runoff were examined for six US cities by Roesner et al. (1991) using the storage, treatment, overflow, runoff model (STORM). The six cities were Butte, MT; Chattanooga, TE; Cincinnati, OH; Detroit, MI; San Francisco, CA; and Tucson, AZ. STORM is a simplified hydrologic model that translates a time series of hourly rainfall to runoff, then routes the runoff through detention storage. Hourly precipitation records of 40 to 60 years were processed by Roesner et al. (1991) for a variety of detention basin sizes for the six cities. These simulations were performed using the characteristics of the most typically-occurring urban- developments found in each city. Table C3-S6-1 lists the average annual rainfall and the area weighted runoff coefficient at each of the study watersheds. Runoff capture efficiencies of detention basins were tested using an outflow discharge rate that emptied or drained the design storage volume in 24 hours, based on field study findings by Grizzard et al. (1986). The findings by Roesner et al. (1991) are illustrated in Figure C3-S6-4.

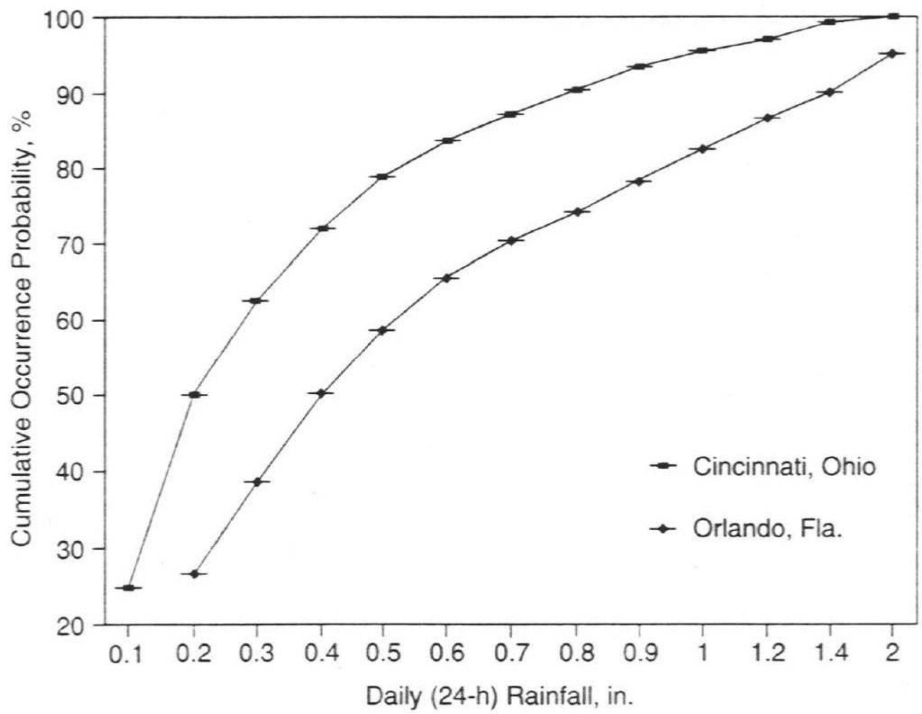

Figure C3-S6-3: Cumulative probability distribution of daily precipitation for two cities in the US
Source: Roesner et al., 1991

Table C3-S6-1: Hydrologic parameters used at six study watersheds

City	Average annual rainfall (inches)	Watershed runoff coefficient (C)
Butte, MT	14.6	0.44
Chattanooga, TN	29.5	0.63
Cincinnati, OH	39.9	0.50
Detroit, MI	35.0	0.47
San Francisco, CA	19.3	0.65
Tucson, AZ	11.6	0.50

Source: Roesner et. al, 1991

A cost-effective basin size can be represented as that which is located on the “knee of the curve” for capture efficiency. This “knee” is evident on the six curves in Figure C3-S6-4. Urbonas et al (1990) defined this “knee” as the optimized capture volume, and reported on a sensitivity study they performed relative to this volume for the

Denver, Colorado area. Later, Urbonas and Stahre (1993) redefined this “knee” as the maximized volume, because it is the point at which rapidly- diminishing returns in the number of runoff events captured begin to occur. For each of the six study watersheds previously described, the maximized storage volume values are listed in Table C3-S6-2. The sensitivity investigation by Urbonas et al. (1990) also estimated the average annual stormwater removal rates of total suspended sediments, using the maximized volume as the surcharge storage above a permanent pool of a retention pond. Estimates of total suspended sediment removals were performed using the procedure reported by Driscoll (1983). Similarly, the runoff capture and total suspended sediment removal efficiencies were estimated for capture volumes equal to 70% and 200% of the maximized volume. These findings are summarized in Table C3-S6-3.

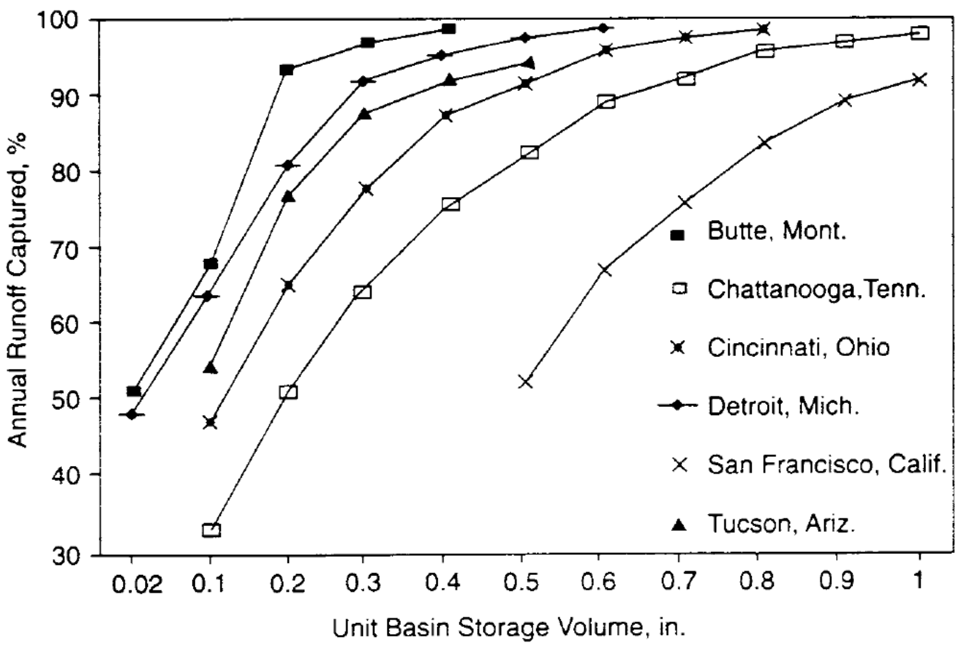

Figure C3-S6-4: Runoff capture rates vs. unit storage volume at six study sites

Source: Roesner et al., 1991

Table C3-S6-2: Maximized unit storage volume at six study watersheds

City	Maximized Storage Volume*	
	inches	acre-feet
Butte, MT	0.25	0.021
Chattanooga, TN	0.50	0.042
Cincinnati, OH	0.40	0.033
Detroit, MI	0.30	0.025
San Francisco, CA	0.30	0.025
Tucson, AZ	0.30	0.025

*Based on the ratio of runoff volume captured from all storms

Source: Roesner et al, 1991

Table C3-S6-3: Sensitivity of the BMP capture volume in Denver, CO

Capture volume to maximized volume ratio	Annual runoff volume captured (%)	No. storms completely captured	Average annual TSS removed (%)
0.7	75	27	86
0.7	85	30	88
2.0	94	33	90

Source: Urbonas et al, 1990

As can be seen from Figure C3-S6-4 and Table C3-S6-2 and Table C3-S6-3, most runoff-producing events occur as a result of the predominant population of smaller storms, namely, less than 0.5-1.25 inches of precipitation. To be effective, stormwater quality management should be designed based on these smaller events. As a result, detention facilities, wetland basins, infiltration facilities, media filters, grass swales, and other treatment BMPs should be sized to accommodate runoff volumes and flows from such storm events to maximize pollution control benefits in a cost-effective manner.

E. Estimating a maximized water quality capture volume

When local resources permit, the stormwater quality capture volume may best be found using continuous hydrologic simulation and local long-term hourly (or 15-minute time increment) precipitation records. However, it is possible to obtain a first-order estimate of the needed capture volume using simplified procedures that target the most typically-occurring population of runoff events. Figure C3-S6-5 contains a map of the contiguous 48 states of the US with the mean annual runoff-producing rainfall depths superimposed (Driscoll et al., 1989). The mean depths are based on a 6-hour inter-event time to define a new storm event, and a minimum depth of 0.10 inches of precipitation for a storm to produce incipient runoff. Guo and Urbonas (1995) found simple regression equations to relate the mean precipitation depths in Figure C3-S6-5 to maximized water quality runoff capture volumes (the "knee" of the cumulative probability curve). The analytical procedure was based on a simple transformation of each storm's volume of precipitation to a runoff volume using a runoff coefficient, C. A third-order regression equation, Equation C3-S6-1 (Urbonas et al., 1990) was derived using data from the 1983 NURP studies of more than 60 urban watersheds over a two- year period (EPA, 1983). Since the data was collected nationally over a two-year period, Equation C3-S6-1 will have broad applicability in the US, for smaller storm events.

Equation C3-S6-6

$$C = 0.858i^3 - 0.774i + 0.004$$

Where:
C = runoff coefficient
i = watershed imperviousness ratio (percent total imperviousness divided by 100)

Equation C3-S6-6 relates mean precipitation depth taken from Figure C3-S6-5 to the maximized detention volume. The coefficients listed in Table C3-S6-4 are based on an analysis of long-term data from seven precipitation- gauging sites located in different meteorological regions of the US. The correlation of determination coefficient, R_2, has a range of 0.80- 0.97.

Equation C3-S6-7

$$P_0 = (a)(C)(P_6)$$

Where:
P_0 = maximized detention volume in watershed-inches (determined using either the event or volume capture ratio "a" from Table C3-S6-4)
a = regression constant from least-squares analysis
C = watershed runoff coefficient
P_6 = mean storm precipitation volume (watershed inches) (Figure C3-S6-5 or local data)

The mean precipitation value, P_6, can be obtained from Figure C3-S6-5 or referenced from local rainfall data.

The maximized detention volume, P_0, can determined using either the event capture ratio or the volume capture ratio as its basis.

Table C3-S6-4 lists the maximized detention volume/mean precipitation ratios based on either the ratio of the total number of storm runoff events captured, or the fraction of the total stormwater runoff volume from a catchment. These can be used to estimate the annual average maximized detention volume at any given site. All that is needed is the watershed's "C" and its mean annual precipitation, P_6. The actual size of the runoff event to target for water quality enhancement should be based on the evaluation of local hydrology and water quality needs. However, examination of Table C3-S6-4 indicates that the use of larger detention volumes does not significantly improve the average annual removal of total suspended sediments or other settleable constituents. It is likely that an extended detention volume equal to a volume between the runoff from a mean precipitation event taken from Figure C3-S6-5 and the maximized event obtained using Equation C3-S6-7 will provide the optimally-sized and most cost-effective BMP facility. A BMP sized to capture such a volume will also capture the leading edge (first flush) of the runoff hydrograph resulting from larger storms. Runoff volumes that exceed the design detention volume either bypass the facility or receive less-efficient treatment than do the smaller volume storms, and have only a minimal net effect on the detention basin's performance. If, however, the design volume is larger and has an outlet to drain it in the same amount of time as the smaller basin, the smallest runoff events will be detained only for a brief interval by the larger outlet. Analysis of long-term precipitation records in the US shows that small events always seem to have the greatest preponderance. Therefore, over-sizing the detention can cause the most frequent runoff events to receive less treatment than provided by properly-designed small basins.

Table C3-S6-4: Values of coefficient "a" in Equation C3-S6-7 for finding the maximized detention storage volume*

		Drain time of capture volume		
		12-hr	**24-hr**	**48-hr**
Event capture ratio	a =	1.109	1.299	1.545
	r_2 =	0.97	0.91	0.85
Volume capture ratio	a =	1.312	1.582	1.963
	r_2 =	0.80	0.93	0.85

*Approximately 85^{th} percentile runoff event (range 82-88%)
Source: Guo and Urbonas, 1995

F. Design example

Water quality capture volume estimate

A project site in the western suburbs of Des Moines, IA has a 280-acre watershed. At full build-out, the project is expected to have about 36% of its area covered by impervious surface. A regional detention basin is one of the alternatives being considered for water quality enhancement. Determine the maximized storage volume. The detention basin needs to be sized and designed to drain its water quality capture volume in 24 hours.

1. Determine the C value for the project watershed, using Equation C3-S6-6 (i = 0.36):

$$C = 0.858i^3 - 0.78i^2 - 0.774i + 0.04$$
$$0.858(0.36)^3 - 0.78(0.36)^2 - 0.774(0.36) + 0.04 = 0.26$$

2. Determine the mean precipitation depth, P_6, for the Des Moines area from Figure C3-S6-5: P_6 = 0.57 inches
3. From Table C3-S6-4, find the coefficient a = 1.299 for the 24-hour drain time. The maximized detention volume is calculated as follows from Equation C3-S6-7:

$$P_0 = (1.299)(C)(P_6) = (1.299)(0.26)(0.57 inches) = 0.192\ watershed\ inches$$

This is equivalent to 0.016 ac-ft/ac. The volume of an extended detention basin for this 280-acre watershed needs to be 4.48 ac-ft. Recommend this volume be increased by at least 20% to account for the loss in volume from sediment accumulation. The final design then can show a total volume for the basin of 5.38 ac-ft with an outlet designed to empty out the bottom 4.5 ac-ft of this volume in approximately 24 hours.

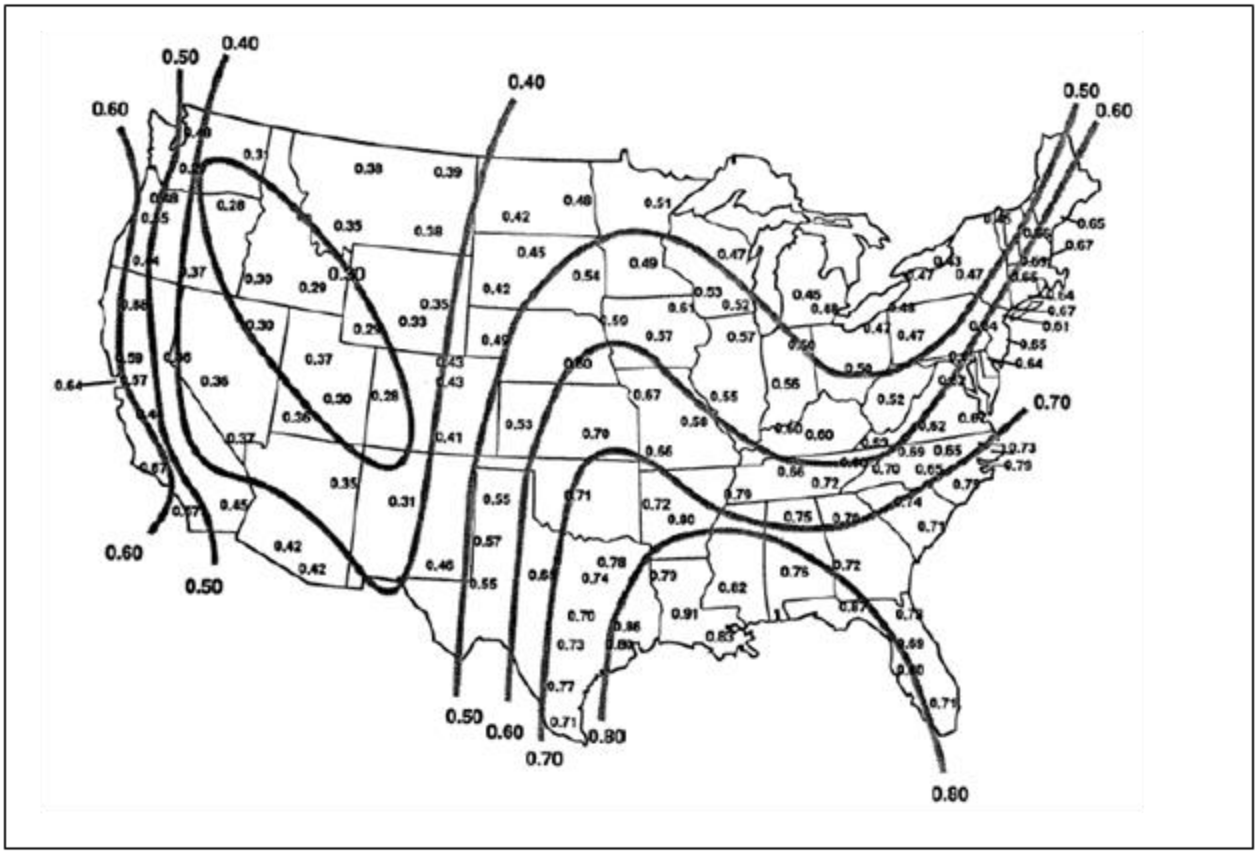

Figure C3-S6-5: Mean storm precipitation depth in the US (inches)
Source: Driscoll et al., 1989