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HYDROLOGIC ANALYSIS

Section 2.1 Estimating Runoff

2.1.1 Introduction to Hydrologic Methods

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

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Rainfall abstraction rates (Initial and continued)
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods available to estimate runoff characteristics for a site or drainage subbasin; however, the following methods have been selected to support hydrologic site analysis for the design methods and procedures included in this Manual:

- Rational Method
- SCS Unit Hydrograph Method
- Snyder's Unit Hydrograph Method
- USGS & TXDOT Regression Equations
- iSWM Water Quality Protection Volume Calculation
- Water Balance Calculations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

Table 2.1.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 2.1.1-2 provides some limitations on the use of several methods.

In general:

The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.

The USGS (U.S. Geological Survey) and TXDOT (Texas Department of Transportation) regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. These equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate general regression equations might not be appropriate.

<u>Method</u>	<u>Manual Section</u>	<u>Rational Method</u>	<u>SCS Method</u>	<u>Modified Rational</u>	<u>Snyder's Unit Hydrograph</u>	<u>USGS / TXDOT Equations</u>	<u>iSWM Water Quality Volume Calculation</u>
Water Quality Protection Volume (WQ _v)	1.4						✓
Streambank Protection Volume (SP _v)	1.4		✓		✓		
Flood Control Discharge (Q _f)	1.4		✓		✓	✓	
Storage Facilities	4.5		✓	✓	✓		
Outlet Structures	4.6		✓		✓		
Gutter Flow and Inlets	3.2	✓					
Storm Drain Pipes	3.2	✓	✓		✓		
Culverts	4.2	✓	✓		✓	✓	
Bridges	4.3		✓		✓		
Small Ditches	4.4	✓	✓		✓		
Open Channels	4.4		✓		✓	✓	
Energy Dissipation	4.7		✓		✓		

Method	Size Limitations¹	Comments
Rational	0 – 200 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Modified Rational	0 – 200 acres	Method can be used for estimating runoff volumes for storage design.
Unit Hydrograph ²	> 100 acres	Method can be used for estimating peak flows and hydrographs for all design applications.
TXDOT Regression Equations	10 to 100 mi ²	Method can be used for estimating peak flows for rural design applications.
USGS Regression Equations	3 – 40 mi ²	Method can be used for estimating peak flows for urban design applications.
iSWM Water Quality Protection Volume Calculation	Limits set for each Structural Control	Method used for calculating the Water Quality Protection Volume (WQ _v)
¹ Size limitation refers to the drainage basin for the storm water management facility (e.g., culvert, inlet). ² This refers to the SCS and Snyder's routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.		

If other hydrologic methods are to be considered and used by a local review authority or design engineer, the method should first be calibrated to local conditions and tested for accuracy and reliability. If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

2.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 2.1.2-1 Symbols and Definitions		
Symbol	Definition	Units
A	Drainage area or area	acres or square feet
B _f	Baseflow	acre-feet
C	Runoff coefficient	-
C _f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Time interval	hours
E	Evaporation	ft
E _t	Evapotranspiration	ft
F	Pond and swamp adjustment factor	-
G _h	Hydraulic gradient	-
I or i	Rainfall intensity	in/hr
I	Percent of impervious cover	%
I	Infiltration	acre-feet
I _a	Initial abstraction from total rainfall	in
k _h	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient	-
NRCS	Natural Resources Conservation Service (formerly SCS)	-
O _f	Overflow	acre-feet
P	Accumulated rainfall	in
P ₂	2-year, 24-hour rainfall	in
P _w	Wetted perimeter	ft
PF	Peaking factor	-
Q	Rate of runoff	cfs (or inches)
Q _i	Peak inflow discharge	cfs
Q _o	Peak outflow discharge	cfs
Q _p	Peak rate of discharge	cfs
Q _{wq}	Water Quality peak rate of discharge	cfs
q	Storm runoff during a time interval	in
q _u	Unit peak discharge	cfs (or cfs/mi ² /inch)
R	Hydraulic radius	ft
R _o	Runoff	acre-feet
R _v	Runoff Coefficient	-
S	Ground slope	ft/ft or %
S	Potential maximum retention	in
S	Slope of hydraulic grade line	ft/ft
SCS	Soil Conservation Service (Now NRCS)	-
SP _v	Streambank Protection Volume	acre-feet
T	Channel top width	ft
T _L	Lag time	hours
T _p	Time to peak	hours
T _t	Travel time	hours
t	Time	min
t _c	Time of concentration	min

Symbol	Definition	Units
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
V _d	Developed runoff volume	in
V _f	Flood control volume	acre-feet
V _r	Runoff volume	acre-feet
V _s	Storage volume	acre-feet
WQ _v	Water quality protection volume	acre-feet

2.1.3 Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs

Depth (inches) – Total amount of rainfall occurring during the storm duration

Intensity (inches per hour) – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedance probability* or *return period*.

Exceedance Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically in years

Return Period – Average length of time between events, which have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100 years.

Rainfall intensities for the 17 counties which participate in the NCTCOG area (see Figure 2.1.3-1) are provided in Appendix A and should be used for all hydrologic analysis within the given county. The values in these tables were derived in the following way:

- New IDF values for the 1-year through 500-year storm return periods were determined for the NCTCOG area on a county by county basis.
- All values were plotted and smoothed to ensure continuity. The values were smoothed by fitting an equation of the form:

$$i = b/(t + d)^e \quad (2.1.1)$$

where *i* is inches per hour and *t* is the rainfall duration in minutes. The parameters *b*, *d* and *e* are found at the top of each of the tables in Appendix A.

- The tabular values in Appendix A were determined from the new IDF curves.

Figure 2.1.3-2 shows an example Intensity-Duration-Frequency (IDF) Curve for Dallas County, for seven storms (1-year – 100-year). These curves are plots of the tabular values. No values are given for times less than 5 minutes. The 500-year values are given for durations no less than 15 minutes.

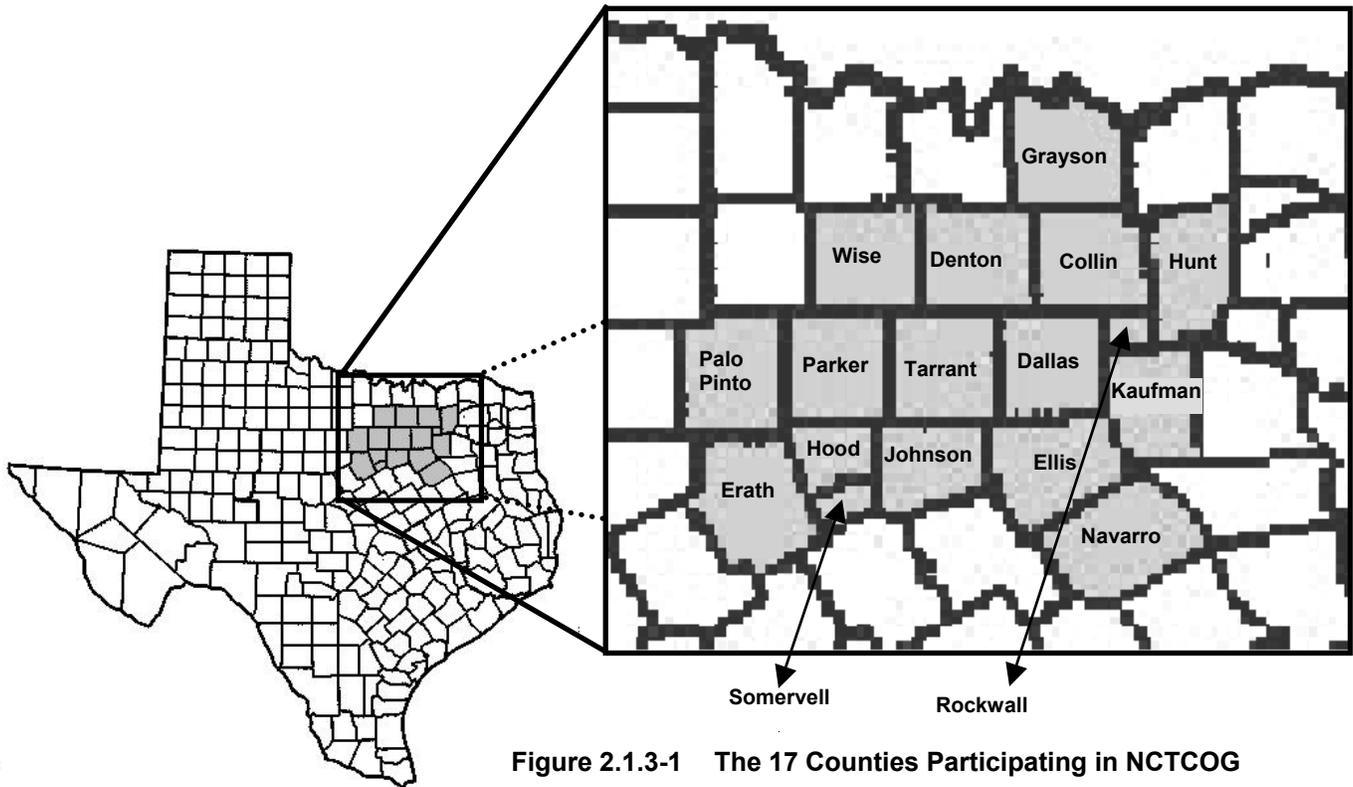


Figure 2.1.3-1 The 17 Counties Participating in NCTCOG

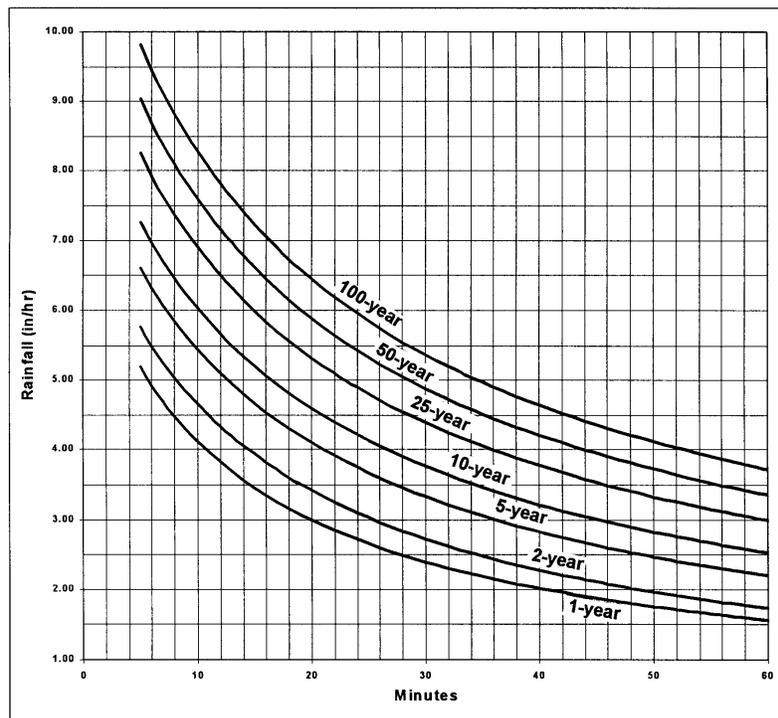


Figure 2.1.3-2 Example IDF Curve (Dallas County, Texas)

2.1.4 Rational Method

2.1.4.1 Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula adheres to the following assumptions:

- The predicted peak discharge has the same probability of occurrence (return period) as the rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate adjustments are appropriate.

2.1.4.2 Application

The Rational Method can be used to estimate storm water runoff peak flows for the design of gutter flows, drainage inlets, storm drainpipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 200 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, the Modified Rational method is used by some for design of small detention facilities, so the method has been included in Subsection 2.1.7. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in Subsection 2.1.7 corrects this deficiency in the method and can be used for detention design for drainage areas up to 200 acres.

The Rational Method should not be used for calculating peak flows downstream of bridges, culverts, or storm sewers that may act as restrictions causing storage, which impacts the peak rate of discharge.

2.1.4.3 Equations

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

The Rational Formula is expressed as follows:

$$Q = CIA \quad (2.1.2)$$

where:

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

The coefficients given in Table 2.1.4-2 are applicable for storms with return periods less than or equal to 10 years. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f . The Rational Formula now becomes:

$$Q = C_f CIA \quad (2.1.3)$$

The C_f values that can be used are listed in Table 2.1.4-1. The product of C_f times C shall not exceed 1.0.

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

2.1.4.4 Time of Concentration

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2.1.4-1 can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter. In urban areas, the length of overland flow distance should realistically be no more than 50 – 100 feet.

Although there is no formula for the graph shown in Figure 2.1.4-1, the formula often used, which seems to match the nomograph very closely, is as follows:

$$T_c = 1.8(1.1 - C)(D)^{0.5}/(S)^{(1/3)} \quad (2.1.4)$$

where:

- T_c = time of concentration (min)
- C = average or composite runoff coefficient
- D = distance from upper end of watershed to outlet (ft)
- S = average slope along distance "D", in percent (ft/100 ft)

Example: Given the following values, determine the time of concentration using (1) Equation 2.1.4, and (2) Figure 2.1.4-1: D = 250 ft, C = 0.7, S = 0.50% slope.

1. Figure 2.1.4-1 gives approximately 15 minutes.
2. $T_c = 1.8(1.1 - 0.7)(250)^{0.5}/(0.50)^{(1/3)} = 14.34 \text{ min}$

Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow travel times that must be added as part of the overall time of concentration. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in Figure 2.1.4-2. More guidance on travel time estimation is given in Section 2.1.5.6.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes. Note that the time of concentration cannot be less than 5 minutes or that which is established by local standards.

The following table gives recommended minimum and maximum times of concentration based on land use categories. The minimum time of concentration should be used for the most upstream inlet (minimum inlet time). Computed downstream travel times will be added to determine times of concentration through the system. For anticipated future upstream development, the time of concentration should be no greater than the maximum.

<u>Land Use</u>	<u>Minimum (minutes)</u>	<u>Maximum (minutes)</u>
Residential Development	15	30
Commercial and Industrial	10	25
Central Business District	10	15

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

2.1.4.5 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in the rainfall tables in Appendix A.

2.1.4.6 Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and 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. Table 2.1.4-2 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2.1.4-2 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients

for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be 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. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

Table 2.1.4-2 Recommended Runoff Coefficient Values	
<u>Description of Area</u>	<u>Runoff Coefficients (C)</u>
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Agricultural	0.30
Forest	0.15
Streams, Lakes, Water Surfaces	1.00
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single Family (1/8 acre lots)	0.65
Single Family (1/4 acre lots)	0.60
Single Family (1/2 acre lots)	0.55
Single Family (1+ acre lots)	0.45
Multi-Family Units, (Light)	0.65
Multi-Family, (Heavy)	0.85
Commercial/Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover:	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

2.1.4.7 Example Problem

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

Site Data

From a topographic map of the City of Arlington (Tarrant County) and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition, the following data were measured:

Average overland slope = 2.0%

Length of overland flow = 50 ft

Length of main basin channel = 2,250 ft

Slope of channel = .018 ft/ft = 1.8%

Roughness coefficient (n) of channel was estimated to be 0.090

From existing land use maps, land use for the drainage basin was estimated to be:

Residential (single family – ¼ acre lots) - 80%

Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 2.1.4-2 to be 0.10.

Time of Concentration

From Figure 2.1.4-1 with an overland flow length of 50 ft, slope of 2% and a C of 0.10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 2.1.4-2 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

$$\text{Flow Time} = \frac{2,250 \text{ feet}}{(3.1 \text{ ft/s}) / (60 \text{ s/min})} = 12.1 \text{ minutes}$$

$$\text{and } t_c = 10 + 12.1 = 22.1 \text{ min (use 22 min)}$$

Rainfall Intensity

From Table A-16 in Appendix A, using a duration equal to 22 minutes,

$$I_{25} \text{ (25-yr return period)} = 5.41 \text{ in/hr}$$

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 2.1.4-2.

<u>Land Use</u>	<u>Percent of Total Land Area</u>	<u>Runoff Coefficient</u>	<u>Weighted Runoff Coefficient</u>
Residential (Single Family – ¼ acre lots)	0.80	0.60	0.48
Graded area	0.20	0.30	0.06
Total Weighted Runoff Coefficient = <u>0.54</u>			
*Column 3 equals column 1 multiplied by column 2.			

Peak Runoff

The estimate of peak runoff for a 25-yr design storm for the given basin is:

$$Q_{25} = C_fCIA = (1.10)(.54)(5.41)(23) = 73.9 \text{ cfs}$$

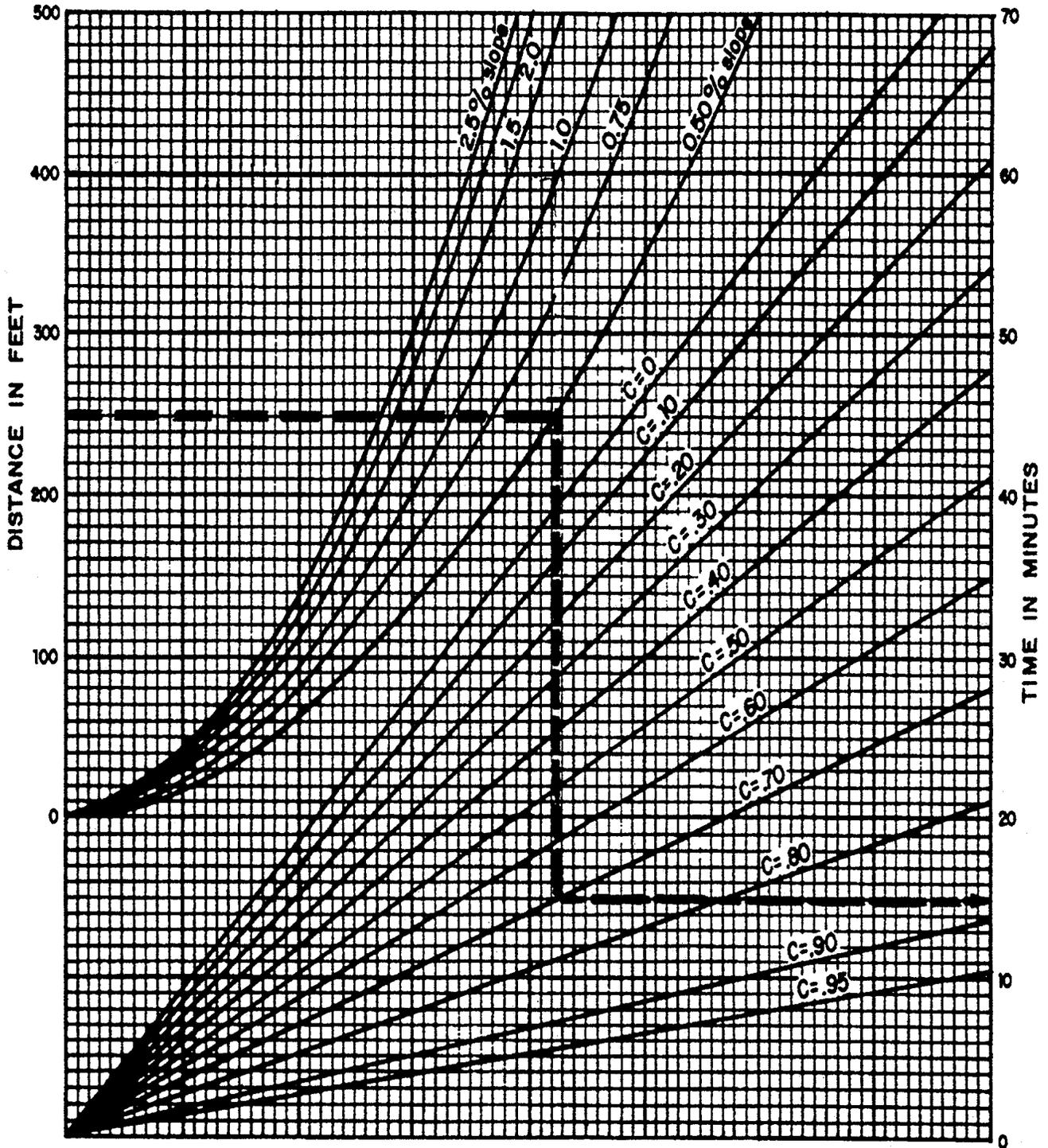
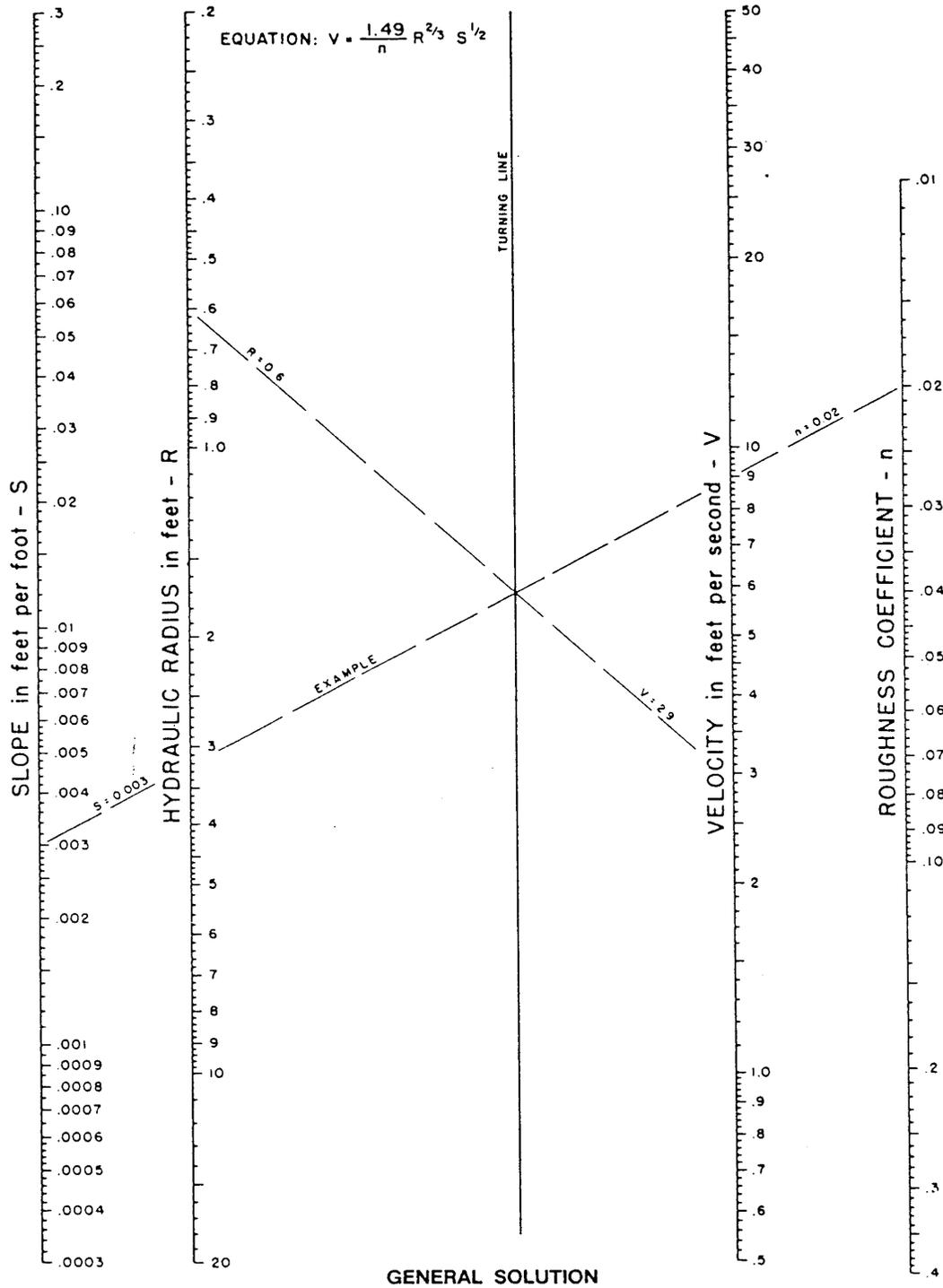


Figure 2.1.4-1 Rational Formula - Overland Time of Flow Nomograph

(Source: Airport Drainage, Federal Aviation Administration, 1965)



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 2.1.4-2 Manning's Equation Nomograph
 (Source: USDOT, FHWA, HDS-3 (1961))

2.1.5 SCS Hydrologic Method

2.1.5.1 Introduction

The Soil Conservation Service¹ (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area.
2. Calculation of time of concentration to the study point.
3. Using the Type II rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 2.1.5-1 for the geographic boundaries for the different SCS rainfall distributions.
4. Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

2.1.5.2 Application

The SCS method can be used for both the estimation of storm water runoff peak rates and the generation of hydrographs for the routing of storm water flows. The simplified method of subsection 2.1.5.7 can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches, open channels, and energy dissipators.

2.1.5.3 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size, and slope are constant, the unit hydrograph approach assumes there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basic concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to North Central Texas is based on a storm event that has a Type II time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 2.1.5-1).

¹ The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

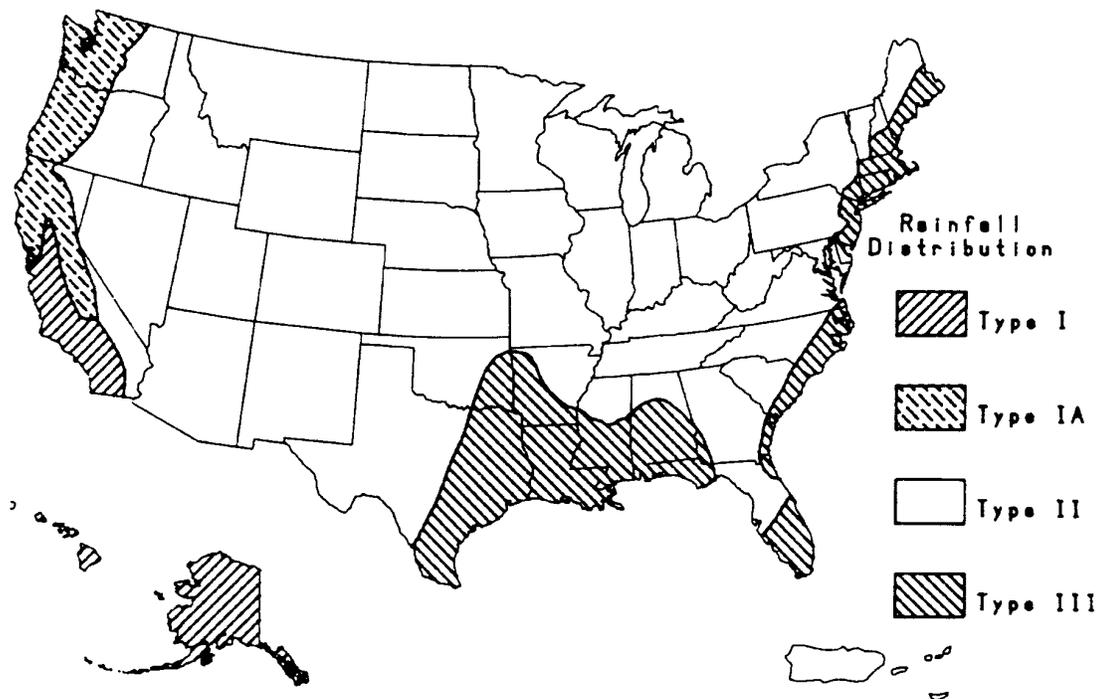


Figure 2.1.5-1 Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = (P - I_a)^2 / [(P - I_a) + S] \quad (2.1.5)$$

where:

Q = accumulated direct runoff (in)

P = accumulated rainfall (potential maximum runoff) (in)

I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)

S = $1000/CN - 10$ where CN = SCS curve number

An empirical relationship used in the SCS method for estimating I_a is:

$$I_a = 0.2S \quad (2.1.6)$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment. Table 2.1.5-3 provides values of I_a for a wide range of curve numbers (CN).

Substituting $0.2S$ for I_a in equation 2.1.5, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \quad (2.1.7)$$

Figure 2.1.5-2 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.

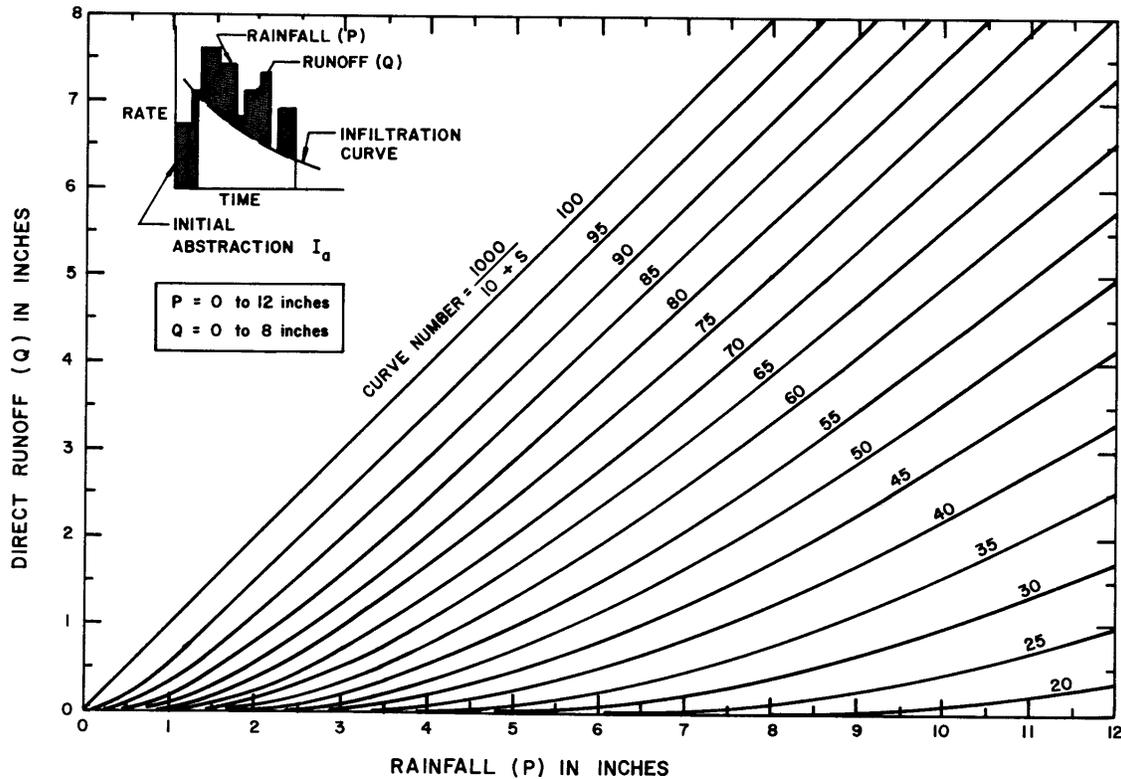


Figure 2.1.5-2 SCS Solution of the Runoff Equation

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

Equation 2.1.6 can be rearranged so the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \tag{2.1.8}$$

2.1.5.4 Runoff Factor (CN)

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups.

- Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.
- Group 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.
- Group 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.

Group 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.

A list of soils throughout the State of Texas and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local USDA Natural Resources Conservation Service offices for use in estimating soil type. Appendix B contains hydrologic soils classification data for North Central Texas. County specific data can be found on-line through NRCS at <http://soils.usda.gov/> and/or www.nctcog.dst.tx.us/.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 2.1.5-1 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

Composite Curve Number Calculation Example			
Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil Group B	0.80	0.85	0.68
Meadow Good condition Soil Group C	0.20	0.71	0.14
Total Weighted Curve Number = 0.68 + 0.14 = 0.82			

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

2.1.5.5 Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2.1.5-1 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over

pervious areas and then into a drainage system. It is possible for curve number values from urban areas to be reduced by not directly connecting impervious surfaces in the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

The CNs provided in Table 2.1.5-1 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. Pervious urban areas are equivalent to pasture in good hydrologic condition, and
2. Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2.1.5-1 are not applicable, use Figure 2.1.5-3 to compute a composite CN. For example, Table 2.1.5-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2.1.5-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

<u>Cover Description</u>	<i>Average percent impervious area²</i>	<u>Curve numbers for hydrologic soil groups</u>			
		A	B	C	D
<i>Cover type and hydrologic condition</i>					
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)³					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93

Cover Description	Average percent impervious area ²	Curve numbers for hydrologic soil groups			
		A	B	C	D
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

¹ Average runoff condition, and $I_a = 0.2S$

² The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

³ CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

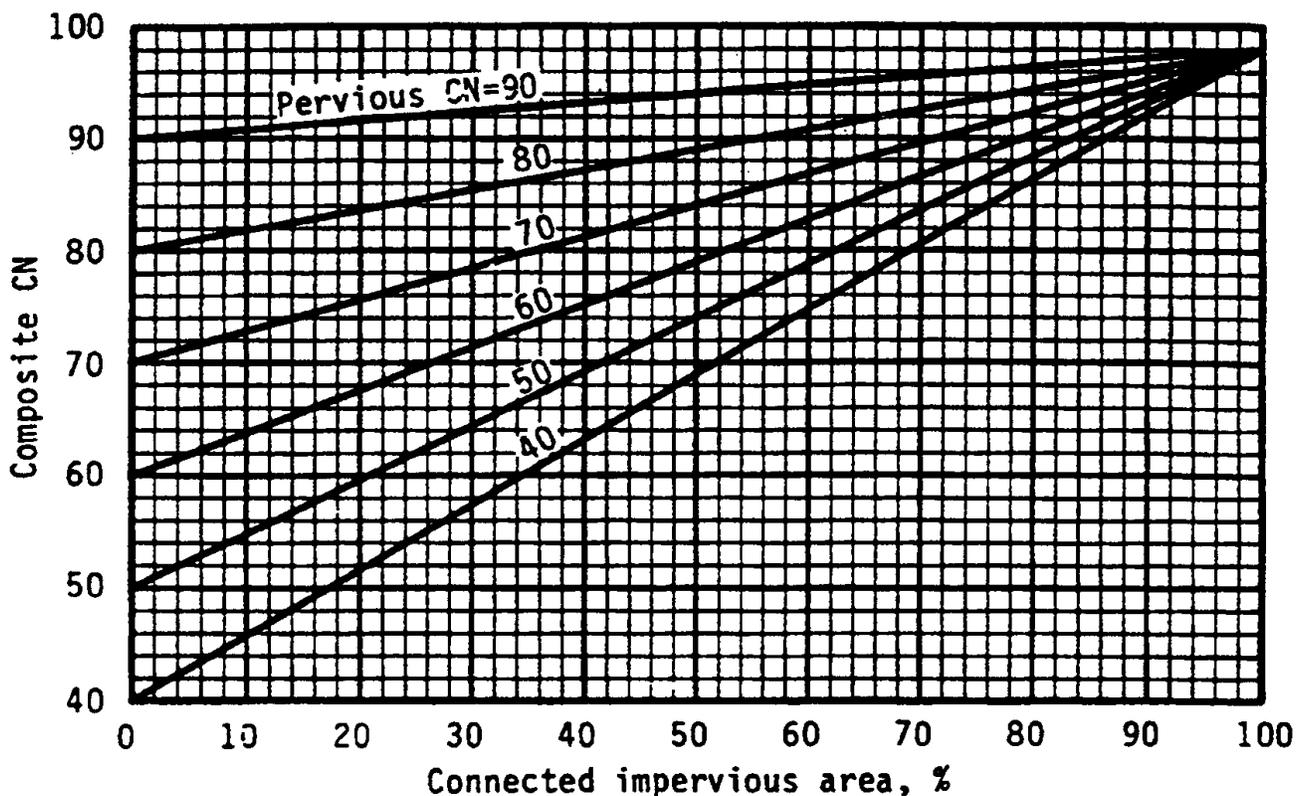
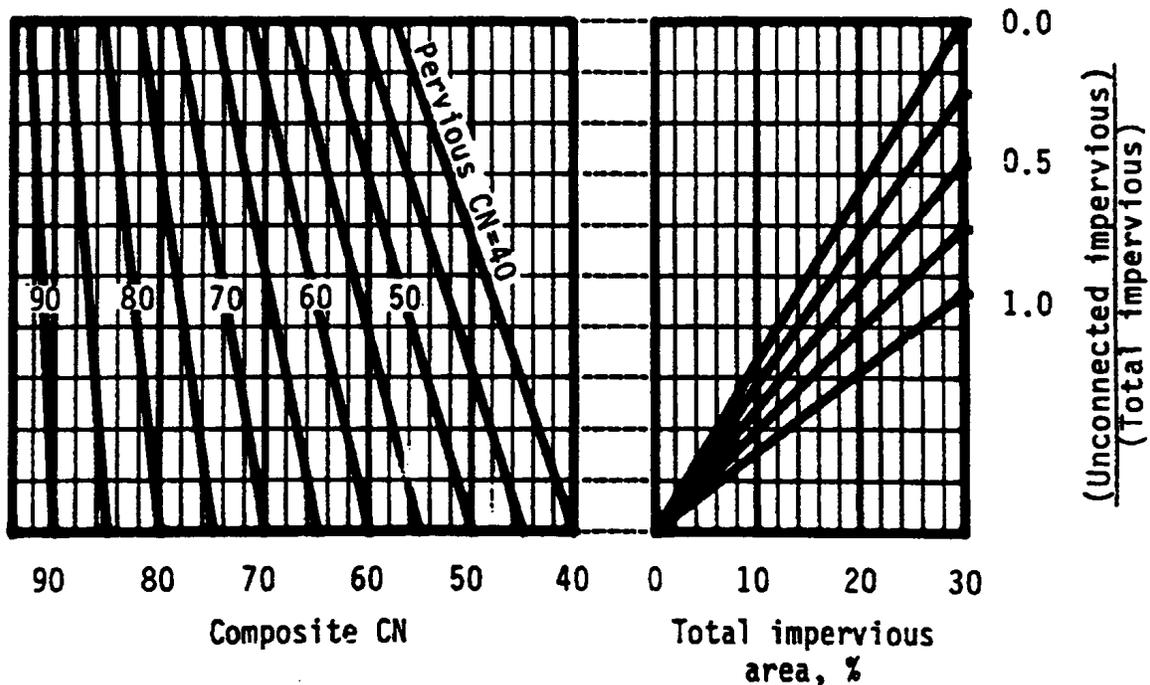


Figure 2.1.5-3 Composite CN with Connected Impervious Areas
 (Source: SCS, TR-55, Second Edition, June 1986)

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2.1.5-4 if total impervious area is less than 30% or (2) use Figure 2.1.5-3 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When the impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 2.1.5-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 2.1.5-4 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2.1.5-3) would be 68.



**Figure 2.1.5-4 Composite CN with Unconnected Impervious Areas
(Total Impervious Area Less Than 30%)**
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.6 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/3600V \quad (2.1.9)$$

where:

- T_t = travel time (hr)
- L = flow length (ft)
- V = average velocity (ft/s)
- 3600 = conversion factor from seconds to hours

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42 (nL)^{0.8}}{60 (P_2)^{0.5} (S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} (S)^{0.4}} \quad (2.1.10)$$

where:

- T_t = travel time (hr)
- n = Manning roughness coefficient (see Table 2.1.5-2)
- L = flow length (ft),
- P_2 = 2-year, 24-hour rainfall
- S = land slope (ft/ft)

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils: Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass: Short grass prairie	0.15
Dense grasses ²	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods³ Light underbrush	0.40
Dense underbrush	0.80

¹ The n values are a composite of information by Engman (1986).
² Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures.
³ When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.
 Source: SCS, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2.1.5-5, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2.1.5-5, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

$$\text{Unpaved } V = 16.13(S)^{0.5} \quad (2.1.11)$$

$$\text{Paved } V = 20.33(S)^{0.5} \quad (2.1.12)$$

where:

V = average velocity (ft/s)

S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 2.1.5-5 or equations 2.1.11 or 2.1.12, use equation 2.1.9 to estimate travel time for the shallow concentrated flow segment.

Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where stream designations appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is

$$V = (1.49/n) (R)^{2/3} (S)^{1/2} \quad (2.1.13)$$

where:

V = average velocity (ft/s)

R = hydraulic radius (ft) and is equal to A/P_w

A = cross sectional flow area (ft²)

P_w = wetted perimeter (ft)

S = slope of the hydraulic grade line (ft/ft)

n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 2.1.13, T_t for the channel segment can be estimated using equation 2.1.9.

Limitations

- Equations in this section should not be used for sheet flow longer than 50 feet for impervious surfaces.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c .
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

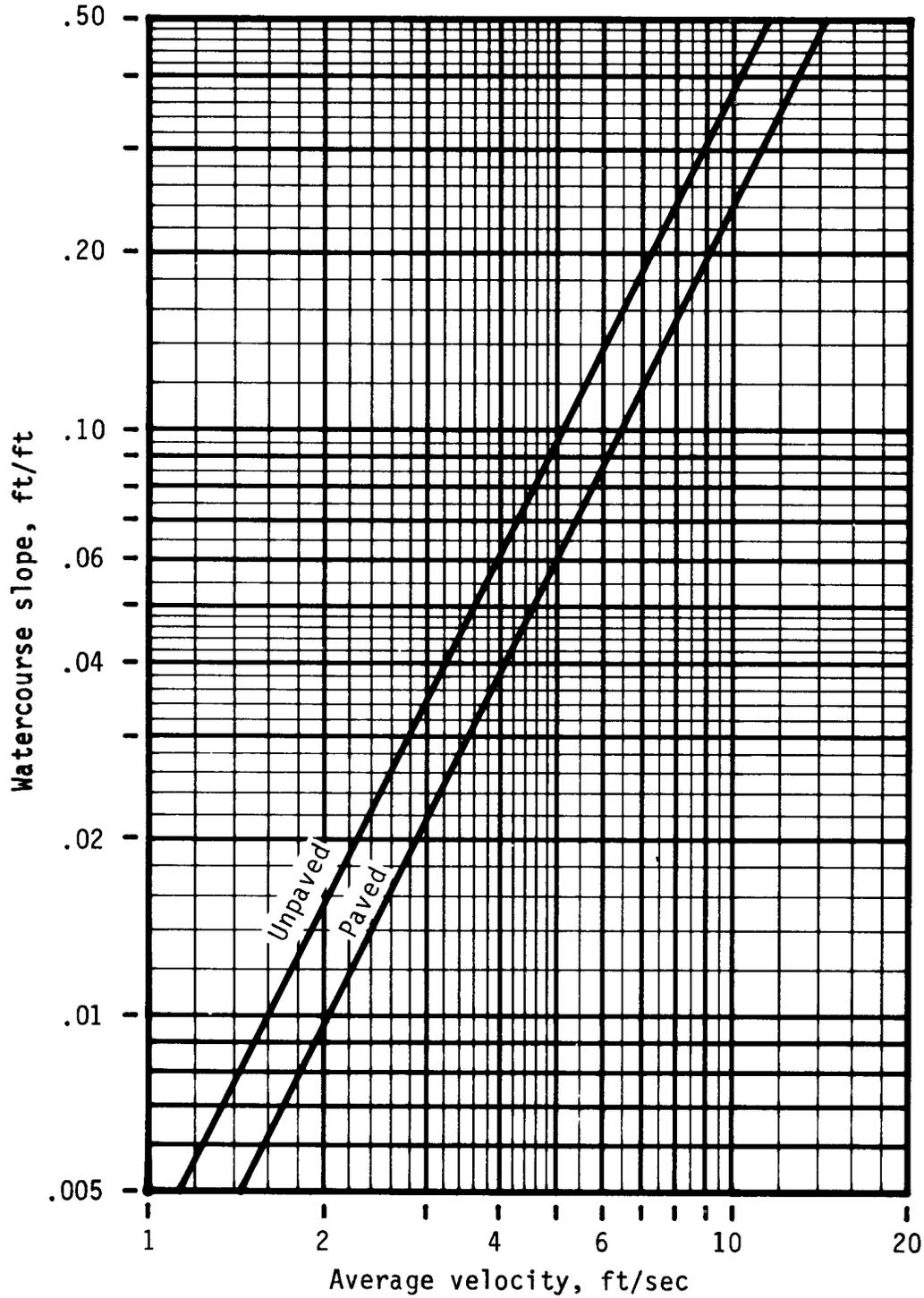


Figure 2.1.5-5 Average Velocities - Shallow Concentrated Flow
 (Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.7 Simplified SCS Peak Runoff Rate Estimation

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses, which can be described by a single CN value. The peak discharge equation is:

$$Q_p = q_u A Q F_p \quad (2.1.14)$$

where:

- Q_p = peak discharge (cfs)
- q_u = unit peak discharge (cfs/mi²/in)
- A = drainage area (mi²)
- Q = runoff (in)
- F_p = pond and swamp adjustment factor

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth (P) is determined from the rainfall tables in Appendix A for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 2.1.5-1 and direct runoff, Q, is calculated using equation 2.1.7.
3. The CN value is used to determine the initial abstraction, I_a , from Table 2.1.5-3, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
4. The watershed time of concentration is computed using the procedures in subsection 2.1.5.6 and is used with the ratio I_a/P to obtain the unit peak discharge (q_u) from Figure 2.1.5-6 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figure 2.1.5-6 is based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used. Peaking factors are discussed further in Section 2.1.5.9.
5. The pond and swamp adjustment factor, F_p , is estimated from below:

<u>Pond and Swamp Areas (%*)</u>	<u>F_p</u>
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

6. The peak runoff rate is computed using equation 2.1.14.

Table 2.1.5-3 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

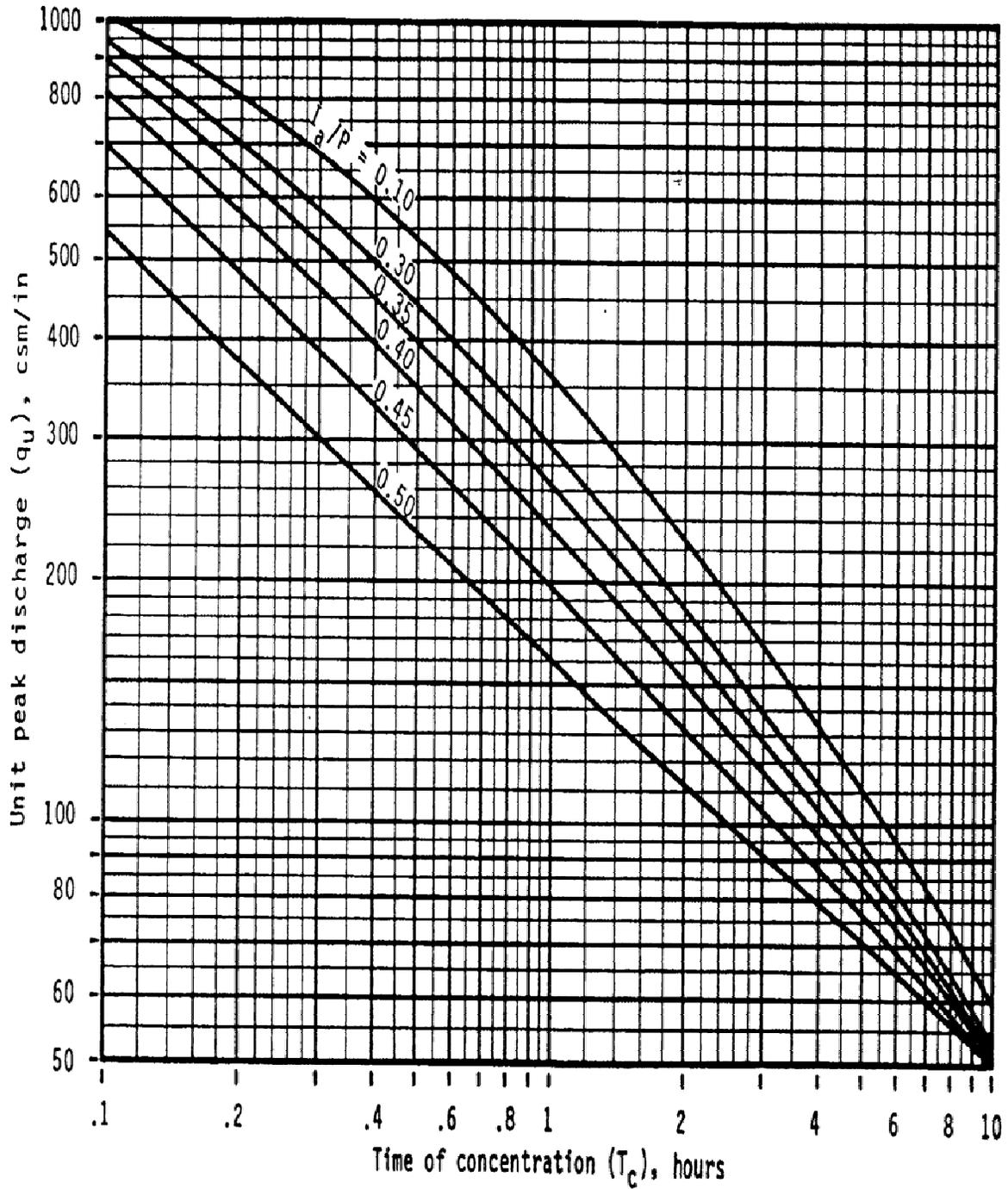


Figure 2.1.5-6 SCS Type II Unit Peak Discharge Graph
 (Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.8 Example Problem 1

Compute the 100-year peak discharge for a 50-acre watershed located in Fort Worth, which will be developed as follows:

1. Pasture / range land - good condition (hydrologic soil group D) = 10 ac
2. Pasture / range land - good condition (hydrologic soil group C) = 10 ac
3. 1/3 acre residential (hydrologic soil group D) = 20 ac
4. Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

Computations

1. Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 9.12 inches (.38 in/hr x 24 hours – From Appendix A, Table A-16).
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	0.20	80	18.2
2	10 ac.	0.20	74	14.8
3	20 ac.	0.40	86	34.4
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00		83

* from Equation 2.1.7 Q (100-year) = 7.1 inches

2. Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland $n = 0.24$	40	2.0
2	Shallow channel (unpaved)	750	1.7
3	Main channel*	1100	0.50

* For the main channel, $n = .06$ (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from equation 2.1.10 with $P_2 = 3.36$ inches (0.14 x 24 – Appendix A, Table A-16)

$$T_t = [0.42(0.24 \times 40)^{0.8}] / [(3.36)^{0.5} (.020)^{0.4}] = 6.69 \text{ minutes}$$

Segment 2 - Travel time from Figure 2.1.5-5 or equation 2.1.11

$$V = 2.1 \text{ ft/sec (from equation 2.1.11)}$$

$$T_t = 750 / 60 (2.1) = 5.95 \text{ minutes}$$

Segment 3 - Using equation 2.1.13

$$V = (1.49/.06) (1.43)^{0.67} (.005)^{0.5} = 2.23 \text{ ft/sec}$$

$$T_t = 1100 / 60 (2.23) = 8.22 \text{ minutes}$$

$$t_c = 6.69 + 5.95 + 8.22 = 20.86 \text{ minutes (.35 hours)}$$

3. Calculate I_a/P for CN = 83 (Table 2.1.5-1), $I_a = .410$ (Table 2.1.5-3)

$$I_a/P = (.410 / 9.12) = .05$$

(Note: Use $I_a/P = .10$ to facilitate use of Figure 2.1.5-6.)

4. Unit discharge q_u (100-year) from Figure 2.1.5-6 = 650 csm/in
5. Calculate peak discharge with $F_p = 1$ using equation 2.1.14

$$Q_{100} = 650 (50/640)(7.1)(1) = 360 \text{ cfs}$$

2.1.5.9 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas.

A value of 484 should be used for most areas of North Texas; however, there are flat areas where a lesser value may be appropriate.

The development of a runoff hydrograph from a watershed is a laborious 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. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other "administrative" parameters, which are peculiar to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

1. Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in subsection 2.1.5.3 is used. This storm is recommended for use in North Central Texas.
2. Development of curve numbers and lag times for the watershed using the methods described in subsections 2.1.5.4, 2.1.5.5, and 2.1.5.6.
3. Development of a unit hydrograph using the standard (peaking factor of 484) dimensionless unit hydrograph. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 2.1.7).
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called "convolution").
6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 484, Figure 2.1.5-7 and Table 2.1.5-4 have been developed. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but, typically, should not be used for areas in North Central Texas.

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$q_u = (PF A) / (T_p) \quad (2.1.15)$$

where:

q_u = unit hydrograph peak rate of discharge (cfs)

PF = peaking factor (484)

A = area (mi^2)

d = rainfall time increment (hr)

T_p = time to peak = $d/2 + 0.6 t_c$ (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by the equation:

$$\frac{q}{q_u} = \left(\frac{t}{T_p} e^{[1-(t/T_p)]} \right)^X \quad (2.1.16)$$

where X is 3.79 for the PF=484 unit hydrograph.

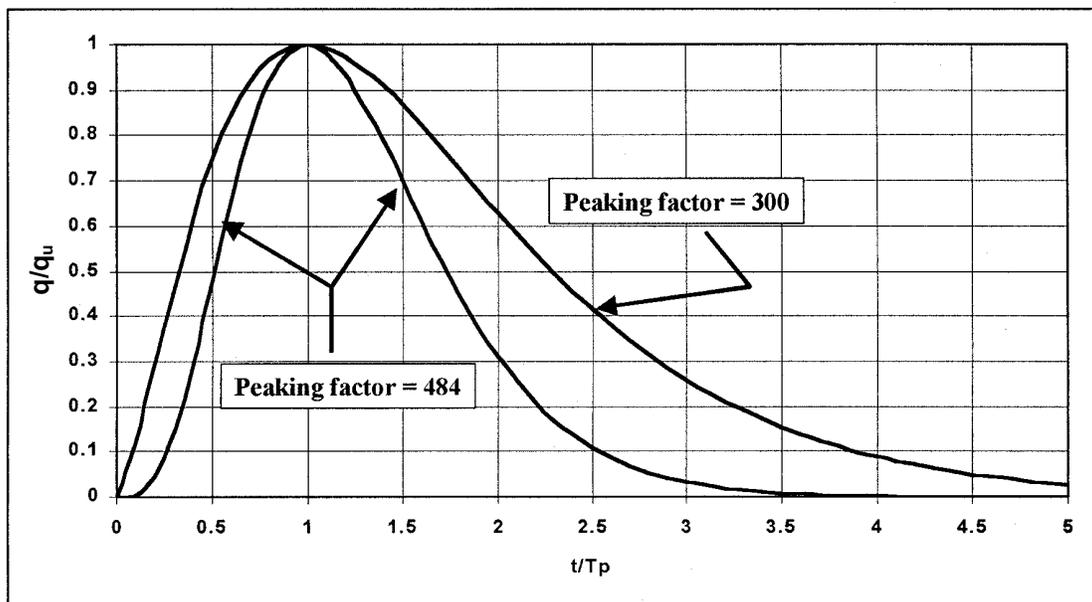


Figure 2.1.5-7 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

t/T_t	484	
	q/q_u	Q/Q_p
0.0	0.0	0.0
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

2.1.5.10 Example Problem 2

Compute the unit hydrograph for the 50-acre watershed in example 2.1.5.8.

Computations

7. Calculate T_p and time increment

The time of concentration (t_c) is calculated to be 20.86 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

$$T_p = d/2 + 0.6t_c = 3/2 + 0.6 * 20.86 = 14.02 \text{ minutes (0.234 hrs)}$$

8. Calculate q_{pu}

$$q_u = PF A/T_p = (484 * 50/640) / (0.234) = 162 \text{ cfs}$$

9. Calculate unit hydrograph.

Based on spreadsheet calculations using equations 2.1.15 and 2.1.16, the table below has been derived.

Time		484	
t/T_p	time (min)	q/q_u	Q
0	0	0	0.00
0.21	3	0.06	9.23
0.43	6.0	0.35	56.77
0.64	9.0	0.72	117.29
0.86	12.0	0.96	155.09
1.00	14.02	1.00	162.00
1.07	15.0	0.99	160.57
1.28	18.0	0.88	142.42
1.50	21.0	0.70	113.52
1.71	24.0	0.52	83.69
1.93	27.0	0.36	58.12
2.14	30.0	0.24	38.51
2.35	33.0	0.15	24.56
2.57	36.0	0.09	15.18
2.78	39.0	0.06	9.14
3.00	42.0	0.03	5.38
3.21	45.0	0.02	3.10
3.42	48.0	0.01	1.76
3.64	51.0	0.01	0.99
3.85	54.0	0.00	0.54
4.07	57.0	0.00	0.30
4.28	60.0	0.00	0.16
4.49	63.0	0.00	0.09
4.71	66.0	0.00	0.05
4.92	69.0	0.00	0.02
5.14	72.0	0.00	0.01
5.35	75.0	0.00	0.01
5.56	78.00	0.00	0.00

2.1.5.11 Hydrologic Stream Routing

Water requires a certain amount of time to travel down a stream or channel reach. A flood wave is attenuated by friction and channel storage as it passes through the reach. The process of computing the travel time and attenuation of water flowing in the reach is often called routing.

Hydrologic routing involves the balancing of inflow, outflow, and volume of storage through the use of the continuity equation. The relation between the outflow rate and storage in the system is also required.

Travel time and attenuation characteristics vary widely between different streams. The travel time is dependent on characteristics such as length, slope, friction, and flow depth. Attenuation is also dependent on friction, in addition to other characteristics such as channel storage. Many routing methods have been developed under different assumptions and for different stream types. Some of the routing methods include: kinematic wave, lag, modified Puls, Muskingum, Muskingum-Cunge 8-point section, and Muskingum-Cunge standard section.

The routing methods selected for use in North Central Texas are the Modified Puls and the Muskingum-Cunge methods (USACE, HEC-HMS, 2000 and Bedient and Huber, 1988).

2.1.6 Snyder's Unit Hydrograph Method

2.1.6.1 Introduction

Snyder's unit hydrograph method is the primary method utilized by the Corps of Engineers Fort Worth District for the majority of hydrologic studies in the region, and is also commonly used by consultants and other entities within the NCTCOG region. It is similar in nature to the SCS method, in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm.

2.1.6.2 Application

Snyder's unit hydrograph method may be used for drainage areas 100 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations* and The Bureau of Reclamation's "Flood Hydrology Manual, A Water Resources Technical Publication," utilizes the following equations:

$$t_p = C_t (L L_{ca})^{0.3} \quad (2.1.17)$$

$$t_r = t_p \div 5.5 \quad (2.1.18)$$

$$q_p = C_p 640 \div t_p \quad (2.1.19)$$

$$t_{pR} = t_p + 0.25(t_R - t_r) \quad (2.1.20)$$

$$q_{pR} = C_p 640 \div t_{pR} \quad (2.1.21)$$

$$q_{pR} = q_p t_p \div t_{pR} \quad (2.1.22)$$

$$Q_p = q_p A \quad (2.1.23)$$

The terms in the above equations are defined as:

- t_r = The standard unit rainfall duration, in hours.
- t_R = The unit rainfall duration in hours other than standard unit, t_r , adopted in specific study.
- t_p = The lag time from midpoint of unit rainfall duration, t_r , to peak of unit hydrograph in hours.
- t_{pR} = The lag time from midpoint of unit rainfall duration, t_R , to peak of unit hydrograph in hours.
- q_p = The peak rate of discharge of unit hydrograph for unit rainfall duration, t_r , in cfs/sq. mi.
- q_{pR} = The peak rate of discharge in cfs/sq. mi. of unit in hydrograph for unit rainfall duration, t_R .

- Q_p = The peak rate of discharge of unit hydrograph in cfs.
 A = The drainage area in square miles.
 L_{ca} = The river mileage from the design point to the centroid of gravity of the drainage area.
 L = The river mileage from the given station to the upstream limits of the drainage area.
 C_t = Coefficient depending upon units and drainage basin characteristics.
 C_p = Coefficient depending upon units and drainage basin characteristics.

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t range from 0.4 to 2.3 and average about 1.1. The value of C_t for the East Fork Trinity River is 2.0. C_t for a watershed can be estimated if the lag time, t_p , stream length, L , and distance to the basin centroid, L_{ca} , are known. The coefficient C_p is the peaking coefficient, which typically ranges from 0.3 to 1.2 with an average value of 0.8, and is related to the flood wave and storage conditions of the watershed. The C_p value for the East Fork Trinity River is 0.69. Larger values of C_p are generally associated with smaller values of C_t . Typical values of C_p are listed in Table 2.1.6-1.

Table 2.1.6-1 Typical Values of C_p	
Typical Drainage Area Characteristics	Value of C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

2.1.6.3 Urbanization Curves

To account for the effects of urbanization, another method was developed by the Corps of Engineers to adjust the t_p coefficient. Urbanization curves allow for the determination of t_p based on the percent urbanization and the type of soil in the study area. Urbanization curves for the Dallas-Fort Worth area were determined from the equation below:

$$t_p = 10^{[0.3833 \log_{10}(L \cdot L_{ca} / (S_{st})^{0.5}) + (\log_{10}(I_p)) - BW \cdot (\% \text{Urb} / 100)]} \quad (2.1.24)$$

$$S_{st} = (e_{l_{85\%}} - e_{l_{15\%}}) / (0.7 \cdot L) \quad (2.1.25)$$

where:

- t_p = The lag time from midpoint of unit rainfall duration, t_r , to peak of unit hydrograph in hours.
 L_{ca} = The river mileage from the design point to the centroid of the drainage area.
 L = The river mileage from the design point to the upstream limits of the drainage area.
 S_{st} = The weighted slope of the flow path (ft/mi)
 I_p = The calibration point, defined as t_p where $(L \cdot L_{ca} / S_{st}^{0.5}) = 1$ and urbanization = 0%.
 BW = The bandwidth, equal to the log of the width between each 20% urbanization line.

- %Urb = A value representative of the degree to which urbanization has occurred in the basin, in percent.
- el_{85%} = The elevation at a location 85% upstream of the given station.
- el_{15%} = The elevation at a location 15% upstream of the given station.

For the Dallas-Fort Worth area, the I_p values used are 0.94 for clay and 1.76 for sand. The bandwidth (BW) value for both of the soil types is 0.266. For a study area that is composed of both sand and clay, a weighted average of the two can be calculated by:

$$t_p \text{ weighted} = \% \text{ sand} * t_p \text{ sand} + \% \text{ clay} * t_p \text{ clay}.$$

Design runoff may be determined for a given watershed by applying the intensity-duration-frequency relationships to the unit hydrograph by multiplying each ordinate of the unit hydrograph by the rainfall intensity.

2.1.6.4 Determination of Percent Urbanization and Percent Sand

The lag time, t_p , is the critical parameter in establishing the timing of the response of a watershed to rainfall. The degree of urbanization is an important variable that determines the value of the lag time. Thomas L. Nelson, Fort Worth District, USACE, defined the general relationship between the lag time, t_p , and the percent of Urbanization, %Urb, and presented a set of Urbanization Curves for the Dallas-Fort Worth area in 1970.

The soil type of a watershed also plays an important role in its response to rainfall. It was found that predominantly sandy soils responded differently to rainfall than predominantly clayey soils. Therefore, two sets of Urbanization Curves were developed to better define the lag time, one set for sandy soils and one set for clayey soils. A paper by Paul K. Rodman, Fort Worth District, USACE presented urbanization curves in 1977 for both "clay loam" and "clay" in the Fort Worth-Dallas area and other Texas locations.

To obtain consistency of computational results, it is necessary to have a logical and routine procedure for the determination of Percent Urbanization (%Urb) and Percent Sand/Clay (%Sand/%Clay). Procedures for their determination are presented below.

Percent Urbanization

Urbanization is defined as the percentage of the basin which has been developed and improved with channelization and/or a storm water collection network. Urbanization of natural and agricultural land converts pervious soils to impervious surfaces. Disturbed soils exhibit a lower infiltration capacity than natural soils. This results in less infiltration which translates to an increased volume of runoff.

Natural flow paths in the watershed may be replaced with prismatic channels. Significant drainage infrastructure may be added in a development composed of streets and gutters, storm sewers, open channels, and other drainage elements. This alteration of the original drainage system changes the watershed's response to precipitation. The addition of drainage infrastructure along with the increase in imperviousness results in significantly increased peak discharges and a greater volume of runoff.

The determination of the percent urbanization (%Urb) as used in the Urbanization Curves defined by Equation 2.1.24 is somewhat subjective, but is related to the type and intensity of development. The U.S. Army Corps of Engineers (USACE) has worked over the years to define the relationship between the type of development and the degree of urbanization. The result of their effort is reflected in Table 2.1.6-2. These are provided for the user's consideration and guidance.

Other techniques to relate the impacts of urbanization on rainfall runoff have been used. Another such technique is presented in Section 2.1.8.2 in the application of USGS regression equations to determine peak flows for urban basins.

Table 2.1.6-2 Percent Urbanization and Imperviousness Summary with Associated Land Use Categories			
Land Use	Description	Percent Imperviousness	Percent Urbanization
Low Density Residential	Single family: ½ – 2 units per acre; average 1 unit per acre.	25	30
Medium Density Residential	Single family: 2 – 3½ units per acre; average 3 units per acre.	41	80
High Density Residential	Single family: greater than 3½ units per acre; average 4 units per acre.	47	90
Multifamily Residential	Row houses, apartments, townhouses, etc.	70	95
Mobile Home Parks	Single family: 5–8 units per acre.	20	40
Central Business District	Intensive, high-density commercial	95	95
Strip Commercial	Low-density commercial; average 3 units per acre.	90	90
Shopping Centers	Grocery stores, drug stores, malls, etc.	95	95
Institutional	Schools, churches, hospitals, etc.	40	50
Industrial	Industrial centers and parks; light and heavy industry.	90	95
Transportation	Major highways, railroads.	35	80
Communication	Microwave towers, etc.	35	50
Public Utilities	Transformer stations, transmission line right-of-way, sewage treatment facilities, water towers, and water treatment facilities.	60	70
Strip Settlement	Densities less than ½ – 2 units per acre; average 1 unit per 3 – 5 acres.	10	20
Parks and Developed Open Space	Parks, cemeteries, etc.	6	10
Developing	Land currently being developed.	15	20
Cropland		3	5
Grassland	Pasture, short grasses.	0	0
Woodlands, Forest		0	0
Water Bodies	Lakes, large ponds.	100	100
Barren Land	Bare exposed rock, strip mines, gravel pits.	0	0

Sources: Determination of Percent Urbanization/Imperviousness in Watersheds, May 1, 1986, U.S. Army Corps of Engineers
SCS, TR-55, Second Edition, June 1986

Percent Sand/Clay

The Fort Worth District, USACE, evaluated methods for determining the percent sand in a watershed and concluded that the permeability rate method was the best method. The procedure was described in the referenced report as follows.

“The permeability rate method uses the range of permeabilities found in the table of physical and chemical properties in the SCS soil surveys for multiple soil classifications and assigns a percent sand to each of the seven ranges. A percent sand of 0 is given to any soil with a permeability less than 0.06

inches per hour which corresponds to the permeability of the Houston Blackland clay upon which the clay urban curves are based. Also, a percent sand of 100 is given to any soil with a rate of 0.6 to 2.0 inches per hour which corresponds to the Crosstell series soil upon which the sandy loam curves are based. The percent sand for the permeability ranges 0.06 to 0.2 inches, 0.2 to 0.6 inches, 2.0 to 6.0 inches, 6.0 to 10.0 inches, and greater than 20 inches are 33, 66, 133, 166, 200 percent sand, respectively. Each soil in the watershed is assigned a percent sand based upon its permeability and a weighted average is computed." (USACE, 1986).

<u>Permeability (inches/hr)</u>	<u>Percent Sand Assignment (%)</u>
< 0.06	0
0.06 to 0.20	33
0.20 to 0.60	66
0.60 to 2.00	100
2.00 to 6.00	133
6.00 to 20.00	166
> 20.00	200

The Houston Black soil series consists of moderately well-drained, deep, cyclic, clayey soils on wetlands. This series formed in alkaline, marine clay, and material weathered from shale. Land slopes range from 1 to 4 percent. The permeability is less than 0.06 inches per hour. This soil is the predominate series found in watersheds used to develop the Dallas-Fort Worth Clay Urbanization Curves. Therefore this soil has a percent sand of 8 for use with the urban curves. The Crosstell soil series consists of moderately well-drained, deep loamy soils on uplands that formed in shaley and clayey sediment containing thin strata of weakly cemented sandstone. Land slopes range from 1 to 6 percent. The permeability for this soil is in the range between 0.6 and 2.0 inches per hour. The Crosstell series is the major soil contained in watersheds used to derive the Dallas-Fort Worth Sandy Loam Urbanization Curves. This soil, therefore, has a percent sand of 100 for use with the urban curves.

Example: Procedure for the Determination of Percent Sand (%Sand).

Given the percent sand assignments below, determine the percent sand for Watershed B.

<u>Watershed</u>	<u>Soil Type No.</u>	<u>Percent Sand</u>	<u>% of Area</u>	<u>% Sand * % Area</u>
B	13	66	2.6	171.6
	23	33	39.7	1310.1
	32	133	31.4	4176.2
	51	33	1.7	56.1
	64	133	17.9	2380.7
	85	33	<u>6.7</u>	<u>221.1</u>
		100		8315.8

Weighted %Sand = 8315.8/100 = 83.2%

There is the possibility of computing greater than 100 percent sand for areas that are very sandy. Soil disturbances during development (urbanization) usually diminish the natural permeability of the soil. Often there is no data reflecting the permeability rate for an urban soil. Therefore, care should be used in applying this method. The percent sand assignment should be that of the controlling sublayer of the soil profile. Consideration should also be given to other factors affecting the initial and time rates of rainfall abstractions. For example, well-vegetated clayey soils may respond hydrologically more like a sandy soil.

Urban lands are usually taken one step down (lower percent sand) from soil types shown in the SCS soil report. The engineer should evaluate all factors bearing on the soil response and determine whether there is a need to make adjustments.

Loss Rates

Several loss rate methodologies are acceptable for use with the Snyder's Unit Hydrograph Method including:

- Block and Uniform
- Holtan
- SCS Curve Number
- Green and Ampt
- Exponential

Block and uniform loss rates developed by the Corps of Engineers during the development of the urbanization curves are listed by clay and sand categories. Losses for a specific basin are determined by a weighting procedure. Adjustments to these values are allowed based on historic storm reproductions.

Frequency	Losses			
	Clay		Sand	
	Block (in)	Uniform (in/hr)	Block (in)	Uniform (in/hr)
2-year	1.5	0.20	2.1	0.26
5-year	1.3	0.16	1.8	0.21
10-year	1.12	0.14	1.5	0.18
25-year	0.95	0.12	1.3	0.15
50-year	0.84	0.1	1.1	0.13
100-year	0.75	0.07	0.9	0.10

Stream Routing

The Modified Puls and Muskingum-Cunge are acceptable routing methods. See Section 2.1.5.11 for an explanation of routing methods and references for further information.

2.1.7 The Modified Rational Method

2.1.7.1 Introduction

For drainage areas of *less than 200 acres*, a modification of the Rational Method can be used for the estimation of storage volumes for detention calculations.

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. There are many variations on the approach. Figure 2.1.7-1 illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (t_c). An allowable target outflow is set (Q_a) based on pre-development conditions. The storm duration is t_d , and is varied until the storage volume (shaded gray area) is maximized. It is normally an iterative process done by hand or on a spreadsheet. Downstream analysis is not possible with this method as only approximate graphical routing takes place.

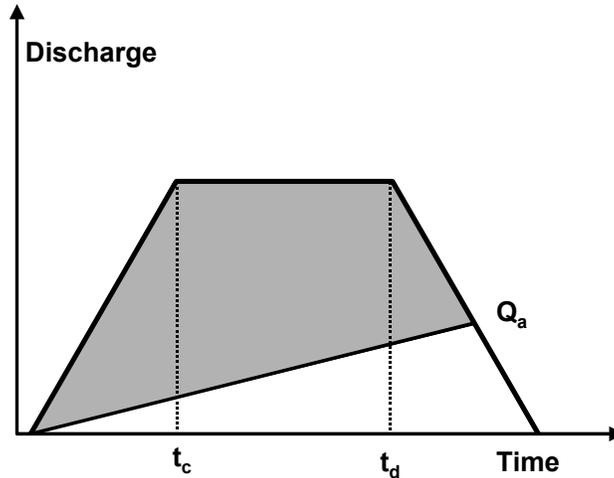


Figure 2.1.7-1 Modified Rational Definitions

2.1.7.2 Design Equations

The design of detention using the Modified Rational Method is presented as a non-iterative approach suitable for spreadsheet calculation (Debo & Reese, 2003).

The allowable release rate can be determined from:

$$Q_a = C_a i A \tag{2.1.26}$$

where:

- Q_a = allowable release rate (cfs)
- C_a = predevelopment Rational Method runoff coefficient
- i = rainfall intensity for the corresponding time of concentration (in/hr)
- A = area (acres)

The critical duration of storm, the time value to determine rainfall intensity, at which the storage volume is maximized, is:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \tag{2.1.27}$$

where:

- T_d = critical storm duration (min)
- Q_a = allowable release rate (cfs)
- C = developed condition Rational Method runoff coefficient
- A = area (acres)
- a, b = rainfall factors dependent on location and return period taken from Table 2.1.7-1

The required storage volume, in cubic feet can be obtained from the equations below:

$$V_{\text{preliminary}} = 60 [CAa - (2CabAQ_a)^{1/2} + (Q_a/2) (b-t_c)] \tag{2.1.28a}$$

$$V_{\text{max}} = V_{\text{preliminary}} * P_{180}/P_{td} \tag{2.1.28b}$$

where:

- $V_{\text{preliminary}}$ = preliminary required storage (ft³)

- V_{\max} = required storage (ft³)
 t_c = time of concentration for the developed condition (min)
 P_{180} = 3-hour (180-minute) storm depth (in)
 P_{td} = storm depth for the critical duration (in)
 all other variables are as defined above

The equations above include the use of an adjustment factor to the calculated storage volume to account for under sizing. The factor (P_{180}/P_{td}) is the ratio of the 3-hour storm depth for the return frequency divided by the rainfall depth for the critical duration calculated in equation 2.1.27.

The Modified Rational Method also often under sizes storage facilities in flat and more sandy areas where the target discharge may be set too large, resulting in an oversized orifice. In these locations modifications to the C factor or time of concentration should be considered in the design of the orifice.

County		Return Interval						
		1	2	5	10	25	50	100
Collin	a	101.14	129.51	177.49	209.08	250.52	283.13	320.81
	b	14.214	16.634	20.174	21.668	22.821	23.455	24.502
Dallas	a	99.8	128.85	178.58	210.73	253.77	288.56	327.75
	b	14.114	16.624	20.352	21.785	23.03	23.866	24.893
Denton	a	97.258	124.47	173.1	205.74	248.54	283.99	325.18
	b	13.788	16.121	19.754	21.358	22.615	23.508	24.822
Ellis	a	101.94	129.3	181.43	214.61	259.34	295.76	336.3
	b	14.511	16.697	20.792	22.384	23.744	24.681	25.818
Erath	a	90.53	113.9	159.31	189.97	228.79	260.81	298.07
	b	13.32	14.99	18.439	19.981	20.955	21.65	22.712
Grayson	a	100.87	128.89	175.74	208.17	250.17	285.35	325.63
	b	14.086	16.567	20.006	21.751	22.993	24.027	25.322
Hood	a	93.351	117.38	163	194.75	235.56	269.71	309.25
	b	13.654	15.308	18.65	20.281	21.438	22.299	23.508
Hunt	a	107.65	131.48	178.92	209.36	249.71	282.05	318.9
	b	15.348	16.855	20.456	21.855	22.995	23.713	24.744
Johnson	a	94.751	120.21	168.39	198.98	240.45	275.19	313.38
	b	13.414	15.543	19.272	20.676	21.847	22.804	23.875
Kaufman	a	104.54	132.07	183.2	216.62	260.03	295.03	334.63
	b	14.637	16.912	20.837	22.424	23.65	24.42	25.496
Navarro	a	108.66	132.42	185.55	221.63	268.93	306.83	350.06
	b	15.326	16.758	20.945	22.903	24.437	25.402	26.665
Palo Pinto	a	91.031	115.97	164.22	196.59	242.51	281.03	326.0
	b	13.127	15.264	19.05	20.714	22.468	23.769	25.388
Parker	a	95.164	118.64	166.17	198.53	242.46	279.34	321.89
	b	13.848	15.396	18.999	20.608	22.048	23.123	24.527
Rockwall	a	107.9	131.23	179.89	212.63	254.36	287.68	325.96
	b	15.671	16.882	20.467	22.064	23.178	23.891	24.906
Somervell	a	92.245	116.25	162.12	193.36	232.22	265.8	303.15
	b	13.091	14.967	18.503	20.102	21.066	22.001	23.039
Tarrant	a	95.835	121.96	170.81	203.93	247.1	282.6	322.07
	b	13.425	15.704	19.435	21.09	22.366	23.302	24.388
Wise	a	93.326	118.05	165.95	200.22	247.21	287.89	334.11
	b	13.491	15.315	18.974	20.889	22.662	24.112	25.784

2.1.7.3 Example Problem

A 5-acre site is to be developed in Dallas. Based on site and local information, it is determined that streambank protection is not required and that limiting the 25-year and 100-year storm is also not required. The local government has determined that the development must detain the 2-year and 10-year storms. Rainfall values are taken from Appendix A. The following key information is obtained:

Area = 5 acres

Slope is about 5%

Pre-development t_c = 21 minutes and C factor = 0.22

Post-development t_c = 10 minutes and C factor = 0.80

<u>Steps</u>	<u>2 - year</u>	<u>10 - year</u>
t_c (min)	21	21
i (in/hr)	3.35	4.79
Q_a (equation 2.1.25) (cfs)	3.69	5.27
a (from Table 2.1.7-1)	128.85	210.73
b (from Table 2.1.7-1)	16.624	21.785
V_{pre} (equation 2.1.28a) (ft ³)	16,570	26,042
P_{180} (in)	2.28	3.60
T_d (equation 2.1.27) (min)	51.52	61.69
P_{td} (in)	1.65	2.66
V_{max} (equation 2.1.28b) (ft ³)	22,897	35,245

2.1.8 USGS and TxDOT Regression Methods

2.1.8.1 Introduction

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed.

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas. For this manual, the USGS regression equations are used to determine peak flows in urban drainage areas, and the TxDOT regression equations are used to determine peak flows in rural drainage areas. It may be difficult to choose the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies. Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

The following suggestions should be considered when using regression equations:

- Conduct a field visit to compare and assess the watershed characteristics for comparison with other watersheds.
- Collect all available historical flood data.
- Use the gathered data to interpret any discharge values.

2.1.8.2 USGS Equations for Urban Basins

Regression equations developed by the USGS for urban streams in Dallas-Fort Worth are for estimating peak discharges (QT) having recurrence intervals (T) that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (DA), in square miles, and an urbanization index (UI), which is evaluated as described in the report by Land and others (U.S.G.S., 1982).

The urbanization index is an attempt to more accurately quantify the degree of urbanization by incorporating the factors of storm sewers, curbs and gutters, and channel rectifications. The index is developed by considering these alterations in the upper, middle, and lower third of the drainage basin. Values are assigned to each factor in each one-third of the basin on the basis of the percentage of the subbasin containing that factor. Each factor carries an equal weight regardless of location within the subbasin. The values of each factor vary from 1 to 4, based on the degree of development. The sum of the 9 factors can vary from 9 to 36 and is the value of the urbanization index. The factor values and corresponding percentages of the subbasin affected are listed below:

<u>Percent</u>	<u>Value</u>
0 – 24	1
25 – 49	2
50 – 74	3
75 – 100	4

The following example is given to illustrate the determination of the urbanization index.

<u>Urbanization Index</u>				
<u>Sub area</u>	<u>Factors</u>			<u>Total</u>
	<u>Storm Sewers</u>	<u>Curbs and Gutters</u>	<u>Channel Rectifications</u>	
Upper	4	4	2	10
Middle	3	4	1	8
Lower	3	4	1	8
<u>Urbanization Index</u>				26

Source: Techniques for Estimating the Magnitude and Frequency of Floods in the Dallas-Fort Worth Metropolitan Area, Texas, U.S. Geological Survey, Water Resources Investigation 82-18

2.1.8.3 Application of USGS Equations

The USGS regression equations were developed from peak-discharge records from drainage areas in the Dallas-Fort Worth area ranging from 1.25 to 66.4 square miles with results considered applicable to drainage areas between 3 and 40 square miles having urbanization indexes between 12 and 33. The standard errors of estimate of the regression equations are about 30 percent. As such, the USGS regression method should only be used for calculating peak discharge in urban drainage areas as described.

The USGS method can be used for several design applications, including storm drain systems, culverts, small drainage ditches and open channels, and energy dissipaters.

For a complete description of the USGS regression equations presented below, consult the USGS publication *Techniques for estimating the magnitude and frequency of floods in the Dallas-Fort Worth metropolitan area, Texas: U.S. Geological Survey Water-Resources Investigations Report 82-18, 55 p.* Table 2.1.8-1 gives the USGS regression equations for urban streams in the Dallas-Fort Worth area.

2.1.8.4 Peak Discharge Limitations for Urban Basins

Following are the limitations of the variables within the peak discharge equations. These equations should not be used on drainage areas which have physical characteristics outside the limits listed below:

Physical Characteristics	Minimum	Maximum	Units
A - Drainage Area	3	40	mi ²
UI – Urbanization Index	12	33	

Table 2.1.8-1 USGS Peak Flow Regression Equations for Dallas-Fort Worth Urban Areas

Frequency	Equations
2-year	$Q_2 = 42.83(DA)^{0.704} (UI)^{0.836}$
5-year	$Q_5 = 82.92(DA)^{0.724} (UI)^{0.751}$
10-year	$Q_{10} = 120.7(DA)^{0.735} (UI)^{0.697}$
25-year	$Q_{25} = 184.8(DA)^{0.745} (UI)^{0.632}$
50-year	$Q_{50} = 246.4(DA)^{0.752} (UI)^{0.587}$
100-year	$Q_{100} = 362.1(DA)^{0.752} (UI)^{0.510}$

For these equations: DA = drainage area in mi², UI = urbanization index
 Source: USGS, 1982

2.1.8.5 TxDOT Equations for Rural (or Undeveloped) Basins

The Texas Department of Transportation (TxDOT) has a regression method for estimating peak discharges for rural basins. For a complete discussion of the development of these equations consult Chapter 5, Section 11 of the TxDOT Hydraulic Design Manual, available online at <http://manuals.dot.state.tx.us/docs/colbridg/forms/hyd.pdf> or the reference USGS, 1997.

2.1.8.6 Rural (or Undeveloped) Basin Application

Equation 2.1.29 applies to rural, uncontrolled watersheds. Figure 2.1.8-1 presents the geographic extents of each region. Note that most of the NCTCOG region lies within Region 7, with small portions of Region 3 and 4. Table 2.1.8-2 presents the coefficients and limits of applicability for Regions 3, 4, and 7. Generally, use this equation to compare with the results of other methods, check existing structures, or where it is not practicable to use any other method, keeping in mind the importance of the facility being designed.

$$Q_T = aA^b SH^c SL^d \tag{2.1.29}$$

where:

Q_T = T-year discharge (cfs)

A = contributing drainage area (sq. mi.)

SH = basin-shape factor defined as the ratio of main channel length squared to contributing drainage area (sq. mi./sq. mi.)

SL = mean channel slope defined as the ratio of headwater elevation of longest channel minus main channel elevation at site to main channel length (ft./mi.). Note: This differs from previous rural regression equations in which slope was defined between points 10 and 85 percent of the distance along the main channel from the outfall to the basin divide.

a, b, c, d = multiple linear regression coefficients dependent on region number and frequency.

The equations to be used for Regions 3, 4, and 7 are found in Table 2.1.8-2.

Regions 3, 4, and 7 have two sets of coefficients. For these regions, if the drainage area is between 10 and 100 sq. mi., determine a weighted discharge (Q_w) as shown in Equation 2.1.30.

$$Q_w = (2 - \log(A/z))Q_1 + (\log(A/z)-1)Q_2 \quad (2.1.30)$$

where:

Q_w = weighted discharge (cfs)

A = contributing drainage area (sq. mi.)

z = 1.0 for English measurements units

Q_1 = discharge based on regression coefficients for $A < 32$ sq. mi. (cfs)

Q_2 = discharge based on regression coefficients for $A \geq 32$ sq. mi. (cfs)

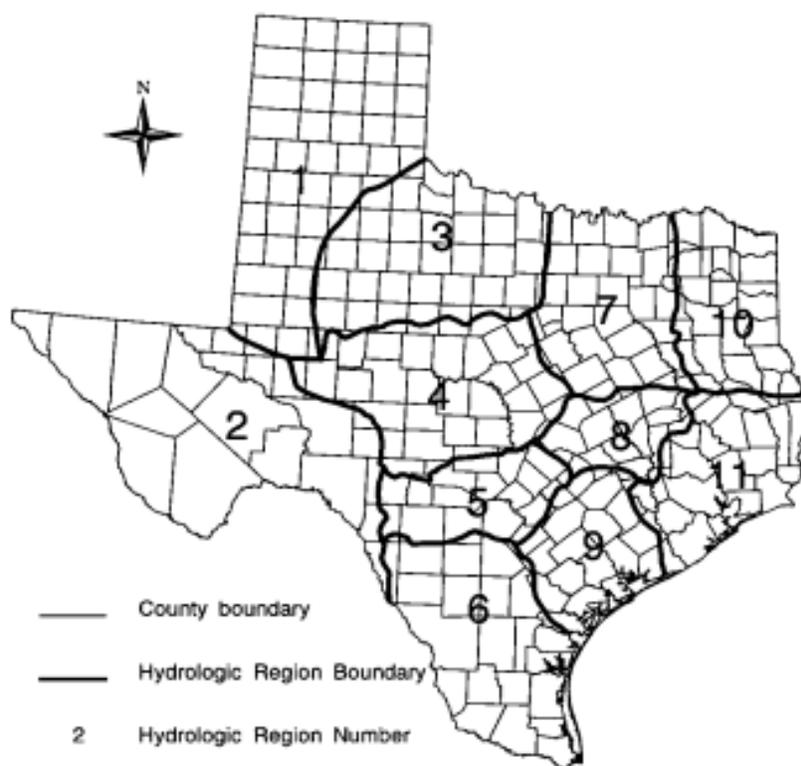


Figure 2.1.8-1 Hydrologic Regions for Statewide Rural Regression Equations

Source: TxDOT, 2002

Table 2.1.8-2 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas ¹		
[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]		
Hydrologic region and recurrence interval	Weighted least-squares regression equation for corresponding recurrence interval	Range of indicated independent variables in corresponding region (units as noted)
Region 3 (sites with contributing drainage area less than 32 square miles) ²		
2 yr	$Q_2 = 119 A^{.592}$	A: 0.10 to 97.0
5 yr	$Q_5 = 252 A^{.629}$	
10 yr	$Q_{10} = 373 A^{.652}$	SH: 0.16 to 9.32
25 yr	$Q_{25} = 566 A^{.679}$	
50 yr	$Q_{50} = 743 A^{.698}$	SL: 10.7 to 105
100 yr	$Q_{100} = 948 A^{.715}$	
Region 3 (sites with contributing drainage area greater than 32 square miles) ²		
2 yr	$Q_2 = 8.05 A^{.668} SL^{.659} SH^{.189}$	A: 11.8 to 14,635
5 yr	$Q_5 = 4.20 A^{.626} SL^{.574}$	
10 yr	$Q_{10} = 91.9 A^{.579} SL^{.537}$	SH: 1.71 to 75.0
25 yr	$Q_{25} = 233 A^{.523} SL^{.476}$	
50 yr	$Q_{50} = 448 A^{.484} SL^{.425}$	SL: 4.81 to 36.3
100 yr	$Q_{100} = 835 A^{.447} SL^{.372}$	
Region 4 (sites with contributing drainage area less than 32 square miles) ²		
2 yr	$Q_2 = 97.1 A^{.626}$	A: 0.19 to 81.1
5 yr	$Q_5 = 196 A^{.650} SH^{.257}$	
10 yr	$Q_{10} = 293 A^{.697} SH^{.281}$	SH: 0.05 to 6.52
25 yr	$Q_{25} = 455 A^{.741} SH^{.311}$	
50 yr	$Q_{50} = 53 A^{.927} SL^{.558} SH^{.333}$	SL: 13.5 to 226
100 yr	$Q_{100} = 51 A^{.968} SL^{.627} SH^{.353}$	
Region 4 (sites with contributing drainage area greater than 32 square miles) ²		
2 yr	$Q_2 = 0.00660 A^{1.29} SL^{2.09}$	A: 12 to 19,819
5 yr	$Q_5 = 0.0212 A^{1.24} SL^{2.18}$	
10 yr	$Q_{10} = 0.0467 A^{1.20} SL^{2.18}$	SH: 0.49 to 19.7
25 yr	$Q_{25} = 0.102 A^{1.16} SL^{2.18}$	
50 yr	$Q_{50} = 0.166 A^{1.13} SL^{2.19}$	SL: 3.52 to 36.1
100 yr	$Q_{100} = 0.252 A^{1.11} SL^{2.19}$	
Region 7 (sites with contributing drainage area less than 32 square miles) ²		
2 yr	$Q_2 = 832 A^{.568} SL^{-.285}$	A: 0.20 to 78.7
5 yr	$Q_5 = 584 A^{.610}$	
10 yr	$Q_{10} = 831 A^{.592}$	SH: 0.037 to 36.6
25 yr	$Q_{25} = 1196 A^{.576}$	
50 yr	$Q_{50} = 1505 A^{.566}$	SL: 7.25 to 116
100 yr	$Q_{100} = 1842 A^{.558}$	

Table 2.1.8-2 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas ¹		
[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]		
Hydrologic region and recurrence interval	Weighted least-squares regression equation for corresponding recurrence interval	Range of indicated independent variables in corresponding region (units as noted)
Region 7 (sites with contributing drainage area greater than 32 square miles) ²		
2 yr	$Q_2 = 129 A^{.578} SL^{.364}$	A: 13 to 2,615
5 yr	$Q_5 = 133 A^{.605} SL^{.578}$	
10 yr	$Q_{10} = 178 A^{.644} SL^{.699} SH^{-.239}$	SH: 1.66 to 36.6
25 yr	$Q_{25} = 219 A^{.651} SL^{.776} SH^{-.267}$	
50 yr	$Q_{50} = 261 A^{.653} SL^{.817} SH^{-.291}$	SL: 3.85 to 31.9
100 yr	$Q_{100} = 313 A^{.654} SL^{.849} SH^{-.316}$	
1. Source: U.S.G.S., 1997, pp. 62-65.		
2. Use Equation 2.1.29 to calculate a weighted discharge for streams with contributing drainage area falling within the arrange of 10 to 100 square miles.		

2.1.8.7 Example Problem

For the 100-year flood, calculate the peak discharge for a rural drainage area located in Region 7 on Timber Creek near Collinsville, Texas.

- Drainage Area = 38.8 mi²
- Main Channel Slope = 13.13 ft/mi
- Main Channel Length= 14.24 mi.
- Shape Factor = (channel miles)² divided by Area = 5.23

Peak Discharge Calculations

The 100-year Rural Peak Discharge determination will necessitate the use of Equations 2.1.29 and 2.1.30 because the drainage is in the range of 10-100.

$$Q_1 = 1842 A^{.558} = 1842(38.8)^{.558} = 14,186 \text{ cfs}$$

$$\begin{aligned} Q_2 &= 313 A^{.654} SL^{.849} SH^{-.316} \\ &= 313(38.8)^{.654} (13.13)^{.849} (5.23)^{-.316} \\ &= 18,072 \text{ cfs} \end{aligned}$$

$$\begin{aligned} Q_{100} &= (2 - \log(A))Q_1 + (\log(A) - 1)Q_2 \\ &= (2 - \log(38.8))14,186 + (\log(38.8) - 1) 18,072 \text{ cfs} \\ &= 16,474 \text{ cfs} \end{aligned}$$

2.1.9 Downstream Hydrologic Assessment

The purpose of the Streambank Protection and Flood Control criteria is to protect downstream properties from flood and erosion impacts due to upstream development. Some Local Criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff.

Due to a site's location within a watershed, there may be very little reason for requiring flood control from a particular site. In certain circumstances where detention is in place or a master drainage plan has been adopted, a development may receive or plan to receive less than ultimate developed flow conditions from upstream. This might be considered in the detention needed and its influence on the downstream assessment. Any consideration in such an event would be with the approval of the local authority. This section outlines a suggested procedure for determining the impacts of post-development storm water peak flows and volumes that a community may require as part of a developer's storm water management site plan.

2.1.9.1 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 2.1.9-1. The peak flow from the site is reduced appropriately, but 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.

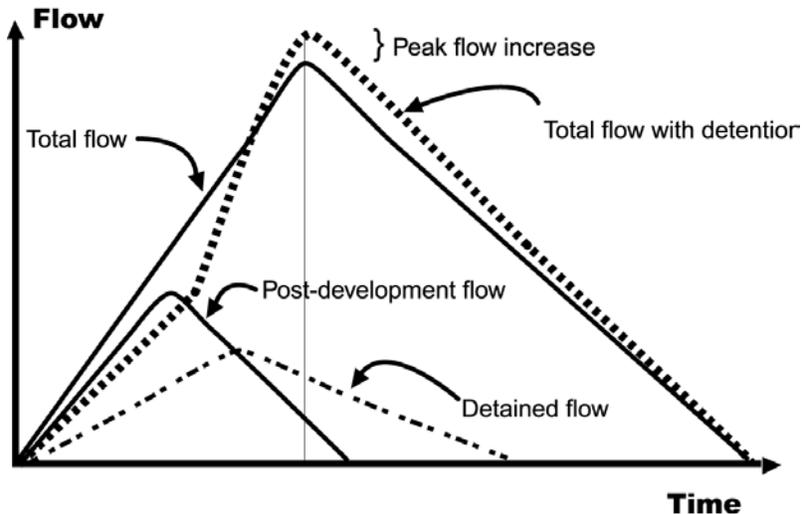


Figure 2.1.9-1 Detention Timing Example

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 bottom 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 of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

Figure 2.1.9-2 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.

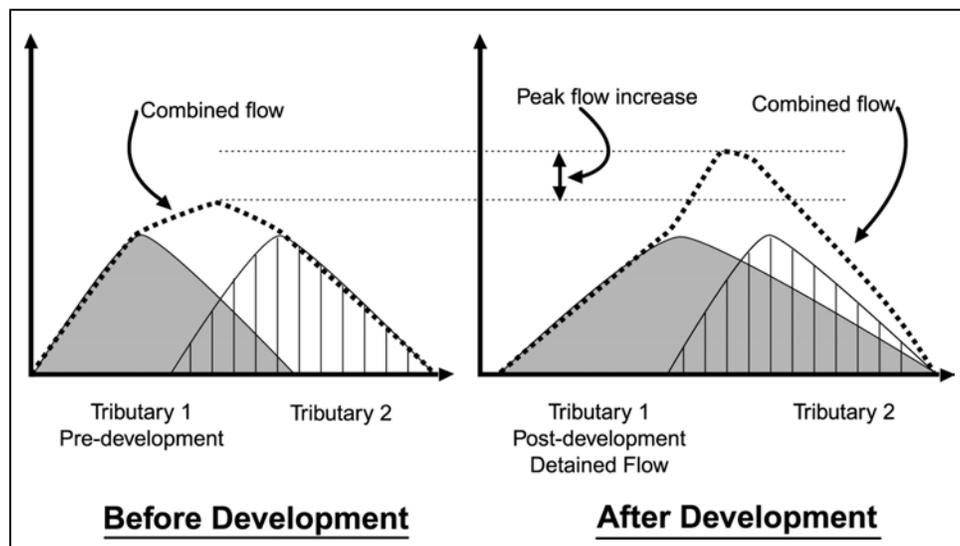


Figure 2.1.9-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

2.1.9.2 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on storm water peak flows and velocities are evaluated downstream. The assessment extends from an outfall of a development to a point downstream, determined by one of two methods:

- *Zone of Influence* – Point downstream where the discharge from a proposed development no longer has a significant impact upon the receiving stream or storm drainage system
- *Adequate Outfall* – Location of acceptable outfall that does not create adverse flooding or erosion conditions downstream

These methods recognize the fact that a structural control providing detention has a “zone of influence” downstream where its effectiveness can be felt. Beyond this zone of influence the storm water effects of a structural control become relatively small and 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 be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. This is known as the *10% Rule*. As an example, if a structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in a downstream assessment include:

1. Determine the outfall location of the site and the pre- and post-development site conditions.
2. Using a topographic map determine a preliminary lower limit of the zone of influence (approximately 10% point).
3. Using a hydrologic model determine the pre-development peak flows and velocities at each junction beginning at the development outfall and ending at the next junction beyond the 10% point. Undeveloped off-site areas are modeled as “full build-out” for both the pre- and post-development analyses. The discharges and velocities are evaluated for three storms:
 1. “Streambank Protection” storm, either the 1- or 2- year, 24-hour event
 1. “Conveyance” storm of either the 5-, 10-, or 25-year, 24-hour event
 1. 100-year, 24-hour storm event
4. Change the land use on the site to post-development conditions and rerun the model.
5. Compare the pre- and post-development peak discharges and velocities at the downstream end of the model. If the post-developed flows are higher than the pre-developed flows for the same frequency event, or the post-developed velocities are higher than the allowable velocity of the downstream receiving system, extend the model downstream. Repeat steps 3 and 4 until the post-development flows are less than the pre-developed flows, and the post-developed velocities are below the allowable velocity. Allowable velocities are given in Table 4.4-3 in Chapter 4 of this manual.
6. If shown that no peak flow increases occur downstream, and post-developed velocities are allowable, then the control of the flood protection volume (Q_f) can be waived by the local authority. The developer saves the cost of sizing a detention basin for flood control. In this case the downstream assessment saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
7. If peak discharges are increased due to development, or if downstream velocities are erosive, one of the following options are required.
 - Document that existing downstream conveyance is adequate to convey post-developed storm water discharges (Option 1 for Streambank Protection and Flood Control)
 - Work with the local government to reduce the flow elevation and/or velocity through channel or flow conveyance structure improvements downstream. (Option 2 for Streambank Protection and Flood Control)
 - Design an on-site structural control facility such that the post-development flows do not increase the peak flows, and the velocities are not erosive, at the outlet and the determined junction locations.

Even if the results of the downstream assessment indicate that no downstream flood or erosion protection is required, the water quality steps of the *integrated* Design Approach will still need to be addressed.

2.1.9.3 Example Problem

Figure 2.1.9-3 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) storm water pond. The flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked “80 acres.” The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase or erosive velocities at the 80-acre point then the same will be true through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single, "full build-out" condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase, and velocities are not erosive, at junction points downstream to the 80-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 TR-20 model of the watershed. The model shows a detention facility, in this case, will actually increase the peak flow in the stream.

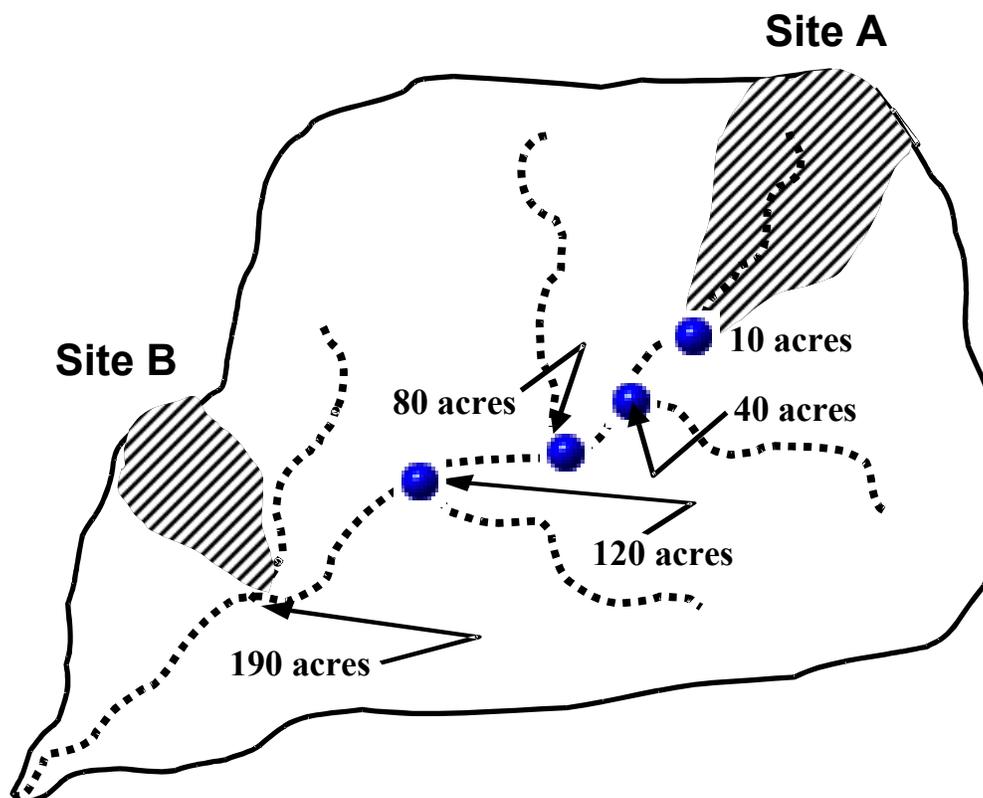


Figure 2.1.9-3 Example of the Ten-Percent Rule

2.1.10 Water Quality Protection Volume and Peak Flow

2.1.10.1 Water Quality Protection Volume Calculation

The Water Quality Protection Volume (WQ_v) is the treatment volume required to remove a significant percentage of the storm water pollution load, defined in this Manual as a 70 - 80% removal of the average annual post-development total suspended solids (TSS) load. This is achieved by intercepting and

treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year. The WQ_v should not be discharged in a period less than 24 hours. Detention times greater than 24 hours would be more effective. Procedures for computing the release rate are found in Section 4.6.3.

The water quality protection volume is calculated by multiplying the 85th percentile annual rainfall event by the volumetric runoff coefficient (R_v) and the site area. R_v is defined as:

$$R_v = 0.05 + 0.009(I) \quad (2.1.31)$$

where:

I = percent of impervious cover (%)

For North Central Texas, the average 85th percentile annual rainfall event is 1.5 inches. Therefore, WQ_v is calculated using the following formula:

$$WQ_v = \frac{1.5 R_v A}{12} \quad (2.1.32)$$

where:

WQ_v = water quality protection volume (acre-feet)

R_v = volumetric runoff coefficient

A = total drainage area (acres)

WQ_v can be expressed in inches (Q_{wv}) using the following formula:

$$Q_{wv} = 1.5(R_v) \quad (2.1.33)$$

where:

Q_{wv} = water quality protection volume (inches)

2.1.10.2 Water Quality Protection Volume Peak Flow Calculation

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as for sand filters and infiltration trenches. An arbitrary storm would need to be chosen using the Rational Method, as conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the treatment practice due to an inadequately sized diversion structure and leads 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 simplified peak flow estimating method above. A brief description of the calculation procedure is presented below.

(Step 1) Using Q_{wv} and equation 2.1.8, a corresponding Curve Number (CN) is computed utilizing the following equation:

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

where:

P = rainfall, in inches (use 1.5 inches for the iSWM Water Quality Storm)

Q_{wv} = Water Quality Protection Volume, in inches ($1.5R_v$)

(Step 2) Once a CN is computed, the time of concentration (t_c) is computed (based on the methods described in this section).

(Step 3) Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the water quality storm event is computed using a slight modification of the Simplified SCS Peak Runoff Rate Estimation technique of subsection 2.1.5.7. Use Type II rainfall distribution for North Central Texas.

(Step 4) Read initial abstraction (I_a) from Table 2.1.5-3, compute I_a/P

(Step 5) Read the unit peak discharge (q_u) from Figure 2.1.5-6 for appropriate t_c

- Using Q_{wv} , compute the peak discharge (Q_{wq})

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

where:

Q_{wq} = the water quality peak discharge (cfs)

q_u = the unit peak discharge (cfs/mi²/inch)

A = drainage area (mi²)

Q_{wv} = Water Quality Protection Volume, in inches ($1.5R_v$)

2.1.10.3 Example Problem

Using the data and information from the example problem in subsection 2.1.5.8 calculate the water quality volume and the water quality peak flow.

Calculate water quality protection volume (Q_{wv})

Compute volumetric runoff coefficient, R_v

$$R_v = 0.05 + (0.009)(I) = 0.05 + (0.009)(18/50 \times 100\%) = 0.37$$

Compute water quality protection volume, Q_{wv}

$$Q_{wv} = 1.5(R_v)(A)/12 = 1.5(.37)(50)/12 = 2.31 \text{ acre-feet}$$

Calculate water quality peak flow

Compute runoff volume in inches, Q_{wv} , where $P = 1.5$ inches:

$$Q_{wv} = 1.5 R_v = 1.5 * 0.37 = 0.56 \text{ inches}$$

Compute curve number:

$$\begin{aligned} CN &= 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25 Q_{wv} P)^{1/2}] \\ CN &= 1000/[10 + 5*1.5 + 10*0.56 - 10(0.56^2 + 1.25*0.56*1.5)^{1/2}] \\ &= 88 \end{aligned}$$

$t_c = 0.35$ (computed previously)

$$S = 1000/CN - 10 = 1000/84 - 10 = 1.36 \text{ inches}$$

$$0.2S = I_a = 0.27 \text{ inches}$$

$$I_a/P = 0.27/1.5 = 0.18$$

Find q_u :

$$\text{From Figure 2.1.5-6 for } I_a/P = 0.18 \quad q_u = 580 \text{ cfs/mi}^2/\text{in}$$

Compute water quality peak flow:

$$Q_{wq} = q_u * A * Q_{wv} = 580 * 50/640 * 0.56 = 25.4 \text{ cfs}$$

2.1.11 Streambank Protection Volume Estimation

2.1.11.1 Streambank Protection Volume Calculation

The Simplified SCS Peak Runoff Rate Estimation approach (see subsection 2.1.5.7) can be used for estimation of the Streambank Protection Volume (SP_v) for storage facility design.

This method should not be used for standard detention design calculations. See the modified rational method in Section 2.1.7 for preliminary detention calculations without formal routing or the SCS Hydrologic Method in Section 2.1.5.

For SP_v estimation, using Figure 2.1.5-6 in Section 2.1.5, the unit peak discharge (q_u) can be determined based on I_a/P and time of concentration (t_c). Knowing q_u and T (extended detention time, typically 24 hours), the q_o/q_i ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 2.1.11-1.

Using the following equation from TR-55 for a Type II rainfall distribution, V_s/V_r can be calculated.

Note: Figure 2.1.11-2 can also be used to estimate V_s/V_r .

$$V_s/V_r = 0.682 - 1.43 (q_o/q_i) + 1.64 (q_o/q_i)^2 - 0.804 (q_o/q_i)^3 \quad (2.1.34)$$

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)

The required storage volume can then be calculated by:

$$V_s = \frac{(V_s/V_r)(Q_d)(A)}{12} \quad (2.1.35)$$

where:

- V_s and V_r are defined above
- Q_d = the developed runoff for the design storm (inches)
- A = total drainage area (acres)

While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 25-year storm.

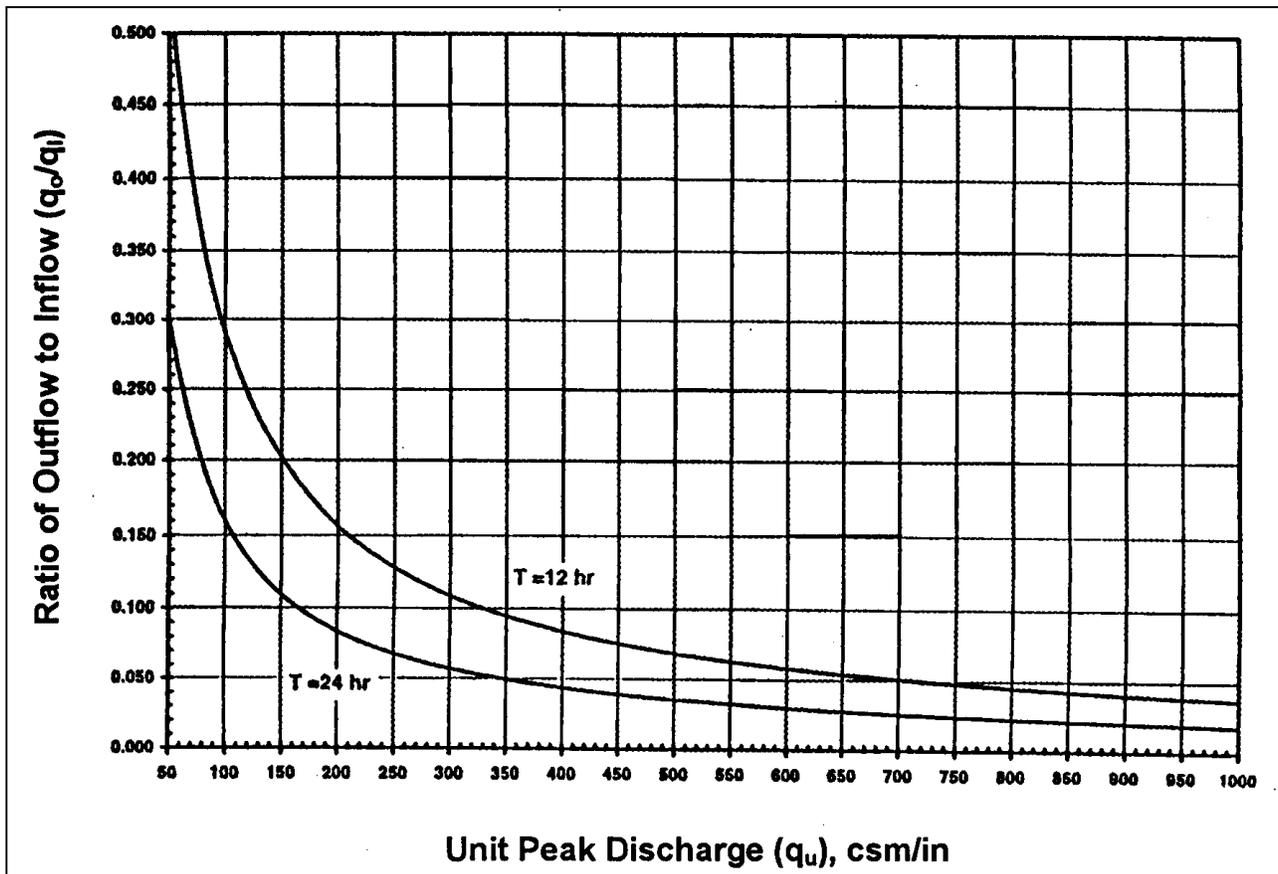


Figure 2.1.11-1 Detention Time vs. Discharge Ratios
 (Source: MDE, 1998)

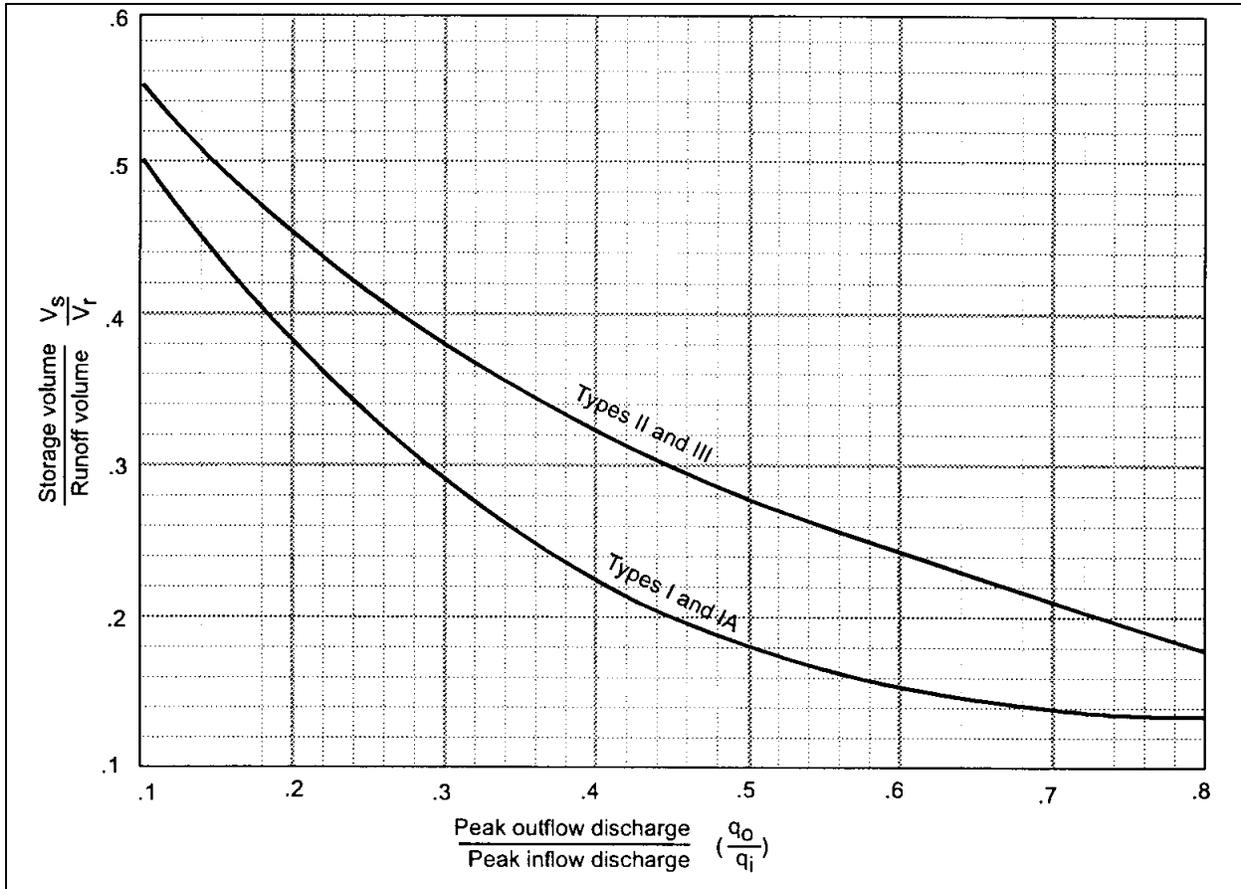


Figure 2.1.11-2 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
 (Source: TR-55, 1986)

2.1.11.2 Example Problem

Compute the Streambank Protection Volume (SP_v) for the 50-acre watershed in example 2.1.5.8.

Computations

8. Calculate rainfall excess:

- The 1-year, 24 hour rainfall is 2.64 inches (0.11 in/hr x 24 hours – From Appendix A, Table A-16).
- Composite area-weighted Curve Number is 83.
- From equation 2.1.7, Q_d (1-year developed) = 1.2 inches

9. Calculate time of concentration

$t_c = 20.86$ minutes (.35 hours)

10. Calculate I_a/P for CN = 83; $I_a = .410$ (Table 2.1.5-3)

$I_a/P = (.410 / 2.64) = .155$ (Note: Use straight-line interpolation to facilitate use of Figure 2.1.5-6.)

11. Find unit discharge q_u :

From Figure 2.1.5-6 for $I_a/P = .155$ and $t_c = .35$ hr
 $q_u = 600$ csm/in

12. Find discharge ratio q_o/q_i :

From Figure 2.1.11-1 for $q_u = 600$ csm/in and $T = 24$ hr
 $q_o/q_i = 0.03$

13. Calculate streambank protection volume ($SP_v = V_s$)

For a Type II rainfall distribution,

$$V_s/V_r = 0.682 - 1.43 (q_o/q_i) + 1.64 (q_o/q_i)^2 - 0.804 (q_o/q_i)^3$$

$$V_s/V_r = 0.682 - 1.43 (0.03) + 1.64 (0.03)^2 - 0.804 (0.03)^3 = 0.64$$

Therefore, streambank protection volume with Q_d (1-year developed) = 1.2 inches, from Step 1, is

$$SP_v = V_s = (V_s/V_r)(Q_d)(A)/12 = (0.64)(1.2)(50)/12 = 3.20 \text{ acre-feet}$$

2.1.12 Water Balance Calculations

2.1.12.1 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 may be advisable for retention pond and wetland 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 pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

2.1.12.2 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential):

$$\Delta V = \Sigma I - \Sigma O \quad (2.1.36)$$

where:

- Δ = "change in"
- V = pond volume (ac-ft)
- Σ = "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, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 2.1.36 can be changed to reflect these factors.

$$\Delta V = P + R_o + B_f - I - E - E_t - O_f \quad (2.1.37)$$

where:

- V = volume (ac-ft)
- P = precipitation (ac-ft) = (Rainfall in Inches times area in acres divided by 12)
- R_o = runoff (ac-ft)
- B_f = baseflow (ac-ft)
- I = infiltration (ac-ft) (Use Equation 2.1.33)

- E = evaporation (ac-ft) (Surface evaporation in feet times surface area)
- E_t = evapotranspiration (ac-ft)
- O_f = overflow (ac-ft)
- Δ = “change in” (+ gain; - loss)

Rainfall (P) – Monthly rainfall values can be obtained from National Weather Service climatology data at:

<http://www.srh.noaa.gov/fwd/ntexclima.html>

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) and divided by 12, it becomes acre-feet of volume. Table 2.1.12-1 shows monthly rainfall rates for the Dallas-Fort Worth area based on a 30-year period of record at Dallas-Fort Worth International Airport.

Runoff (R_o) – 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. 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. Two other methods have been proposed.

Equation 2.1.31 gives the volumetric coefficient (R_v) of the drainage area. If it can be assumed that the average storm producing runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called “initial abstractions”) are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Texas, this is equivalent of about a 10% runoff volume loss. Thus a factor of 0.9 should be applied to the calculated R_v value to account for storms producing no runoff. Equation 2.1.38 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$R_o = 0.9(P/12)R_vA \tag{2.1.38}$$

where:

- P = precipitation (in)
- R_o = runoff volume (acre-ft)
- R_v = volumetric runoff coefficient [see equation 2.1.31]
- A = Area in acres

	<u>Jan</u>	<u>Feb</u>	<u>Mar</u>	<u>Apr</u>	<u>May</u>	<u>Jun</u>	<u>Jul</u>	<u>Aug</u>	<u>Sep</u>	<u>Oct</u>	<u>Nov</u>	<u>Dec</u>
Precipitation (in)	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57
Annual Precipitation (in)								34.73				

Source: National Weather Service, 2002

Baseflow (B_f) – Most storm water ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks. Consideration may also have to be given to irrigation return flow in certain areas.

Infiltration (I) – Infiltration is a very complex subject and cannot be covered in detail here. 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. The infiltration rate is governed by the Darcy equation as:

$$I = A k_h G_h \quad (2.1.39)$$

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

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate Table 2.1.12-2 can be used.

Table 2.1.12-2 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 Storm Water Management," 1990

Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, 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. Pan evaporation methods are also used. A pan coefficient of 0.7 is commonly used to convert the higher pan value to the lower lake values.

Table 2.1.12-3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information for Grapevine, Texas. Figure 2.1.12-1 depicts a map of annual free water surface (FWS) evaporation averages for Texas based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural storm water ponds and wetlands being designed in Texas. Total annual values can be estimated from this map and distributed according to Table 2.1.12-3.

Table 2.1.12-3 Evaporation Monthly Distribution – Grapevine, Texas as a % of Annual Total											
Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
3.1%	4.0%	7.2%	8.7%	10.3%	12.4%	14.5%	13.9%	9.8%	7.4%	4.9%	3.9%

Evapotranspiration (E_t) Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of E_t for crops in Texas is well documented and has become standard practice. However, for wetlands the estimating methods are not 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 typical hydrology textbooks or from the web site mentioned above.

Overflow (O_f) – Overflow is considered as excess runoff, and in water balance design is either not considered, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in pond design.

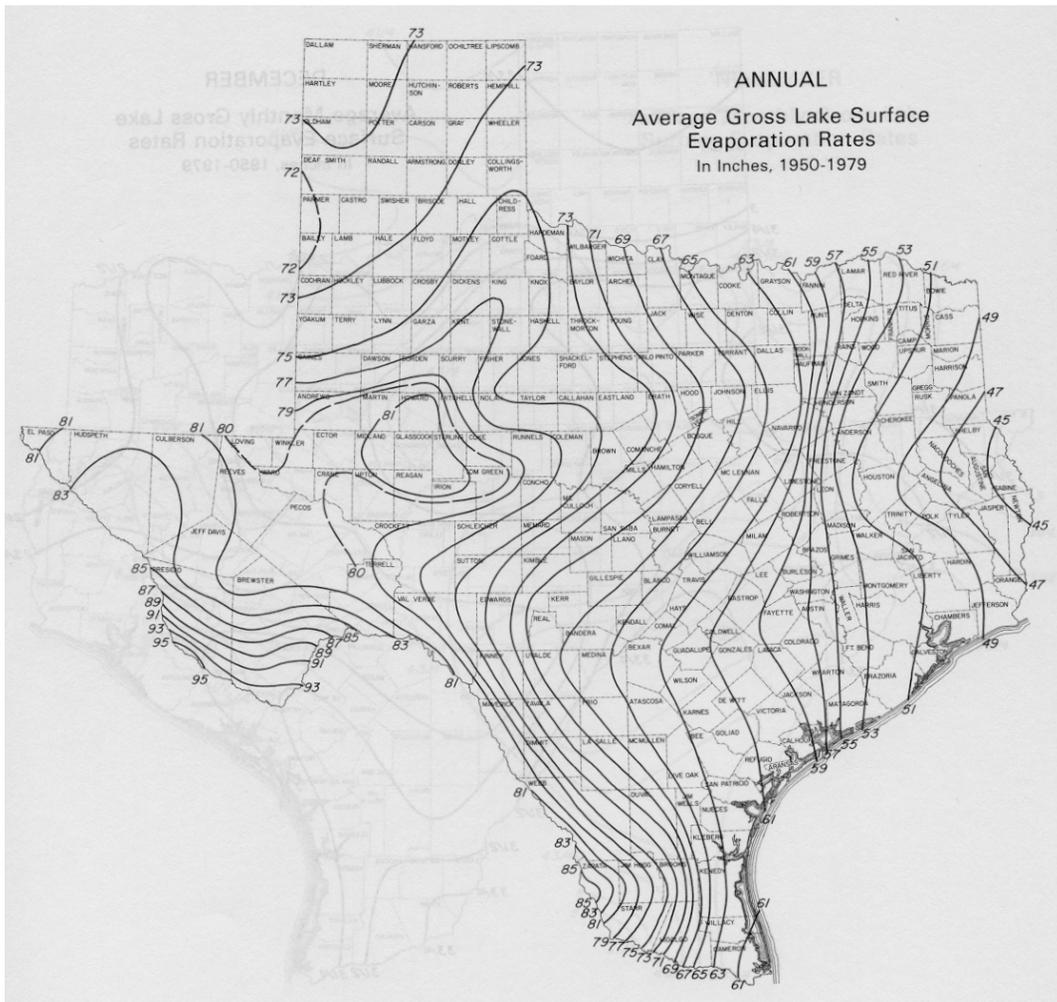


Figure 2.1.12-1 Average Annual Free Water Surface Evaporation (in inches)
 (Source: NOAA, 1982)

2.1.12.3 Example Problem

A 26-acre site in North Dallas is being developed along with an estimated 0.5-acre surface area pond. There is no baseflow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data, we find the site is 75% impervious with clay loam soil.

- From equation 2.1.31, $R_v = 0.05 + 0.009 (75) = 0.73$. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 2.1.12-1 is about 64 inches.
- For a clay loam the infiltration rate is $I = 0.18$ ft/day (Table 2.1.12-2).
- From a grading plan, it is known that about 10% of the total pond area is sloped greater than 1:4.
- Monthly rainfall for Dallas was found from a Web site similar to the one provided above.

Table 2.1.12-4 shows summary calculations for this site for each month of the year.

Drainage Area (Acres)	26
Pond Surface (Acres)	0.5
Volume at Overflow (Ac-Ft)	2
Watershed Efficiency	0.65
Annual Rainfall	34.73
Infiltration Rate (In/Day)	0.18 Clay Loam
% Pond Bottom Flat (Acres)	90
% Pond Bottom > 1:4 (Acres)	10
Annual Lake Evaporation (in)	64 Assume Pond Starts Full

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	1.9	2.37	3.06	3.2	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57
4	Evaporation - % of Yr	3.1	4	7.2	8.7	10.3	12.4	14.5	13.9	9.8	7.4	4.9	3.9
5	Runoff (Ac-Ft)	2.68	3.34	4.31	4.51	7.25	4.55	2.99	2.86	3.41	5.79	3.62	3.62
6	Precipitation (Ac-Ft)	0.08	0.10	0.13	0.13	0.21	0.13	0.09	0.08	0.10	0.17	0.11	0.11
7	Evaporation (Ac-Ft)	0.08	0.11	0.19	0.23	0.27	0.33	0.39	0.37	0.26	0.20	0.13	0.10
8	Infiltration (Ac-Ft)	2.65	2.39	2.65	2.57	2.65	2.57	2.65	2.65	2.57	2.65	2.57	2.65
9													
10	Balance (Ac-Ft)	0.02	0.94	1.59	1.84	4.54	1.79	0.04	-0.08	0.68	3.11	1.03	0.97
11	Running Balance (Ac-Ft)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.92	2.00	2.00	2.00	2.00

Explanation of Table line number:

1. Months of year
2. Days per month in percent
3. Monthly precipitation
4. Distribution of evaporation by month
5. 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.
6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet

7. Evaporation equals the monthly percentage of the annual gross lake evaporation in inches converted to acre-feet
8. Infiltration equals infiltration rate times 90% of the surface area plus infiltration rate times 0.5 (banks greater than 1:4) times 10% of the pond area converted to acre-feet
10. Lines 5 and 6 minus lines 7 and 8
11. Accumulated total from line 10 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design. Each pond has a unique volume at which overflows occur and it would be used for line 11. The pond volume in January should be set equal to the expected end-of-year volume.

It can be seen that, for this example, the pond has potential to maintain a wet pond in all months. Had the soil been a sandy clay loam with an infiltration rate of 0.34 inches per day, the pond would have been dry most months of the year. Excessive infiltration rates may be remedied in a number of ways including compacting the pond bottom, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area.

Climatic data for North Texas, as that in Figure 2.1.12-2, can be obtained from the following web site: <http://www.srh.noaa.gov/fwd/CLIMO/dfw/normals/dfwann.html>.

		DFW Annual Summary of Normal, Means, and Extremes													
		Rain (in.)													
	POR	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	YEAR	
Normal	30	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57	34.73	
Monthly Maximum		5.07	7.40	6.69	12.19	13.66	8.75	11.13	6.85	9.52	14.18	6.23	8.75	14.18	
Year of Occurrence	45	1998	1997	1995	1957	1982	1989	1973	1970	1964	1981	1964	1991	Oct 1981	
Minimum Monthly		T	0.15	0.10	0.11	0.95	0.40	0	T	0.09	T	0.20	0.17	0	
Year of Occurrence	45	1986	1963	1972	1987	1996	1964	1993	1980	1984	1975	1970	1981	Jul 1993	
Max in 24 hours		3.15	4.06	4.39	4.55	5.34	3.15	3.76	4.05	4.76	5.91	2.83	4.22	5.91	
Year of Occurrence	45	1998	1965	1977	1957	1989	1989	1975	1976	1965	1959	1964	1991	Oct 1959	
Number of Days with...	Precipitation > Tr.	30	6.7	6.3	7.3	7.6	8.7	6.4	4.7	4.6	7.1	6.2	6.0	6.5	78.1
	Precipitation > 0.99	30	0.3	0.5	0.7	1.2	1.4	0.9	0.7	0.8	1.1	1.4	0.6	0.4	10.0

Figure 2.1.12-2 Dallas/Fort Worth Precipitation Information

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