

# HYDROLOGIC ANALYSIS

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## 4.1 Introduction to Hydrologic Analysis

This Section addresses surface water hydrology and % TSS Removal Calculation only; it does not address groundwater hydrology. Therefore all references in this Manual to "hydrology" means surface water hydrology. Also, in this chapter the terms "subbasin" and "catchment" (used interchangeably) refer to a single drainage area being analyzed by the various techniques, independently of any other subbasins or catchments that make up the basin or watershed being analyzed. For example, the peak flow computed by one of the peak flow only methods (Regression or Rational), or a unit hydrograph developed for a specific drainage area (e.g., NRCS), are applied to subbasins or catchments. Subbasins or catchments make up basins or watersheds, along with channels, reservoirs, and other hydrologic features as applicable.

Hydrology deals with estimating peak flows, runoff volumes, and time distributions of runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater controls. In the hydrologic analysis of a development/redevelopment site, there are a number of factors that affect the nature of stormwater runoff from the site. Some of the factors to be considered are:

- Rainfall amount and distribution;
- Drainage area size and shape;
- Ground cover and soil type;
- Slopes of terrain, streams and channels;
- Antecedent moisture (or runoff) conditions;
- Rainfall abstraction or loss rates (initial and continued);
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.);
- Watershed development; and,
- Characteristics of the local drainage system.

There are a number of hydrologic methods available to estimate runoff characteristics for a site or drainage subbasin. Some of those methods provide estimates of peak discharge only. Some provide estimates of runoff volume only. Special methods estimate the runoff volume for small storms that are not readily analyzed using more conventional methods. Others provide estimates of the full hydrograph (discharge versus time, and thus the distribution of runoff volume). In addition, there are methods available for estimating long-term runoff for use in water balance applications.

Rainfall, of course, is the primary driver for all of the methods mentioned above. The depth and distribution of rainfall during storm events is critical to estimating peak flows as well as the distribution of discharge over time.

For some applications, it is necessary to “route” flow through a control structure such as a stormwater pond and outlet. The temporary storage of inflowing water (detention) tends to attenuate the inflow hydrograph, resulting in outflow with a lower peak but increased duration. Thus, routing is fundamental to the design of stormwater storage facilities.

This chapter provides descriptions of the hydrologic methods to be used to implement the requirements of this Manual.

## 4.2 Rainfall

A rainfall event-based hydrologic analysis requires an estimate of the amount (depth or intensity) of rainfall that will occur on the site during a specified duration for a specified average return period. The average return period (sometimes referred to as average return interval or frequency) is expressed in years. The average annual exceedance probability of the event is the reciprocal of the average return period, and is expressed as the probability that the event will be equalled or exceeded in any given year. (The probability lies between 0 and 1.) For example, an event with an average return interval of 100 years has an average annual probability of being equalled or exceeded of  $1/100 = 0.01$ , or 1%.

For methods discussed in this Manual requiring a distributed rainfall event, the NRCS 24-hour Type 2 rainfall distribution (Figure 4-1) will be used. Table 4-1 lists the 24-hour point rainfall depths for various frequency storm events. These values were derived from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 8, Version 2.0 and the Precipitation Frequency Data Server (PFDS) from the Hydrometeorological Design Studies Center. (Note that the -year through 10-year values are partial durations series depths and the 25-year through 1000-year values are annual maximum series depths. For the larger storms, there is no significant difference between the partial duration and annual series.)

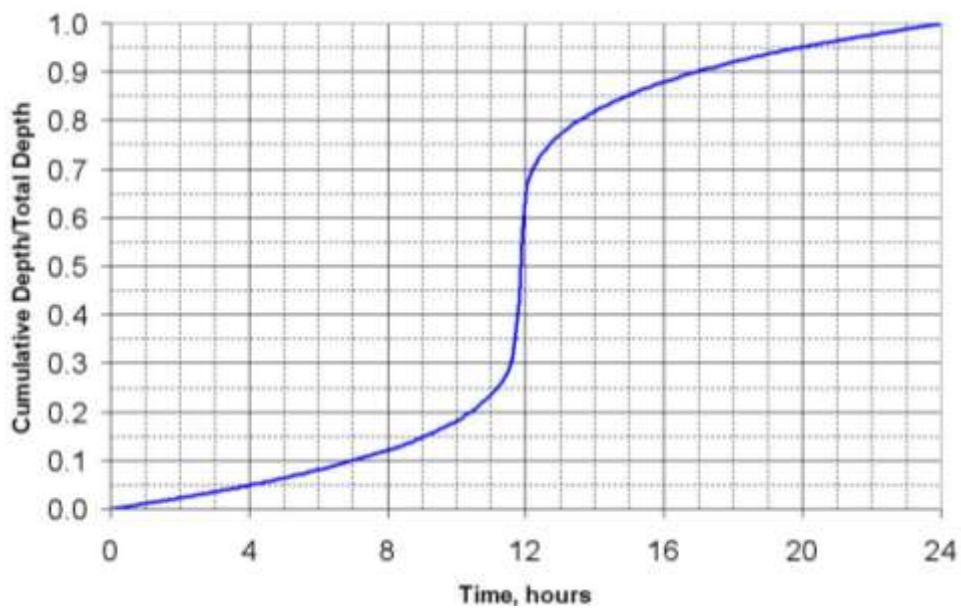


Figure 4-1 NRCS 24-Hour, Type 2 Rainfall Distribution

**Table 4-1 Point Rainfall Depths for 24-Hour Design Storms**

Return Period	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr	1000-Yr
Depth (Inches)	2.90	3.39	4.24	4.98	6.03	6.91	7.83	8.78	10.10	11.10

For methods requiring the rainfall intensity of specific durations, a table of point rainfall intensities for Wichita and Sedgwick County has been derived from a regression analysis of the data obtained from NOAA Atlas 14, Volume 8, Version 2.0 and the PFDS, presented in Appendix B. The table is for durations of 15 to 120 minutes, and average return periods of 1 to 1000 years.

The depths and intensities provided herein are point values. Theoretically, these values may be adjusted for actual basin area. Figure 15 of the National Weather Service TP-40 rainfall atlas may be used for that purpose. However, it is general practice to use point values for drainage areas of 10 square miles or less. The reduction is minor (approximately 3% or less) for basins up to 20 square miles. Practically, for the majority of studies performed under the provisions of this Manual, aerial reduction will not be required.

## 4.3 Rainfall Losses

### 4.3.1 Curve Number Method

Except for unusual conditions, not all of the rainfall that falls on a subbasin is discharged as direct runoff. A portion of the rainfall may be retained in depressions, while some may infiltrate into the soil. Other portions of the rainfall may be returned to the atmosphere by vegetative interception and ultimately by transpiration and/or evaporation. The portions of rainfall that do not become direct runoff are referred to collectively as "rainfall losses" or "abstractions" while the rainfall that does become direct runoff is referred to as "excess rainfall" or "direct runoff."

Rainfall loss is a function of soil characteristics, land-use, antecedent moisture (or runoff) conditions, and other factors. There are many methods used for estimating rainfall losses, with varying degrees of complexity from the standpoint of ease of use as well as data requirements.

For the applications of the hydrology methods presented in this Manual, the NRCS (formerly SCS) "Curve Number" (CN) method will be used. The CN indicates the runoff potential. The greater the CN value, the higher the runoff potential.

An approximate relationship between cumulative rainfall and cumulative runoff was derived by NRCS from experimental plots for numerous soils and cover conditions. The following NRCS runoff equation (SCS, 1986) is used to estimate direct runoff from storm rainfall:

$$\text{Equation 4-1} \quad Q = \frac{(P - I_a)^2}{[(P - I_a) + S]}$$

where:

Q	=	cumulative direct runoff (in)
P	=	cumulative rainfall (in)
I <sub>a</sub>	=	initial abstraction including surface storage, interception, evaporation, and infiltration prior to any runoff occurring (in)
S	=	(1000/CN) - 10 where CN = NRCS Curve Number

An empirical relationship used in the NRCS method (SCS, 1986) for estimating I<sub>a</sub> is:

$$I_a = 0.2S \quad (\text{Note that for } P \leq I_a, Q = 0)$$

This is an average value that could be adjusted if there are calibration data to substantiate the adjustment. Table 4-9 provides values of I<sub>a</sub> for a range of CNs.

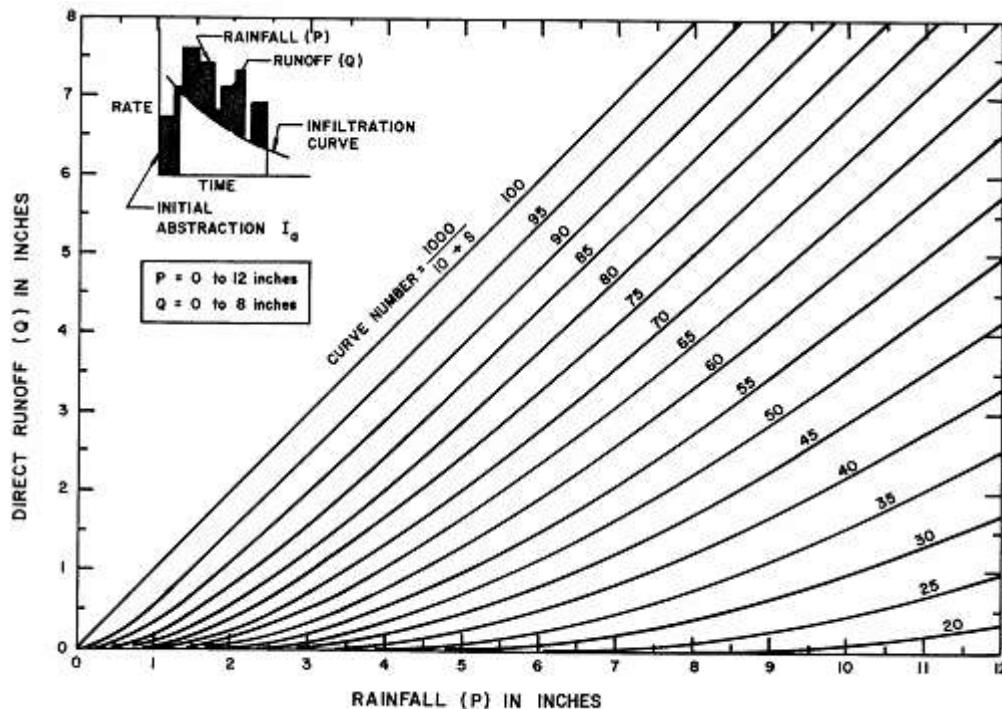
Substituting 0.2S for I<sub>a</sub> in Equation 4-1, the equation becomes:

$$\text{Equation 4-2} \quad Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Figure 4-2 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a curve number of 85.

Equation 4-2 can be rearranged so the CN can be estimated if rainfall and runoff volume are known (Pitt, 1994). The equation then becomes:

$$\text{Equation 4-3} \quad CN = \frac{1000}{\left[ 10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2} \right]}$$



**Figure 4-2 SCS Solution of the Runoff Equation**

(Source: SCS, TR-55, Second Edition, June 1986)

### 4.3.2 Hydrologic Soil Groups

The Curve Number method uses the combination of soil conditions and land uses (ground cover and treatment) to assign a CN to an area.

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the NRCS has divided soils into four hydrologic soil groups (HSGs).

- HSG A: Soils having a low runoff potential due to high infiltration rates: These soils consist primarily of deep, well-drained sands and gravels.
- HSG B: Soils having a moderately low runoff potential due to moderate infiltration rates: These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.
- HSG C: Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.
- HSG D: Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

## Section 4.3 - Time of Concentration

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Site plans with subbasin boundaries can be overlain on the hydrologic soil group map to aid in the determination of Curve Numbers. Please note that the map includes a soil group identified as "Urban." These areas should be considered as HSG D soils unless the reviewing authority approves an alternate group classification based on a review of field tests provided by the design engineer.

### 4.3.3 Required Curve Numbers

The CN values in Table 4-2 shall be used for all pre- and post-development hydrologic calculations. The pre-developed CN values for pervious areas are an equal blending of pasture in fair condition and cultivated small grain in good condition. The City of Valley Center and Sedgwick County elect to use this land use as a realistic basis for the condition of local watersheds prior to development. The detrimental effect of grading on infiltration rates is acknowledged in the disturbed pervious land use values. This effect can be reduced through preferred site design practices that minimize the grading footprint. The impervious CN of 98 accounts for the near total runoff of rainfall from impervious surfaces. A CN of 100 is to be used for permanent water surfaces such as lakes and ponds.

**Table 4-2 Pre- and Post-Development Curve Numbers**

Land Use	Hydrologic Soil Group			
	A	B	C	D
Pre-Developed or Undisturbed Pervious	55	71	80	84
Developed or Disturbed Pervious	71	80	84	88
Impervious	98	98	98	98

### 4.3.4 Composite Curve Numbers

For a subbasin containing subareas of varying CNs, a composite CN is computed. It should be noted that when composite CNs are used, the analysis does not take into account the location of the specific land uses within the subbasin, but simplifies the drainage area conceptually as a uniform land use represented by the composite CN. The composite CN for a subbasin shall be calculated by using an area-weighted averaging procedure as illustrated in the following example:

**Table 4-3 Calculation of a Composite Curve Number**

Land Use	Fraction of Total Land Area (1)	CN (2)	Weighted CN (1 x 2)
Disturbed Pervious, "B" Soil	0.30	80	24.0
Undisturbed Pervious, "B" Soil	0.70	71	49.7
Total Weighted Composite Curve Number = 24.0 + 49.7 = 73.7			

Except when the simplified method presented in Section 4.8.4 is used, runoff analysis using hydrograph methods shall be performed on pervious and impervious areas separately, rather than using a composite CN that includes the impervious areas for a given subbasin. The separation of pervious and impervious areas ensures the most accurate estimate of runoff peaks and volumes. Free modern software packages like HEC-1, HEC-HMS and EPA-SWMM allow the entry of a pervious CN and % imperviousness for a subbasin.

For large developed watersheds Table 4-4, may be used to estimate the percentage of subbasin area that is impervious.

**Table 4-4 Average Imperviousness per Land Use (Source NRCS, TR-55)**

Land Use	Average % Impervious
<b>Urban Districts:</b>	
Commercial and business	85%
Industrial	72%
<b>Residential districts by average lot size:</b>	
1/8 acre or less (town house)	65%
1/4 acre	38%
1/3 acre	30%
1/2 acre	25%
1 acre	20%
2 acres	12%

#### 4.4 Time of Concentration

The “time of concentration” is equal to the cumulative time for runoff to travel from the hydraulically most remote point of the subbasin being analyzed, to the design point (outlet of the subbasin). In this Manual, the time of concentration is assumed to consist of three types of flow:

- Sheet flow;
- Shallow concentrated flow;
- Channel flow.

Time of concentration ( $T_c$ ) is computed by summing the travel times for these consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the outlet of the subbasin. The following is a discussion of the required procedures and equations (USDA/SCS, 1986 and FHWA, 2001).

##### ***Travel Time***

Travel time is the ratio of flow length to flow velocity, as shown by Equation 4-4 (HEC-22, 2001):

$$\text{Equation 4-4} \quad T_T = \frac{L}{60V}$$

where:

$T_T$  = travel time (min)  
 $L$  = flow length (ft)  
 $V$  = average velocity (ft/s)  
 60 = conversion factor from seconds to minutes

#### **Sheet Flow**

Sheet flow shall be calculated using the Equation 4-5, which uses sheet flow roughness coefficients found in Appendix A: (SCS, 1986)

$$\text{Equation 4-5} \quad T_{T(sheet)} = \frac{0.42(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}}$$

where:

$T_{T(sheet)}$  = travel time (min)  
 $n$  = Manning sheet flow roughness coefficient (see Appendix Table A-1)  
 $L$  = flow length (ft): 100' maximum  
 $P_2$  = 2-year, 24-hour rainfall (3.4 inches from Table 4-1)  
 $S$  = land slope (ft/ft)

#### **Shallow Concentrated Flow**

Overland sheetflow often becomes shallow concentrated flow as it progress down the drainage area. By definition, sheet flow occurs only over plane surfaces at the head of the drainage area. Due to surface irregularities, sheet flow will eventually transition to shallow concentrated flow. The NRCS has determined that sheet flow will never occur for more than 300 feet, regardless of the evenness of the surface. When sheet flow conditions exist (or when the design exceeds the permitted length) the designer will generally use additional segments of shallow flow or channel flow to handle the remainder of the flow path. The average velocity for shallow concentrated flow shall be computed using Equation 4-6 (HEC 22, 2001). The equation uses Table 4-5 to determine K-values.

$$\text{Equation 4-6} \quad V = KS^{0.5}$$

where:

$V$  = average velocity (ft/s)  
 $K$  = shallow concentrated flow velocity coefficient (see Table 4-5).  
 $S$  = slope of hydraulic grade line approximated by the watercourse slope (ft/ft)

**Table 4-5 Velocity Coefficient (K value) for Shallow Concentrated Flow**

Shallow Concentrated Flow Cover	K value
Wooded w/heavy litter	2.5
Fallow or no-till cultivation	4.7
Wooded w/light litter	5.0

Shallow Concentrated Flow Cover	K value
Short grass pasture	7
Cultivated straight row crop	9
Nearly bare and untilled	10
Grassed waterway	15
Unpaved (as defined by TR-55)	16.3
Paved	20.3

After determining the average velocity using, use Equation 4-4 to calculate travel time for the shallow concentrated flow segments ( $T_{T(\text{shallow})}$ ).

### ***Open Channels***

Velocity in channels should be calculated using the Manning equation, Equation 4-7. It is recommended that the upslope beginning of channels be determined in the field. Otherwise, open channels may be assumed to begin where channels are visible on aerial photographs, where channel characteristics are indicated on contour maps, where channels have been identified by the local municipality, or where stream designations appear on United States Geological Survey (USGS) quadrangle sheets. Average flow velocity for travel time calculations shall be determined for bankfull conditions (with the exception discussed below) using Equation 4-7 (HEC-22, 2001):

$$\text{Equation 4-7} \quad V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

where:

V	=	average velocity (ft/s)
R	=	hydraulic radius (ft) and is equal to $A/P_w$
A	=	cross sectional flow area ( $\text{ft}^2$ )
$P_w$	=	wetted perimeter (ft)
S	=	slope of the hydraulic grade line approximated by the channel slope (ft/ft)
n	=	Manning's roughness coefficient for open channel flow

Bankfull flow conditions typically represent flow for the 1 to 2-year flood event which is considered to be a reasonable flow regime for estimating  $T_c$ . However, for streams that are significantly enlarged or incised due to long-term erosion, or for a constructed channel with bankfull flood capacity greater than the 1 to 2-year flow, bankfull conditions may overestimate the hydraulic radius for 1 to 2-year event capacity. Judgement should be exercised in selecting a representative value. In addition, open channel flow may pass through a closed conduit. The conduit should be treated the same as any other channel provided the conduit does not pressurize. If the conduit pressurizes, an estimate of flow velocity based on hydraulic pressure flow principles is appropriate (see Volume 2, Chapter 5).

After average velocity is computed using Equation 4-7,  $T_T$  for the channel segment ( $T_{T(\text{open})}$ ) can be calculated using Equation 4-5.

#### **Total Time of Concentration for Subbasin or Catchment**

The time of concentration for a subbasin or catchment is the sum of all sheet flow, shallow concentrated flow, and open channel flow travel times, as shown in Equation 4-8: (SCS, 1986)

$$\text{Equation 4-8} \quad T_C = \sum T_{T(\text{sheet})} + \sum T_{T(\text{shallow})} + \sum T_{T(\text{open})}$$

#### **Limitations**

- Post-development overland flow paths and conveyances will probably not be the same as before development. Time of concentration calculations shall consider the effects of development on flow paths and conveyances.
- Sheet flow length shall be limited to 100 feet.
- The minimum total time of concentration value to be used with the Rational Method (see Section 4.6) for the design of on-site conveyances (ditches, gutters, inlets, pipes, etc.) is 15 minutes.

## **4.5 USGS Regression Methods**

### **4.5.1 Introduction**

Regional regression equations are commonly used to estimate peak flows for ungaged sites (or for sites with insufficient data for a statistical derivation of the peak discharge versus flood frequency relationship). Regression analyses use stream gage data to define hydrologic regions having similar flood frequency relationships, and are typically based on watershed, channel, and meteorological characteristics. In Wichita and Sedgwick County, the USGS regression equations provided below may be used to estimate peak flows for unregulated rural drainage areas between 0.17 mi<sup>2</sup> and 30 mi<sup>2</sup> in size. It should be noted that these equations have relatively large standard errors and should be used accordingly. Unless approved by the appropriate review authority, regression estimates of peak flow shall be limited to preliminary drainage designs and as a “reasonableness” check of the results of more rigorous analyses. Equations 4-9 through 4-14 provide the regional regression equations for the Sedgwick County area (USGS, 2004). Figure 4-3 can be used to determine the mean annual precipitation for use in Equations 4-9 through 4-14.

$$\text{Equation 4-9} \quad Q_2 = 0.0126(CDA)^{0.579}(P)^{2.824}$$

$$\text{Equation 4-10} \quad Q_5 = 0.300(CDA)^{0.600}(P)^{2.138}$$

$$\text{Equation 4-11} \quad Q_{10} = 1.224(CDA)^{0.611}(P)^{1.844}$$

$$\text{Equation 4-12} \quad Q_{25} = 4.673(CDA)^{0.622}(P)^{1.572}$$

$$\text{Equation 4-13} \quad Q_{50} = 10.26(CDA)^{0.628}(P)^{1.415}$$

$$\text{Equation 4-14} \quad Q_{100} = 19.80(CDA)^{0.634}(P)^{1.288}$$

where:

$Q_x$  = Peak flow for the  $x$  year storm (cfs)  
 CDA = Contributing drainage area (square miles)  
 $P$  = Mean annual precipitation (inches); see Figure 4-3

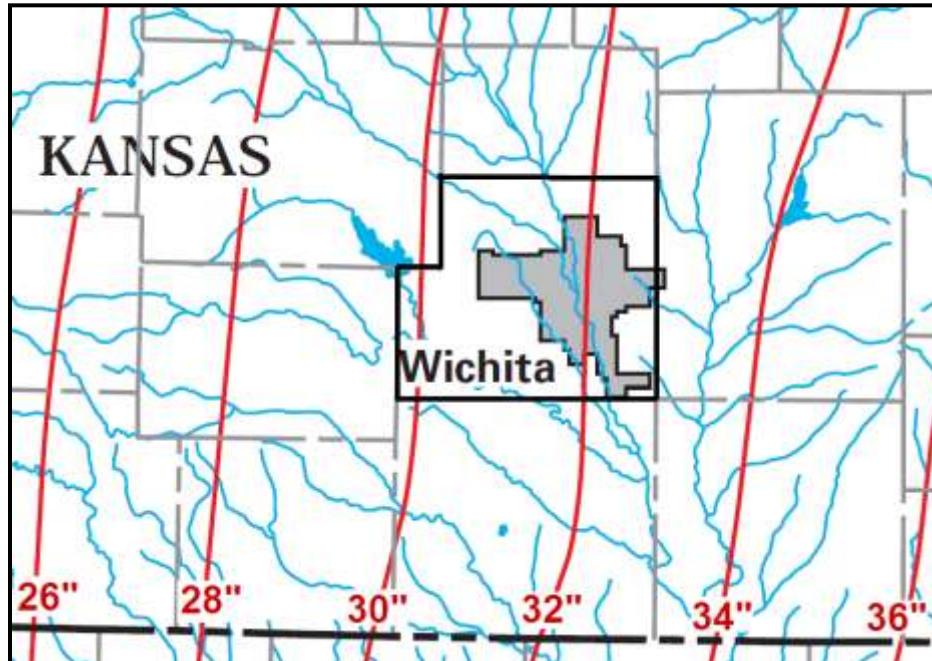


Figure 4-3 Mean Annual Precipitation

### Example Problem

Estimate the peak 100-year discharge for a 15.1 square mile rural drainage area located in western Sedgwick County.

Mean Annual Precipitation = 30.0"

#### Peak Discharge Calculations

Using Equation 4-14:

$$\begin{aligned}
 Q_{100} &= 19.80(CDA)^{0.634}(P)^{1.288} \\
 Q_{100} &= 19.80(15.1)^{0.634}(30)^{1.288} \\
 Q_{100} &= 19.80(5.59)(79.90) \\
 Q_{100} &= 8843 \text{ cfs}
 \end{aligned}$$

## 4.6 Rational Formula Method

### 4.6.1 Introduction

The Rational Formula is applicable to relatively small areas. The maximum drainage area that may be analyzed with the Rational Formula is 40 acres when used for the applications in this Manual.

The Rational Formula cannot be used for storage design or any other application where a runoff volume or routing is required (see Section 4.8). The Rational Formula should not be used for calculating peak flows downstream of bridges, culverts, or storm sewers if they act as flow restrictions that cause significant backwater storage, thus attenuating the peak rate of discharge, unless such storage effects are being purposely neglected. In general, the method may be used for computing design flows for gutters, inlets, storm sewers, and ditches for small drainage areas.

### 4.6.2 Equations

The Rational Formula method estimates the peak rate of runoff at a location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity for a rainfall duration equal to the time of concentration (i.e., the time required for water to flow from the hydraulically most remote point of the catchment to the location being analyzed).

The Rational Formula is expressed as shown in Equation 4-15 follows (HEC-22, 2001).

$$\text{Equation 4-15 } Q = CIA$$

where:

Q	=	maximum rate of runoff (cfs)
C	=	coefficient representing ratio of peak runoff to rainfall intensity (see Appendix C)
I	=	rainfall intensity for a duration equal to the $T_c$ (in/hr)
A	=	drainage area contributing to the design location (acres)

### 4.6.3 Runoff Coefficient (C)

The runoff coefficient (C) is a variable of the Rational Formula method which requires judgment and experience understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Appendix C gives the recommended runoff coefficients for the Rational Formula.

It is often necessary to develop a composite runoff coefficient based on the percentage of different types of surfaces in the contributing drainage area. Composite C values may be computed using values from Appendix C by using percentages of different land uses. This results in an area-weighted composite C value. The compositing procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide in the selection of reasonable coefficient values for an entire area.

It should be noted that the Rational Formula method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to consider the location of a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated for separate catchments, and routed through the drainage system. Also, it is possible that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site in the calculation wherein peak flows may be underestimated. This should be checked, particularly in areas with relatively large sheet flow and shallow concentrated flow portions (e.g., grassy areas) which typically yield a long  $T_c$ .

#### 4.6.4 Rainfall Intensity (I)

The rainfall intensity (I) is the rate of rainfall expressed in inches/hour for the duration equal to the time of concentration for a selected average return period. Once a particular return period has been selected for design and a time of concentration has been calculated for the drainage area, the rainfall intensity shall be selected from the rainfall intensity-duration-frequency data given in Appendix B.

---

#### Example Problem – Weighted Runoff Coefficient

Estimate the 25-year peak runoff rate at the inlet of a proposed culvert draining 20 acres. The land use for the drainage basin was estimated to be:

Residential (single family -  $\frac{1}{4}$  acre lots) - 80%  
Agricultural - 20%

To calculate the total weighted runoff coefficient (C), start with the values from Appendix C. Multiply the fraction of total land area by the runoff coefficient for the respective land uses to obtain the individual weighted runoff coefficients. Sum all of the individual weighted runoff coefficients to obtain the total weighted runoff coefficient for the drainage basin. This method is demonstrated in Table 4-6.

**Table 4-6 Calculate Weighted Runoff Coefficients (C)**

Land Use	Fraction of Total Land Area (1)	Runoff Coefficient (2)	Weighted Runoff Coefficient (1 x 2)
Residential (Single Family – $\frac{1}{4}$ acre lots)	0.80	0.60	0.48
Agricultural	0.20	0.30	0.06
<b>Total Weighted Runoff Coefficient = <math>0.48 + 0.06 = 0.54</math></b>			

Based on the project map and a site inspection, the hydraulically longest flow path was then identified and the required hydraulic data were collected. In addition, the 2-year, 24-hour rainfall depth (3.5 inches) was determined from Table 4-1. The travel times for the sheet flow, shallow concentrated flow and channel flow were then computed using the methods described in Section 4.4.

- Sheet flow = 4 minutes (based on travel length, slope, 2-year 24-hour rainfall, and surface roughness);
- Shallow Concentrated flow time = 6 minutes (based on travel length, slope and surface roughness);
- Channel flow time = 12 minutes (based on travel length, slope, channel geometry and surface roughness);
- Therefore, time of concentration =  $T_c = 22$  minutes.

Next, the rainfall intensity was found in the intensity-duration-frequency table of Appendix B, using a return interval of 25 years and a  $T_c$  of 22 minutes.

$$I_{25} = (25\text{-yr return period}) = 4.61 \text{ in/hr}$$

Finally, the estimate of peak runoff for the 25-yr storm for the given basin was calculated using the Rational Method:

$$Q_{25} = CIA = (0.54)(4.61)(20) = 49.8 \text{ cfs}$$

---

## **4.7 Hydrograph Methods**

### **4.7.1 General**

Hydrograph methods are procedures used to estimate the hydrograph of runoff (flow versus time) for a subbasin based on the rainfall and physical characteristics of the subbasin. Hydrograph methods require three elements:

- A time distribution for the design rainfall event: For the methods presented in this Manual, this element shall be the 24-hour rainfall depth for the design return period as provided in Table 4-1, distributed in time in accordance with the NRCS Type 2 rainfall distribution pattern.
- A runoff versus rainfall relationship to account for rainfall losses (i.e., rainfall that does not become direct runoff): For the hydrograph procedures presented in this Manual, the NRCS “Curve Number” (CN) method, assuming Antecedent Moisture (or Runoff) Condition 2, will be used to determine runoff (or “rainfall excess”) volume.
- An analytical method for converting the time distribution of “rainfall excess” to a hydrograph of runoff: Three methods may be used:
  - the NRCS synthetic unit hydrograph;
  - the Clark synthetic unit hydrograph;
  - the RUNOFF block of the EPA-SWMM model.

## 4.8 NRCS Unit Hydrograph Method

### 4.8.1 Description

The development of a runoff hydrograph is a complex process not normally done by hand calculation. For that reason, only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program.

The development of a runoff hydrograph for a subbasin involves the following steps:

- Develop or select a design storm hyetograph (i.e., rainfall versus time distribution);
- Estimate “lag time” (see discussion below) and CN;
- Develop a “unit hydrograph” using the standard (peaking factor of 484) dimensionless unit hydrograph (see discussion below);
- Perform step-wise computation of the rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation;
- Apply each increment of excess rainfall to the unit hydrograph to develop a series of elemental runoff hydrographs, one for each increment of rainfall excess; and,
- Sum the flows from each of the elemental hydrographs (keeping proper track of time steps) to form a composite runoff hydrograph for that subbasin.

Figure 4-4 illustrates the unit hydrograph concept:

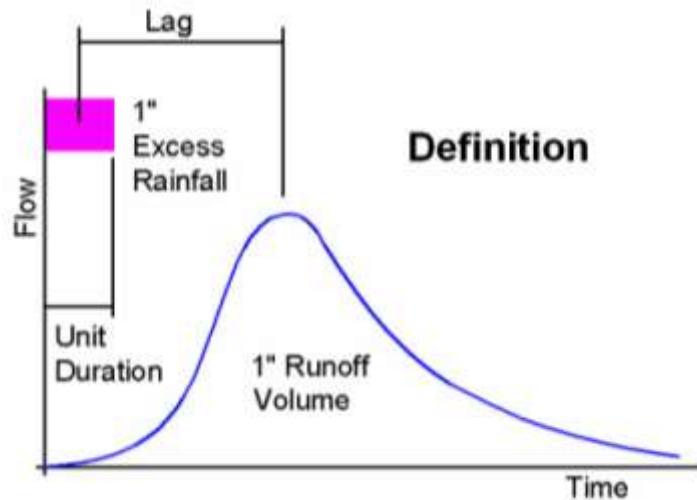


Figure 4-4 Unit Hydrograph

The unit hydrograph equations used in the NRCS method for generating hydrographs include a constant to account for the “peakiness” or “flatness” of the runoff response to excess rainfall in the watershed. The value of 484 is an “average” value that may not be representative of

some flat or steep areas, or areas with significant transient runoff surface storage. For flat terrain or surface storage intensive areas, the factor can be as low as 300 or less. It is recognized that in some watersheds in Sedgwick County, the peaking factor of 484 is probably too high. However, in the absence of detailed hydrologic studies of County watersheds, a value of 484 shall be used for all computations unless the site design engineer provides sufficient technical basis to the local jurisdiction for the use of an alternative peaking factor.

Figure 4-5 shows dimensionless unit hydrographs for peaking factors of 300 (for comparative purposes only) and 484. Note that these graphs are dimensionless. If scaled to the actual time and flow values, the 300 curve would be much broader and would have a lower peak than the 484 curve.

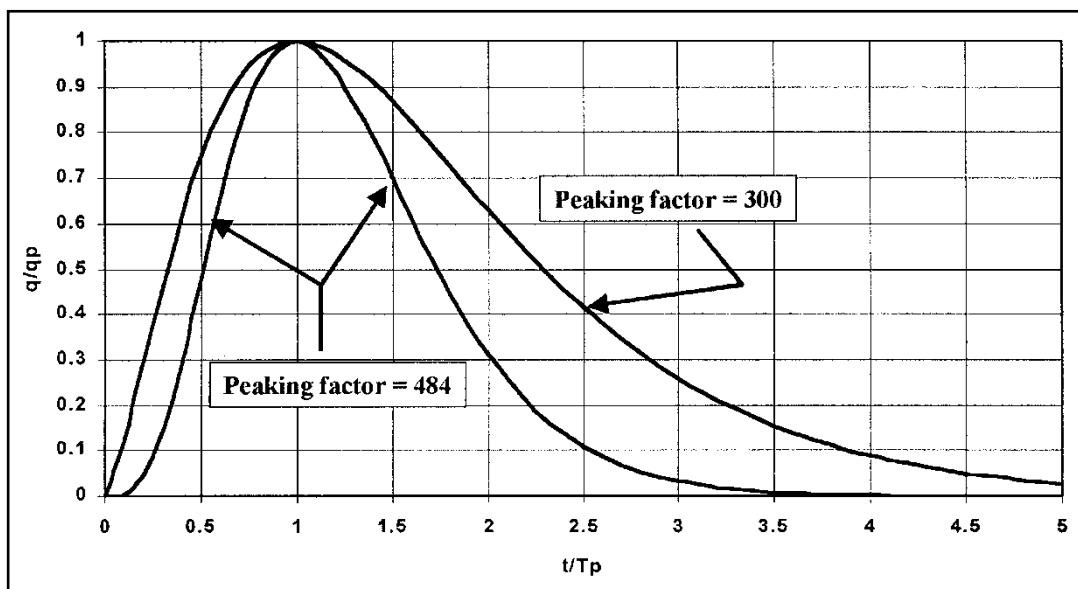


Figure 4-5 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 4-7 provides a tabulation of the dimensionless unit hydrograph for a factor of 484.

Table 4-7 Dimensionless Unit Hydrograph with Peaking Factor of 484

$t/T_p$	$q/q_p$	$t/T_p$	$q/q_p$
0.0	0.0	2.1	0.257
0.1	0.005	2.2	0.210
0.2	0.046	2.3	0.170
0.3	0.148	2.4	0.137
0.4	0.301	2.5	0.109
0.5	0.481	2.6	0.087
0.6	0.657	2.7	0.069
0.7	0.807	2.8	0.054
0.8	0.916	2.9	0.042

$t/T_p$	$q/q_p$	$t/T_p$	$q/q_p$
0.9	0.980	3.0	0.033
1.0	1.000	3.1	0.025
1.1	0.982	3.2	0.020
1.2	0.935	3.3	0.015
1.3	0.867	3.4	0.012
1.4	0.786	3.5	0.009
1.5	0.699	3.6	0.007
1.6	0.611	3.7	0.005
1.7	0.526	3.8	0.004
1.8	0.447	3.9	0.003
1.9	0.376	4.0	0.002
2.0	0.312	--	--

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph presented in Table 4-7 is to multiply each time ratio value by the time-to-peak ( $T_p$ ) and each value of  $q/q_p$  by  $q_p$ . Equation 4-16 presents the calculation for  $q_p$ . (HEC-22, 2001).

$$\text{Equation 4-16} \quad q_p = \frac{PF * A}{T_p}$$

where:

$q_p$	=	unit hydrograph peak rate of discharge (cfs)
PF	=	peaking factor (484)
A	=	area ( $mi^2$ )
$T_p$	=	time to peak = $d/2 + 0.6 T_c$ (hr)
$d$	=	time increment (min)

The parameter  $0.6T_c$  is referred to as the "lag time" ( $T_L$ ) and is the time between the centroid of unit excess rainfall and the peak of the unit hydrograph.

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by Equation 4-17. In the equation,  $X = 3.79$  for when using a 484 peaking factor.

$$\text{Equation 4-17} \quad \frac{q}{q_p} = \left[ \frac{t}{T_p} e^{\left[ 1 - \left( \frac{t}{T_p} \right) \right]} \right]^X$$

where:

$X$	=	3.79 for a unit hy
$t$	=	time (hr)
$q$	=	peak rate of discharge (cfs)

The example problem below provides the calculation of a unit hydrograph using the information provided previously.

### Example Problem – NRCS Unit Hydrograph

Compute the unit hydrograph for a 5-acre example watershed.

- Calculate  $T_p$  and time increment

The time of concentration ( $T_c$ ) was calculated to be 12 minutes for this watershed. If we assume a computer calculation time step ( $d$ ) of 2 minutes then:

$$T_p = \frac{d}{2} + (0.6T_c) = \frac{2}{2} + (0.6*12) = 8.2 \text{ minutes (0.14 hours)}$$

- Calculate  $q_u$

$$q_p = \frac{PF * A}{T_p} = \frac{484 * \sqrt{5/640}}{0.14} = 27 \text{ cfs}$$

- Calculate unit hydrograph.

Based on spreadsheet calculations and interpolations using Equation 4-16, Equation 4-17, and the table below is derived.

**Table 4-8 Dimensionless Unit Hydrograph with Peaking Factor of 484**

Time (min)	t/Tp	q/qp	q (cfs)	Time (min)	t/Tp	q/qp	q (cfs)
0	0.000	0.000	0.00	16	2.145	0.235	6.3
1	0.134	0.013	0.35	17	2.279	0.178	4.8
2	0.268	0.109	2.9	18	2.413	0.133	3.6
3	0.402	0.305	8.2	19	2.547	0.098	2.6
4	0.536	0.546	14.7	20	2.681	0.072	1.9
5	0.670	0.766	21.0	21	2.815	0.052	1.4
6	0.804	0.920	24.8	22	2.949	0.037	1.0
7	0.938	0.993	26.8	23	3.083	0.027	0.73
8	1.072	0.991	26.8	24	3.217	0.019	0.51
9	1.206	0.931	25.1	25	3.351	0.013	0.35
10	1.340	0.835	22.5	26	3.485	0.009	0.24
11	1.475	0.721	19.5	27	3.619	0.006	0.16
12	1.609	0.604	16.3	28	3.753	0.004	0.11
13	1.743	0.492	13.3	29	3.887	0.003	0.08
14	1.877	0.392	10.6	30	4.021	0.002	0.05
15	2.011	0.306	8.3				

#### 4.8.2 Selection of Time Step

The “time step”,  $d$ , should be selected as large as possible without exceeding  $0.29T_L$  or  $0.17T_c$ . The time step (also called the “computation interval” or the “unit duration”) is 2 minutes in the above example.

#### 4.8.3 Application

Once the unit hydrographs for all subbasins have been determined, the process described in Section 4.8.1 is completed using computer hydrologic models. Computer models convolute the rainfall, loss rates, and unit hydrographs to create hydrographs for each subbasin. The subbasins are then “routed” downstream through stream reaches and reservoirs to the watershed outlet. Routing is discussed in Section 4.11.

The details of setting up and applying hydrologic modeling are beyond the scope of this Manual. However, the manuals for the major public domain models HEC-HMS and EPA-SWMM provide instructions on the use of the software.

#### 4.8.4 Simplified NRCS Peak Runoff Rate Estimation

The following NRCS procedure (SCS TR-55, USDA, 1986) is a simplified method for calculating peak discharges. It is included here because it is based on the NRCS hydrograph method. While this procedure is applicable to drainage areas with relatively homogeneous land use, and which can be satisfactorily described by a single CN value. Such drainage areas can then be combined to develop a composite CN that includes both pervious and any impervious areas. It is important to note that the use of this method is limited to estimating peak flows for off-line WQ<sub>v</sub> treatment and for estimating peak inflow for CP<sub>v</sub> facilities when the simplified CP<sub>v</sub> method is used. For these applications, a composite CN that includes both pervious and any impervious areas is used.

Using the simplified NRCS procedure, the peak discharge is estimated using Equation 4-18.

$$\text{Equation 4-18} \quad Q_p = q_u A Q F_p$$

where:

$Q_p$	=	peak discharge (cfs)
$q_u$	=	unit peak discharge (cfs/mi <sup>2</sup> -in of runoff)
$A$	=	drainage area (mi <sup>2</sup> )
$Q$	=	runoff (in)
$F_p$	=	pond and swamp adjustment factor

Computations using the simplified NRCS peak discharge method generally follow the steps defined below.

1. Select the desired rainfall depth,  $P$ .
2. Select or otherwise determine the subbasin runoff curve number, CN.

3. Determine the initial abstraction,  $I_a$ , from Table 4-9, and compute the ratio  $I_a/P$ .

**Table 4-9  $I_a$  Values for Runoff Curve Numbers**

Curve Number	$I_a$ (in)	Curve Number	$I_a$ (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.74	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899	--	--

Source: SCS, TR-55, Second Edition, June 1986

4. The watershed time of concentration is computed using the procedures in Section 4.4 and is used with the ratio  $I_a/P$  to obtain the unit peak discharge ( $q_u$ ) from Figure 4-6 for the Type 2 rainfall distribution. When using Figure 4-6, if the ratio  $I_a/P$  lies outside the range shown in the figure, either the limiting values or another peak discharge method should be used. Also, note that Figure 4-6 is based on a standard peaking factor of 484. The standard peaking shall be used for all analyses unless specifically allowed otherwise by the reviewing authority. If a peaking factor other than 484 is used, this figure is not applicable and the simplified SCS method should not be used.

5. The pond and swamp adjustment factor,  $F_p$ , is estimated from Table 4-10 below. If the drainage area contains between 5% and 10% permanent water surface a value of 0.70

may be assumed. Please note that these adjustments are less accurate for small rainfall events and judgement should be exercised in the use of the simplified method for design.

**Table 4-10 Pond and Swamp Adjustment Factors,  $F_p$**

Pond and Swamp Areas (%*)	$F_p$
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

\* Percent of drainage area

6. Finally, the peak runoff rate is computed using Equation 4-18.

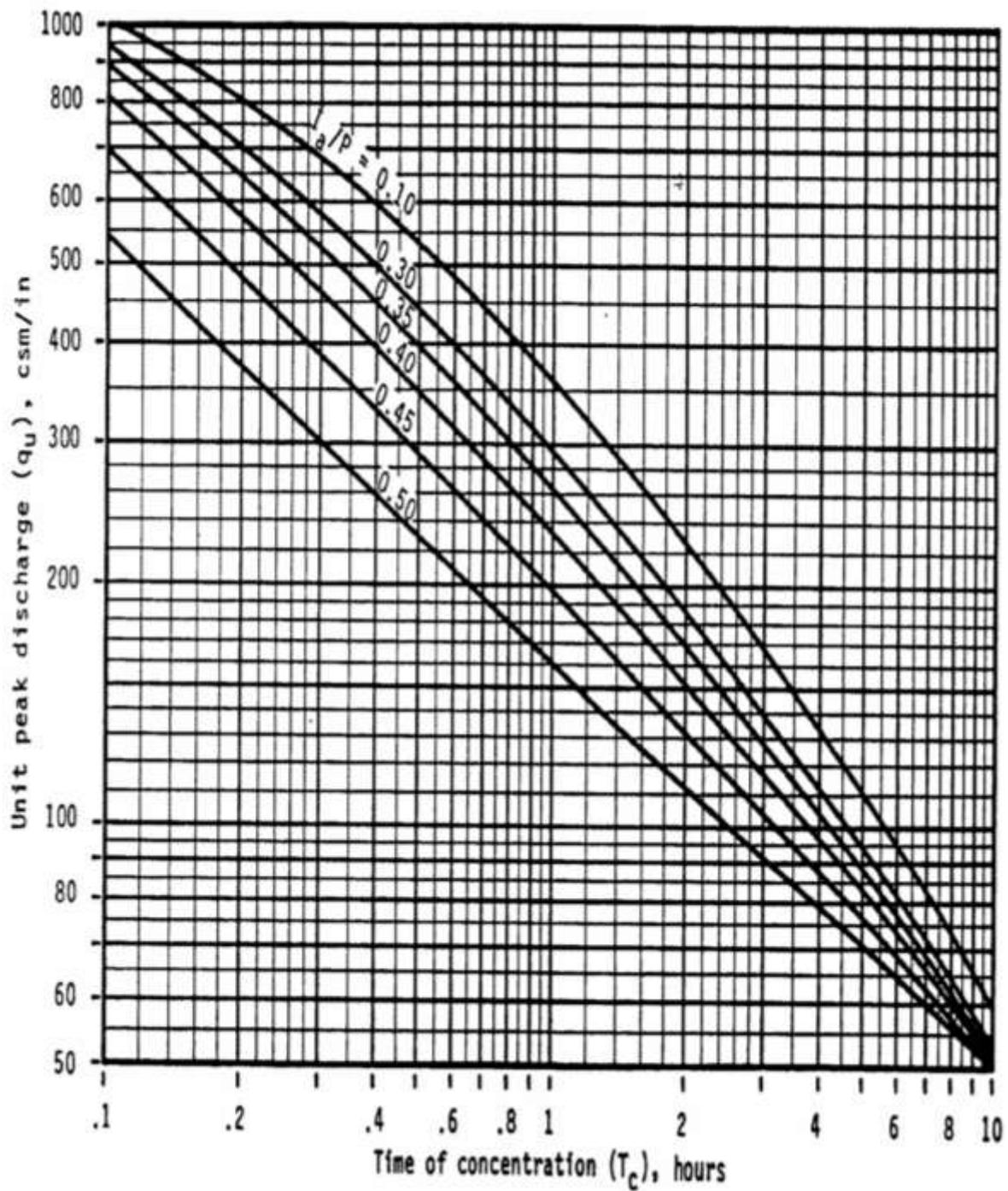


Figure 4-6 SCS Type II Unit Peak Discharge Graph

(Source: SCS, TR-55, Second Edition, June 1986)

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#### Example Problem – Simplified NRCS Peak Flow Method

Compute the 100-year peak discharge for a 28-acre site located in Wichita.

Separate the planned development into relatively homogeneous land uses to estimate a weighted CN for the entire site. For the example, the site will be developed as follows:

- Disturbed pervious area (hydrologic soil group B) = 10 ac
- Impervious area = 8 ac
- Undisturbed area (hydrologic soil group C) = 10 ac
- % pond surfaces = 1

Calculate rainfall excess:

- Look up the 100-year, 24-hour rainfall in Table 4-1; rainfall = 7.8"
- Compute the weighted CN:

Land Use	Area (ac.)	Area Fraction	Curve Number	Weighted Curve Number (Area Fraction x CN)
Disturbed Pervious Area Soil Group B	10	0.36	80	28.8
Impervious Area	8	0.28	98	27.4
Undisturbed Area Soil Group C	10	0.36	71	25.6

$$\text{Weighted CN} = 28.8 + 27.4 + 25.6 = 81.8$$

- Look up the initial abstraction ( $I_a$ ) from Table 4-9 based on the curve number.  $I_a = 0.439"$ .
- Determine the potential maximum retention (S) from Equation 4-1.  $S = 2.2"$ .
- Calculate the rainfall excess (Q) from  $I_a$ , S and rainfall depth.

$$Q = \frac{(P - I_a)^2}{[(P - I_a) + S]} = \frac{(7.8 - 0.439)^2}{[(7.8 - 0.439) + 2.2]} = \frac{54.18}{9.56} = 5.7"$$

Calculate time of concentration

Segment	Type of Flow	Length (ft)	Slope (%)
1	Sheet flow through pasture	100	1.5
2	Shallow grassed channel	450	1.0
3	Main riprap channel (10'W x 2'D)	600	0.5

Calculate sheet flow travel time using Equation 4-5.

$$T_{T_{(sheet)}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(0.13*100)^{0.8}}{(3.5)^{0.5}(0.015)^{0.4}} = 0.156 \text{ hr} = 9 \text{ min.}$$

## Section 4.8 - Clark Method

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Calculate shallow concentrated flow travel time using Equation 4-6 and Equation 4-4

$$V = KS^{0.5} = 15 * (0.01)^{0.5} = 1.5 \text{ ft/s}$$

$$T_{T(\text{shallow})} = \frac{L}{60V} = \frac{450}{60 * 1.5} = 5 \text{ min}$$

Calculate open channel flow travel time using Equation 4-7 and Equation 4-5.

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.04} \left( \frac{2 * 10}{10 + 2 + 2} \right)^{2/3} 0.005^{1/2} = 3.3 \text{ ft/s}$$

$$T_{T(\text{open})} = \frac{L}{60V} = \frac{600}{60 * 3.3} = 3 \text{ min}$$

Calculate total time of concentration using Equation 4-8.

$$T_C = \sum T_{T(\text{sheet})} + \sum T_{T(\text{shallow})} + \sum T_{T(\text{open})} = 9 + 5 + 3 = 17 \text{ min}$$

Calculate  $I_a/P$

$$I_a/P = 0.439/7.8 = 0.056$$

Determine unit peak discharge  $q_u$  (100-year) from Figure 4-6.

$$q_u = 700 \text{ cfs/mi}^2\text{-inch} \text{ (limit of graph)}$$

Look up the pond/swamp adjustment factor for the swamp area of 1% from Table 4-10.

$$F_p = 0.87$$

Calculate peak discharge using Equation 4-18.

$$Q_p = q_u A Q F_p = 700 \left( \frac{28}{640} \right) (6.0) (0.87) = 160 \text{ cfs}$$

---

## 4.9 Clark Method

### 4.9.1 Introduction

The Clark method defines a unit hydrograph for a given subbasin using the concept of the instantaneous unit hydrograph (IUH). Using this method, the subbasin is subdivided by a series of isochrones. An isochrone is a line on the ground wherein all points on the line flow to the subbasin outlet in an equal amount of time. Runoff from the subbasin can thus be routed to the subbasin outlet based on a time-area diagram. At the outlet, runoff transient storage effects are then routed through a theoretical linear reservoir. The resulting hydrograph is then adjusted to represent the hydrograph for a unit of rainfall excess for a specified unit duration. The Clark Method is shown in Figure 4-7.

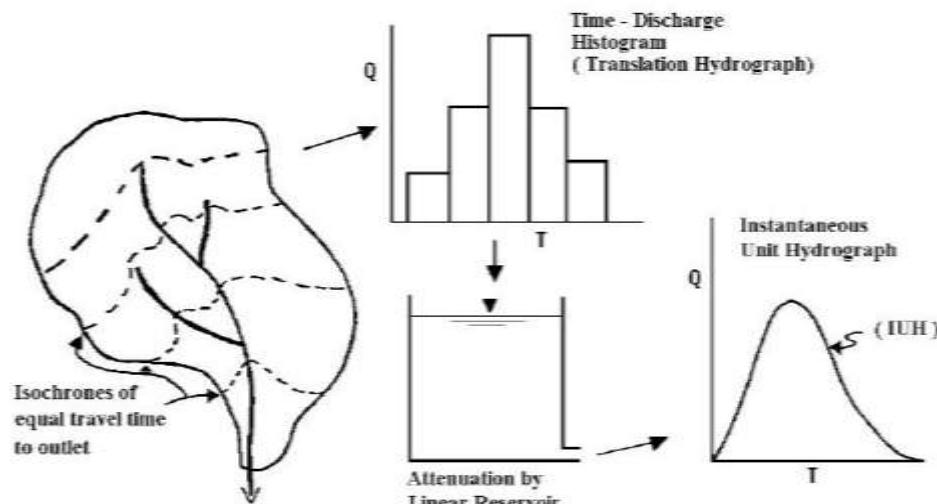


Figure 4-7 Clark Concept (US Army Corps of Engineers EM 1110-2-1417)

It should be noted that the theoretical linear reservoir as applied here is not an actual reservoir. It is a mathematical concept used to represent the distributed attenuation effects occurring in the subbasin runoff. The linear reservoir is represented by Equation 4-19 (USACE, 1994):

$$\text{Equation 4-19 } S = RO$$

where:

$S$	=	Storage
$R$	=	Attenuation (Storage) Constant
$O$	=	Outflow

### 4.9.2 Dimensionless Time-Area Curve

Studies have indicated that it is not necessary to produce detailed time-area curves in order to produce reasonably accurate synthetic unit hydrographs. The dimensionless time-area curve included in the HEC-1 and HEC-HMS hydrologic models (developed by the United States

Army Corps of Engineers) produce reasonably accurate synthetic unit hydrographs. The dimensionless curve, as implemented in HEC-HMS, is shown in Figure 4-8. For computer models having this method available (e.g., HEC-1 and HEC-HMS), it is not necessary to compute the time-area curve or unit hydrograph since those steps are performed by the program. However the user must provide two input parameters for each subbasin, as discussed below.

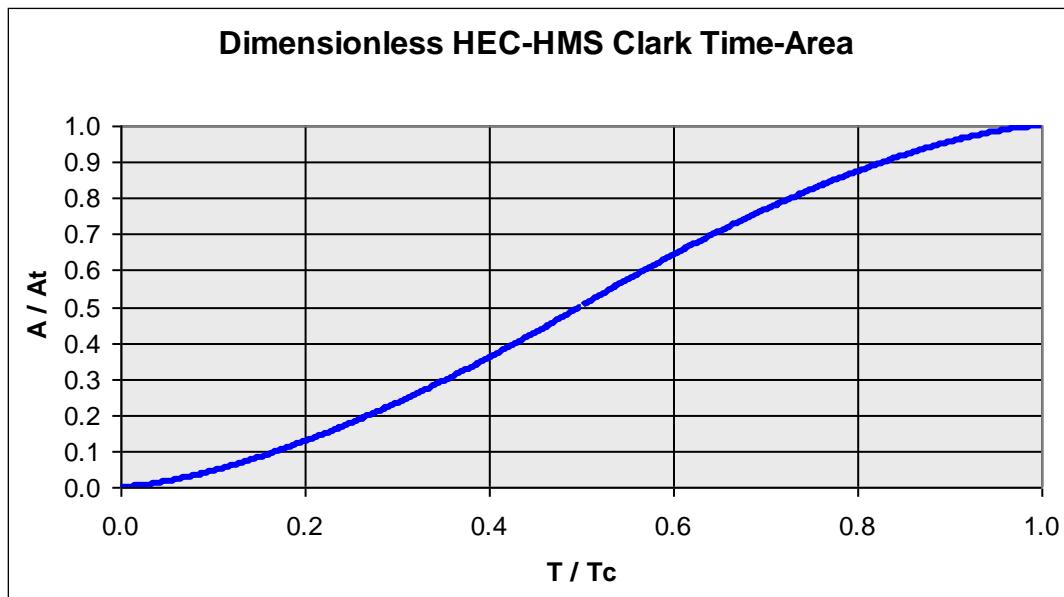


Figure 4-8 Dimensionless Clark Time-Area

#### 4.9.3 Storage Constant and Time of Concentration

In order to apply the Clark method in a model using the dimensionless time-area curve, the time of concentration ( $T_c$ ) and the watershed storage constant ( $R$ ) are required as inputs. The time of concentration can be estimated using the method provided in this Manual. However, the value of  $R$  is not as readily determined. The best procedure for determining the storage constant is to derive the parameter from a number of observed hydrographs in the region and determine the ratio shown in Equation 4-20 that is representative of the region of interest. Equation 4-20 is shown with a value of 0.5, however ratio values of 0.3 to 0.7 are common (USACE, 1994):

$$\text{Equation 4-20} \quad \frac{R}{T_c + R} = 0.5$$

where:

$R$  = Attenuation (Storage) Constant  
 $T_c$  = Time of concentration (minutes)

#### 4.9.4 Application

The unit hydrograph derived by the Clark method is applied to general hydrologic analysis in the same manner as the NRCS unit hydrograph. The NRCS rainfall distribution and Curve Number method for losses are used for the Clark method when implementing the requirements of this Manual. Typically the Clark method is applied using appropriate modeling software such as HEC-HMS, developed by the United States Army Corps of Engineers. The time of concentration ( $T_c$ ) and the watershed storage constant ( $R$ ) are required as inputs.

### 4.10 EPA-SWMM "RUNOFF" Method

#### 4.10.1 Introduction

EPA-SWMM was developed by the Environmental Protection Agency (EPA) to analyze stormwater quantity and quality problems associated with runoff from urban areas. EPA-SWMM has become the model of choice of many engineers for simulation of drainage systems primarily composed of closed conduits. The model can simulate both single-event scenarios and continuous rainfall/runoff processes. The model does not use the unit hydrograph approach previously discussed, but instead uses a deterministic-based RUNOFF module for generating flows. EPA-SWMM has the capability to model both wet and dry weather flow. The basic output from EPA-SWMM consists of runoff hydrographs, pollutographs, storage volumes and flow stages and depths. EPA-SWMM's hydraulic computations are link-node based. EPA-SWMM solves the complete dynamic flow routing equations to simulate backwater, looped pipe connections, manhole surcharging and pressure flow. It is a comprehensive model in its capabilities to simulate urban storm flow and many cities have used EPA-SWMM successfully for stormwater, sanitary, or combined sewer system modeling.

EPA-SWMM also contains routines for hydrologic and water quality modelling. A large suite of hydrologic processes affecting the quantity and quality of runoff from drainage areas can be modelled. Pollutant transport is simulated in tandem with hydrologic and hydraulic computations and consists of calculation of pollutant buildup and washoff from land surfaces and pollutant routing, scour and in-conduit suspension in flow conduits and channels. EPA-SWMM is a public domain model. Because of the popularity of the model, commercial third party enhancements to EPA-SWMM have become common, making the model a strong choice for drainage modeling.

#### 4.10.2 Approved EPA-SWMM Hydrologic Procedures

EPA-SWMM is approved for both hydrologic and hydraulic modelling within Wichita and Sedgwick County. Commercial third-party models based on EPA-SWMM must be pre-approved by the City and County, not only on the basis of technical acceptance but also on the basis of the cost for the City and County to maintain a license for the product for review purposes.

A detailed review of the use of the model is beyond the scope of this Manual. Details may be found in the manual distributed with the EPA-SMMM software. However, an overview of the application of the basic model options as they relate to hydrology follows. Guidelines addressing hydraulic flow principles are found in Volume 2, Chapter 5.

- The Curve Number method shall be chosen as the infiltration model;
- Dynamic wave routing shall be employed;
- The “allow ponding” option shall not be used. Rather, local ponding shall be modelled within storage nodes;
- The reporting time step shall be no greater than 5 minutes;
- The routing time step shall be small enough to prevent significant instabilities;
- Inertial terms of the dynamic wave equations shall be kept or damped, not ignored;
- Conduit lengthening shall be employed;
- The model shall be run for 48-hours or until site outflow is less than 5% of peak flow, whichever is longer;
- Rainfall shall be applied to each subcatchment as a NRCS Type 2 distribution of the local 24-hour storm depth;
- Subcatchment width shall be defined as the subcatchment area in square feet divided by the longest flow path that does not enter a curb and gutter, lined channel or storm sewer;
- Curve numbers for pervious areas shall be determined per the guidance found in Section 4.3.3;
- The percent slope shall be calculated along the longest flow path;
- Area-weighted saturated hydraulic conductivity values for pervious areas shall be taken from the table below, or substantiated by on-site testing. These values are not critical for single event analysis, but are necessary for proper continuous simulation. Table 4-10 provides hydraulic conductivity values by hydrologic soil group.

**Table 4-11 Hydraulic Conductivity by Hydrologic Soils Group**

NRCS Hydrologic Soil Group	Saturated Hydraulic Conductivity (in/hr)
A	0.45
B	0.2
C	0.1
D	0.02
Disturbed A	0.2
Disturbed B	0.1
Disturbed C	0.05
Disturbed D	0.015

- Drying time shall be 4 days. This value is also important only for continuous simulation models. Depression storage shall be as indicated in Table 4-11 below.

**Table 4-12 Depression Storage Depths**

Surface	Depression Storage Depth (in)
Impervious	0.1
Lawn	0.3
Cropland	0.3
Wooded Areas and Pasture	0.4

- Manning's roughness values for subcatchment sheet flow shall be taken from Appendix Table A.
- Subarea flows shall be routed to the outlet unless specifically allowed by the local jurisdiction as a reduction for impervious disconnection.
- The percent of runoff routed shall be 100%.

## 4.11 Routing

### 4.11.1 Routing Methods

As water flows along a channel or through a reservoir, the hydrograph changes due to translation and attenuation. Translation reflects the time required for water to move from one location to another location downstream. For simple translation, the downstream hydrograph is identical to the upstream hydrograph except it occurs later in time due to the travel time between the two locations. Attenuation reflects the change in shape of the hydrograph as it moves downstream, primarily due to the effects of water temporarily stored in the stream, floodplain, and/or reservoir. The downstream hydrograph characteristics (its timing and shape) are determined by both translation and attenuation effects. "Routing" is the procedure used to analyze the upstream hydrograph and the downstream channel and floodplain or reservoir to predict the change in hydrograph characteristics as it travels through the "routing reach" (section of stream and floodplain or reservoir), and is transformed into the downstream hydrograph.

There are a number of methods used to route flows through streams, floodplains and reservoirs. For use in implementing the requirements of this Manual, three methods may be used for channel and floodplain routing, and one method will be used for reservoir routing. Those methods are discussed briefly herein. For more detailed treatment of these methods, the reader is referred to the documentation readily available from a number of sources, including the manuals that are distributed with HEC-1, HEC-HMS and EPA-SWMM.

#### 4.11.1.1 Stream and Floodplain Routing

##### ***Muskingum-Cunge Method***

The Muskingum-Cunge method is a hydraulic routing technique that uses conservation of mass and the diffusion form of the conservation of momentum. The method considers the shape and roughness of the channel and floodplain, the energy gradient (approximated by the channel slope), and total energy loss. Input is relatively straightforward: channel shape, slope, length and roughness. The method is applicable to most flow conditions, with two notable exceptions. The method shall not be used if the flow is significantly affected by backwater conditions from downstream culverts, bridges, reservoirs, or other restrictive features if they tend to cause ponding effects in the reach of stream or floodplain being routed. Secondly, the method shall not be used for rapidly rising hydrographs on relatively flat streams. The Muskingum-Cunge method is a computation-intense procedure and is performed using computer models such as HEC-1 and HEC-HMS. Details of the application of this method are covered in numerous standard hydraulic routing references.

##### ***Dynamic Wave Method***

The dynamic wave method uses the full equation of flow (Saint Venant's equation) to incorporate conservation of energy and momentum to perform a rigorous hydraulic analysis of flow in channels and streams. Some of the terms in the full equation are often neglected or damped for computational stability purposes. This method is extremely computation-intense and is performed with computer models such as EPA-SWMM. The most common use of this method for urban hydrology applications is when EPA-SWMM is also used to compute runoff hydrographs and route those flows through pipe, storage and channel systems in the dynamic node. Details of the application of this method are covered in numerous standard hydraulic routing references.

##### ***Stream and Floodplain Routing by Modified Puls Method***

The modified Puls stream routing method is a hydrologic routing procedure wherein the procedure used for flat reservoir routing (see below) is adapted for use in analyzing streams and floodplains with sloping hydraulic grade lines. With this method, the stream is segmented into routing reaches and a storage versus outflow relationship is developed for each reach based on steady state profile analyses for a range of flows. The storage-discharge relationship for each reach is then used to route the inflow hydrograph through the reach using the storage-indication method common to the modified Puls method of flat pool reservoir routing. The process is repeated until the hydrology and hydraulics models are "balanced". Although this method can be performed manually, it is much more efficient to use computer programs such as HEC-1 or HEC-HMS and HEC-2 or HEC-RAS for the analysis. Details of this method are covered in standard references.

#### 4.11.1.2 Reservoir Routing

##### ***Modified Puls Method***

For flat pools (ponds and lakes), flood routing is by the well-known and widely used modified Puls method. This is a hydrologic routing method that considers conservation of mass only. Basically, over a specified time interval the increase in volume stored in the reservoir is equal to the inflow volume minus the outflow volume during that time interval. These factors are dependent on the inflow hydrograph, the stage-storage relationship for the reservoir, and the stage-discharge relationship for the outlet(s) serving the reservoir. (Stage is the water surface elevation). Routing is performed in a time step-wise method that tracks inflow, change in volume, and outflow.

Since reservoir routing is a fundamental procedure for implementing many of the requirements of this Manual, the following provides a more detailed review of this method, including general guidance on stormwater runoff storage for meeting stormwater management control objectives (i.e., water quality protection, downstream channel protection, and flood control).

##### ***Storage Facilities***

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows for water quality protection and downstream channel protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural stormwater controls and/or nonstructural features. Examples of storage facilities include:

- Lakes, ponds, wetlands;
- Bioretention areas, swales, channels and other water quality management facilities when engineered for detention;
- Underground detention; and,
- Rooftop and parking lot detention when permitted and approved.

##### ***Storage Classification***

Stormwater storage can be classified as detention, extended detention or retention. Some facilities include one or more types of storage.

Conventional detention is used to reduce the peak discharge by detaining runoff for whatever time period is necessary to accomplish the desired peak outflow. The time period is not critical. Detention volumes are designed to completely drain after the storm has passed without regard to drain time. This type of detention can be used to meet flood criteria where required. Examples of this type of detention are conventional dry detention ponds, or the flood control pools located above the water quality and channel protection pools in ponds or wetlands.

Extended detention (ED) is used to detain a specified volume of runoff for a specified time. This type of detention can be used to provide water quality treatment and channel protection detention. Examples of this type of detention are extended dry detention ponds and extended wet detention ponds/wetlands.

Retention facilities are designed to contain a permanent pool of water, such as stormwater ponds and wetlands, which is used for water quality protection or other purposes. Routing is not applied to the permanent pool. However, detention or extended detention pools located above the permanent pool would be subject to routing.

Storage facilities are often classified on the basis of their location. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are designed to manage stormwater runoff from multiple projects and/or properties. A discussion of regional stormwater controls is found in Chapter 3.

Storage can also be categorized as *on-line* or *off-line*. *On-line* storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. *Off-line* storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 4-9 illustrates *on-line* versus *off-line* storage.

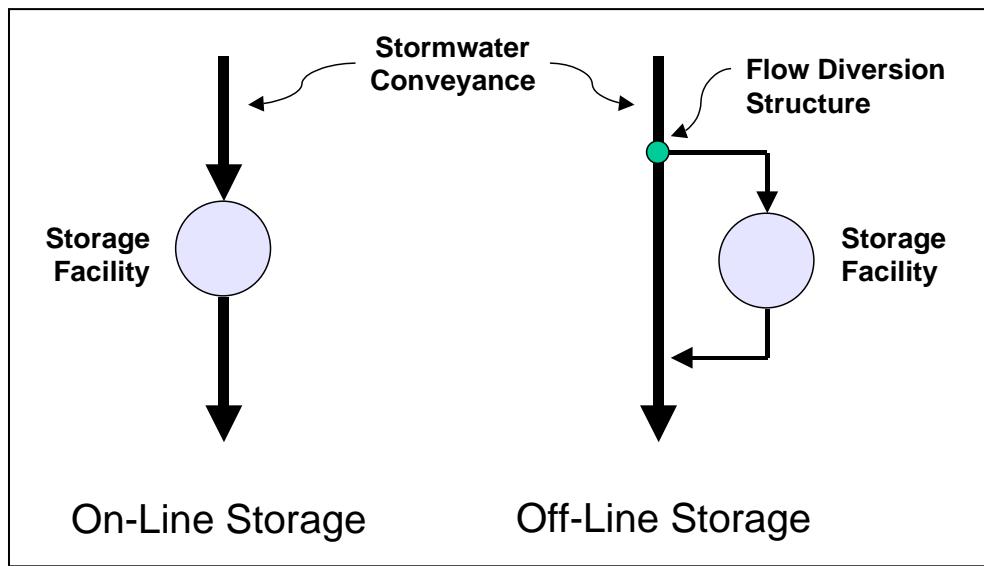


Figure 4-9 On-Line versus Off-Line Storage

#### **Stage-Storage Relationship**

A stage-storage curve or table defines the relationship between the elevation of water and the storage volume in a storage facility (see Figure 4-10).

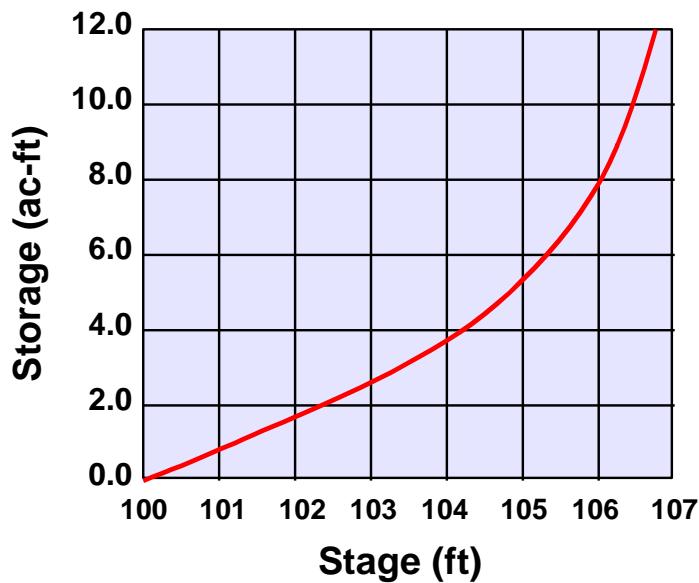


Figure 4-10 Example Stage-Storage Curve

The storage volume for basins may be developed using a topographic map, grading plan, or other suitable geometric definition, and either the average-end area method or the conic method formulas. The average-end area method is shown graphically in Figure 4-11.

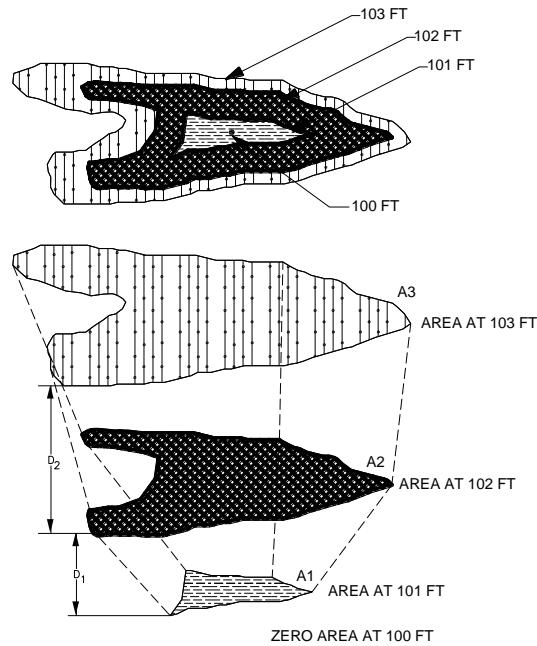


Figure 4-11 Average-End Area Method

The average-end area formula presented in Equation 4-21.

$$\text{Equation 4-21} \quad V_{1,2} = \left[ \frac{A_1 + A_2}{2} \right] * d$$

where:

$V_{1,2}$	=	storage volume ( $\text{ft}^3$ ) between elevations 1 and 2
$A_1$	=	surface area at elevation 1 ( $\text{ft}^2$ )
$A_2$	=	surface area at elevation 2 ( $\text{ft}^2$ )
$d$	=	change in elevation between points 1 and 2 (ft)

The conic method formula is presented in Equation 4-22.

$$\text{Equation 4-22} \quad V_{1,2} = [d / 3](A_1 + A_2 + [A_1 A_2]^{0.5})$$

where:

$V_{1,2}$	=	storage volume ( $\text{ft}^3$ ) between elevations 1 and 2
$A_1$	=	surface area at elevation 1 ( $\text{ft}^2$ )
$A_2$	=	surface area at elevation 2 ( $\text{ft}^2$ )
$d$	=	change in elevation between points 1 and 2 (ft)

### ***Stage-Discharge Relationship***

A stage-discharge (or rating) curve defines the relationship between the elevation of the water surface in the storage facility and the rate of total discharge or outflow from the facility. A typical storage facility has two outlets: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey a relatively large design flow (for example, the 24-hour, 25-year flood event) without allowing flow to enter the emergency spillway. A riser, pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. For normal operating conditions, the stage-discharge curve may take into account the discharge characteristics of both the principal spillway and the emergency spillway. However, for emergency conditions, the stage-discharge curve should take into account only the capacity of the emergency spillway, assuming the principal spillway is blocked. See Volume 2, Chapters 3 and 5 for more details on pond spillway requirements.

Since the total discharge rating for a storage facility usually consists of several orifices and weirs in the principal outlet, plus the emergency overflow, the total rating will be a composite of the individual ratings. The discharge rating for each orifice and weir, and the emergency overflow, are developed and then the flows for each increment of elevation are summed to obtain the total rating. See Volume 2, Chapter 5 for more detail on outlet hydraulics.

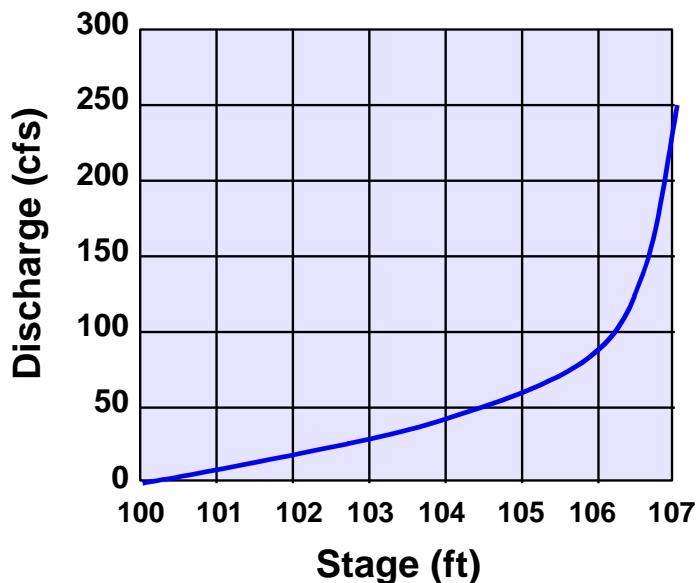


Figure 4-12 Example Stage-Discharge Rating Curve

#### ***Routing through Storage Facility***

Given the hydrograph of inflow to the reservoir, the stage-storage relationship, and the stage-discharge relationship, the hydrograph of outflow may be determined. This is accomplished by first combining the stage-storage and stage-discharge curves into a single curve relating storage and discharge (possible since stage is common to both curves). This is put in the form of:  $O$  versus  $(2S/T + O)$  where  $O$  is outflow,  $S$  is storage, and  $T$  is the routing time interval. From this, and conservation of mass, the routing equation may be developed. (USACE, 1994)

$$\text{Equation 4-23} \quad (2S_2/T) + O_2 = I_1 + I_2 + (2S_1/T) - O_1$$

where:

$S_1, S_2$	=	storage volume at beginning and end of time step
$O_1, O_2$	=	outflow at beginning and end of time step
$I_1, I_2$	=	inflow at beginning and end of time step
$T$	=	duration of time step (the routing interval)

Since the terms on the right side of the equation are known, the left side can be computed. Then, given the storage and outflow relationship developed as previously described, the outflow may be determined, and then the storage at the end of the interval may be computed. The routing continues one routing interval at a time, until the routing is complete. Figure 4-13 illustrates flat pool reservoir routing.

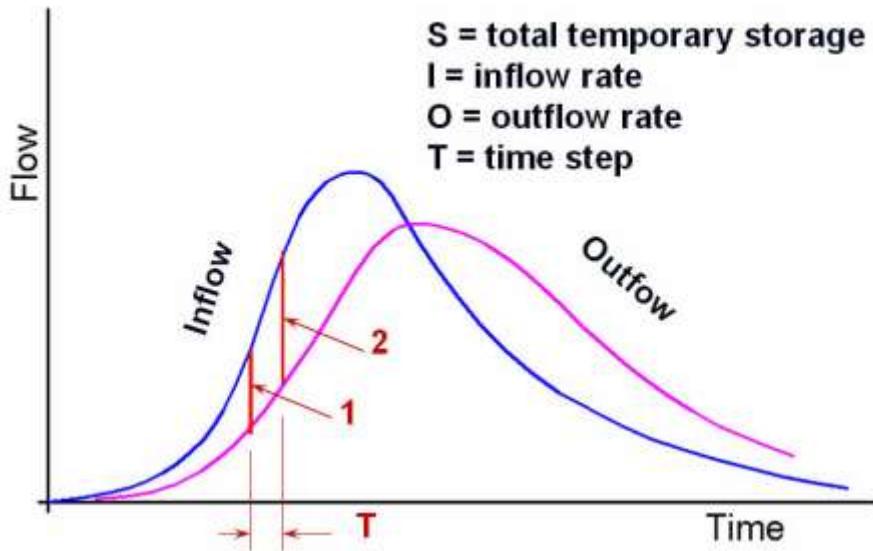


Figure 4-13 Illustration of Flat-Pool Reservoir Routing

## 4.12 Downstream Hydrologic Assessment

The purpose of peak flow control is to protect downstream properties from increased flooding and increased erosion impacts due to upstream development. Controlling peak flow at the outlet of a site such that post-development peak discharge does not exceed pre-development peak discharge does not necessarily accomplish this goal. It has been shown that in certain cases this does not always provide effective flood control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this are related to: (1) the timing of the flow peaks; and, (2) the increase in total volume of runoff.

### 4.12.1 Reasons for Downstream Problems

#### *Flow Timing*

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 4-14. The peak flow from the site is reduced appropriately, but in this example case the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required.

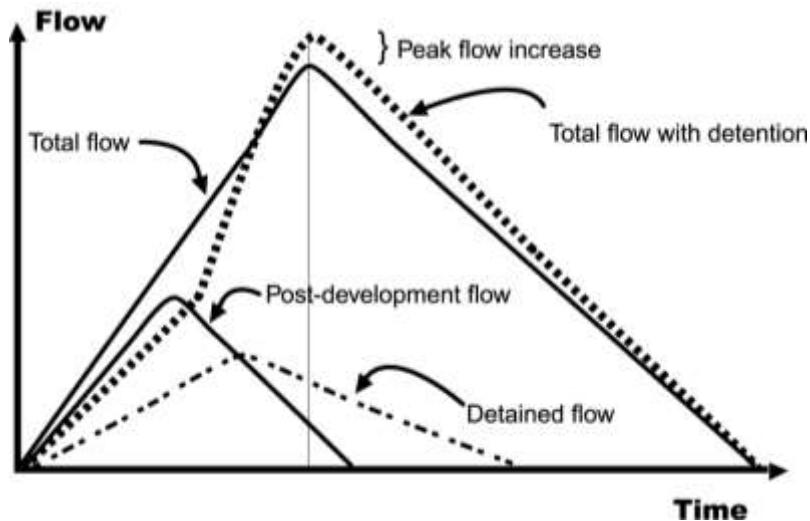


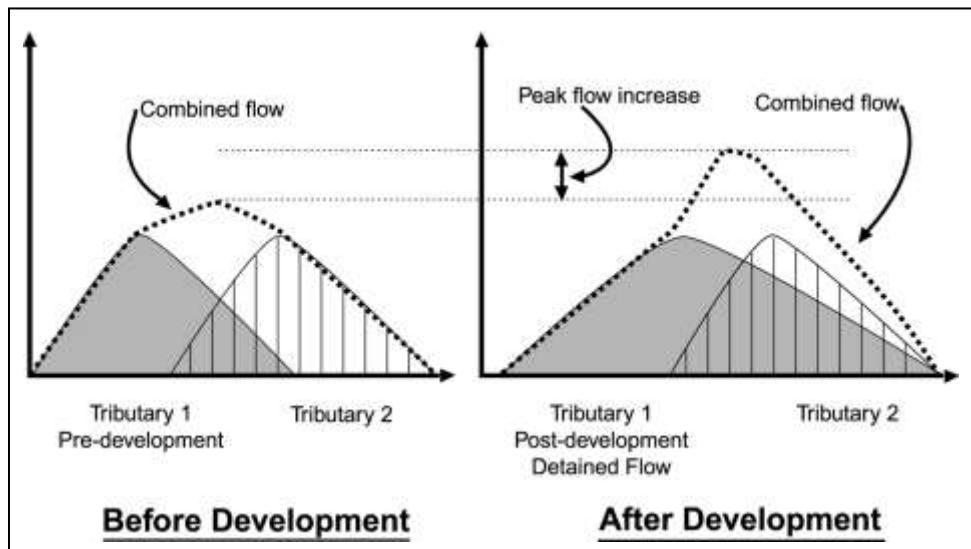
Figure 4-14 Detention Timing Example

In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained. This is most likely to happen if detention is placed on tributaries towards the downstream portion of the watershed, holding back peak flows and adding them as the peak from the upper reaches of the watershed arrives.

#### ***Increased Volume***

An important impact of new development is an increase in the total runoff volume. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributary flows to increase the downstream peak flows.

Figure 4-15 illustrates this concept. The figure shows the pre and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.



**Figure 4-15 Effect of Increased Post-Development Runoff Volume w/ Detention on a Downstream Hydrograph**

#### 4.12.2 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on stormwater peak flows and velocities are evaluated downstream. The assessment begins at the outfall of a development and extends to a point downstream where the discharge from a proposed development no longer has a significant impact upon the receiving stream or storm drainage system. This is called the “zone of influence.”

Beyond the zone of influence the stormwater effects of a structural control become relatively insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, a general rule of thumb is that the zone of influence can be considered to end at the point where the drainage area controlled by the detention or storage facilities for a project comprise 10% of the total drainage area if the facilities are located on a tributary to the main stream (i.e., is *off-line* relative to the main receiving stream.) This is known as the *10% Rule*. As an example, if a structural control (or group of controls for a project) drains 10 acres, the zone of influence may be assumed to end at the point where the total drainage area is 100 acres.

##### 4.12.2.1 Off-line

The following downstream assessment procedure shall be employed for proposed projects with *off-line* detention

Determine the outfall location of the site and the pre- and post-development site conditions.

- Using a topographic map determine the location of the 10% point.

- Using a hydrologic model determine the pre-development peak flows and velocities at the project discharge point, and at each tributary junction or significant hydraulic structure (i.e., bridge or culvert) beginning at the location of the planned development outfall and ending at the next junction or significant structure (whichever is encountered first) beyond the 10% point. Undeveloped off-site areas are modeled in their existing condition for both the pre- and post-development analyses. The discharges and velocities are evaluated for the 2, 5, 10, 25 and 100-year, 24-hour storm events.
- Change the land use and/or stormwater facilities on the site and create a post-development model.
- Compare the pre- and post-development peak discharges and velocities at each junction and significant structure in the model. If the post-developed flows or velocities are higher than the pre-developed flows and velocities for the same frequency event, change the detention design. Repeat steps 3 and 4 until the post-development flows and velocities do not increase relative to pre-development conditions.
- If modelling of a site shows that flood protection storage is not necessary for downstream peak flow and velocity control, then the developer may request to the reviewing authority that the flood control detention requirement be waived. The request must be accompanied by an analysis demonstrating that detention is not required. In the event that a request to waive detention for flood control is approved, extended detention for long-term channel protection and water quality protection are still required.

Note that the requirement to not exceed existing peak flood flows or velocities may not be protective downstream if existing peak flood flows cause flooding or existing velocities are erosive. Such situations are beyond the scope of this Manual.

#### 4.12.2.2 On-line Detection Analysis

The above procedure addresses *off-line* ponds. That is, ponds located on a tributary to the main stream. If the detention pond serving a project is located *on-line*, the above procedure is modified. An *on-line* pond is located on the main receiving stream, and has significant off-project inflow from the upstream watershed passing through it in addition to the project runoff. In that case, the zone of influence is considered to the 10% point, as described above, or the point where the pre- and post-development flows are the same, whichever is further downstream.

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#### Example Problem – 10% Rule

Figure 4-16 illustrates the concept of the ten-percent rule for two sites in a watershed.

##### ***Discussion***

Site A is a development of 10 acres, all draining to a wet Extended Detention (ED) stormwater pond. The design will incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is

determined that the analysis should end at the tributary marked "120 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The designer constructs a simple HEC-HMS model of the 120-acre area using existing condition subbasins for each tributary. Key detention structures existing in any tributary must be modeled. An approximate CN number is used for each off-site subbasin since the actual peak flow is not key for the analysis; only the increase or decrease is important. The accuracy in CN number determination for those subbasins is not as significant as an accurate estimate of the time-of-concentration. A more detailed subbasin model for the development site is required, using design values for CNs, time-of-concentration, etc. Since flooding and erosion are issues downstream, the pond is designed (through several iterations) until the peak flow and velocities do not increase at any location down to the 120-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time-of-concentration for input into the HEC-HMS model of the watershed. The model shows a detention facility, in this case, will actually increase the peak flow in the stream. Flood control detention will not be required for this site, though water quality and channel protection requirements must still be met.

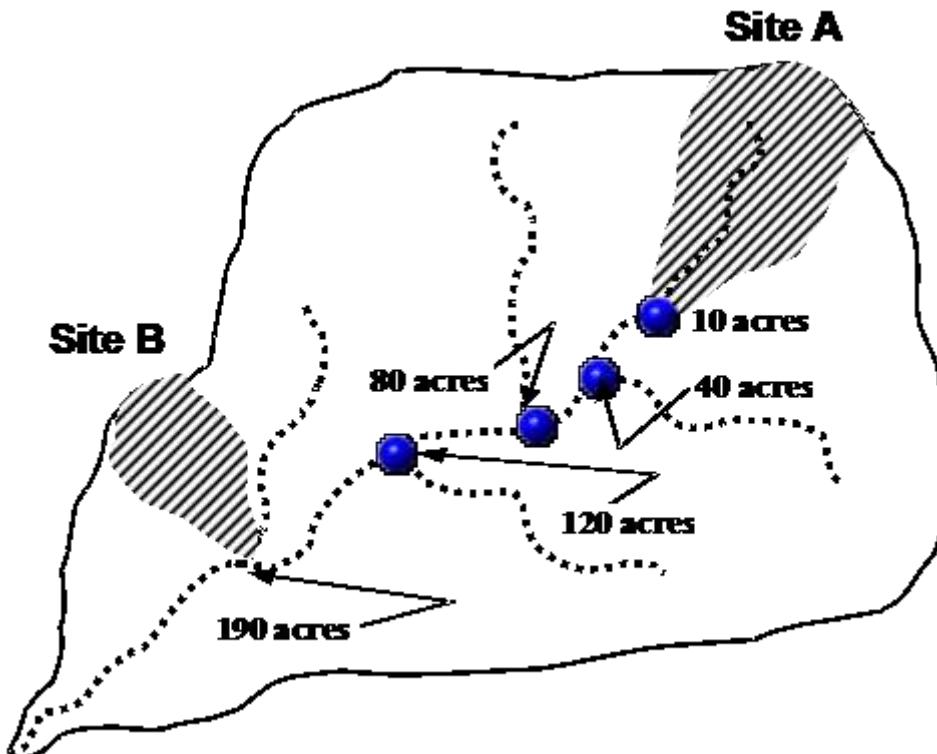


Figure 4-16 Example of the Ten-Percent Rule

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## 4.13 Water Quality Protection Volume and Peak Flow

### 4.13.1 Water Quality Protection Volume Calculation

The Water Quality Protection Volume ( $WQ_v$ ) is the treatment volume that must be treated to remove 80% of the post-development total suspended solids (TSS) load. This is achieved by intercepting and treating all of the runoff from 85% of the storms that occur, and a portion of the runoff from all larger storms. See Volume 1, Chapter 3 for a detailed presentation of the 80% TSS removal policy.

The  $WQ_v$  is calculated by multiplying the 85<sup>th</sup> percentile annual rainfall event (1.2") by an area weighted volumetric runoff coefficient ( $R_v$ ) and the effective site area. Equation 4-24 presents the  $WQ_v$  calculation.

$$\text{Equation 4-24} \quad WQ_v = \frac{PR_v A}{12}$$

where:

$WQ_v$	=	water quality protection volume (acre-feet)
$P$	=	water quality rainfall depth (1.2 inches for Wichita/Sedgwick County)
$R_v$	=	volumetric runoff coefficient
$A$	=	drainage area (acres) adjusted for $WQ_v$ Reductions per Volume 2, Chapter 2 of this Manual

When a development project contains or is divided into multiple outfalls,  $WQ_v$  should be calculated and addressed separately for each outfall. For any site design, it is possible to treat the  $WQ_v$  using multiple treatment facilities. (Section 4.14 of this chapter presents the calculation of the 80% TSS removal standard).

When desired by the design engineer,  $WQ_v$  can be expressed in inches over the drainage area ( $Q_{wv}$ ) using the following formula:

$$\text{Equation 4-25} \quad Q_{wv} = PR_v$$

where:

$Q_{wv}$	=	water quality protection volume (inches)
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The volumetric runoff coefficient ( $R_v$ ) relates total runoff to total rainfall for the range of small storms that dominate the total volume of precipitation. The value of the coefficient depends largely on the fraction of the site that is impervious, and to a lesser degree on the texture of the soil (i.e., sandy versus clayey) and whether or not the soil has been disturbed. If more than one category of runoff area is present, an area-weighted value of  $R_v$  should be used in the  $WQ_v$  calculation.  $R_v$  is calculated as follows (Schueler, 2008):

$$\text{Equation 4-26} \quad R_v = R_{vU}U + R_{vD}D + R_{vI}I$$

where:

$R_v$	=	volumetric runoff coefficient
$R_{vU}$	=	runoff coefficient for undisturbed wooded, meadow or agricultural cover

## Section 4.13 - Water Balance Calculations

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U	=	fraction of site in wooded cover
$R_{VD}$	=	runoff coefficient for turf cover or disturbed soils
D	=	fraction of site in turf cover or disturbed soils
$R_{VI}$	=	runoff coefficient for impervious cover
I	=	fraction of site in impervious cover

The values of the runoff coefficients to be used in the calculation of  $R_v$  are presented in Table 4-13.

**Table 4-13 Volumetric Runoff Coefficients by Land Use and Hydrologic Soil Group**

Land Use	Hydrologic Soil Group			
	A	B	C	D
Undisturbed Woods, Meadow or Ag. Land ( $R_{VU}$ )	0.02	0.03	0.04	0.05
Turf or Disturbed Soils ( $R_{VD}$ )	0.15	0.20	0.22	0.25
Impervious Cover ( $R_{VI}$ )	0.95	0.95	0.95	0.95

An example of the calculation of  $R_v$  and  $WQ_v$  is presented below.

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### Example Problem 1 – Calculating the $WQ_v$ for a New Development

Compute the water quality protection volume for a 20 acre undisturbed meadow on HSG “C” soils that is converted to a development containing:

- 4 acres undisturbed natural area not held in reserve as a “reduction”;
- 10 acres disturbed pervious area;
- 6 acres impervious area.

First look up the appropriate  $R_v$  coefficients from Table 4-13.

$$\begin{aligned}R_{VU} &= 0.04 \\R_{VD} &= 0.22 \\R_{VI} &= 0.95\end{aligned}$$

Then calculate the overall volumetric runoff coefficient per Equation 4-26.

$$\begin{aligned}R_v &= R_{VU}U + R_{VD}D + R_{VI}I \\R_v &= 0.04\left(\frac{4}{20}\right) + 0.22\left(\frac{10}{20}\right) + 0.95\left(\frac{6}{20}\right) \\R_v &= 0.008 + 0.110 + 0.285 \\R_v &= 0.403\end{aligned}$$

Finally, calculate the water quality protection volume per Equation 4-24.

$$WQ_v = \frac{PR_v A}{12}$$

$$WQ_v = \frac{1.2(0.403)20}{12}$$

$$WQ_v = 0.81 \text{ acre - feet}$$

### Example Problem 2 – Calculating the WQv for a Redevelopment

Compute the water quality protection volume for a 40-acre site on HSG “C” soils. The site as it currently exists has:

- 18 acres of impervious area comprised of office buildings and parking;
- 2 acres of disturbed, pervious green space, consisting of lawn and a few trees; and,
- an adjacent 20 acres of previously undisturbed pervious green space consisting of meadow, light forest and associated bushy undergrowth.

The proposed plan is to expand the office space by using the existing buildings and adding new buildings, parking and associated green spaces. The proposed plan is described below.

- The 18-acre existing site will be “refinished”, updating the interior and exterior of the old buildings and resurfacing the parking area. The impervious area of this 18 acres will remain intact,
- The 2-acres of previously undisturbed green space on the existing site will become an impervious parking area.
- The adjacent, previously undisturbed 20 acres will be developed to contain:
  - 10 acres of disturbed pervious green space which will consist of picnic tables, a pervious walking trail, lawn and trees; and,
  - 10 acres impervious area, consisting of an additional office building and associated parking.

#### Solution:

The 20-acre area that has existing development is considered a redevelopment site. First determine which one of the six Stormwater Quality Standards for Redevelopment (from Volume 1, Section 3.2.1 of this manual) will be used. For this example, it is assumed that the developer desires to use standard #2:

*Stormwater runoff from at least thirty percent (30%) of the site's existing impervious cover and for one-hundred percent (100%) of the impervious cover for any newly disturbed area must be treated for water quality prior in accordance with the standards and criteria presented in the manual.*

Using this standard, water quality treatment must be applied to 30% of the 18 acre existing impervious area and 100% of the new 2-acre impervious (parking) area.

The  $R_v$  coefficient for impervious area from Table 4-13, is:

$$R_{VI} = 0.95.$$

## Section 4.13 - Water Balance Calculations

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Calculate the water quality protection volume for 30% of the 18 acre existing development using Equation 4-24.

$$WQ_{v1} = \frac{PR_v(0.3A)}{12}$$
$$WQ_{v1} = \frac{1.2(0.95)(0.3*18)}{12}$$
$$WQ_{v1} = 0.51 \text{ acre - feet}$$

Calculate the water quality protection volume for 100% of the new 2 acre parking area, again using Equation 4-24.

$$WQ_{v2} = \frac{PR_v A}{12}$$
$$WQ_{v2} = \frac{1.2(0.95)(2)}{12}$$
$$WQ_{v2} = 0.19 \text{ acre - feet}$$

Now, the water quality protection volume must be calculated for the 20-acres of previously undeveloped/undisturbed area. This area is considered new development.

First, look up the appropriate  $R_v$  coefficients for this newly developing area from Table 4-13.

$$R_{vD} = 0.22$$
$$R_{vI} = 0.95$$

Then calculate the overall volumetric runoff coefficient for this area using Equation 4-26.

$$R_v = R_{vU}U + R_{vD}D + R_{vI}I$$
$$R_v = 0.04\left(\frac{0}{20}\right) + 0.22\left(\frac{10}{20}\right) + 0.95\left(\frac{10}{20}\right)$$
$$R_v = 0 + 0.11 + 0.48$$
$$R_v = 0.58$$

Now, calculate the water quality protection volume for this area using Equation 4-24.

$$WQ_{v3} = \frac{PR_v A}{12}$$
$$WQ_{v3} = \frac{1.2(0.58)20}{12}$$
$$WQ_{v3} = 1.16 \text{ acre - feet}$$

Finally, calculate the overall water quality protection volume for the entire 40 acre area by adding the WQ<sub>v</sub>'s calculated for the existing redeveloped and the newly developed areas.

$$WQ_v = WQ_{v1} + WQ_{v2} + WQ_{v3}$$

$$WQ_v = 0.51 + 0.19 + 1.16$$

$$WQ_v = 1.86 \text{ acre-feet}$$

#### 4.13.2 Water Quality Peak Flow Calculation

When designing off-line stormwater management facilities, the peak discharge of the water quality storm ( $Q_{wq}$ ) can be estimated using the method described in this section. The peak rate of discharge for the water quality design storm is needed for hydraulic sizing of off-line treatment facilities, such as for sand filters and infiltration trenches, and their respective hydraulic diversion structures. Conventional SCS methods have been found to be unsatisfactory for estimating 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 by-passes the treatment practice due to an inadequately sized diversion structure, as well as lead to the design of undersized bypass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the Water Quality Protection Volume and the previously described NRCS simplified peak flow estimating method.

**Step 1** Using  $Q_{wv}$  and Equation 4-3, compute a corresponding Curve Number (CN) utilizing the following equation:

$$CN = \frac{1000}{\left[ 10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25Q_{wv}P)^{1/2} \right]}$$

where:

$P$  = rainfall, in inches (use 1.2 inches for the Water Quality Storm)  
 $Q_{wv}$  = Water Quality Protection Volume, in inches (1.2R<sub>v</sub>)

**Step 2** Compute S from the Curve Number

**Step 3** Compute  $I_a$  from S

**Step 4** Compute  $I_a/P$

**Step 5** Read the unit peak discharge ( $q_u$ ) from Figure 4-6 for appropriate  $t_c$  and  $I_a/P$

**Step 6** Compute the peak discharge ( $Q_{wq}$ ) using  $q_u$ , drainage area and  $Q_{wv}$  using Equation 4-18

$$Q_{wq} = q_u * A * Q_{wv} * F_p$$

where:

$Q_{wq}$  = the water quality peak discharge (cfs)  
 $q_u$  = the unit peak discharge (cfs/mi<sup>2</sup>/inch)  
 $A$  = drainage area (mi<sup>2</sup>)  
 $Q_{wv}$  = Water Quality Protection Volume, in inches (1.2R<sub>v</sub>)  
 $F_p$  = pond and swamp adjustment factor

### Water Quality Peak Flow Example Problem

Using the data and information from the example problem 1 in Section 4.13.1, calculate the water quality peak flow for a site having a time of concentration is 0.35 hours.

- Compute runoff volume in inches ( $Q_{wv}$ ) using Equation 4-26, where  $P = 1.2$  inches:

$$Q_{wv} = 1.2 * R_v = 1.2 * 0.403 = 0.48 \text{ inches}$$

- Compute curve number using equation 4-3:

$$CN = \frac{1000}{\left[ 10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25Q_{wv}P)^{1/2} \right]}$$

$$CN = \frac{1000}{\left[ 10 + (5 * 1.2) + (10 * 0.48) - 10(0.48^2 + 1.25 * 0.48 * 1.2)^{1/2} \right]} = 90.5$$

- Compute S:

$$S = \frac{1000}{CN} - 10 = \frac{1000}{90.5} - 10 = 1.05 \text{ in}$$

- Compute  $I_a$ :

$$I_a = 0.2S = 0.2(1.05) = 0.21 \text{ in}$$

- Compute  $I_a/P$ :

$$\frac{I_a}{P} = \frac{0.21}{1.2} = 0.175$$

- Find  $q_u$  from Figure 4-6 for  $I_a/P = 0.175$  and  $t_c = 0.35$  hours:

$$q_u = 600 \text{ cfs}/\text{mi}^2/\text{in}$$

- Compute water quality peak flow using equation 4-18:

$$Q_{wq} = q_u * A * Q_{wv} = 600 * \frac{20}{640} * 0.48 = 9.0 \text{ cfs}$$


---

#### 4.13.3 Water Quality Volume Extended Detention

The WQ<sub>v</sub> for a site must be detained for 24 hours to achieve the design TSS removal efficiencies for extended detention treatment. After the 24 hour detention period, this volume must be released over a reasonable period of time (e.g., 2 to 4 days) to return the pond to a dry condition (or its static pool elevation if a wet pond). Because the maximum pollutant load is carried by the first flush from the initial storm event, these drainage criteria do not apply to successive rain events within a 4-day period.

The WQ<sub>v</sub> for the entire site may be detained in a single detention pond at the downstream portion of the site, or the runoff may be detained in several facilities located within the development, including "treatment trains" as discussed in Volume 3, Chapter 3 of this Manual. The primary requirement is that all areas of the site (unless it is reserved as a "reduction" area per Volume 2, Chapter 2) must receive the required extended detention of their respective WQ<sub>v</sub>. Please refer to Chapter 2 of Volume 2 for more details on this requirement.

Normally, a hydrograph for the water quality event would be routed through the treatment facilities, and the detention time could be determined as the time lapse between the centroid of the inflow hydrograph and the centroid of the outflow hydrograph. However, for the small events associated with the WQ<sub>v</sub> rainfall, this is not feasible. Instead, a simplified method is used to set controls (usually an orifice) in the facility to achieve the required detention time, as described below.

1. Develop a volume versus water surface elevation curve or table for the storage facility. (In this discussion, the term pond will be used for storage facilities. However, the procedure applies as well to wetlands and other storage features that may be used for extended detention (ED) for WQ<sub>v</sub> treatment.)
2. Using the computed WQ<sub>v</sub>, determine the elevation of the pool with the WQ<sub>v</sub> added. (For a dry extended detention pond, this would be the elevation in the pond if it contains only the WQ<sub>v</sub>. For a wet extended detention pond where  $\frac{1}{2}$  of the WQ<sub>v</sub> lies above the permanent pool, this would be the elevation of the pond if it contains the permanent pool plus  $\frac{1}{2}$  of the WQ<sub>v</sub>.)
3. Select a trial orifice size to control the drain time for the WQ<sub>v</sub> held in temporary storage (full WQ<sub>v</sub> for dry ED pond,  $\frac{1}{2}$  WQ<sub>v</sub> for wet ED pond). The invert of the orifice will be located at the pond bottom for a dry ED pond, or at the surface of the permanent pool for a wet ED pond.
4. Using the hydraulic characteristics of the orifice and the volume-elevation curve for the pond, compute the time for approximately 90% of the WQ<sub>v</sub> in temporary storage to be in the pond at the 24<sup>th</sup> hour and for the drainage facility to discharge the WQ<sub>v</sub> with a reasonable period of time afterward (e.g., 2 to 4 days) (see Volume 2, Chapter 5 for guidance on analyzing orifices). The reason 90% is used is that the simplified drawdown method for sizing orifices tends to over-estimate the orifice size and thus under-predict actual ED time. The additional 10% of drawdown after the end of the 24-hour period is intended to help compensate for that effect.

Repeat steps 3 and 4 until a satisfactory orifice size has been selected.

## 4.14 Calculation of %TSS Removal

As previously stated, the site design engineer must utilize one or more stormwater treatment facilities that achieve (or exceed) the 80% TSS removal standard. The % TSS removal is an area-weighted value for the site (excluding  $WQ_V$  reductions), based upon the % TSS removal value of the structural facility(s) located on-site and the drainage area treated by each facility. Table 4-14 provides the % TSS Removal value for each structural stormwater treatment facility that is presented in this Manual.

**Table 4-14 % TSS Removal Values for Structural Stormwater Treatment Facilities**

Structural Facility	TSS Removal %
Stormwater Pond	80
Dry Extended Detention Pond	60
Enhanced Swale	90
Grass Channel	50
Infiltration Trench	90
Soakage Trench	90
Vegetative Filter Strip	50
Surface Sand Filter	80
Organic Filter	80
Bioretention Area	85
Stormwater Wetland	75
Proprietary Manufactured Device	device-specific
Gravity Oil/Water Separator	device-specific
Alum Treatment	90
Green Roof	installation-specific

If a single treatment facility is used for a site, the calculation of the % TSS removal is straightforward – the % TSS removal for the site equals the % TSS removal value for the stormwater treatment facility that is used. However, multiple facilities are often used at a site to achieve the overall required removal rate. The following sections provide guidance on calculating the % TSS Removal for sites with multiple stormwater management facilities.

### 4.14.1 Calculation of % TSS Removal for a site (Controls in Parallel)

The percent TSS removal (%TSS) that is achieved on a site can be calculated using Equation 4-27 (parallel treatment). Any untreated area is to be assigned a null control with a TSS removal rate of 0%.

$$\text{Equation 4-27} \quad TSS_{site} = \frac{\sum_1^n (TSS_1 A_1 + TSS_2 A_2 + \dots + TSS_n A_n)}{\sum_1^n (A_1 + A_2 + \dots + A_n)}$$

where:

$TSS_n$  = TSS removal (Table 3-2) for each water quality control located on-site (%)  
 $A_n$  = the area draining to each control, excluding reduction areas (acres)

#### 4.14.2 Calculation of % TSS Removal for a site (Controls in Series)

The site designer will often want to use two or more controls (structural and/or non-structural) in series, where stormwater treated in one control is discharged directly into another control for further treatment. Where controls are used in series, the total % TSS removal for the combination of controls shall be used for  $TSS_n$  in Equation 4-27 for the area treated by the series of controls.

To calculate the total % TSS removal for two controls in series, Equation 4-28 shall be used.

$$\text{Equation 4-28} \quad TSS_{train} = TSS_A + TSS_B - \frac{(TSS_A * TSS_B)}{100}$$

where:

$TSS_{train}$  = total TSS removal for series (%)  
 $TSS_A$  = % TSS removal of the first (upstream) control (%)  
 $TSS_B$  = % TSS removal of the second (downstream) control (%)

For more than two controls in series, use the combined removal for the first two controls as  $TSS_A$ , and the removal for the third control as  $TSS_B$ . Repeat this procedure as necessary to account for all controls in the series. For development sites where the series of controls (also called a “treatment train”) provides the only stormwater treatment on the site,  $TSS_{train}$  must be greater than or equal to 80%. For development sites that have other controls for stormwater treatment that are not included in the treatment train,  $TSS_{train}$  must be included in Equation 4-27 in the calculation of the overall % TSS removal for the site. An example application of the latter situation is presented below.

---

#### Example Problem – Calculation of %TSS when Controls in Series are Used

A stormwater management system located on a 30 acre development site consists of a dry extended detention pond, a water quality swale, and a shallow wetland. The extended detention pond and swale are located in series, with the pond as the upstream control. The treatment train treats stormwater runoff from 20 acres of the site. The shallow wetland treats 10 acres. What is the % TSS removal rate for the site?

The % TSS removal value for each control located on the site is determined from Table 4-14, as follows:

**Control A** (dry extended detention pond) = 60% TSS removal

**Control B** (water quality swale) = 90% TSS removal

**Control C** (shallow wetland) = 75% TSS removal

**Step 1** Calculate  $TSS_{train}$ :

$$TSS_{train} = TSS_A + TSS_B - (TSS_A \times TSS_B)/100 = 60 + 90 - (60 \times 90)/100 = 96\% \text{ removal}$$

**Step 2** Calculate % TSS removal for the site:

$$\%TSS = ((TSS_{train} \times 20 \text{ acres}) + (%TSS_{wetland} \times 10 \text{ acres})) \div 30 \text{ acres}$$

$$\%TSS = ((96\% \times 20 \text{ acres}) + (75\% \times 10 \text{ acres})) \div 30 \text{ acres} = 89\%$$

Therefore, the % TSS removal for the site is 89%, which exceeds the minimum standard of 80% TSS removal. No other water quality controls are required.

---

#### 4.14.3 Calculation of % TSS Removal for Flow-through Situations

Water quality controls within a treatment train may sometimes be separated by a contributing drainage area. (That is, runoff requiring treatment may enter the train between two other controls located in series.) In this case, Equation 4-28 cannot be used, since some of the flow entering the downstream control has not been treated by the upstream control. This section presents the calculation of the total % TSS removal for flow-through situations.

To calculate the total % TSS removal for a treatment train separated by a contributing drainage area, Equation 4-29 shall be used (Knox County TN, 2008).

$$\text{Equation 4-29} \quad TSS_{train} = \frac{TSS_A A_A + TSS_B A_B + \frac{TSS_B A_A (100 - TSS_A)}{100}}{A_A + A_B}$$

where:

$TSS_{train}$ =	total TSS removal for treatment train (%)
$TSS_A$ =	% TSS removal of the first (upstream) control (%)
$TSS_B$ =	% TSS removal of the second (downstream) control (%)
$A_A$ =	Area draining to control A
$A_B$ =	Area draining to control B

For development sites where the treatment train provides the only stormwater treatment on the site,  $TSS_{train}$  must be greater than or equal to 80%. If other areas of the site are treated by other controls, then the removal rate computed by Equation 4-29 is used in Equation 4-27 for its treatment area. An example application of Equation 4-29 is shown below.

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#### Example Problem – Calculation of %TSS in a Flow-through Situation

A stormwater management system located on a 9 acre development site consists of a dry extended detention pond, and a bioretention cell. Five acres drain to the bioretention cell, which then drains to a pipe system. The pipe system also drains an additional 4 acres that have not been treated for water

quality. The pipe system leads to a dry extended detention pond, that is used for final treatment. What is the % TSS removal rate for the site?

The % TSS removal value for each control located on the site is determined from Table 4-14, as follows:

Control A (bioretention cell) = 85% TSS removal

Control B (dry extended detention pond) = 60% TSS removal

Calculate TSS<sub>train</sub>:

$$TSS_{strain} = \frac{TSS_A A_A + TSS_B A_B + \frac{TSS_B A_A (100 - TSS_A)}{100}}{A_A + A_B}$$

$$TSS_{strain} = \frac{85 * 5 + 60 * 4 + \frac{60 * 5 (100 - 85)}{100}}{5 + 4}$$

$$TSS_{strain} = 79\%$$

The % TSS removal for the site is 79%, which is slightly below the minimum standard of 80% TSS removal. The conversion of the stormwater pipe system to a grass swale would add additional pollutant removal and help the site meet the 80% criteria.

#### 4.14.4 Application of WQ<sub>v</sub> Reductions

The WQ<sub>v</sub> reductions discussed in Chapter 2 are primarily intended to reduce treatment volume requirements by excluding those areas from the WQ<sub>v</sub> calculations. However, reductions 1 through 4 also serve as water quality controls with TSS removal benefits. These benefits should be included in the TSS removal calculation if used in the project. The TSS removal efficiencies are provided in their respective reduction descriptions in Chapter 2. The TSS efficiencies should be included in the TSS calculations if used in the project.

It is important to note that site WQ<sub>v</sub> and site TSS removal computations, although related, are performed independently. For example, the area draining to a qualifying stream buffer (see Chapter 2) is subtracted from the site area for WQ<sub>v</sub> calculation purposes, but is still included in the site TSS removal calculation. However, the area that drains to the buffer receives 80% TSS removal in the site TSS calculation.

### 4.15 Channel Protection Volume

#### 4.15.1 Description

Although the runoff associated with large, relatively infrequent storms can induce significant erosion in streams, the long-term channel forming storms are associated with smaller, more frequent events. Specifically, it has been shown that the 1 to 2-year storm events create the predominant channel forming runoff that causes long-term sustained channel erosion. These

flows are commonly referred to as the “bankfull” flows that just fill the stream channel without significantly overflowing to the floodplain. (Note that the term “bankfull” would not apply to streams that are already deeply incised or enlarged from previous erosion or to engineered channels with capacity larger than the 1 to 2-year runoff.)

In order to provide a degree of protection from long-term erosion associated with these frequent channel forming events, the Channel Protection Volume ( $CP_v$ ) concept is applied. This criteria requires (with certain exceptions) that the runoff from the project for the 24-hour, 1-year storm event (defined as the  $CP_v$ ) be detained for a minimum of 24 hours. The  $CP_v$  can then be drained within a reasonable amount of time (e.g., 2 to 4 days).

The  $CP_v$  for the entire site may be detained in a single detention pond at the downstream portion of the site, or the runoff may be detained in several storage facilities located within the development, as discussed in Chapter 3 of this Manual. The primary requirement is that all areas of the site, including any  $WQ_v$  reduction areas, must receive the required extended detention of their respective  $CP_v$ .

If  $CP_v$  control is being provided by a single facility located at or near the project boundary (for example, an extended detention pond), or for facilities located in parallel, the design is relatively straightforward. The individual controls may be designed by applying the “centroid” method (see Section 4.15.2) to the facility inflow and outflow hydrographs, or by using the simplified methods for facilities that are not readily modeled by routing procedures.

Since  $CP_v$  controls are detention facilities, they affect the rate and duration of flows downstream. Therefore, the effect of  $CP_v$  controls placed in series is hydrologically complex since the upstream controls affect the downstream controls. The use of  $CP_v$  facilities in series is generally inefficient and should be avoided if possible.

If  $CP_v$  controls are placed in series, only the local inflow between the upstream and downstream controls is considered to be regulated by the downstream control. In addition, the designer must demonstrate that the outflow from the upstream control does not adversely affect the function of the downstream control.

#### **4.15.2 Channel Protection Volume Extended Detention – Centroid Method**

For ponds, wetlands, or other storage facilities used for extended detention (ED) of the  $CP_v$  where routing procedures are practicable, the following procedure applies:

1. Develop a 24-hour, 1-year runoff storm event hydrograph for the catchment area draining to the storage facility using the previously described methods. This is the inflow hydrograph for the ED facility.
2. Select a trial hydraulic control (usually an orifice or weir) to control the  $CP_v$  detention time for the facility. The invert of the control is located at the top of the  $WQ_v$  pool for facilities that also treat the  $WQ_v$ .

3. Using the routing procedure previously described for ponds and reservoirs (i.e., flat pool), route the inflow hydrograph through the storage facility and thus determine the outflow hydrograph.
4. Compute the centroids of the inflow and outflow hydrographs. The time lapse between the two centroids is the extended detention time.
5. Repeat steps 3 and 4 until the ED time is at least 24 hours and the  $CP_v$  will drain within a reasonable time period (e.g., 2 to 4 days).

An example calculation of the ED time using the Centroid Method is provided below.

### Example Problem – $CP_v$ Outlet Sizing Using the Centroid Method

An ED pond is being designed for a 5 acre development site. Design the ED pond orifice using the centroid method to meet the  $CP_v$  requirements. Relevant information is below:

Catchment area: 5 acres

24-hour, 1-year rainfall depth: 2.9 inches (from Table 4-1)

Use a NRCS Type II Rainfall Distribution

Pre-development curve number = 80 (from Table 4-1)

Post-development curve number = 90

Antecedent Moisture Condition II

Standard Peaking Factor 484

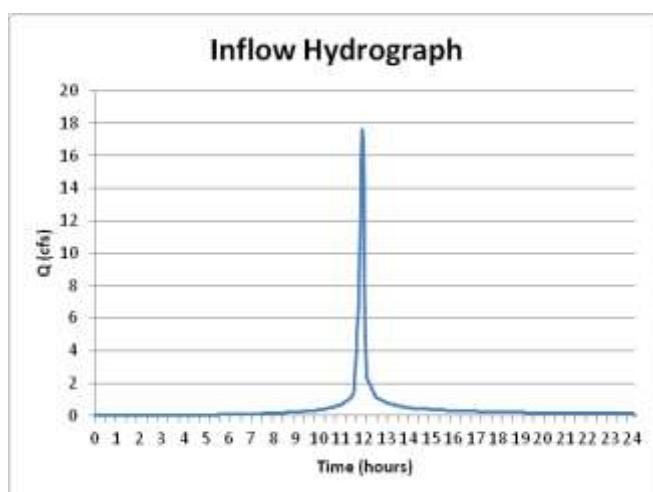
Travel time for sheet flow =  $T_{T_{sheet}} = 1.16$  minutes

Travel time for shallow concentrated flow =  $T_{T_{shallow}} = 0.72$  minutes

Travel time for channel flow =  $T_{T_{channel}} = 1.84$  minutes

**Step 1** Develop the inflow hydrograph for the 1-year, 24-hour storm event that will discharge to the ED pond from the 5 acre site.

For the purpose of this example, the NRCS synthetic unit hydrograph was used to determine the 24-hour, 1-year runoff storm event hydrograph.



**Step 2** Select a trial hydraulic control (usually an orifice or weir) to control the CP<sub>v</sub> detention time for the facility. The invert of the control is located at the top of the WQ<sub>v</sub> pool for facilities that will also treat the WQ<sub>v</sub>.

The selected hydraulic control for this example is an orifice to be installed through the chosen outlet structure with the invert at the bottom of the facility. The initial orifice diameter chosen is 3 inches.

**Step 3** Using the routing procedure previously described for ponds and reservoirs (i.e., flat pool), route the inflow hydrograph through the storage facility and thus determine the outflow hydrograph.

The detention facility is a dry pond with bottom dimensions of 30' x 90' with 3:1 side slopes.

Using the Average-End Area Method, the Stage-Storage Relationship for the pond is as follows:

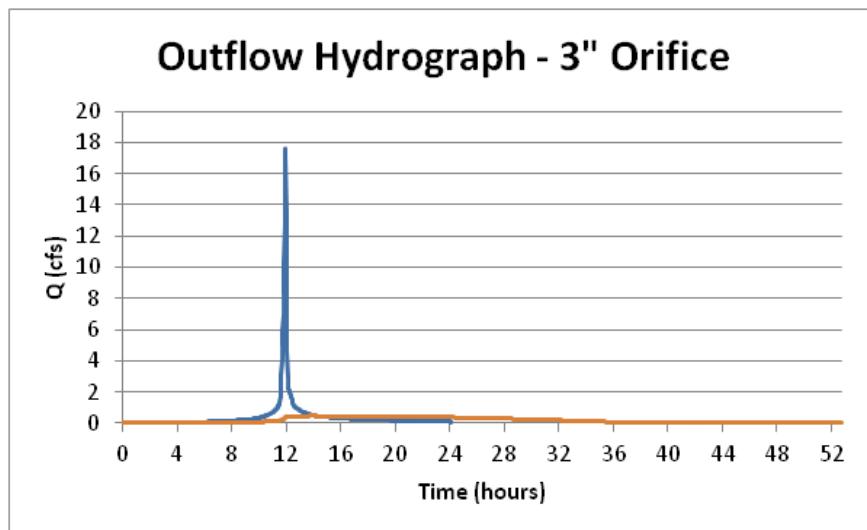
Stage	Elevation	Contour Area	Incremental Storage	Total Storage
0	100	2700	0	0
1	101	3456	3070	3070
2	102	4284	3862	6932
3	103	5184	4726	11659
4	104	6156	5662	17321
5	105	7200	6671	23992
6	106	8316	7751	31742
7	107	9504	8903	40645
8	108	10764	10126	50771

Then, using Equation 5-36 and the information given below, compute the Stage-Discharge Relationship.

Given: 3 inch diameter orifice (0.25 ft)  
 Discharge coefficient (C) = 0.80  
 $A = 0.05 \text{ ft}^2$   
 $G = 32.2 \text{ ft/s}^2$   
 Diameter (D) = 0.25 ft  
 H = varies with stage

Stage	Elevation	Discharge
0	100	0.00
1	101	0.32
2	102	0.45
3	103	0.56
4	104	0.64
5	105	0.72
6	106	0.79
7	107	0.85
8	108	0.91

The outflow hydrograph is shown below.



**Step 4** Compute the centroids of the inflow and outflow hydrographs. The time lapse between the two centroids is the extended detention (ED) time.

The centroid of a hydrograph ( $T_m$ ) is computed using the equation given below. For each point on the inflow and outflow hydrographs, multiply the flow value by the time step value between the flows.

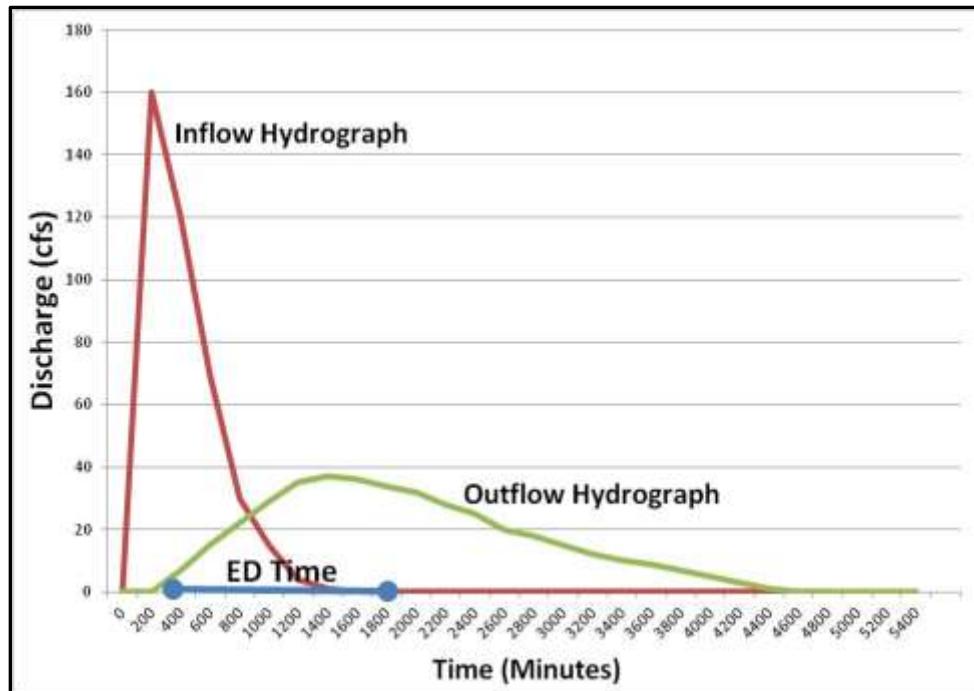
$$T_m = \frac{\sum_0^t qt}{\sum_0^t q}$$

Using this equation, the centroid of the Inflow Hydrograph ( $T_{mi}$ ) = 13 hours  
 The centroid of the Outflow Hydrograph ( $T_{mo}$ ) = 21.5 hours

The ED time of the pond (ED) =  $T_{mo} - T_{mi}$  = 21.5 hrs – 13 hrs = 8.5 hours.

The ED time of the pond must be at least 24 hours. Because ED for the 3 inch orifice is only 8.5 hours, the 3 inch orifice is too large.

**Step 5** Repeat steps 3 and 4 until the ED time is at least 24 hours, but not more than 48 hours. If we repeat steps 3 and 4 using a 1.75 inch orifice, the ED time is 26 hours. Therefore a 1.75 inch orifice is suitable for this pond. An example of inflow and outflow hydrographs with a suitable ED time is shown graphically below.



#### 4.15.3 Channel Protection Volume Extended Detention – Simplified Method

For some  $CP_v$  facilities, routing procedures may not be practicable. For example, some swales, pocket wetlands, and bioretention areas do not lend themselves to routing. For those facilities, a simplified method may be used.

The simplified method consists of determining the volume of runoff using the CN method, estimating the peak runoff rate using the previously described NRCS simplified peak flow method, and then estimating the volume of storage required using procedures derived from the NRCS TR-55 detention pond storage estimating method. Finally, the ED control (orifice or weir) is sized.

1. Using the simplified peak flow procedures described in Section 4.8, estimate the peak inflow ( $q_i$ ) to the storage facility.
2. Using the unit peak discharge ( $q_u$ ) determined in Step 1, determine the ratio of peak outflow to peak inflow ( $q_o/q_i$ ) using the curve labelled “T = 24 hr” in Figure 4-17.
3. Using the ratio  $q_o/q_i$ , determine the ratio of the storage volume to runoff volume ( $V_s/V_r$ ) using the curve labelled “Types II and III” in Figure 4-18.
4. Using the ratio  $V_s/V_r$ , and the runoff volume for the 1-year, 24-hour storm ( $V_r$ ), compute the storage volume ( $V_s$ ). This is equivalent to the  $CP_v$ .
5. Size the outflow hydraulic controls based on peak outflow  $q_o$  and peak head (i.e. when pool contains  $CP_v$ ).

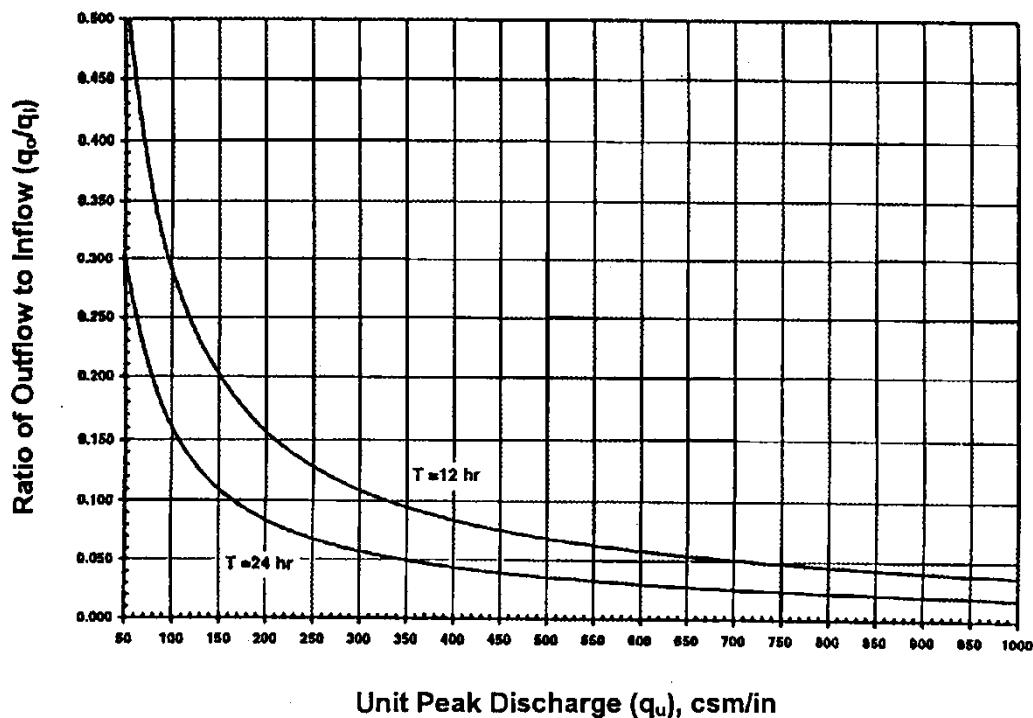


Figure 4-17 Detention Time vs Discharge Ratios

(Source: MDE, 1998)

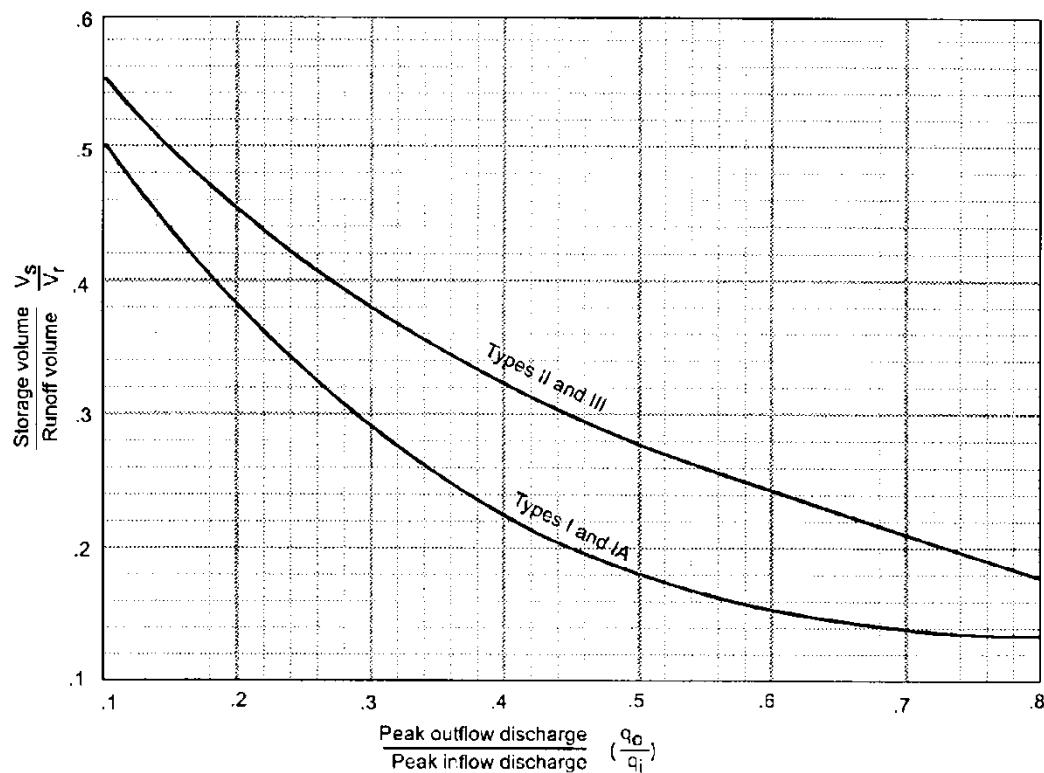


Figure 4-18 Approximate Detention Basin Routing for Rainfall Types I, IA, II and III

(Source: TR-55, 1986)

### Example Problem - Simplified Channel Protection Volume ED

Compute the Channel Protection Volume ( $CP_v$ ) for the following catchment.

#### Computations

Calculate rainfall excess for the 1-year storm using equation 4-1:

P	=	2.9 inches (the 1-year, 24-hour rainfall)
CN	=	81.8 (from previous calculation)
S	=	2.2 inches
$I_a$	=	0.439 (from previous calculation)
$T_c$	=	11.6 minutes
A	=	28 acres

$$Q = \frac{(P - I_a)^2}{[(P - I_a) + S]} = \frac{(2.9 - 0.439)^2}{[(2.9 - 0.439) + 2.2]} = 1.3 \text{ in}$$

Calculate  $I_a/P$

$$I_a/P = 0.439/2.9 = 0.15$$

Find unit discharge  $q_u$  from Figure 4-6 for  $I_a/P = 0.15$  and  $T_c = 11.6$  min (0.19 hours)

$$q_u = 780 \text{ cfs/mi}^2\text{-in}$$

Find discharge ratio  $q_o/q_i$  from Figure 4-17 for  $q_u = 780 \text{ csm/in}$  and  $T = 24 \text{ hr}$

$$q_o/q_i = 0.02$$

From Figure 4-18, using  $q_o/q_i = 0.02$  determine the ratio  $V_s/V_r$  using the Type II curve. Note that the lower limit of the curve is  $q_o/q_i = 0.1$ , which is greater than the computed value. Although the accuracy of the estimate is diminished by extrapolating the curve, it is customary to do so for the purposes of analyzing CPv requirements. The extrapolation may be made using the following equation: (SCS, 1986)

$$\text{Equation 4-30} \quad \frac{V_s}{V_r} = 0.682 - 1.43 \left( \frac{q_o}{q_i} \right) + 1.64 \left( \frac{q_o}{q_i} \right)^2 - 0.804 \left( \frac{q_o}{q_i} \right)^3$$

where:

$V_s$	=	required storage volume (acre-feet)
$V_r$	=	runoff volume (acre-feet)
$q_o$	=	peak outflow discharge (cfs)
$q_i$	=	peak inflow discharge (cfs)

Using this equation,  $V_s/V_r = 0.65$ .

Therefore,  $CP_v$  for  $Q = 1.3$  inches is

$$V_s = \frac{\left(\frac{V_s}{V_r}\right) Q A}{12} = \frac{(0.65)(1.3)(28)}{12} = 1.97 \text{ acre-feet}$$

The simplified  $CP_v$  detention control sizing procedure is as follows:

- The peak discharge from the pond is computed using the method described in Section 4.8.4 and the previously computed value of  $q_o/q_i$  (i.e., ratio of peak pond outflow to inflow).
- Wet Ponds with Permanent  $WQ_v$  Pools: For ponds where 100% of the  $WQ_v$  is in a permanent pool, the  $CP_v$  orifice is placed at the top of the permanent pool. The top of the  $CP_v$  pool is determined based on the storage versus elevation curve for the pond and the combined  $CP_v$  and  $WQ_v$ . The maximum head on the  $CP_v$  orifice is computed as the difference between the  $CP_v$  pool elevation and the elevation of the center of the  $CP_v$  orifice. The orifice size is then determined based on the orifice equation, given the peak discharge from the pond and the maximum head.
- Extended Detention Wet Pond and Extended Detention Micropool Pond: For wet extended detention ponds, the  $CP_v$  orifice is usually located at the top of the permanent part of the  $WQ_v$  pool. As with the wet pond, the preferred method is to rout the 1-year, 24-hour inflow hydrograph through the pond, and select the orifice size that provides 24 hours of detention between the centroids of the inflow and outflow hydrographs. However, in some cases, the simplified method may be used. The simplified method is strictly applicable to the situation where the active pool (in this case the  $CP_v$  pool and the extended detention portion of the  $WQ_v$  pool) is controlled by one or more orifices at the same elevation. However it is rare that the  $WQ_v$  orifice alone can serve to regulate both the  $WQ_v$  and  $CP_v$ . In most cases a  $CP_v$  orifice is also required. If the centers of the  $WQ_v$  and  $CP_v$  orifices are “approximately” the same then the simplified method is applicable. Until further information is available, the elevation of the centers of the orifices will be considered “approximately” the same if the difference between the center elevations does not exceed 20% of the height of the active pool. The procedure is basically the same as for wet ponds except the maximum head at peak discharge is computed for both the  $WQ_v$  and  $CP_v$  orifices, and the  $CP_v$  orifice is sized such that the total discharge through both orifices is equal to the pond peak discharge.
- Dry Extended Detention Pond: The procedure is the same as for the Wet Extended Detention Pond.

#### 4.16 Backwater Conditions

The control outlets for all stormwater management facilities are vulnerable to being inundated by backwater from the downstream conveyance. Stormwater ponds and wetlands located in floodplains and backwater streams are particularly susceptible. Such facilities must perform

as specified for peak flow protection for any tailwater condition, up to and including the floodplain of the 100-year flood elevation. The potential for backflow into the facility must be addressed with flap gates or by providing sufficient volume to receive the backflow up to the 100-year flood elevation, and still provide peak flow control surcharge volume in the pond above the 100-year flood elevation. A joint probability analysis will not be accepted as justification to waive this requirement.

Due to the complexity of backwater conditions, it is often useful to use a dynamic model. EPA-SWMM is capable of analyzing these types of conditions, including the application of flap-gates and backflow.

## 4.17 Water Balance Calculations

### 4.17.1 Introduction

Water balance calculations can help determine if a drainage area is large enough, or has the right characteristics, to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation is advisable for a storage facility with a permanent pool that is solely dependent on rainfall and runoff to sustain the pool. This is particularly important for wetland and stormwater pond design.

The details of a rigorous water balance are beyond the scope of this Manual. However, a simplified procedure is described herein to provide an estimate of the feasibility of a permanent pool and an indication as to whether a more rigorous water balance analysis is needed. (Water balance can also be used to help establish planting zones in a wetland design.)

### 4.17.2 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow, as shown in Equation 4-31.

$$\text{Equation 4-31} \quad \Delta V = \Sigma I - \Sigma O$$

where:

$\Delta$	=	"change in"
$V$	=	pond volume (ac-ft)
$\Sigma$	=	"sum of"
$I$	=	Inflows (ac-ft)
$O$	=	Outflows (ac-ft)

The inflows consist of rainfall, runoff, and baseflow into the pond. The outflows consist of infiltration (seepage losses), evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 4-32 reflects these factors.

$$\text{Equation 4-32} \quad \Delta V = P + R_o + B_f - I - E - E_t - O_f$$

where:

V	=	pool volume (ac-ft)
P	=	precipitation (ac-ft) = rainfall in inches times pool area in acres divided by 12
R <sub>o</sub>	=	runoff into pond from its catchment drainage area (ac-ft)
B <sub>f</sub>	=	baseflow into pond (ac-ft)
I	=	pool bottom and sides infiltration losses (ac-ft); use Equation 4-34
E	=	pool evaporation (ac-ft); surface evaporation in feet times pool surface area
E <sub>t</sub>	=	plant evapotranspiration (ac-ft)
O <sub>f</sub>	=	overflow (ac-ft)
Δ	=	“change in” (+ gain; - loss)

Rainfall (P): Mean monthly values of rainfall depth are commonly used for simplified water balance calculations. Rainfall depth is the direct amount that falls on the pond surface for the month. When multiplied by the pool surface area (in acres) and divided by 12, the volume of P, in acre-feet, is determined. Table 4-15 shows mean monthly rainfall for Sedgwick County as a percent of mean annual precipitation. Rainfall values for the site may be determined by multiplying these percentages by the mean annual depth at the site as provided in Figure 4-3.

**Table 4-15 Monthly Precipitation Values as % of Average Annual Precipitation**

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation (% of Annual)	2.8	3.4	8.9	8.4	13.7	14.0	10.9	9.7	9.7	8.1	6.0	4.4

Source: National Oceanic and Atmospheric Administration, “Comparative Climatic Data”, 2006

Runoff (R<sub>o</sub>): Runoff is equivalent to the rainfall for the period times the “efficiency” of the watershed, which is equal to the ratio of runoff to rainfall (Q/P). In lieu of gage information, Q/P can be estimated one of several ways.

The best method would be to perform long-term simulation modeling using rainfall records and a watershed model. However, this procedure is seldom justified for a small stormwater management facility and is not described herein.

Alternatively, Equation 4-33 calculates runoff based on the volumetric coefficient (R<sub>v</sub>) of the drainage area. Not all monthly rainfall produces runoff. Thus a factor of 0.9 is applied to the calculated R<sub>v</sub> value to account for monthly rainfall producing no runoff. Equation 4-33 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$\text{Equation 4-33} \quad R_O = 0.9 \left( \frac{P}{12} \right) R_v A$$

where:

R <sub>o</sub>	=	runoff volume (acre-ft)
P	=	precipitation (inches)
R <sub>v</sub>	=	volumetric runoff coefficient (see Equation 4-26)
A	=	area (acres)

**Baseflow (B<sub>f</sub>):** Most stormwater ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so, baseflow must be estimated from observation or through theoretical estimates. Methods of estimating baseflow can be found in most hydrology textbooks. Consideration may also have to be given to irrigation return flows.

**Infiltration (I):** Infiltration (seepage loss from the pond) is a complex subject and is not covered in detail in this Manual. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. For ponds with permeable bottoms and sides, the infiltration rate is governed by the Darcy equation (Equation 4-34) as:

$$\text{Equation 4-34} \quad I = Ak_h G_h$$

where:

I	=	infiltration (ac-ft/day)
A	=	cross sectional area through which the water infiltrates (ac)
$k_h$	=	saturated hydraulic conductivity or infiltration rate (ft/day)
$G_h$	=	hydraulic gradient = pressure head/distance

For approximate analysis,  $G_h$  can be set equal to 1.0 for pond bottoms and sides no steeper than 4:1, and 0.5 for pond sides steeper than 4:1. Infiltration rate can be established through testing. An approximate rate can be obtained from Table 4-16.

**Table 4-16 Saturated Hydraulic Conductivity**

Material	Hydraulic Conductivity	
	in/hr	ft/day
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08
Clay	0.02	0.04

Source: Ferguson and Debo, "On-Site Stormwater Management," 1990

**Evaporation (E):** Evaporation rates for water surfaces are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, sunlight, water

purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks.

Table 4-17 gives the estimated average free water surface evaporation rate for Sedgwick County for a typical 12-month period. This table is based on a pan coefficient of 0.7, and class A pan evaporation rates estimated by the Penman method. (NOAA Technical Report NWS 33 and 34, 1982.)

**Table 4-17 Average Monthly Evaporation (inches)**

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1.16	1.47	2.90	4.12	5.25	6.13	6.76	6.42	4.20	3.28	1.84	1.39

**Evapotranspiration ( $E_t$ ):** Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of  $E_t$  for plants in a wet pool environment are not well documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating  $E_t$  only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based  $E_t$  estimates and a decision made. Crop-based  $E_t$  estimates can be obtained from hydrology references.

**Overflow ( $O_f$ ):** Overflow is excess runoff lost from cumulative pool volumes in excess of the maximum available permanent storage. (Obviously, for continuous simulations of rainfall-runoff, overflow from large storms would play an important role in pond storage design. However, for the purposes of this simplified method, those large storms are not analyzed.)

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### Example Problem – Simplified Water Balance

A 50-acre site is being developed along with an estimated 0.5-acre surface area water quality pond. There is no baseflow. The desired permanent pool volume is 2 acre-feet. Will the site be able to support the pond volume?

From the basic site data, we determine that  $R_v = 0.72$  and the pond bottom and sides are clay-loam soil.

- With the adjustment factor of 0.9 the watershed efficiency is  $0.9 \times 0.72 = 0.648$ .
- For a clay loam the approximate infiltration rate is  $I = 0.18 \text{ ft/day}$  (Table 4-16).
- From a grading plan, none of the normal full pool pond area is sloped steeper than 4:1.
- Monthly rainfall, based on 32 inches per year, is derived from Table 4-15.
- Neglect evapotranspiration due to limited wetland plants.

## Section 4.17 - Approved Hydrology Models

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The table to follow shows summary calculations for this site for each month of the year, beginning in January.

**Table 4-18 Data for Water Balance Example**

Drainage Area (acres)	50
Pond Surface (acres)	0.5
Volume at Overflow (ac-ft)	2
Watershed Efficiency	0.648
Infiltration Rate (ft/day)	0.18 Clay Loam
% Pond Bottom and Slopes 4:1 or Flatter	100
% Pond Slopes 4:1 or Steeper	0

**Table 4-19 Water Balance Calculation**

1	Months of Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2	Days Per Month	31	28	31	30	31	30	31	31	30	31	30	31
3	Monthly Precipitation (in)	0.90	1.09	2.85	2.69	4.38	4.48	3.49	3.10	3.10	2.59	1.92	1.41
4	Monthly Evaporation (in)	1.16	1.47	2.90	4.12	5.25	6.13	6.76	6.42	4.20	3.28	1.84	1.39
5	Monthly Evapotranspiration (in)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	Runoff (ac-ft)	2.43	2.94	7.70	7.26	11.83	12.10	9.42	8.37	8.37	6.99	5.18	3.81
7	Precipitation (ac-ft)	0.04	0.05	0.12	0.11	0.18	0.19	0.15	0.13	0.13	0.11	0.08	0.06
8	Evaporation (ac-ft)	0.05	0.06	0.12	0.17	0.22	0.26	0.28	0.27	0.18	0.14	0.08	0.06
9	Infiltration (ac-ft)	2.79	2.52	2.79	2.70	2.79	2.70	2.79	2.79	2.70	2.79	2.70	2.79
10	Evapotranspiration (ac-ft)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
11	Balance (ac-ft)	-0.37	0.41	5.31	6.50	11.00	11.33	8.50	7.44	7.62	6.17	4.49	3.02
12	Running Balance (Ac-ft)	0.00	0.41	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00

### Explanation of Table Line Numbers:

1. Months of year
2. Days per month
3. Monthly precipitation
4. Monthly evaporation
5. Monthly evapotranspiration
6. In the example, watershed efficiency of 0.65 times the rainfall and area (in acres) and converted to acre-feet. The watershed efficiency must be determined for each watershed.
7. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet.
8. Evaporation equals the monthly percentage of the annual gross lake evaporation in inches converted to acre-feet, excluding ice covered months.

9. Infiltration equals infiltration rate for the bottom of the pond plus infiltration rate for the inundated pond slopes that are steeper than 4:1 (none in this example), converted to acre-feet.
10. Evapotranspiration from wetland type plants.
11. The sum of lines 6 and 7, plus the line 12 from the previous month, minus the sum of lines 8, 9 and 10.
12. Accumulated total from line 11 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design.

In practice, the balance in lines 11 and 12 should be continued for a 2 to 3 year period to adjust for the assumed starting conditions inherent in the simplified method (dry pond at the start of January). For this example, the pond has the potential to maintain its pool in all months except possibly January and February when rainfall depths are low. In this case, a liner may be required.

It should be noted that this type of analysis does not account for departures from average values. For marginal results, it is suggested that a sensitivity analysis, using conservatively low rainfall values and conservatively high infiltration values, be prepared. Excessive infiltration rates may be remedied in a number of ways including placing a clay liner or installing a synthetic liner in the pond.

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## 4.18 Approved Hydrology Models

The following public domain software are approved for use in preparing hydrology analyses. Only the approved hydrology methods discussed above shall be employed within these software packages.

HEC-1: Flood Hydrograph Package, U.S. Army Corps of Engineers

HEC-HMS: Hydrologic Modeling System, U.S. Army Corps of Engineers

EPA-SWMM: Stormwater Management Model, U.S. Environmental Protection Agency

Other public domain hydrology software must be pre-approved by the local jurisdiction.

Commercial third-party software must be pre-approved by the local jurisdiction, not only on the basis of technical acceptance but also on the basis of the cost for the local jurisdiction to maintain a license for the product for review purposes.