

# Hydrology:

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# 1.0 Hydrological Analysis

## 1.1 Estimating Runoff

### 1.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater controls. In the hydrologic analysis of a development/redevelopment site, there are a number of variable factors that affect the nature of stormwater 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 1.1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 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.

**Table 1.1 Applications of the Recommended Hydrologic Methods**

Method	Technical Manual Section	Rational Method	SCS Method	Modified Rational	Snyder's Unit Hydrograph	USGS / TXDOT Equations	iSWM Water Quality Volume Calculation
Water Quality Protection Volume (WQ <sub>v</sub> )	Section 1.2 of Water Quality						✓
Streambank Protection Volume (SP <sub>v</sub> )	Section 3.0 of Hydrology		✓		✓		
Flood Mitigation Discharge (Q <sub>f</sub> )	Section 1.3 of Criteria Manual		✓		✓	✓	
Storage Facilities	Section 2.0 of Hydraulics		✓	✓	✓		✓
Outlet Structures	Section 2.2 of Hydraulics		✓		✓		
Gutter Flow and Inlets	Section 1.2 of Hydraulics	✓					
Storm Drain Pipes	Section 1.1 of Hydraulics	✓	✓		✓		
Culverts	Section 3.3 of Hydraulics	✓	✓		✓	✓	
Bridges	Section 3.4 of Hydraulics		✓		✓		
Small Ditches	Section 3.2 of Hydraulics	✓	✓		✓		
Open Channels	Section 3.2 of Hydraulics		✓		✓	✓	
Energy Dissipation	Section 4.0 of Hydraulics		✓		✓		

<b>Method</b>	<b>Size Limitations<sup>1</sup></b>	<b>Comments</b>
Rational	0 – 100 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Modified Rational <sup>2</sup>	0 – 200 acres	Method can be used for estimating runoff volumes for storage design.
Unit Hydrograph (SCS) <sup>3</sup>	Any Size	Method can be used for estimating peak flows and hydrographs for all design applications.
Unit Hydrograph (Snyder's) <sup>4</sup>	1 acre and larger	Method can be used for estimating peak flows and hydrographs for all design applications.
TXDOT Regression Equations	10 to 100 mi <sup>2</sup>	Method can be used for estimating peak flows for rural design applications.
USGS Regression Equations	3 – 40 mi <sup>2</sup>	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 can be used for calculating the Water Quality Protection Volume (WQ <sub>v</sub> ).
<sup>1</sup> Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet). <sup>2</sup> Where the Modified Rational Method is used for conceptualizing, the engineer is cautioned that the method could underestimate the storage volume. <sup>3</sup> This refers to SCS routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology. <sup>4</sup> This refers to the Snyder's 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.*



### 1.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 1.3 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.

<b>Symbol</b>	<b>Definition</b>	<b>Units</b>
A	Drainage area or area	acres or square feet
B <sub>f</sub>	Baseflow	acre-feet
C	Runoff coefficient	-
C <sub>f</sub>	Frequency factor	-
CN	SCS-runoff curve number	-
D	Time interval	hours
E	Evaporation	ft
E <sub>t</sub>	Evapotranspiration	ft
F	Pond and swamp adjustment factor	-
G <sub>h</sub>	Hydraulic gradient	-
I or i	Rainfall intensity	in/hr
I	Percent of impervious cover	%
I	Infiltration	acre-feet
I <sub>a</sub>	Initial abstraction from total rainfall	in
k <sub>h</sub>	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient	-
NRCS	Natural Resources Conservation Service (formerly SCS)	-
O <sub>f</sub>	Overflow	acre-feet
P	Accumulated rainfall	in
P <sub>2</sub>	2-year, 24-hour rainfall	in
P <sub>w</sub>	Wetted perimeter	ft
PF	Peaking factor	-
Q	Rate of runoff	cfs (or inches)
Q <sub>i</sub>	Peak inflow discharge	cfs
Q <sub>o</sub>	Peak outflow discharge	cfs
Q <sub>p</sub>	Peak rate of discharge	cfs
Q <sub>wq</sub>	Water Quality peak rate of discharge	cfs
q	Storm runoff during a time interval	in
q <sub>u</sub>	Unit peak discharge	cfs (or cfs/mi <sup>2</sup> /inch)
R	Hydraulic radius	ft
R <sub>o</sub>	Runoff	acre-feet
R <sub>v</sub>	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 <sub>v</sub>	Streambank Protection Volume	acre-feet
T	Channel top width	ft
T <sub>L</sub>	Lag time	hours
T <sub>p</sub>	Time to peak	hours
T <sub>t</sub>	Travel time	hours
t	Time	min
t <sub>c</sub>	Time of concentration	min
TIA	Total impervious area	%

<b>Symbol</b>	<b>Definition</b>	<b>Units</b>
V	Velocity	ft/s
V	Pond volume	acre-feet
V <sub>d</sub>	Developed runoff volume	in
V <sub>f</sub>	Flood control volume	acre-feet
V <sub>r</sub>	Runoff volume	acre-feet
V <sub>s</sub>	Storage volume	acre-feet
WQ <sub>v</sub>	Water quality protection volume	acre-feet

### 1.1.3 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 16 counties which participate in the NCTCOG area (see Figure 1.1) are provided in [Section 5.0](#) 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:

$$i = b/(t + d)^e \quad (1.1)$$

where:

i = rainfall intensity (inches per hour)

t = rainfall duration (minutes)

b, d and e = parameters found at the top of each of the tables in Section 5.0.

- The tabular values in [Section 5.0](#) Rainfall Tables were determined from the new IDF curves.

Figure 1.2 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.

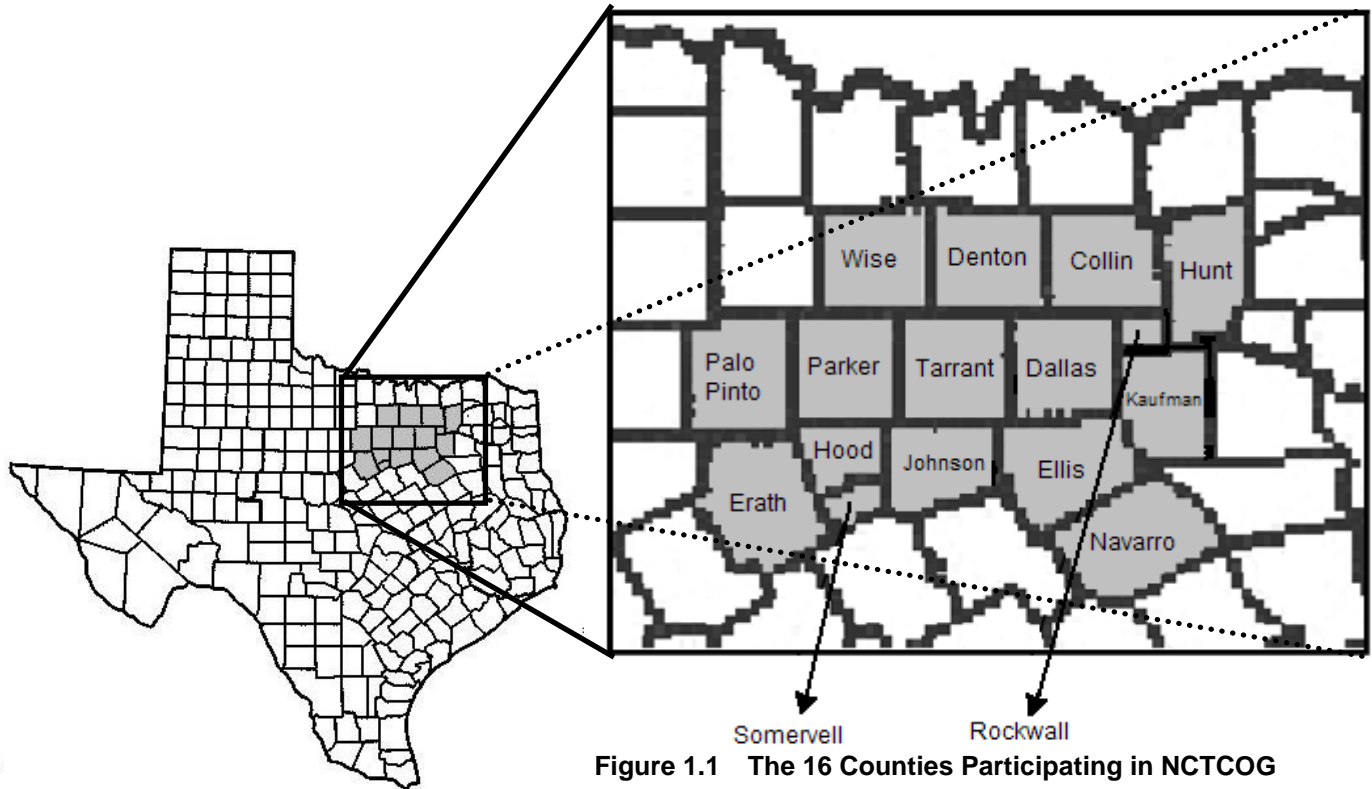


Figure 1.1 The 16 Counties Participating in NCTCOG

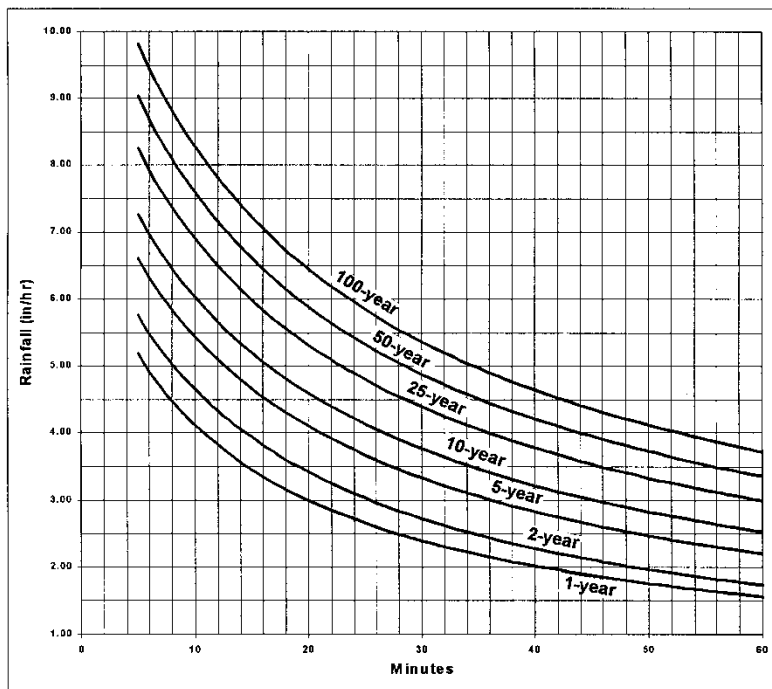


Figure 1.2 Example IDF Curve (Dallas County, Texas)

## 1.2 Rational Method

### 1.2.1 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.

### 1.2.2 Application

The Rational Method can be used to estimate stormwater 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 [Section 1.5](#). The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in [Section 1.5](#) 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.

### 1.2.3 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:

$$Q = CIA \quad (1.2)$$

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 1.6 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:

$$Q = C_f CIA \quad (1.3)$$

The  $C_f$  values that can be used are listed in Table 1.4. 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

### 1.2.4 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 1.3 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 1.3, the formula often used, which seems to match the nomograph very closely, is as follows:

$$T_c = 1.8(1.1 - C)(D)^{0.5}/(S)^{(1/3)} \quad (1.4)$$

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 1.4, and (2) Figure 1.3:  $D = 250$  ft,  $C = 0.7$ ,  $S = 0.50\%$  slope.

- Figure 1.3 gives approximately 15 minutes.
- $T_c = 1.8(1.1 - 0.7)(250)^{0.5}/(0.50)^{(1/3)} = 14.34$  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 1.4. More guidance on travel time estimation is given in [Section 1.3.6](#).

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.

Table 1.5 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.

Land Use	Minimum (minutes)	Maximum (minutes)
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.

### 1.2.5 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 [Section 5.0](#).

### 1.2.6 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 1.6 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 1.6 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.

### 1.2.7 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 1.6 to be 0.10.

**Table 1.6 Recommended Runoff Coefficient Values**

Description of Area	Runoff Coefficients (C)
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



## Time of Concentration

From Figure 1.3 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 1.4 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 5.15 in [Section 5.0](#), using a duration equal to 22 minutes,

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

## Runoff Coefficient

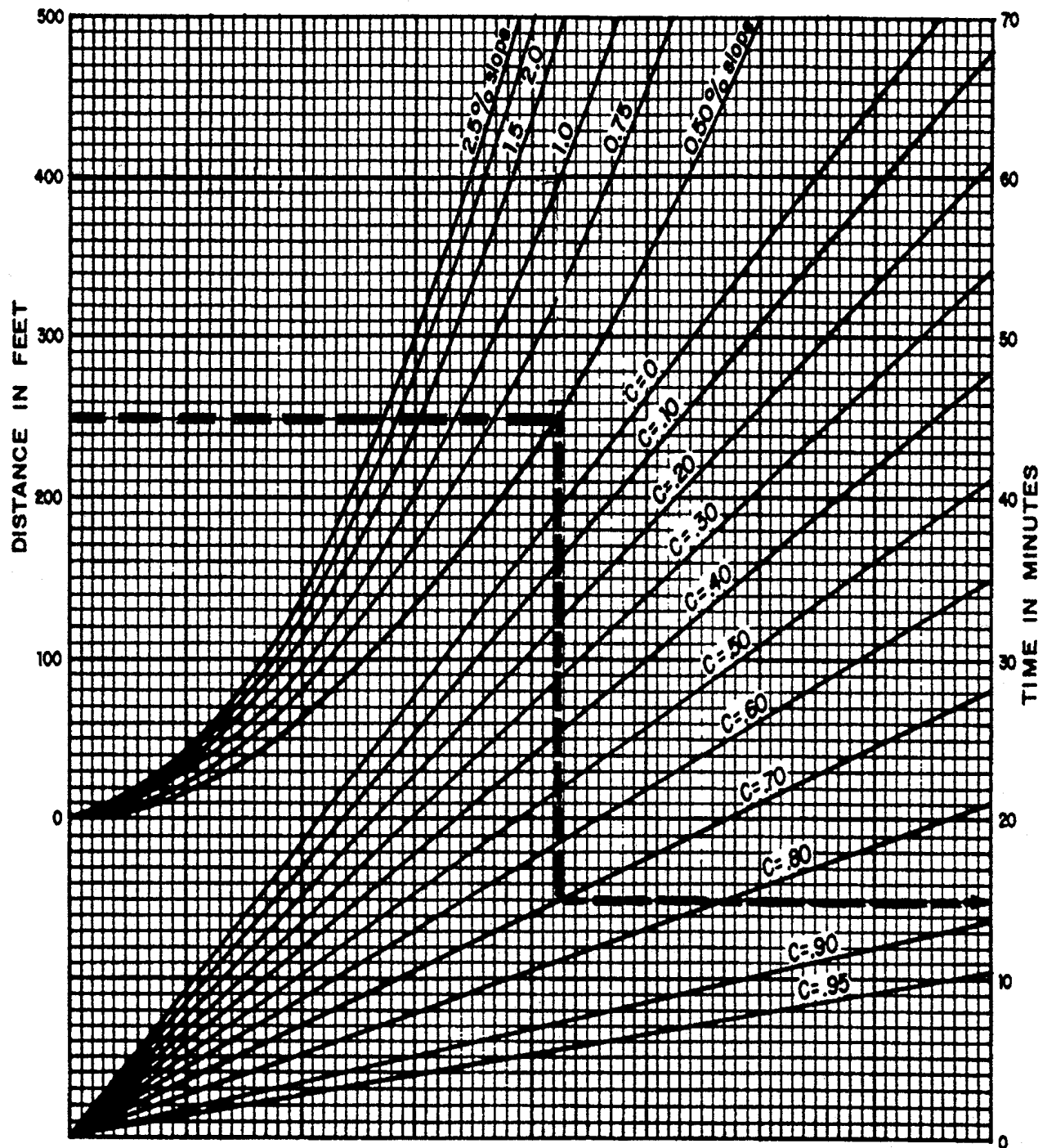
A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 1.7.

<b>Table 1.7 Weighted Runoff Coefficient</b>			
1	2	3	4
Land Use	Percent of Total Land Area	Runoff Coefficient	Weighted Runoff Coefficient*
Residential (Single Family – ¼ acre lots)	0.80	0.60	0.48
Graded area	0.20	0.30	0.06
Total Weighted Runoff Coefficient =			0.54
*Column 4 equals column 2 multiplied by column 3.			

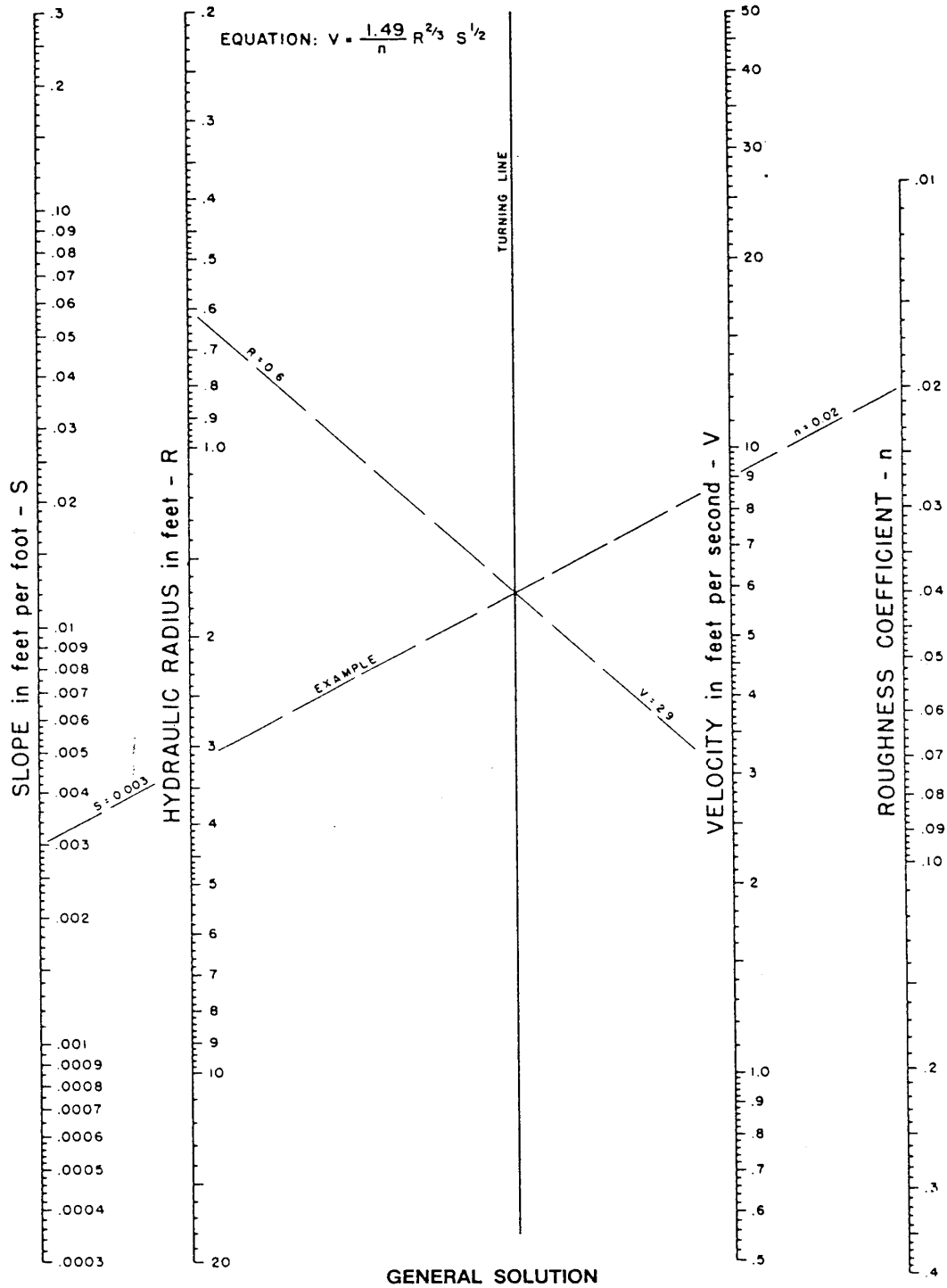
## Peak Runoff

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

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



**Figure 1.3 Rational Formula - Overland Time of Flow Nomograph**  
 (Source: Airport Drainage, Federal Aviation Administration, 1965)



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

**Figure 1.4 Manning's Equation Nomograph**  
 (Source: USDOT, FHWA, HDS-3 (1961))

## 1.3 SCS Hydrological Method

### 1.3.1 Introduction

The Soil Conservation Service<sup>1</sup> (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 1.5 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.

### 1.3.2 Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified method of [Section 1.3.7](#) 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 dissipaters.

### 1.3.3 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.

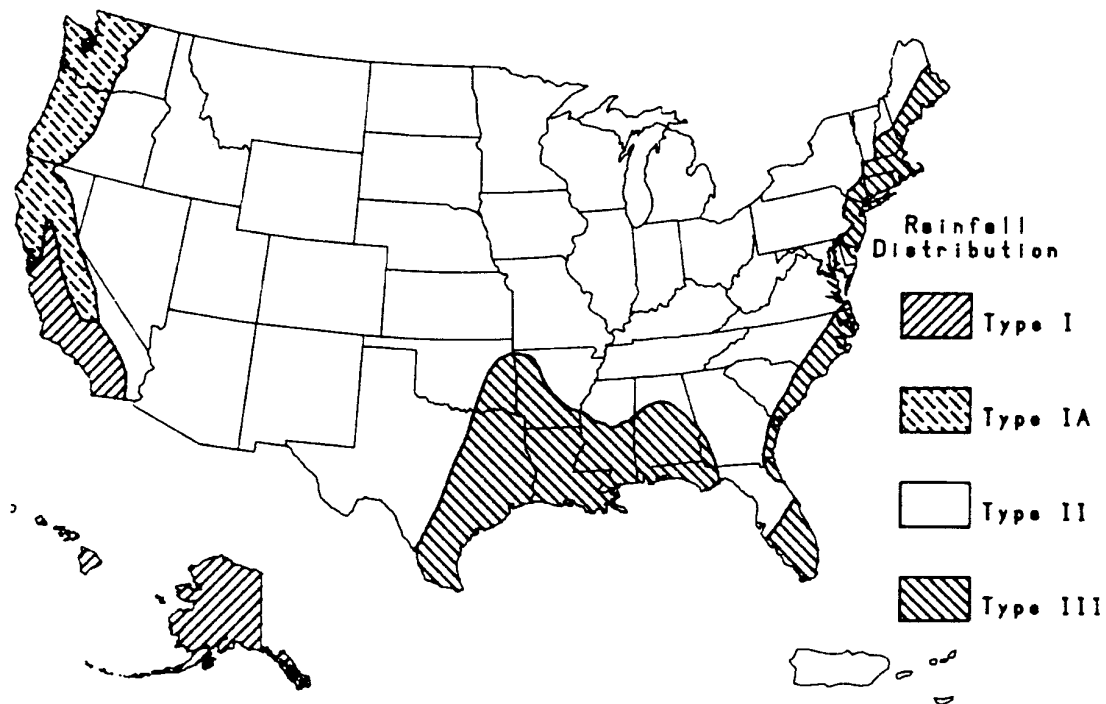
The following discussion outlines the equations and basic concepts used in the SCS method.

**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 1.5).

---

<sup>1</sup> The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)



**Figure 1.5 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:

$$Q = (P - I_a)^2 / [(P - I_a) + S] \quad (1.5)$$

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:

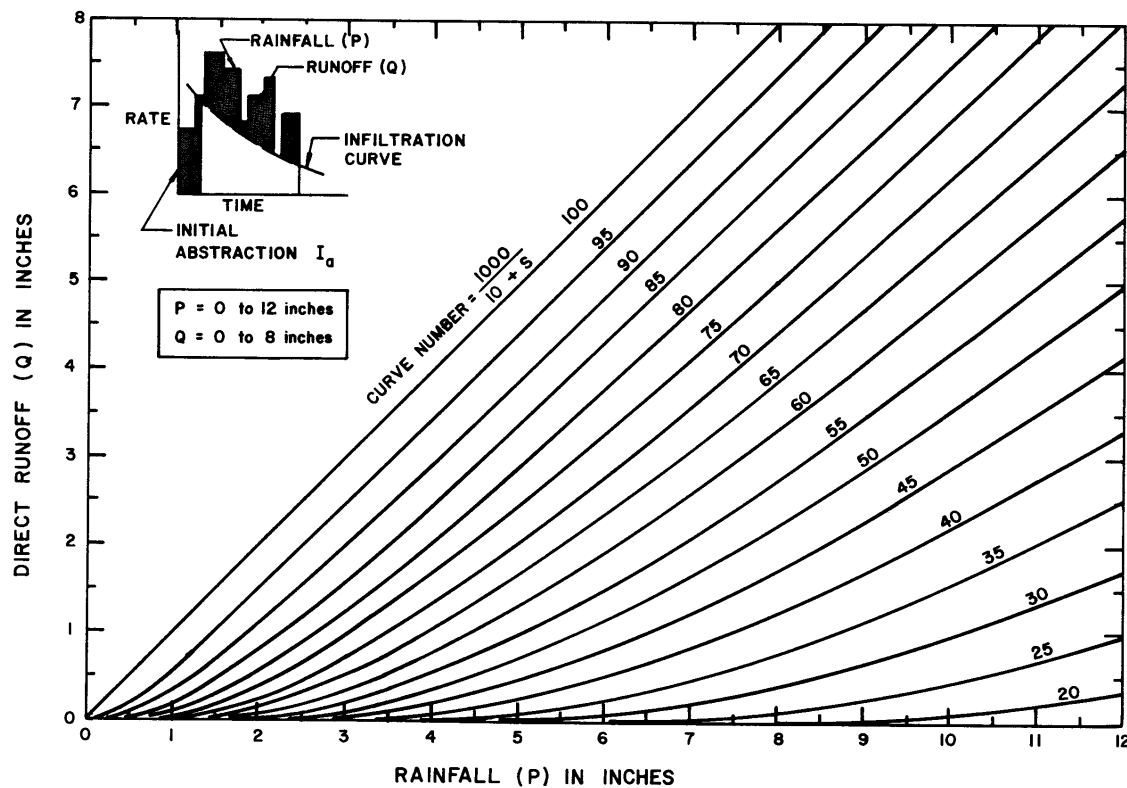
$$I_a = 0.2S \quad (1.6)$$

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 1.11 provides values of  $I_a$  for a wide range of curve numbers (CN).

Substituting  $0.2S$  for  $I_a$  in Equation 1.6, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \quad (1.7)$$

Figure 1.6 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 1.6 SCS Solution of the Runoff Equation**

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

Equation 1.7 can be rearranged so the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \quad (1.8)$$

### 1.3.4 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, 2<sup>nd</sup> 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. *Section 6.0 - Hydrologic Soils Data* 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/](http://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 1.9 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 in Table 1.8.

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.

### 1.3.5 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 1.9 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 1.9 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 1.9 are not applicable, use Figure 1.7 to compute a composite CN. For example, Table 1.9 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 1.7 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

### 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 1.8 if total impervious area is less than 30% or (2) use Figure 1.7 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 1.8 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 1.8 is 66. If all of the impervious area is connected, the resulting CN (from Figure 1.7) would be 68.

### 1.3.6 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:

$$T_t = L/3600V \quad (1.9)$$

where:

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



Cover Description		Curve numbers for hydrologic soil groups			
		A	B	C	D
<i>Cover type and hydrologic condition</i>	<i>Average percent impervious area<sup>2</sup></i>				
<b>Cultivated Land:</b>					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
<b>Pasture or range land:</b>					
Poor condition		68	79	86	89
Good condition		39	61	74	80
<b>Meadow:</b>					
Good condition		30	58	71	78
<b>Wood or forest land:</b>					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
<b>Open space (lawns, parks, golf courses, cemeteries, etc.)<sup>3</sup></b>					
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
<b>Impervious areas:</b>					
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
<b>Urban districts:</b>					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
<b>Residential districts by average lot size:</b>					
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
<b>Developing urban areas and newly graded areas (previous areas only, no vegetation)</b>					
		77	86	91	94

<sup>1</sup> Average runoff condition, and  $I_a = 0.2S$

<sup>2</sup> 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.

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

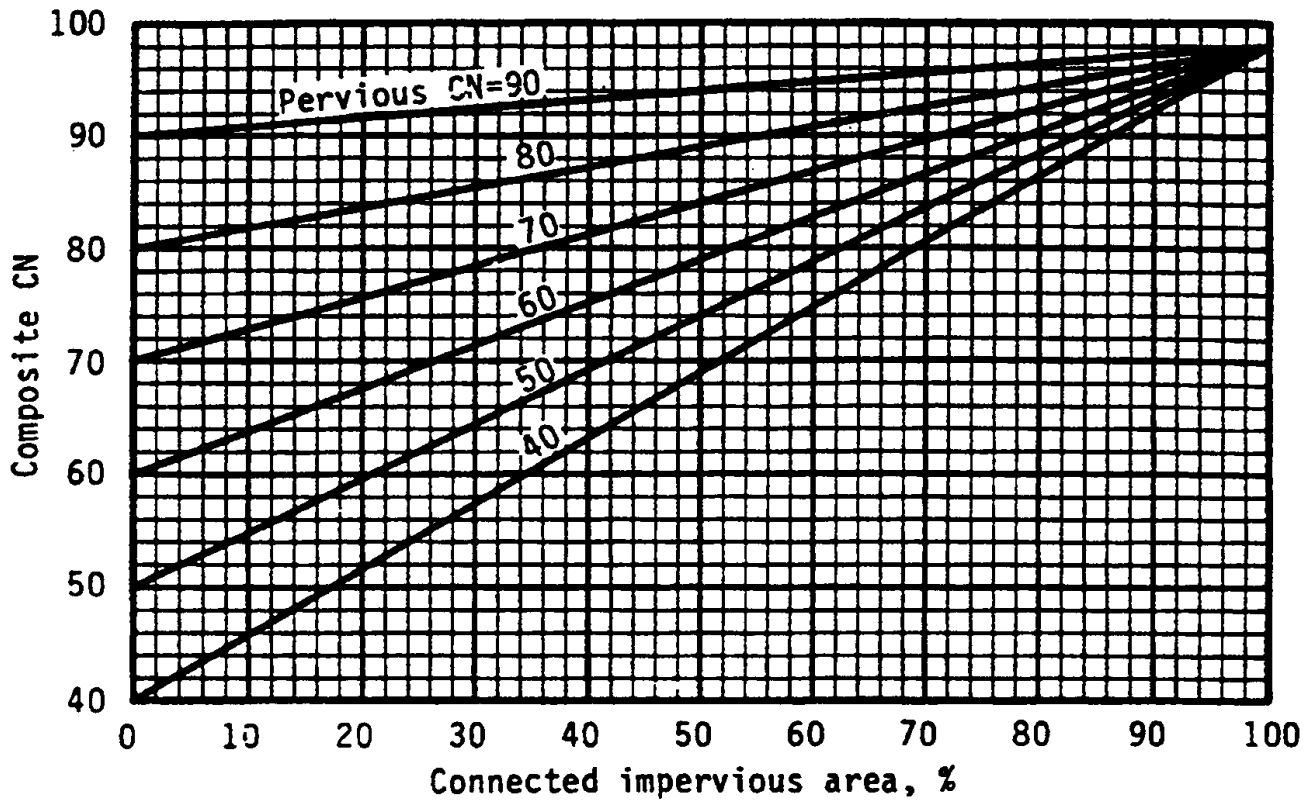


Figure 1.7 Composite CN with Connected Impervious Areas  
 (Source: SCS, TR-55, Second Edition, June 1986)

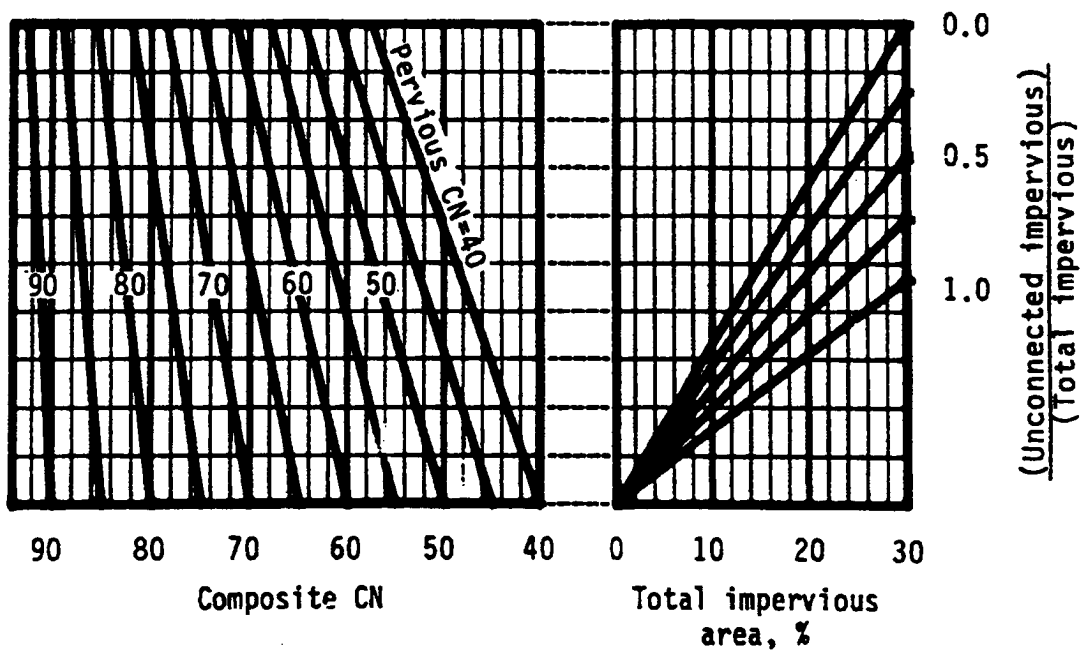


Figure 1.8 Composite CN with Unconnected Impervious Areas  
 (Total Impervious Area Less Than 30%)  
 (Source: SCS, TR-55, Second Edition, June 1986)

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42 (nL)^{0.8}}{60 (P_2)^{0.5} (S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} (S)^{0.4}} \quad (1.10)$$

where:

- T<sub>t</sub> = travel time (hr)
- n = Manning roughness coefficient (see Table 1.10)
- L = flow length (ft),
- P<sub>2</sub> = 2-year, 24-hour rainfall (in)
- S = land slope (ft/ft)

Table 1.10 Roughness Coefficients (Manning's n) for Sheet Flow <sup>1</sup>	
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 <sup>2</sup>	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods <sup>3</sup>	
Light underbrush	0.40
Dense underbrush	0.80

<sup>1</sup> The n values are a composite of information by Engman (1986).  
<sup>2</sup> Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures.  
<sup>3</sup> 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 1.9, 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 1.9, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

$$\text{Unpaved} \quad V = 16.13(S)^{0.5} \quad (1.11)$$

$$\text{Paved} \quad V = 20.33(S)^{0.5} \quad (1.12)$$

where:

- V = average velocity (ft/s)
- S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 1.9 or Equations 1.11 or 1.12, use Equation 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

$$V = (1.49/n) (R)^{2/3} (S)^{1/2} \quad (1.13)$$

where:

- V = average velocity (ft/s)
- R = hydraulic radius (ft) and is equal to  $A/P_w$
- A = cross sectional flow area (ft<sup>2</sup>)
- $P_w$  = 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 1.13,  $T_t$  for the channel segment can be estimated using Equation 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.

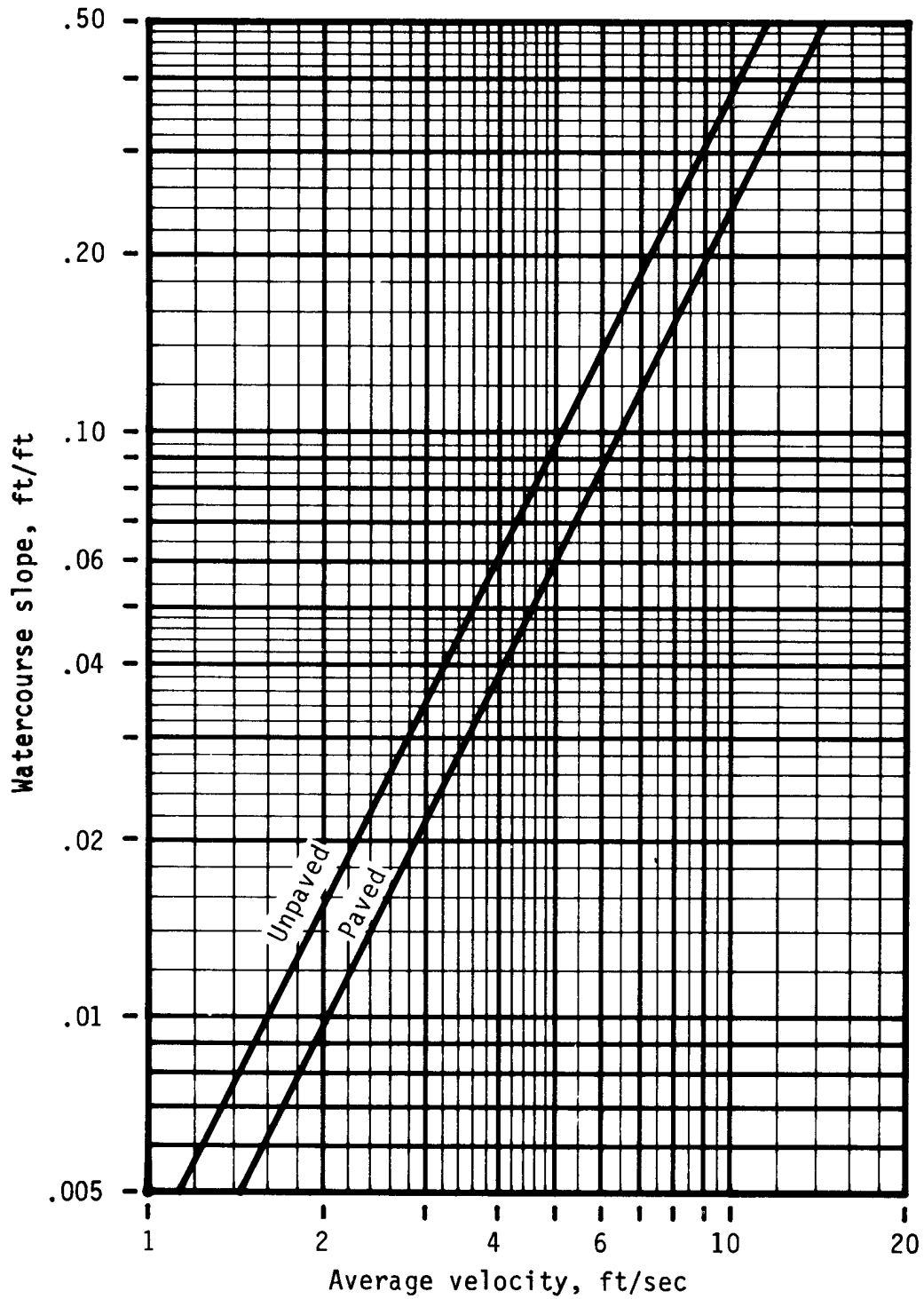
### *1.3.7 Simplified SCS Peak Runoff Rate Estimation*

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:

$$Q_p = q_u A Q F_p \quad (1.14)$$

where:

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



**Figure 1.9 Average Velocities - Shallow Concentrated Flow**  
 (Source: SCS, TR-55, Second Edition, June 1986)

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth (P) is determined from the rainfall tables in [Section 5.0](#) for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 1.9 and direct runoff, Q, is calculated using Equation 1.7.
3. The CN value is used to determine the initial abstraction,  $I_a$ , from Table 1.11, 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 [Section 1.3.6](#) and is used with the ratio  $I_a/P$  to obtain the unit peak discharge ( $q_u$ ) from Figure 1.10 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 1.10 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 1.3.9](#).
5. The pond and swamp adjustment factor,  $F_p$ , is estimated from below:

<u>Pond and Swamp Areas (%*)</u>	<u><math>F_p</math></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 1.14.

### 1.3.8 Example Problem 1

Compute the flood mitigation storm 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:

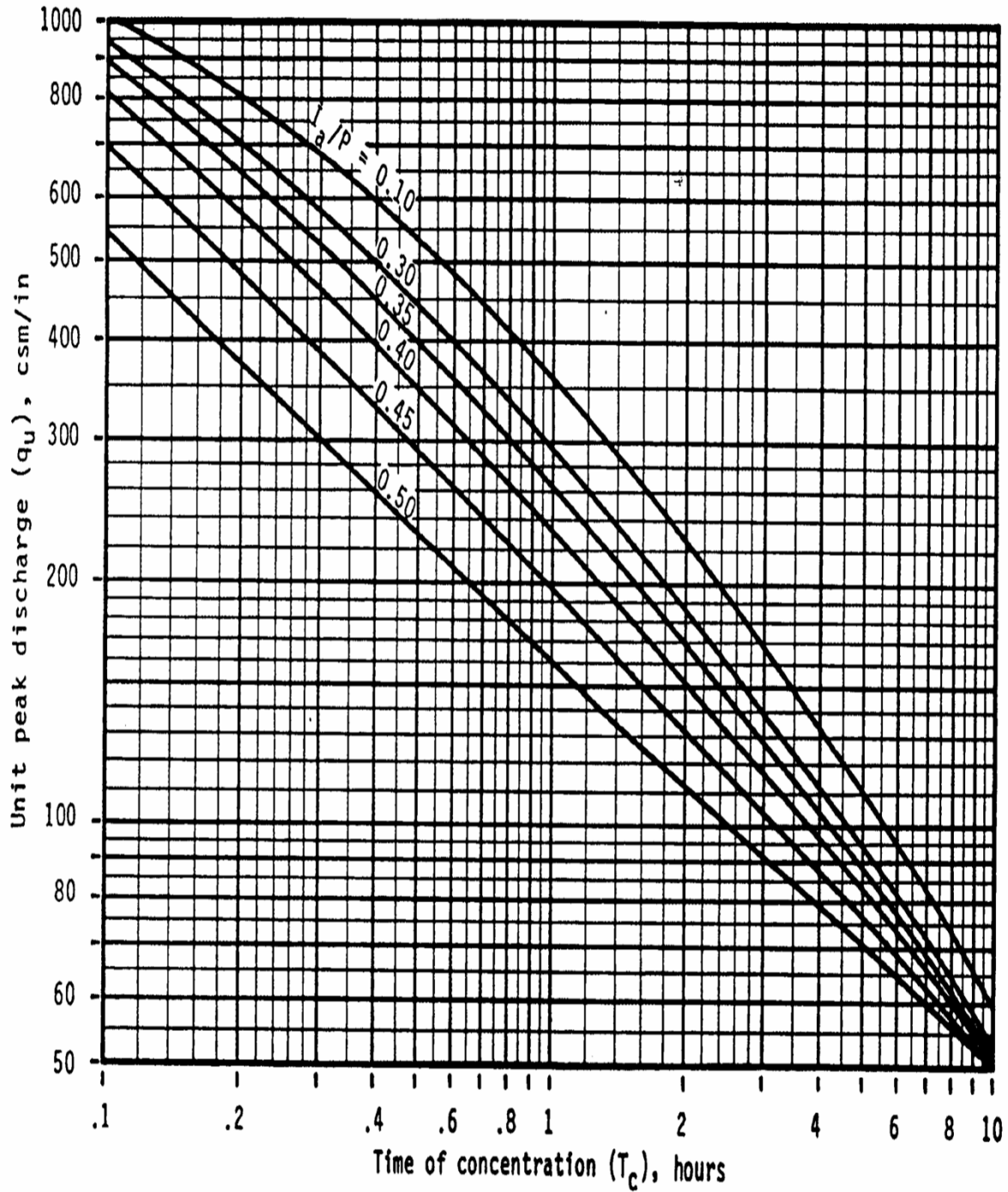
- The flood mitigation storm, 24-hour rainfall is 9.12 inches (.38 in/hr x 24 hours – From [Section 5.0](#), Table 5.15).
- Composite weighted runoff coefficient is:

<u>Dev. #</u>	<u>Area</u>	<u>% Total</u>	<u>CN</u>	<u>Composite CN</u>
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  $Q$  (flood mitigation storm) = 7.1 inches

<b>Table 1.11 I<sub>a</sub> Values for Runoff Curve Numbers</b>			
<b>Curve Number</b>	<b>I<sub>a</sub> (in)</b>	<b>Curve Number</b>	<b>I<sub>a</sub> (in)</b>
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 1.10 SCS Type II Unit Peak Discharge Graph**

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



## 2. Calculate time of concentration

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

<u>Segment</u>	<u>Type of Flow</u>	<u>Length (ft)</u>	<u>Slope (%)</u>
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 1.10 with  $P_2 = 3.36$  inches  
(0.14 x 24 – [Section 5.0](#), Table 5.15)

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

Segment 2 - Travel time from Figure 1.9 or Equation 1.10

$$V = 2.1 \text{ ft/sec (from Equation 1.11)}$$

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

Segment 3 - Using Equation 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 1.9),  $I_a = .410$  (Table 1.11)

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

(Note: Use  $I_a/P = .10$  to facilitate use of Figure 1.10.)

4. Unit discharge  $q_u$  (flood mitigation storm) from Figure 1.10 = 650 csm/in5. Calculate peak discharge with  $F_p = 1$  using Equation 1.14

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

### 1.3.9 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 [Section 1.3.3](#) 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 [Sections 1.3.4, 1.3.5, and 1.3.6](#).
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 1.8).
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 1.11 and Table 1.12 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:

$$q_u = (PF \cdot A) / (T_p) \quad (1.15)$$

where:

$q_u$  = unit hydrograph peak rate of discharge (cfs)

PF = peaking factor (484)

A = area (mi<sup>2</sup>)

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:

$$\frac{q}{q_u} = \left( \frac{t}{T_p} e^{[1-(t/T_p)]} \right)^X \quad (1.16)$$

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

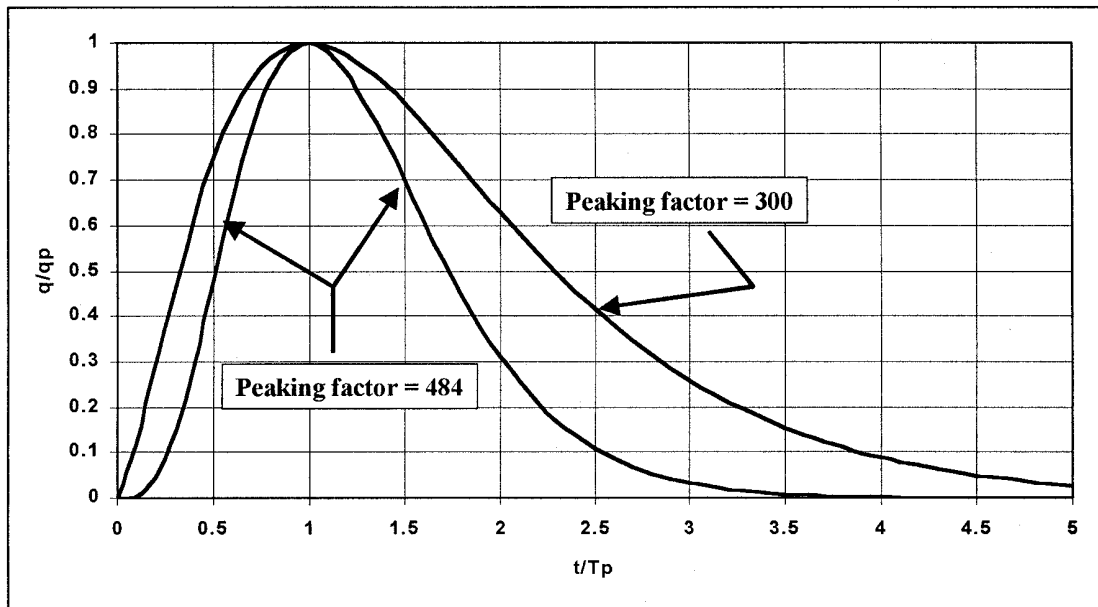


Figure 1.11 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

t/T <sub>t</sub>	484	
	q/q <sub>u</sub>	Q/Q <sub>p</sub>
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

### 1.3.10 Example Problem 2

Compute the unit hydrograph for the 50-acre watershed in Example Problem 1 (*Section 1.3.8*).

#### Computations

1. 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:

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

2. Calculate  $q_{pu}$

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

3. Calculate unit hydrograph.

Based on spreadsheet calculations using Equations 1.15 and 1.16, Table 1.13 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

### 1.3.11 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).

## 1.4 Snyder's Unit Hydrograph Method

### 1.4.1 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.

### 1.4.2 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:

$$t_p = C_t (L L_{ca})^{0.3} \quad (1.17)$$

$$t_r = t_p \div 5.5 \quad (1.18)$$

$$q_p = C_p 640 \div t_p \quad (1.19)$$

$$t_{pR} = t_p + 0.25(t_r - t_p) \quad (1.20)$$

$$q_{pR} = C_p 640 \div t_{pR} \quad (1.21)$$

$$Q_p = q_p t_p \div t_{pR} \quad (1.22)$$

$$Q_p = q_p A \quad (1.23)$$

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 1.14.

Table 1.14 Typical Values of $C_p$	
Typical Drainage Area Characteristics	$C_p$
<b>Undeveloped Areas w/ Storm Drains</b>	
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
<b>Moderately Developed Area</b>	
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
<b>Highly Developed/Commercial Area</b>	
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

### 1.4.3 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:

$$t_p = 10^{[0.3833 \log_{10}(L \cdot L_{ca} / (S_{st})^{0.5}) + (\log_{10}(I_p)) - BW \cdot (\%Urb/100)]} \quad (1.24)$$

$$S_{st} = (el_{85\%} - el_{15\%}) / (0.7 \cdot L) \quad (1.25)$$

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.

#### 1.4.4 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 stormwater 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 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 1.15. 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 1.6](#) in the application of USGS regression equations to determine peak flows for urban basins.

##### 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 (Table 1.16) 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)

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

**Table 1.16 Permeability Rating for the Determination of Percent Sand**

Permeability (inches/hr)	Percent Sand Assignment (%)
< 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

$$\text{Weighted \%Sand} = 8315.8/100 = \underline{83.2\%}$$

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, as shown in Table 1.17, 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 1.3.11](#), for an explanation of routing methods and references for further information.

## 1.5 Modified Rational Method

### 1.5.1 Introduction

For drainage areas of *less than 200 acres*, a modification of the Rational Method can be used for the estimation or design 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 1.12

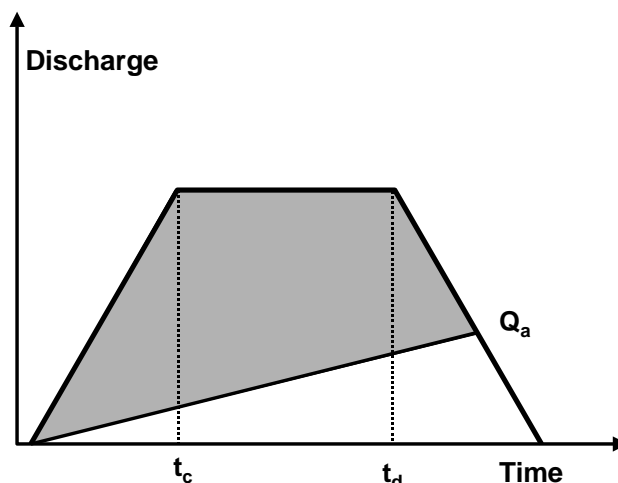


Figure 1.12 Modified Rational Definitions

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.

### 1.5.2 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:

$$Q_a = C_a i A \quad (1.26)$$

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:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \quad (1.27)$$

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 1.18

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

$$V_{\text{preliminary}} = 60 [CAa - (2CabAQ_a)^{1/2} + (Q_a/2)(b-t_c)] \quad (1.28a)$$

$$V_{\text{max}} = V_{\text{preliminary}} * P_{180}/P_{td} \quad (1.28b)$$

where:

$V_{\text{preliminary}}$  = preliminary required storage (ft<sup>3</sup>)

$V_{\text{max}}$  = required storage (ft<sup>3</sup>)

$t_c$  = 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 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

### 1.5.3 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 flood mitigation 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 [Section 5.0](#). 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 1.26) (cfs)	3.69	5.27
$a$ (from Table 1.18)	128.85	210.73
$b$ (from Table 1.18)	16.624	21.785
$V_{pre}$ (Equation 1.28a) (ft <sup>3</sup> )	16,570	26,042
$P_{180}$ (in)	2.28	3.60
$T_d$ (Equation 1.27) (min)	51.52	61.69
$P_{td}$ (in)	1.65	2.66
$V_{max}$ (Equation 1.28b) (ft <sup>3</sup> )	22,897	35,245

## 1.6 USGS and TxDOT Regression Methods

### 1.6.1 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.

### 1.6.2 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.

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

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

### 1.6.3 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 1.19 gives the USGS regression equations for urban streams in the Dallas-Fort Worth area.

### 1.6.4 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:

<u>Physical Characteristics</u>	<u>Minimum</u>	<u>Maximum</u>	<u>Units</u>
A - Drainage Area	3	40	mi <sup>2</sup>
UI – Urbanization Index	12	33	

Table 1.19 USGS Peak Flow Regression Equations for Dallas-Fort Worth Urban Areas	
Frequency	Equations
2-year	$Q_2 = 42.83(A)^{0.704} (UI)^{0.836}$
5-year	$Q_5 = 82.92(A)^{0.724} (UI)^{0.751}$
10-year	$Q_{10} = 120.7(A)^{0.735} (UI)^{0.697}$
25-year	$Q_{25} = 184.8(A)^{0.745} (UI)^{0.632}$
50-year	$Q_{50} = 246.4(A)^{0.752} (UI)^{0.587}$
100-year	$Q_{100} = 362.1(A)^{0.752} (UI)^{0.510}$
For these equations: A = drainage area in mi <sup>2</sup> , UI = urbanization index Source: USGS, 1982	

### 1.6.5 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.

### 1.6.6 Rural (or Undeveloped) Basin Application

Equation 1.29 applies to rural, uncontrolled watersheds. Figure 1.13 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 1.20 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.

$$Q_T = aA^bSH^cSL^d \quad (1.29)$$

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 1.20.

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 1.30.

$$Q_w = (2 - \log(A/z))Q_1 + (\log(A/z) - 1)Q_2 \quad (1.30)$$

where:

$Q_w$  = weighted discharge (cfs)

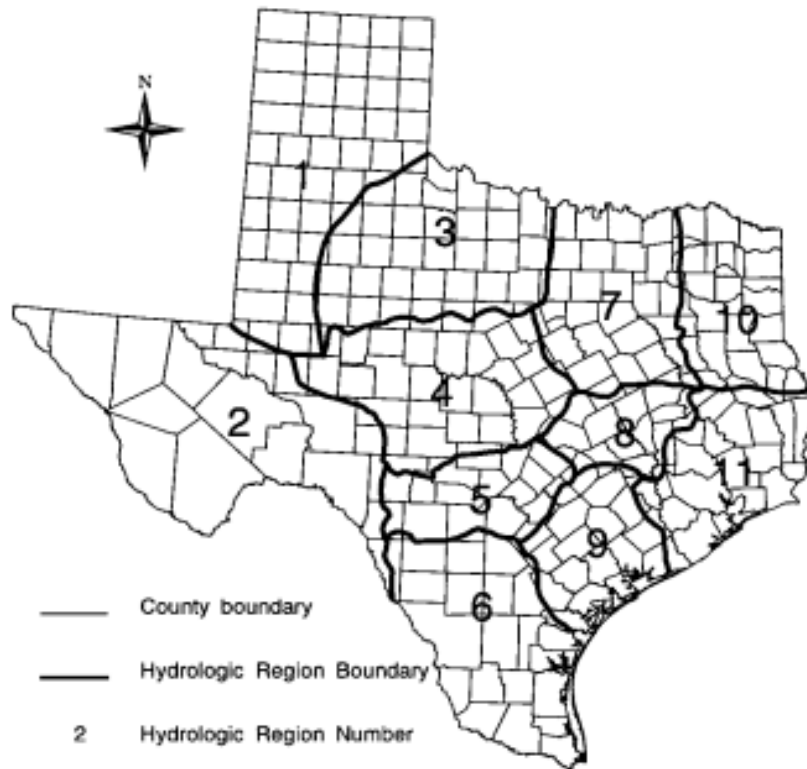
$A$  = contributing drainage area (sq. mi.)

$z$  = 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 1.13 Hydrologic Regions for Statewide Rural Regression Equations**  
 Source: TXDOT, 2002

Table 1.20 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas <sup>1</sup>		
[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) <sup>2</sup>		
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) <sup>2</sup>		

<b>Table 1.20 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas<sup>1</sup></b>		
[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]		
<b>Hydrologic region and recurrence interval</b>	<b>Weighted least-squares regression equation for corresponding recurrence interval</b>	<b>Range of indicated independent variables in corresponding region (units as noted)</b>
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) <sup>2</sup>		
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) <sup>2</sup>		
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) <sup>2</sup>		
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	$Q_{100} = 1842 A^{.558}$	
Region 7 (sites with contributing drainage area greater than 32 square miles) <sup>2</sup>		
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 1.29 to calculate a weighted discharge for streams with contributing drainage area falling within the arrange of 10 to 100 square miles.		

### 1.6.7 Example Problem

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

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

### Peak Discharge Calculations

The 100-year storm Rural Peak Discharge determination for Region 7 will necessitate the use of Equation 1.30 because the drainage area is in the range of 10-100 square miles. The first step is to determine the discharge based on regression coefficients for areas greater than 32 square miles and less than 32 square miles. Table 1.20 provides the regression equations for Region 7 as follows;

For contributing drainage area less than 32 square miles,

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

For contributing drainage area greater than 32 square miles,

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

Equation 1.30 is then used to determine the 100-year storm Rural Peak Discharge.

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

## 1.7 References

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## 2.0 Downstream Assessment

### 2.1 Introduction

The downstream impacts of development must be carefully evaluated. The purpose of the downstream assessment is to protect downstream properties from increased flooding and downstream channels from increased erosion potential due to upstream development. The importance of the downstream assessment is particularly evident for larger sites or developments that have the potential to dramatically impact downstream areas. The cumulative effect of smaller sites, however, can be just as dramatic.

The assessment should extend from the outfall of a proposed development to a point downstream where the discharge from a proposed development no longer has a significant impact on the receiving stream or storm drainage system. The assessment should be a part of the concept, preliminary, and final iSWM plans, and should include the following properties:

- Hydrologic analysis of the pre- and post-development on-site conditions
- Drainage path which defines extent of the analysis.
- Capacity analysis of all existing constraint points along the drainage path, such as existing floodplain developments, underground storm drainage systems culverts, bridges, tributary confluences, or channels
- Offsite undeveloped areas are considered as “full build-out” for both the pre- and post-development analyses
- Evaluation of peak discharges and velocities for three (3) 24-hour storm events
  - “Streambank Protection” storm
  - “Conveyance” storm
  - “Flood Mitigation” storm
- Separate analysis for each major outfall from the proposed development

Once the analysis is complete, the designer should ask the following three questions at each determined junction downstream:

- Are the post-development discharges greater than the pre-development discharges?
- Are the post-development velocities greater than the pre-development velocities?
- Are the post-development velocities greater than the velocities allowed for the receiving system?

These questions should be answered for each of the three storm events. The answers to these questions will determine the necessity, type, and size of non-structural and structural controls to be placed on-site or downstream of the proposed development.

### 2.2 Downstream Hydrologic Assessment

Common practice requires 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.

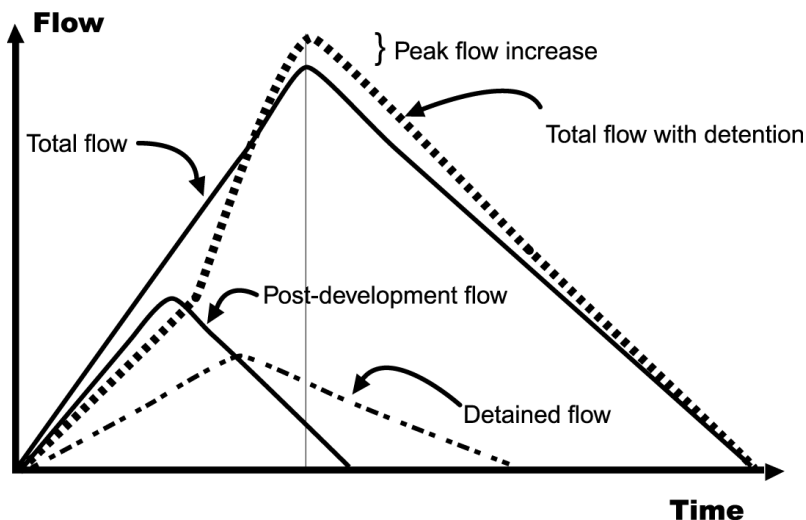
Due to a site’s location within a watershed, there may be very little reason for requiring flood control from a particular site. In certain circumstances where detention is in place or a master drainage plan has been adopted, a development may receive or plan to receive less than ultimate developed flow 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 stormwater peak

flows and volumes that a community may require as part of a developer's stormwater management site plan.

## 2.3 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. 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.



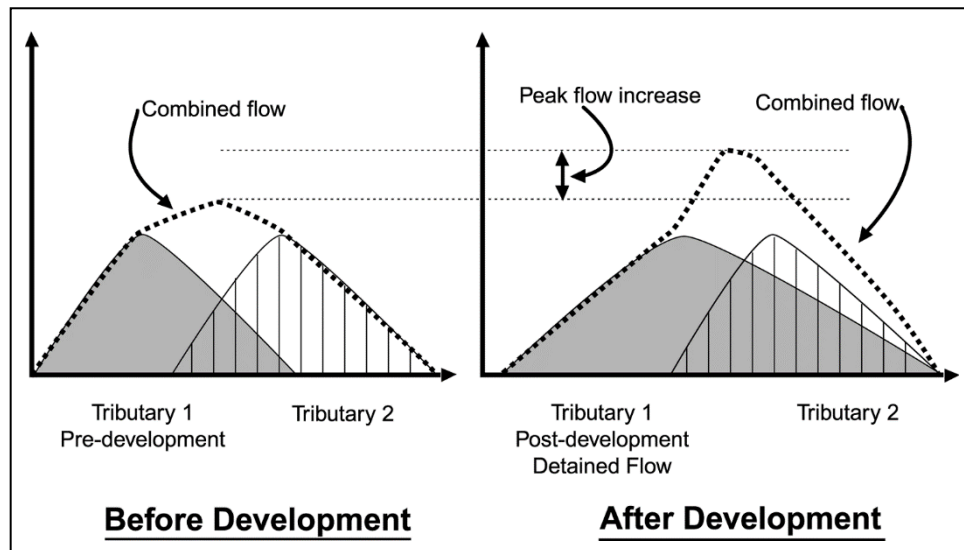
**Figure 2.1 Detention Timing Example**

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 2.2 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 2.2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph**

## 2.4 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on stormwater peak flows and velocities are evaluated downstream. The assessment 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 stormwater 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:
  - “Streambank Protection” storm

- “Conveyance” storm
  - “Flood Mitigation” storm
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 3.2 of the Hydraulics Technical 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 stormwater 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.

## 2.5 Example Problem

Figure 2.3 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) stormwater 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.

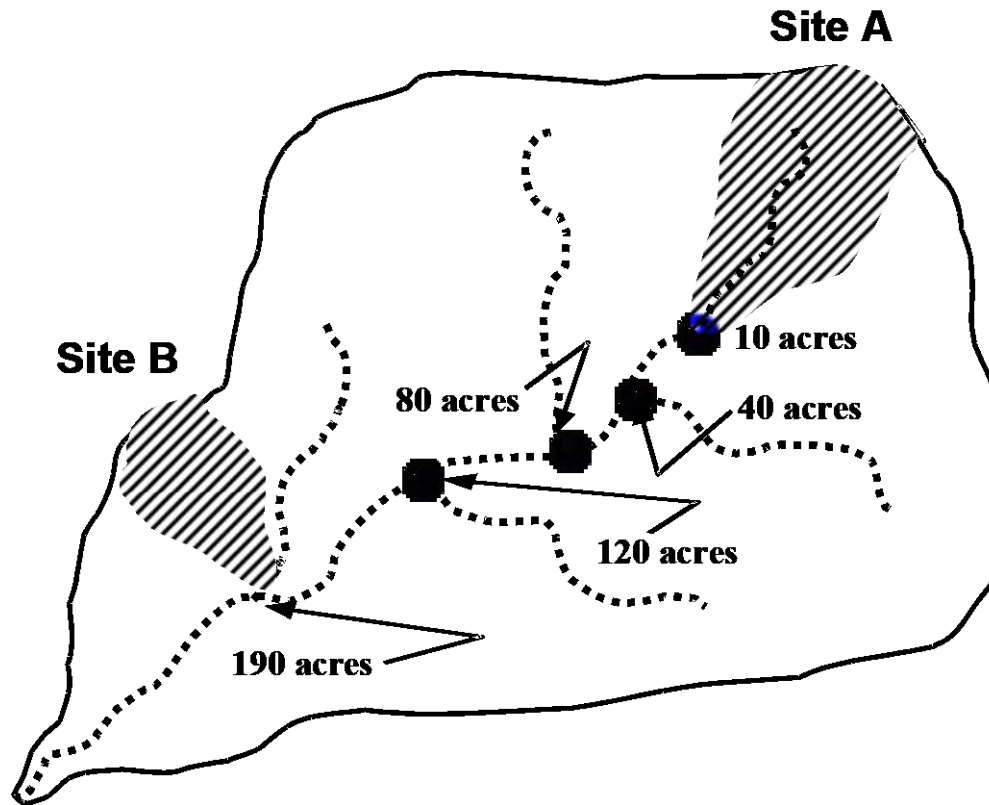


Figure 2.3 Example of the Ten-Percent Rule

## 3.0 Streambank Protection Volume Estimation

### 3.1 Streambank Protection Volume Calculation

The Simplified SCS Peak Runoff Rate Estimation approach (see [Section 1.3.7](#)) 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 1.5](#) for preliminary detention calculations without formal routing or the SCS Hydrologic Method in [Section 1.3](#).

For  $SP_v$  estimation, using Figure 1.10, 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 3.1.

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

*Note: Figure 3.2 can also be used to estimate  $V_s/V_r$ .*

$$V_s/V_r = 0.682 - 1.43 (q_o/q_i) + 1.64 (q_o/q_i)^2 - 0.804 (q_o/q_i)^3 \quad (3.1)$$

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:

$$V_s = \frac{(V_s/V_r)(Q_d)(A)}{12} \quad (3.2)$$

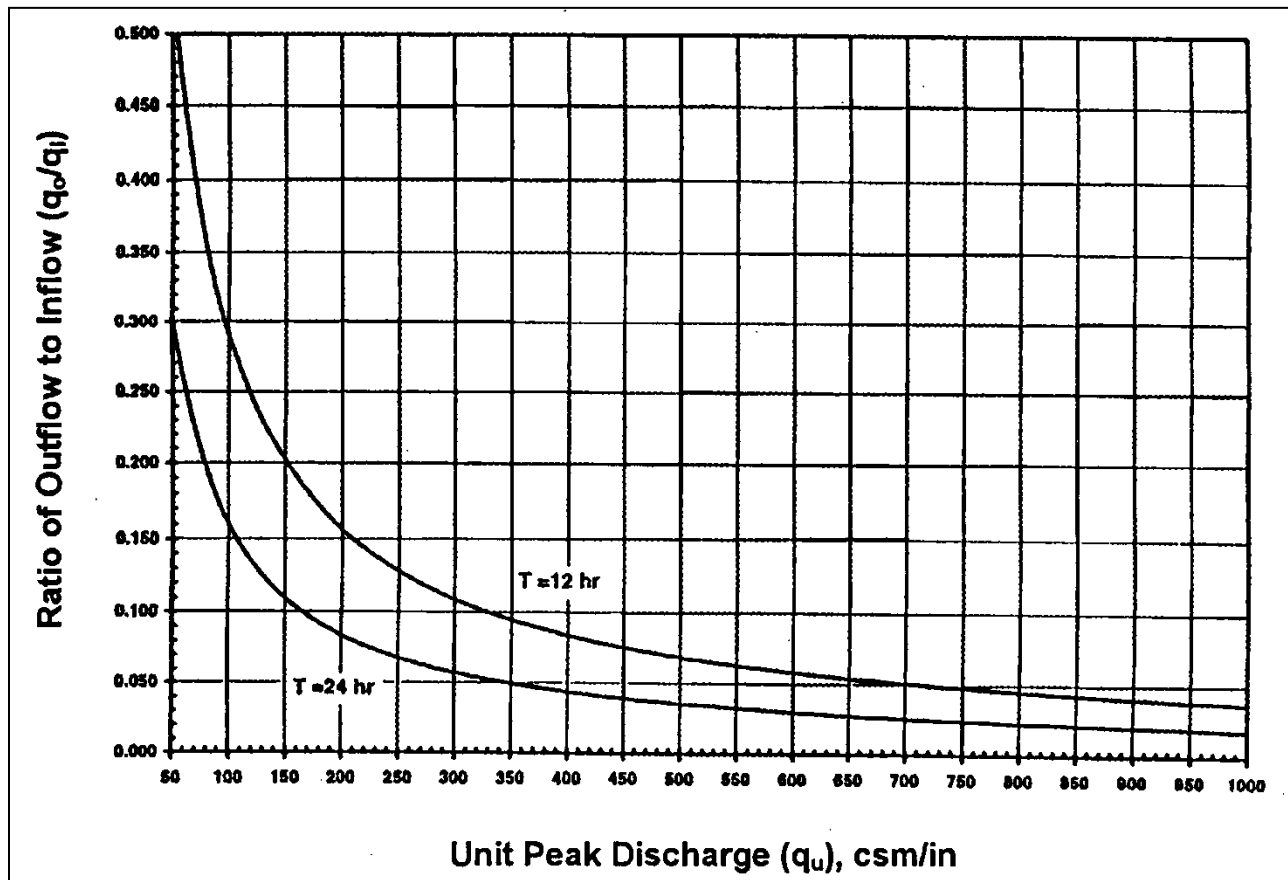
where:

$V_s$  and  $V_r$  are defined above

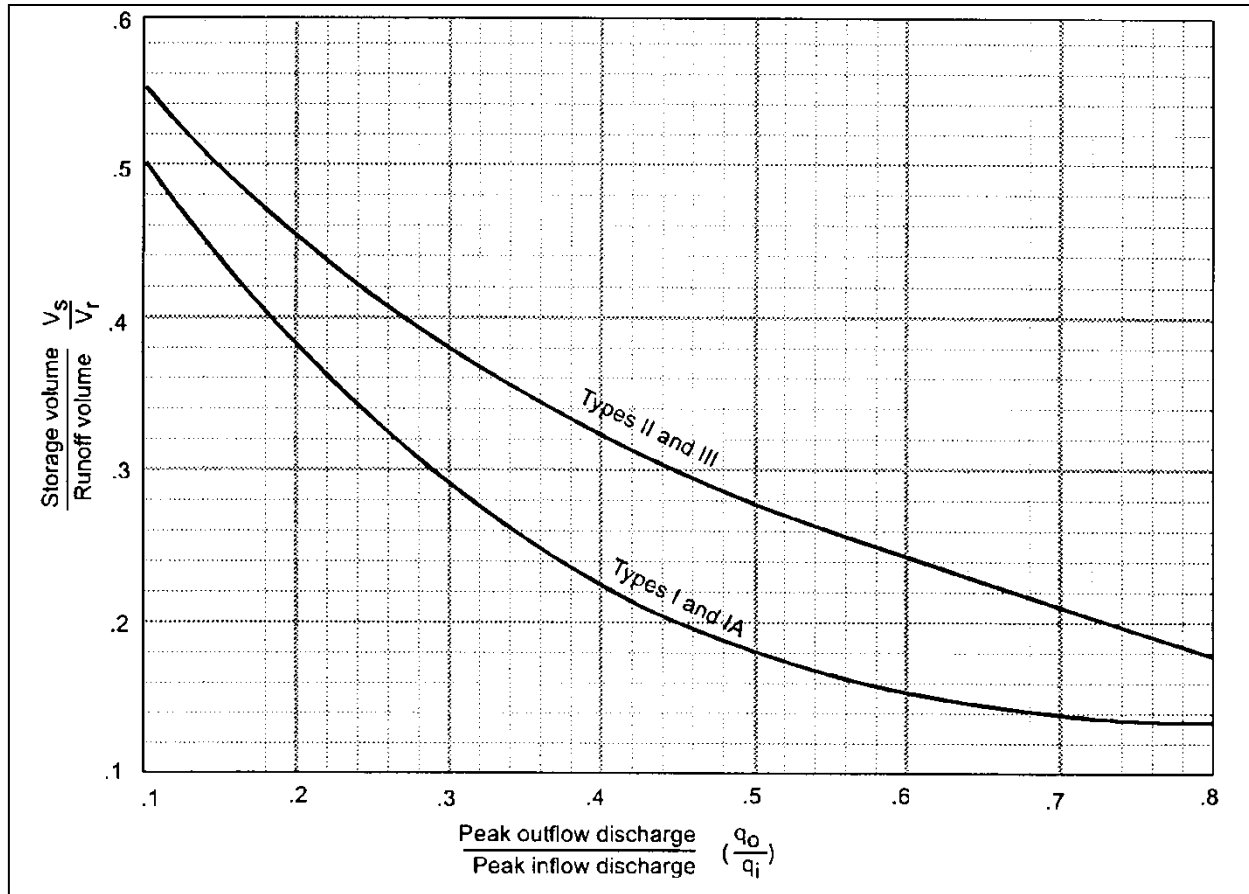
$Q_d$  = the developed runoff for the design storm (inches)

$A$  = 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 3.1 Detention Time vs. Discharge Ratios**  
 (Source: MDE, 1998)



**Figure 3.2 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III**  
(Source: TR-55, 1986)

## 3.2 Example Problem

Compute the Streambank Protection Volume ( $SP_v$ ) for the 50-acre watershed in [Section 1.3.8 Example Problem One](#).

### Computations

1. Calculate rainfall excess:

- The 1-year, 24 hour rainfall is 2.64 inches (0.11 in/hr x 24 hours – From Table 5.16).
- Composite area-weighted Curve Number is 83.
- From Equation 2.1.7,  $Q_d$  (1-year developed) = 1.2 inches

2. Calculate time of concentration

$$t_c = 20.86 \text{ minutes } (.35 \text{ hours})$$

3. Calculate  $I_a/P$  for CN = 83;  $I_a = .410$  (Table 1.11)

$$I_a/P = (.410 / 2.64) = .155 \text{ (Note: Use straight-line interpolation to facilitate use of Figure 1.10)}$$

4. Find unit discharge  $q_u$ :

$$\text{From Figure 1.10 for } I_a/P = .155 \text{ and } t_c = .35 \text{ hr}$$

$$q_u = 600 \text{ csm/in}$$

5. Find discharge ratio  $q_o/q_i$ :

From Figure 3.1 for  $q_u = 600 \text{ csm/in}$  and  $T = 24 \text{ hr}$   
 $q_o/q_i = 0.03$

6. 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

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

## 4.0 Water Balance

### 4.1 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.

### 4.2 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):

$$\Delta V = \Sigma I - \Sigma O \quad (4.1)$$

where:

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

The inflows consist of rainfall, runoff, and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 4.1 can be changed to reflect these factors.

$$\Delta V = P + R_o + B_f - I - E - E_t - O_f \quad (4.2)$$

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 4.4)
- $E$  = evaporation (ac-ft) (Surface evaporation in feet times surface area)
- $E_t$  = evapotranspiration (ac-ft)
- $O_f$  = overflow (ac-ft)
- $\Delta$  = "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 4.1 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 1.1 of the Water Quality Technical Manual* 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 4.3 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$R_o = 0.9(P/12)R_vA \quad (4.3)$$

where:

P = precipitation (in)

$R_o$  = runoff volume (acre-ft)

$R_v$  = volumetric runoff coefficient [see *Equation 1.1 of the Water Quality Technical Manual*]

A = Area in acres

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

Source: National Weather Service, 2002

**Baseflow ( $B_i$ )** – Most stormwater 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:

$$I = A k_h G_h \quad (4.4)$$

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 4.2 can be used.

<b>Table 4.2 Saturated Hydraulic Conductivity</b>		
<b>Material</b>	<b>Hydraulic Conductivity</b>	
	<b>in/hr</b>	<b>ft/day</b>
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08
Clay	0.02	0.04

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

**Evaporation (E)** – Evaporation 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.

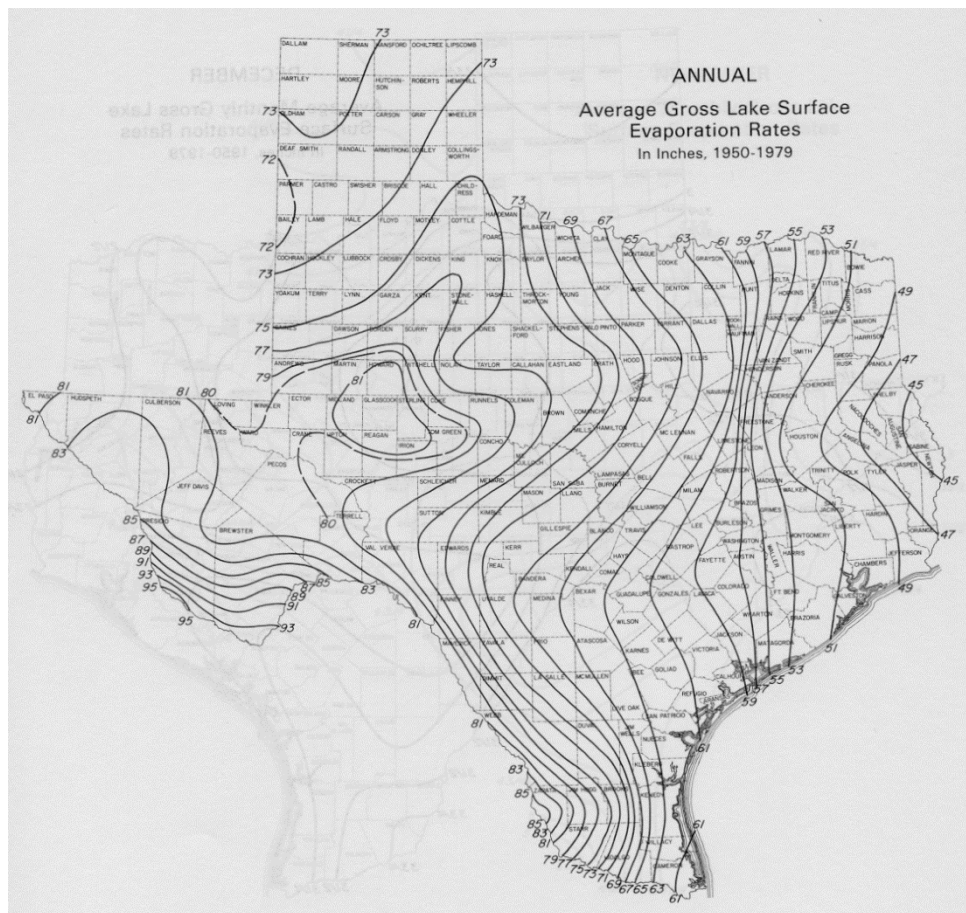
Table 4.3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information for Grapevine, Texas. Figure 4.1 depicts a map of annual free water surface (FWS) evaporation averages for Texas based on a National Oceanic and Atmospheric Administration (NOAA) assessment 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 stormwater ponds and wetlands being designed in Texas. Total annual values can be estimated from this map and distributed according to Table 4.3.



<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>
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 4.1 Average Annual Free Water Surface Evaporation (in inches)**  
 (Source: NOAA, 1982)

### 4.3 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 1.1 of the Water Quality Technical Manual*,  $R_v = 0.05 + 0.009 (75) = 0.73$ . With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 4.1 is about 64 inches.
- For a clay loam the infiltration rate is  $I = 0.18$  ft/day (Table 4.2).
- 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 4.4 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
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 4.2, 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	YEA R	
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...	Precipitati on > 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
	Precipitati on > 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 4.2 Dallas/Fort Worth Precipitation Information**

## 4.4 References

Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14.

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USDA, SCS, 1975. Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas, College Park, Maryland.

U.S. Department of Interior, Bureau of Reclamation, 1978. Design of Small Canal Structures.

## 5.0 Rainfall Tables

### 5.1 Methodology

Rainfall tables are based on the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 for the counties within the jurisdiction of NCTCOG. NOAA Atlas 14 is produced by the NOAA's National Weather Service, Hydrometeorological Design Studies Center and may be considered as the national standard for precipitation frequency estimates. NOAA Atlas 14 is published in volumes for different geographic areas of the US. The final version of NOAA Atlas 14 Volume 11 for Texas was released on September 2018 and has been peer-reviewed extensively. Volume 11 provides precipitation frequency estimates, upper and lower bounds for 90% confidence intervals for durations of 5-minute through 60-day and recurrence intervals of 1, 2, 5, 10, 25, 50, 100, 200, 500 and 1,000-year for the State of Texas.

Precipitation frequency estimates are computed using regional frequency analysis based on L-moment statistics calculated from annual maxima series (AMS). NOAA Atlas 14 employs a regionalization approach wherein the L-moment statistics are calculated by grouping stations within a 60-mile radius. This results in 700 to 1,800 years of data for daily durations and 200 to 700 years for hourly durations. Several distribution functions were examined and ultimately the generalized extreme value (GEV) distribution was adopted for all stations and durations. The upper and lower 90% confidence intervals are based on a Monte-Carlo simulation approach.

Gridded precipitation frequency estimates at a spatial resolution of 30-arc second are also available for all durations and recurrence intervals discussed above. The gridded frequency estimates are generated from grids of mean annual maxima which are derived from at-stations mean annual maxima using the PRISM interpolation methodology (<http://www.prism.oregonstate.edu/>). The precipitation frequency grids are the basis of the NOAA Precipitation Frequency Data Server (PFDS) for retrieval of precipitation frequency estimates by co-ordinates.

Based on the review of the NOAA Atlas 14 methodology, it is apparent that a robust and standardized technique has been adopted for the development of precipitation frequency estimates. The methods and results have also been extensively peer reviewed. In addition, Atlas 14 also uses a long period of data for the development of frequency estimates (average record length of approximately 60 years) and more recent data, as available.

Rainfall tables have been generated at the center of each county within the jurisdiction of the North Central Texas Council of Governments (NCTCOG). AMS based precipitation frequency grids for all available recurrence intervals and durations were first downloaded from the NOAA PFDS<sup>1</sup>. Subsequently, at the centroid of each county, the value associated with each duration and recurrence interval was extracted from the respective AMS frequency grids in ArcGIS. The compiled precipitation frequencies for each county in a tabular format are provided below. Note that the estimates for 1-yr recurrence interval are based on the frequency analysis of partial duration series (PDS).

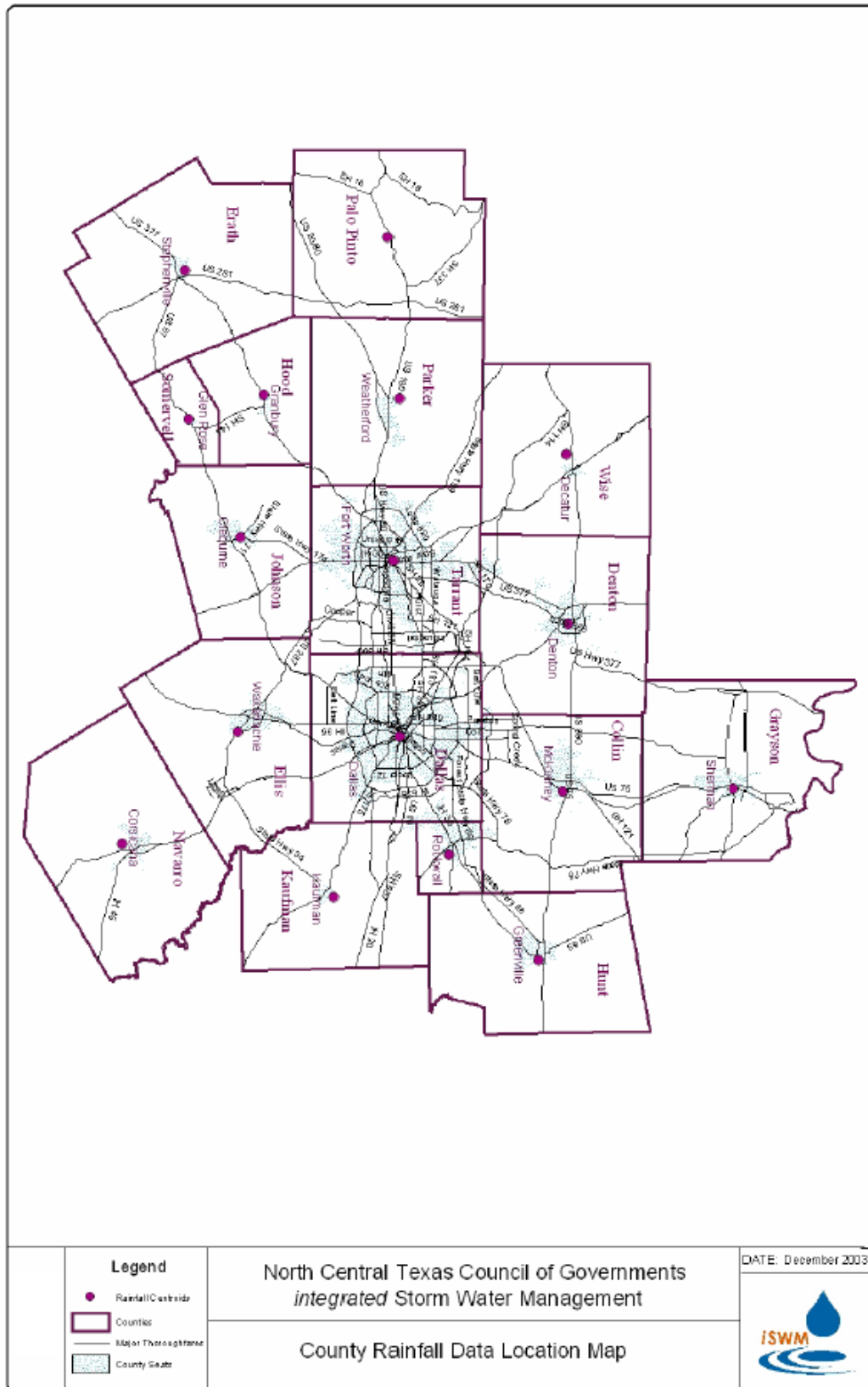
While precipitation frequency estimates at the centroid of a county is a reasonable approach for summarizing the NOAA Atlas 14 data, these estimates may not be representative of smaller areas such as a metropolitan area or census block. For such cases, precipitation frequency estimates may be directly downloaded from the NOAA PFDS<sup>2</sup> by specifying the desired geographic co-ordinates.

### 5.2 References

Sanja Perica, Sandra Pavlovic, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Orlan Wilhite (2018). *NOAA Atlas 14 Volume 11 Version 2, Precipitation-Frequency Atlas of the United States, Texas*. NOAA, National Weather Service, Silver Spring, MD.

<sup>1</sup> [https://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_gis.html](https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_gis.html)

<sup>2</sup> [https://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html](https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html)



**Figure 5.1 County Rainfall Data Location Map**

**Table 5.1 AMS-based precipitation frequency estimates for Collin County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.430	0.462	0.588	0.683	0.806	0.897	0.987	1.077	1.198	1.289
10-min	0.687	0.739	0.943	1.095	1.293	1.441	1.585	1.725	1.904	2.034
15-min	0.858	0.921	1.171	1.358	1.601	1.779	1.956	2.133	2.369	2.547
30-min	1.195	1.282	1.625	1.881	2.212	2.453	2.693	2.942	3.278	3.537
60-min	1.554	1.670	2.128	2.470	2.915	3.240	3.569	3.915	4.390	4.763
2-hr	1.898	2.056	2.666	3.130	3.749	4.215	4.696	5.206	5.915	6.478
3-hr	2.100	2.287	3.001	3.550	4.292	4.861	5.456	6.088	6.968	7.671
6-hr	2.482	2.719	3.615	4.311	5.263	6.005	6.788	7.623	8.791	9.729
12-hr	2.938	3.224	4.302	5.138	6.281	7.171	8.113	9.121	10.539	11.682
24-hr	3.456	3.790	5.054	6.032	7.364	8.395	9.487	10.666	12.334	13.684
48-hr	4.032	4.412	5.855	6.968	8.477	9.634	10.864	12.210	14.135	15.707
3-day	4.407	4.816	6.376	7.578	9.207	10.453	11.778	13.237	15.328	17.040
4-day	4.670	5.105	6.757	8.032	9.767	11.100	12.519	14.078	16.311	18.138
7-day	5.221	5.714	7.568	9.011	10.992	12.537	14.187	15.975	18.518	20.585
10-day	5.708	6.243	8.259	9.827	11.982	13.669	15.461	17.387	20.103	22.297
20-day	7.470	8.096	10.503	12.339	14.795	16.656	18.585	20.636	23.488	25.762
30-day	8.964	9.660	12.384	14.433	17.116	19.099	21.115	23.237	26.151	28.446
45-day	11.030	11.840	15.054	17.445	20.525	22.765	24.986	27.265	30.320	32.668
60-day	12.866	13.782	17.444	20.151	23.603	26.093	28.517	30.945	34.131	36.520

**Table 5.2 AMS-based precipitation frequency estimates for Dallas County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.428	0.462	0.595	0.695	0.824	0.919	1.014	1.111	1.239	1.338
10-min	0.684	0.738	0.952	1.112	1.320	1.474	1.626	1.776	1.971	2.115
15-min	0.856	0.923	1.188	1.385	1.641	1.828	2.014	2.203	2.456	2.650
30-min	1.199	1.291	1.657	1.929	2.280	2.535	2.790	3.054	3.412	3.690
60-min	1.560	1.684	2.169	2.532	3.004	3.348	3.696	4.063	4.567	4.963
2-hr	1.903	2.068	2.702	3.185	3.827	4.310	4.809	5.339	6.072	6.656
3-hr	2.103	2.295	3.027	3.590	4.351	4.935	5.545	6.193	7.097	7.818
6-hr	2.466	2.707	3.613	4.318	5.286	6.043	6.847	7.705	8.913	9.885
12-hr	2.865	3.155	4.240	5.090	6.264	7.190	8.182	9.255	10.781	12.020
24-hr	3.316	3.657	4.930	5.929	7.312	8.401	9.577	10.864	12.713	14.229
48-hr	3.857	4.248	5.714	6.858	8.433	9.658	10.984	12.462	14.611	16.390
3-day	4.215	4.636	6.222	7.455	9.148	10.460	11.878	13.465	15.780	17.700
4-day	4.466	4.910	6.580	7.881	9.667	11.053	12.551	14.220	16.646	18.653
7-day	4.995	5.486	7.327	8.765	10.749	12.303	13.975	15.805	18.428	20.576
10-day	5.460	5.986	7.962	9.504	11.625	13.287	15.063	16.987	19.722	21.945
20-day	7.127	7.725	10.033	11.790	14.130	15.885	17.720	19.723	22.565	24.871
30-day	8.532	9.189	11.776	13.713	16.233	18.064	19.944	21.994	24.883	27.215
45-day	10.454	11.221	14.270	16.534	19.442	21.541	23.635	25.820	28.787	31.097
60-day	12.156	13.029	16.509	19.087	22.385	24.777	27.106	29.424	32.455	34.719



**Table 5.3 AMS-based precipitation frequency estimates for Denton County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.419	0.452	0.584	0.682	0.808	0.901	0.992	1.085	1.207	1.299
10-min	0.671	0.725	0.936	1.094	1.298	1.448	1.595	1.737	1.920	2.055
15-min	0.836	0.902	1.162	1.356	1.606	1.788	1.967	2.148	2.387	2.568
30-min	1.162	1.252	1.610	1.875	2.217	2.463	2.708	2.959	3.297	3.557
60-min	1.508	1.627	2.100	2.452	2.907	3.237	3.569	3.915	4.388	4.757
2-hr	1.842	2.000	2.612	3.076	3.690	4.149	4.621	5.121	5.814	6.363
3-hr	2.040	2.223	2.927	3.465	4.189	4.741	5.315	5.927	6.779	7.460
6-hr	2.407	2.635	3.502	4.172	5.085	5.791	6.536	7.335	8.458	9.363
12-hr	2.833	3.106	4.143	4.944	6.033	6.872	7.760	8.723	10.088	11.194
24-hr	3.310	3.630	4.847	5.785	7.056	8.030	9.062	10.189	11.796	13.105
48-hr	3.838	4.206	5.606	6.684	8.143	9.258	10.440	11.732	13.573	15.074
3-day	4.180	4.580	6.102	7.275	8.867	10.088	11.382	12.794	14.806	16.444
4-day	4.420	4.848	6.465	7.718	9.429	10.753	12.162	13.696	15.880	17.659
7-day	4.922	5.413	7.241	8.677	10.672	12.253	13.954	15.795	18.416	20.548
10-day	5.362	5.901	7.896	9.470	11.665	13.416	15.302	17.335	20.222	22.567
20-day	6.930	7.566	9.970	11.833	14.377	16.347	18.442	20.713	23.939	26.559
30-day	8.254	8.968	11.704	13.798	16.614	18.752	20.998	23.431	26.872	29.657
45-day	10.090	10.933	14.181	16.657	19.966	22.475	25.066	27.801	31.580	34.576
60-day	11.723	12.687	16.403	19.233	23.012	25.891	28.826	31.841	35.918	39.080

**Table 5.4 AMS-based precipitation frequency estimates for Ellis County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.429	0.465	0.607	0.714	0.856	0.963	1.072	1.186	1.341	1.462
10-min	0.685	0.744	0.971	1.143	1.372	1.546	1.721	1.896	2.130	2.307
15-min	0.859	0.932	1.215	1.429	1.709	1.917	2.128	2.349	2.652	2.891
30-min	1.202	1.302	1.691	1.984	2.368	2.651	2.940	3.250	3.680	4.024
60-min	1.559	1.691	2.205	2.595	3.111	3.497	3.897	4.329	4.937	5.427
2-hr	1.887	2.065	2.738	3.259	3.971	4.522	5.108	5.744	6.647	7.381
3-hr	2.074	2.283	3.060	3.671	4.520	5.192	5.917	6.703	7.824	8.739
6-hr	2.420	2.682	3.642	4.408	5.488	6.359	7.312	8.353	9.852	11.082
12-hr	2.806	3.119	4.264	5.179	6.474	7.520	8.674	9.954	11.819	13.366
24-hr	3.255	3.619	4.955	6.019	7.518	8.717	10.045	11.541	13.750	15.599
48-hr	3.831	4.242	5.772	6.973	8.639	9.936	11.371	13.034	15.534	17.655
3-day	4.213	4.651	6.296	7.578	9.341	10.698	12.192	13.934	16.557	18.787
4-day	4.476	4.933	6.655	7.995	9.835	11.252	12.803	14.592	17.263	19.519
7-day	5.020	5.517	7.386	8.842	10.842	12.397	14.075	15.937	18.635	20.862
10-day	5.497	6.024	8.009	9.551	11.661	13.301	15.048	16.949	19.656	21.860
20-day	7.189	7.772	10.042	11.757	14.017	15.686	17.421	19.320	22.014	24.201
30-day	8.622	9.255	11.777	13.646	16.039	17.739	19.469	21.368	24.050	26.217
45-day	10.615	11.354	14.332	16.520	19.280	21.224	23.140	25.146	27.864	29.978
60-day	12.389	13.236	16.653	19.159	22.310	24.542	26.687	28.818	31.592	33.651

**Table 5.5 AMS-based precipitation frequency estimates for Erath County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.417	0.451	0.585	0.687	0.821	0.923	1.024	1.124	1.255	1.354
10-min	0.666	0.722	0.937	1.101	1.317	1.483	1.645	1.799	1.995	2.136
15-min	0.834	0.902	1.166	1.366	1.631	1.832	2.031	2.228	2.484	2.675
30-min	1.165	1.258	1.620	1.894	2.255	2.528	2.801	3.075	3.439	3.716
60-min	1.509	1.634	2.118	2.485	2.971	3.335	3.706	4.088	4.605	5.008
2-hr	1.827	1.997	2.652	3.151	3.820	4.326	4.853	5.416	6.202	6.833
3-hr	2.007	2.209	2.980	3.571	4.369	4.977	5.618	6.315	7.303	8.104
6-hr	2.326	2.581	3.547	4.295	5.315	6.101	6.941	7.864	9.189	10.274
12-hr	2.644	2.951	4.099	4.997	6.237	7.210	8.257	9.407	11.060	12.416
24-hr	3.016	3.371	4.695	5.733	7.173	8.312	9.536	10.868	12.768	14.317
48-hr	3.545	3.936	5.407	6.552	8.125	9.358	10.666	12.064	14.028	15.609
3-day	3.897	4.307	5.858	7.059	8.698	9.973	11.316	12.743	14.735	16.329
4-day	4.135	4.558	6.166	7.407	9.090	10.390	11.754	13.205	15.231	16.853
7-day	4.621	5.069	6.790	8.105	9.868	11.205	12.601	14.101	16.205	17.898
10-day	5.035	5.504	7.319	8.697	10.529	11.901	13.328	14.869	17.036	18.781
20-day	6.415	6.951	9.046	10.624	12.695	14.225	15.791	17.461	19.780	21.624
30-day	7.568	8.163	10.499	12.251	14.535	16.213	17.912	19.696	22.140	24.060
45-day	9.180	9.867	12.581	14.606	17.230	19.149	21.063	23.033	25.681	27.724
60-day	10.615	11.388	14.452	16.732	19.673	21.822	23.941	26.084	28.924	31.080

**Table 5.6 AMS-based precipitation frequency estimates for Hood County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.399	0.434	0.569	0.671	0.805	0.905	1.005	1.108	1.245	1.351
10-min	0.641	0.697	0.915	1.079	1.295	1.457	1.618	1.776	1.983	2.137
15-min	0.792	0.862	1.136	1.342	1.611	1.810	2.007	2.206	2.468	2.665
30-min	1.096	1.191	1.563	1.842	2.205	2.474	2.742	3.018	3.389	3.675
60-min	1.424	1.547	2.030	2.393	2.870	3.223	3.582	3.960	4.477	4.885
2-hr	1.750	1.911	2.530	3.002	3.636	4.119	4.618	5.142	5.866	6.439
3-hr	1.945	2.131	2.839	3.386	4.127	4.701	5.300	5.929	6.800	7.490
6-hr	2.298	2.531	3.406	4.086	5.018	5.747	6.517	7.332	8.469	9.377
12-hr	2.678	2.962	4.038	4.870	6.003	6.877	7.804	8.811	10.242	11.405
24-hr	3.105	3.441	4.722	5.706	7.037	8.050	9.128	10.324	12.048	13.466
48-hr	3.606	3.983	5.425	6.534	8.030	9.166	10.379	11.737	13.708	15.339
3-day	3.944	4.346	5.877	7.056	8.650	9.864	11.161	12.608	14.704	16.435
4-day	4.193	4.614	6.220	7.454	9.124	10.395	11.749	13.250	15.416	17.197
7-day	4.734	5.195	6.958	8.311	10.135	11.524	12.988	14.584	16.850	18.691
10-day	5.186	5.678	7.562	9.006	10.946	12.422	13.967	15.632	17.974	19.862
20-day	6.599	7.161	9.330	10.982	13.183	14.845	16.564	18.391	20.930	22.953
30-day	7.764	8.387	10.805	12.637	15.065	16.887	18.755	20.722	23.433	25.574
45-day	9.387	10.122	12.989	15.153	18.000	20.126	22.279	24.509	27.536	29.892
60-day	10.825	11.669	14.971	17.455	20.710	23.134	25.566	28.052	31.387	33.953

**Table 5.7 AMS-based precipitation frequency estimates for Hunt County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.443	0.474	0.596	0.687	0.806	0.894	0.981	1.069	1.187	1.278
10-min	0.708	0.757	0.954	1.101	1.292	1.435	1.574	1.710	1.886	2.016
15-min	0.885	0.946	1.188	1.368	1.603	1.775	1.946	2.120	2.351	2.528
30-min	1.237	1.321	1.653	1.901	2.222	2.457	2.691	2.936	3.267	3.525
60-min	1.617	1.731	2.177	2.511	2.945	3.261	3.583	3.924	4.394	4.767
2-hr	1.996	2.153	2.763	3.224	3.830	4.278	4.740	5.239	5.939	6.501
3-hr	2.222	2.409	3.134	3.684	4.414	4.959	5.526	6.142	7.012	7.715
6-hr	2.637	2.877	3.799	4.504	5.450	6.167	6.919	7.739	8.902	9.846
12-hr	3.102	3.394	4.505	5.363	6.528	7.426	8.373	9.395	10.837	12.002
24-hr	3.620	3.963	5.262	6.267	7.638	8.700	9.826	11.042	12.761	14.152
48-hr	4.197	4.581	6.046	7.171	8.690	9.842	11.070	12.443	14.432	16.075
3-day	4.585	4.994	6.565	7.767	9.380	10.591	11.885	13.346	15.483	17.259
4-day	4.875	5.306	6.962	8.229	9.931	11.210	12.574	14.105	16.333	18.178
7-day	5.512	5.995	7.838	9.254	11.165	12.624	14.161	15.831	18.198	20.118
10-day	6.064	6.585	8.576	10.103	12.162	13.736	15.380	17.129	19.569	21.519
20-day	7.938	8.537	10.888	12.651	14.954	16.649	18.365	20.165	22.626	24.558
30-day	9.511	10.173	12.819	14.775	17.268	19.052	20.816	22.644	25.108	27.013
45-day	11.700	12.469	15.582	17.860	20.717	22.729	24.667	26.615	29.170	31.084
60-day	13.645	14.516	18.058	20.639	23.847	26.094	28.217	30.293	32.953	34.889

**Table 5.8 AMS-based precipitation frequency estimates for Johnson County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.406	0.442	0.583	0.691	0.836	0.947	1.062	1.182	1.347	1.477
10-min	0.649	0.708	0.934	1.107	1.341	1.521	1.705	1.893	2.146	2.342
15-min	0.809	0.882	1.162	1.376	1.663	1.884	2.111	2.348	2.672	2.928
30-min	1.125	1.226	1.612	1.908	2.304	2.605	2.917	3.247	3.702	4.063
60-min	1.466	1.599	2.108	2.498	3.023	3.423	3.840	4.286	4.908	5.406
2-hr	1.806	1.978	2.631	3.137	3.826	4.361	4.927	5.533	6.386	7.073
3-hr	2.010	2.208	2.954	3.535	4.335	4.962	5.629	6.346	7.358	8.175
6-hr	2.382	2.627	3.540	4.256	5.247	6.030	6.869	7.775	9.059	10.101
12-hr	2.797	3.089	4.182	5.037	6.217	7.144	8.139	9.222	10.768	12.028
24-hr	3.258	3.599	4.883	5.882	7.255	8.323	9.469	10.725	12.526	14.001
48-hr	3.774	4.165	5.645	6.790	8.350	9.551	10.835	12.252	14.291	15.965
3-day	4.117	4.538	6.136	7.370	9.045	10.330	11.699	13.211	15.386	17.171
4-day	4.368	4.811	6.491	7.788	9.550	10.902	12.342	13.924	16.193	18.049
7-day	4.911	5.397	7.238	8.661	10.601	12.099	13.688	15.411	17.857	19.842
10-day	5.378	5.896	7.861	9.379	11.442	13.033	14.715	16.531	19.098	21.175
20-day	6.938	7.520	9.781	11.492	13.755	15.428	17.182	19.134	21.939	24.242
30-day	8.236	8.877	11.402	13.288	15.737	17.498	19.329	21.392	24.376	26.839
45-day	10.011	10.777	13.797	16.054	18.983	21.109	23.276	25.610	28.865	31.468
60-day	11.576	12.468	15.972	18.601	22.034	24.576	27.122	29.728	33.228	35.921

**Table 5.9 AMS-based precipitation frequency estimates for Kaufman County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.437	0.471	0.603	0.701	0.829	0.923	1.017	1.113	1.241	1.339
10-min	0.698	0.751	0.964	1.122	1.328	1.482	1.631	1.778	1.968	2.108
15-min	0.875	0.942	1.203	1.398	1.650	1.835	2.019	2.205	2.455	2.646
30-min	1.229	1.320	1.681	1.949	2.295	2.544	2.794	3.057	3.415	3.696
60-min	1.602	1.725	2.209	2.570	3.039	3.380	3.727	4.099	4.617	5.030
2-hr	1.955	2.124	2.770	3.265	3.929	4.432	4.959	5.529	6.333	6.982
3-hr	2.159	2.360	3.117	3.704	4.509	5.135	5.801	6.523	7.546	8.376
6-hr	2.532	2.788	3.738	4.489	5.536	6.372	7.277	8.261	9.669	10.820
12-hr	2.938	3.251	4.394	5.306	6.593	7.632	8.771	10.023	11.833	13.324
24-hr	3.409	3.775	5.119	6.190	7.702	8.919	10.258	11.742	13.901	15.691
48-hr	4.011	4.423	5.964	7.171	8.842	10.150	11.576	13.179	15.531	17.490
3-day	4.408	4.848	6.506	7.795	9.562	10.925	12.404	14.071	16.516	18.553
4-day	4.681	5.142	6.882	8.234	10.084	11.513	13.056	14.784	17.302	19.390
7-day	5.245	5.751	7.652	9.134	11.172	12.764	14.469	16.327	18.979	21.143
10-day	5.739	6.278	8.308	9.888	12.054	13.747	15.544	17.474	20.195	22.391
20-day	7.473	8.080	10.439	12.223	14.579	16.329	18.138	20.089	22.825	25.024
30-day	8.947	9.609	12.240	14.193	16.698	18.489	20.299	22.251	24.968	27.136
45-day	11.027	11.786	14.849	17.096	19.927	21.921	23.872	25.885	28.582	30.653
60-day	12.889	13.742	17.202	19.729	22.886	25.110	27.224	29.302	31.979	33.939

**Table 5.10 AMS-based precipitation frequency estimates for Navarro County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.431	0.469	0.611	0.720	0.865	0.977	1.091	1.208	1.366	1.489
10-min	0.688	0.748	0.977	1.152	1.387	1.569	1.752	1.931	2.165	2.339
15-min	0.864	0.937	1.219	1.434	1.722	1.941	2.164	2.393	2.700	2.937
30-min	1.215	1.315	1.702	1.996	2.389	2.685	2.990	3.312	3.756	4.107
60-min	1.575	1.710	2.230	2.627	3.159	3.562	3.984	4.442	5.089	5.613
2-hr	1.887	2.076	2.784	3.336	4.095	4.686	5.327	6.046	7.098	7.974
3-hr	2.061	2.287	3.122	3.782	4.706	5.436	6.243	7.162	8.525	9.671
6-hr	2.389	2.677	3.724	4.562	5.752	6.710	7.783	9.014	10.859	12.422
12-hr	2.780	3.119	4.344	5.333	6.745	7.894	9.187	10.665	12.876	14.747
24-hr	3.243	3.630	5.033	6.164	7.777	9.088	10.558	12.229	14.718	16.816
48-hr	3.832	4.266	5.868	7.138	8.917	10.328	11.889	13.661	16.290	18.498
3-day	4.222	4.683	6.404	7.753	9.620	11.077	12.674	14.490	17.178	19.433
4-day	4.491	4.970	6.762	8.163	10.098	11.606	13.249	15.102	17.823	20.093
7-day	5.050	5.563	7.480	8.979	11.048	12.668	14.415	16.335	19.098	21.367
10-day	5.539	6.077	8.097	9.670	11.832	13.520	15.321	17.273	20.045	22.298
20-day	7.270	7.862	10.165	11.905	14.200	15.899	17.658	19.564	22.247	24.410
30-day	8.736	9.373	11.917	13.798	16.200	17.905	19.626	21.487	24.083	26.158
45-day	10.780	11.500	14.427	16.562	19.219	21.059	22.850	24.716	27.229	29.169
60-day	12.601	13.401	16.685	19.062	21.981	23.982	25.874	27.762	30.213	32.027



**Table 5.11 AMS-based precipitation frequency estimates for Palo Pinto County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.392	0.426	0.558	0.656	0.785	0.878	0.974	1.075	1.217	1.329
10-min	0.631	0.685	0.898	1.058	1.265	1.418	1.572	1.729	1.942	2.106
15-min	0.779	0.845	1.106	1.300	1.553	1.736	1.922	2.121	2.399	2.620
30-min	1.069	1.159	1.513	1.777	2.117	2.361	2.611	2.884	3.269	3.580
60-min	1.376	1.493	1.957	2.303	2.753	3.080	3.416	3.783	4.303	4.724
2-hr	1.672	1.826	2.419	2.870	3.474	3.928	4.401	4.909	5.620	6.191
3-hr	1.846	2.024	2.702	3.225	3.935	4.484	5.059	5.668	6.516	7.192
6-hr	2.162	2.385	3.216	3.869	4.773	5.492	6.254	7.057	8.174	9.065
12-hr	2.504	2.776	3.773	4.568	5.689	6.597	7.581	8.633	10.121	11.326
24-hr	2.897	3.220	4.398	5.340	6.676	7.761	8.949	10.240	12.092	13.609
48-hr	3.399	3.767	5.132	6.208	7.706	8.893	10.185	11.613	13.681	15.388
3-day	3.732	4.127	5.604	6.759	8.352	9.596	10.943	12.444	14.624	16.429
4-day	3.965	4.378	5.932	7.143	8.807	10.100	11.498	13.054	15.315	17.187
7-day	4.450	4.902	6.605	7.927	9.738	11.140	12.647	14.317	16.732	18.722
10-day	4.862	5.344	7.167	8.581	10.511	12.002	13.599	15.360	17.896	19.979
20-day	6.202	6.774	8.946	10.623	12.902	14.657	16.517	18.538	21.411	23.744
30-day	7.318	7.963	10.422	12.315	14.880	16.851	18.924	21.152	24.286	26.811
45-day	8.881	9.633	12.513	14.720	17.694	19.969	22.337	24.846	28.330	31.105
60-day	10.273	11.121	14.379	16.869	20.208	22.754	25.382	28.136	31.923	34.912

**Table 5.12 AMS-based precipitation frequency estimates for Parker County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.395	0.430	0.565	0.668	0.806	0.911	1.018	1.126	1.270	1.382
10-min	0.635	0.691	0.909	1.075	1.297	1.467	1.639	1.807	2.029	2.195
15-min	0.785	0.854	1.122	1.325	1.597	1.804	2.015	2.228	2.513	2.732
30-min	1.080	1.175	1.542	1.821	2.192	2.472	2.758	3.050	3.445	3.752
60-min	1.399	1.523	2.002	2.367	2.854	3.225	3.604	3.994	4.523	4.934
2-hr	1.720	1.878	2.475	2.937	3.567	4.062	4.572	5.091	5.791	6.332
3-hr	1.915	2.094	2.766	3.291	4.016	4.596	5.198	5.805	6.621	7.250
6-hr	2.272	2.493	3.311	3.956	4.859	5.591	6.357	7.129	8.168	8.971
12-hr	2.670	2.943	3.953	4.750	5.859	6.754	7.697	8.662	9.978	11.007
24-hr	3.122	3.449	4.672	5.631	6.956	8.009	9.122	10.289	11.906	13.191
48-hr	3.655	4.030	5.442	6.539	8.043	9.215	10.461	11.803	13.699	15.234
3-day	3.997	4.402	5.939	7.128	8.744	9.989	11.313	12.760	14.826	16.511
4-day	4.235	4.664	6.288	7.544	9.252	10.567	11.966	13.498	15.686	17.473
7-day	4.726	5.203	7.008	8.407	10.318	11.798	13.373	15.084	17.519	19.499
10-day	5.144	5.660	7.607	9.117	11.179	12.778	14.478	16.320	18.934	21.057
20-day	6.524	7.129	9.448	11.225	13.616	15.427	17.343	19.451	22.469	24.936
30-day	7.681	8.364	10.996	13.000	15.675	17.679	19.789	22.121	25.465	28.204
45-day	9.318	10.135	13.271	15.669	18.884	21.320	23.870	26.627	30.516	33.656
60-day	10.782	11.726	15.333	18.104	21.848	24.726	27.726	30.896	35.296	38.796

**Table 5.13 AMS-based precipitation frequency estimates for Rockwall County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.438	0.470	0.596	0.690	0.812	0.900	0.988	1.078	1.199	1.291
10-min	0.700	0.752	0.954	1.106	1.301	1.446	1.587	1.725	1.905	2.038
15-min	0.875	0.938	1.188	1.374	1.613	1.788	1.960	2.137	2.373	2.553
30-min	1.223	1.310	1.653	1.908	2.235	2.471	2.706	2.953	3.289	3.552
60-min	1.593	1.709	2.168	2.510	2.951	3.271	3.594	3.938	4.415	4.792
2-hr	1.950	2.108	2.722	3.187	3.803	4.263	4.738	5.244	5.949	6.512
3-hr	2.160	2.348	3.067	3.618	4.358	4.922	5.511	6.138	7.013	7.713
6-hr	2.546	2.785	3.690	4.390	5.346	6.088	6.872	7.712	8.892	9.842
12-hr	2.983	3.273	4.363	5.212	6.376	7.284	8.253	9.307	10.807	12.029
24-hr	3.476	3.817	5.099	6.096	7.464	8.527	9.669	10.930	12.747	14.242
48-hr	4.049	4.436	5.903	7.036	8.576	9.753	11.019	12.444	14.527	16.259
3-day	4.426	4.842	6.426	7.645	9.297	10.551	11.899	13.423	15.655	17.513
4-day	4.693	5.133	6.804	8.092	9.842	11.177	12.610	14.218	16.559	18.500
7-day	5.258	5.749	7.602	9.042	11.017	12.553	14.194	15.979	18.524	20.598
10-day	5.754	6.284	8.282	9.835	11.965	13.629	15.392	17.278	19.928	22.061
20-day	7.520	8.127	10.478	12.263	14.632	16.412	18.245	20.186	22.875	25.010
30-day	9.016	9.685	12.326	14.298	16.851	18.709	20.581	22.547	25.238	27.349
45-day	11.094	11.870	14.979	17.272	20.187	22.272	24.313	26.392	29.156	31.259
60-day	12.945	13.822	17.363	19.959	23.226	25.546	27.770	29.966	32.812	34.912

**Table 5.14 AMS-based precipitation frequency estimates for Somervell County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.409	0.443	0.578	0.679	0.812	0.910	1.009	1.110	1.246	1.351
10-min	0.656	0.711	0.928	1.091	1.305	1.466	1.624	1.780	1.984	2.136
15-min	0.813	0.882	1.150	1.352	1.618	1.817	2.015	2.213	2.472	2.668
30-min	1.127	1.220	1.586	1.861	2.220	2.487	2.754	3.028	3.398	3.684
60-min	1.462	1.585	2.067	2.429	2.903	3.252	3.608	3.986	4.507	4.920
2-hr	1.796	1.959	2.589	3.069	3.711	4.196	4.701	5.239	5.990	6.591
3-hr	1.993	2.184	2.913	3.474	4.233	4.816	5.429	6.084	7.003	7.741
6-hr	2.338	2.579	3.487	4.192	5.155	5.905	6.700	7.558	8.770	9.752
12-hr	2.681	2.974	4.086	4.945	6.116	7.017	7.979	9.036	10.553	11.795
24-hr	3.065	3.409	4.718	5.728	7.097	8.144	9.263	10.508	12.308	13.793
48-hr	3.545	3.927	5.374	6.494	8.021	9.197	10.457	11.853	13.868	15.527
3-day	3.878	4.282	5.810	6.994	8.612	9.862	11.199	12.670	14.785	16.519
4-day	4.132	4.553	6.151	7.387	9.072	10.372	11.758	13.272	15.437	17.204
7-day	4.700	5.159	6.909	8.255	10.075	11.468	12.933	14.512	16.736	18.530
10-day	5.175	5.662	7.528	8.956	10.873	12.332	13.852	15.476	17.744	19.561
20-day	6.652	7.198	9.320	10.924	13.040	14.613	16.229	17.952	20.345	22.251
30-day	7.860	8.457	10.804	12.562	14.851	16.528	18.230	20.036	22.527	24.499
45-day	9.515	10.210	12.972	15.024	17.666	19.581	21.484	23.445	26.083	28.118
60-day	10.970	11.763	14.927	17.268	20.259	22.420	24.527	26.638	29.409	31.489

**Table 5.15 AMS-based precipitation frequency estimates for Tarrant County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405	0.439	0.572	0.673	0.807	0.909	1.012	1.117	1.261	1.371
10-min	0.649	0.704	0.919	1.081	1.297	1.462	1.627	1.791	2.008	2.171
15-min	0.807	0.875	1.137	1.336	1.601	1.801	2.004	2.212	2.493	2.711
30-min	1.117	1.210	1.570	1.842	2.203	2.475	2.752	3.041	3.436	3.746
60-min	1.449	1.572	2.047	2.408	2.888	3.248	3.619	4.011	4.553	4.983
2-hr	1.773	1.935	2.555	3.031	3.671	4.159	4.669	5.215	5.977	6.588
3-hr	1.966	2.154	2.871	3.423	4.173	4.752	5.361	6.013	6.930	7.668
6-hr	2.323	2.558	3.446	4.135	5.077	5.810	6.586	7.419	8.591	9.536
12-hr	2.733	3.016	4.089	4.919	6.051	6.928	7.852	8.840	10.224	11.336
24-hr	3.191	3.522	4.776	5.747	7.069	8.096	9.175	10.321	11.921	13.201
48-hr	3.692	4.063	5.447	6.532	8.033	9.225	10.488	11.816	13.663	15.136
3-day	4.018	4.417	5.901	7.067	8.686	9.976	11.348	12.792	14.805	16.411
4-day	4.258	4.680	6.252	7.488	9.204	10.573	12.030	13.569	15.716	17.433
7-day	4.771	5.249	7.037	8.438	10.375	11.908	13.541	15.281	17.726	19.693
10-day	5.216	5.736	7.688	9.211	11.306	12.950	14.699	16.573	19.213	21.343
20-day	6.739	7.340	9.640	11.404	13.780	15.588	17.492	19.564	22.505	24.891
30-day	8.012	8.684	11.283	13.257	15.883	17.846	19.898	22.138	25.318	27.899
45-day	9.755	10.569	13.712	16.103	19.289	21.690	24.175	26.822	30.508	33.449
60-day	11.294	12.248	15.917	18.720	22.477	25.346	28.296	31.362	35.554	38.840

**Table 5.16 AMS-based precipitation frequency estimates for Wise County (inches)**

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.404	0.437	0.569	0.667	0.795	0.889	0.983	1.078	1.206	1.303
10-min	0.648	0.702	0.914	1.072	1.278	1.431	1.581	1.729	1.922	2.066
15-min	0.804	0.870	1.131	1.326	1.578	1.763	1.946	2.133	2.384	2.576
30-min	1.111	1.201	1.560	1.826	2.170	2.418	2.667	2.925	3.274	3.546
60-min	1.438	1.557	2.027	2.377	2.832	3.165	3.500	3.850	4.327	4.700
2-hr	1.762	1.916	2.508	2.959	3.561	4.018	4.485	4.970	5.633	6.151
3-hr	1.955	2.131	2.803	3.321	4.023	4.568	5.133	5.715	6.509	7.130
6-hr	2.303	2.523	3.346	3.990	4.879	5.587	6.328	7.090	8.129	8.943
12-hr	2.682	2.954	3.963	4.757	5.863	6.753	7.691	8.654	9.969	11.000
24-hr	3.105	3.432	4.645	5.602	6.937	8.011	9.148	10.326	11.948	13.228
48-hr	3.592	3.969	5.376	6.481	8.013	9.231	10.531	11.912	13.850	15.407
3-day	3.920	4.330	5.864	7.066	8.726	10.036	11.437	12.950	15.094	16.834
4-day	4.168	4.604	6.233	7.510	9.277	10.670	12.165	13.787	16.096	17.976
7-day	4.714	5.207	7.040	8.482	10.487	12.076	13.790	15.655	18.322	20.501
10-day	5.173	5.709	7.701	9.269	11.450	13.181	15.049	17.084	19.994	22.373
20-day	6.619	7.255	9.641	11.504	14.067	16.074	18.224	20.567	23.911	26.639
30-day	7.814	8.533	11.244	13.348	16.229	18.468	20.852	23.439	27.114	30.101
45-day	9.485	10.339	13.557	16.059	19.488	22.167	25.003	28.028	32.270	35.679
60-day	10.968	11.949	15.634	18.506	22.455	25.565	28.841	32.283	37.055	40.850

## 6.0 Hydrologic Soils Data

### 6.1 Electronic Soil Maps

Electronic soils data in the Soil Survey Geographic (SSURGO) Database can be obtained free of charge from the National Resource Conservation Service (NRCS) at <http://soildatamart.nrcs.usda.gov>. The data is downloadable by county and includes extensive soil information, including hydrologic soil groups. The data is intended to be imported into a geographic information system (GIS) to allow for site-specific soil analysis of soil characteristics for storm design. All soil survey results can also be accessed online at <http://websoilsurvey.nrcs.usda.gov/app/>. Maps can be created and printed from this site without the use of a GIS. An example SSURGO Soil Map for West Tarrant County is shown in Figure 6.1.

### 6.2 Online Web Soil Survey

Following is a procedure for using the NRCS online Web Soil Survey.

1. Go to <http://websoilsurvey.nrcs.usda.gov/app/>
2. Click **Start WSS**
3. Define your Area of Interest by drawing a box around your site location.
4. Click the **Soil Map** tab
5. Click **Save or Print** in the upper right hand corner. A pdf will open in a new window that you can either print or save. It will show the area of interest along with a legend and the appropriate map units.

### 6.3 Downloading Soil Surveys

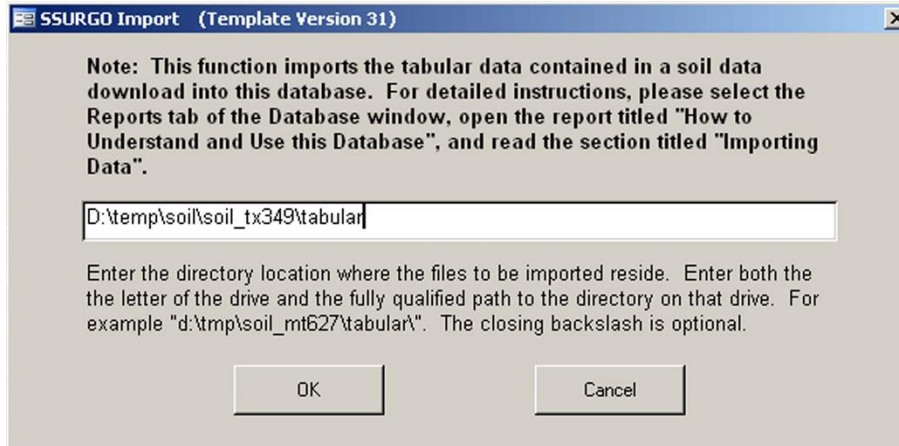
Following is a procedure for downloading data from the NRCS web site and importing it into ArcGIS.

#### Downloading SSURGO Soil Data into ArcInfo 9.x

1. Go to <http://soildatamart.nrcs.usda.gov>
2. Click **Select State**
3. Select State (**Texas**)
4. Select **County** of interest
5. Click **Select Survey Area**
6. Click **Download Data**
7. Enter your e-mail address in the provided form space
8. Click **Submit Request**
9. You will receive an immediate message acknowledging your request and a follow-up e-mail once your request has been processed.
10. The file(s) will be provided in compressed ZIP format, requiring the use of WinZip to extract.
11. Extract the files to a destination directory of your choice. The extracted files contain a spatial sub-folder, a tabular sub-folder, and a zip file containing the SSURGO MS Access template file.

### Importing raw tabular soil data into Microsoft Access

1. Extract the soildb\_US\_2002.zip file into the same destination folder by using the “extract to here” command in WinZip. This will extract the template database.
2. Open the template database and input the path name to the tabular data. This will build the SSURGO database and allow the creation of reports and queries.

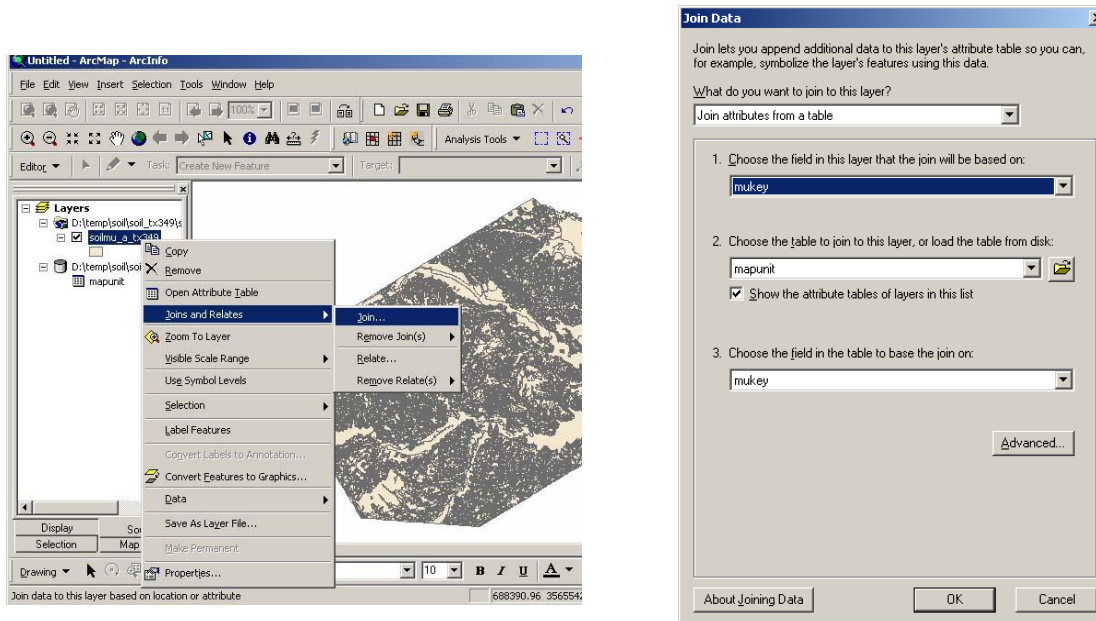


3. Once the data is imported into the database, a report can be run. With the soil reports dialog box up, press the **Select All** option and generate the report. Note: Regardless of what report you wish to run, all reports are simultaneously created. The selected report is displayed on the screen.
4. All the reports are now complete, and the tables can now be added directly into ArcGIS.

### Joining tables to shapefiles in ArcGIS

1. Open **ArcGIS** and add the **Soils** shapefile.
2. Add the “**mapunit**” report to ArcGIS by navigating directly to the MS Access database and opening it (via the add data dialog box). Note: mapunit is only a commonly used example, containing full soil names and prime farmland information.
3. Now that the table is added to the Table of Contents, it is ready to be joined to the existing soils shapefile.
4. Right click on the **soils** shapefile and select **join**.





5. Under the **Join Data** dialog box, select the **mukey** field in Dropdown Box 1 and select **mapunit** in Dropdown Box 2.
6. Now that your shapefile is joined with the appropriate information, the next step is to export the shapefile into a new shapefile with the joins saved permanently. Right click on the **soils** shapefile and choose **Data > Export** and **Save** your file.

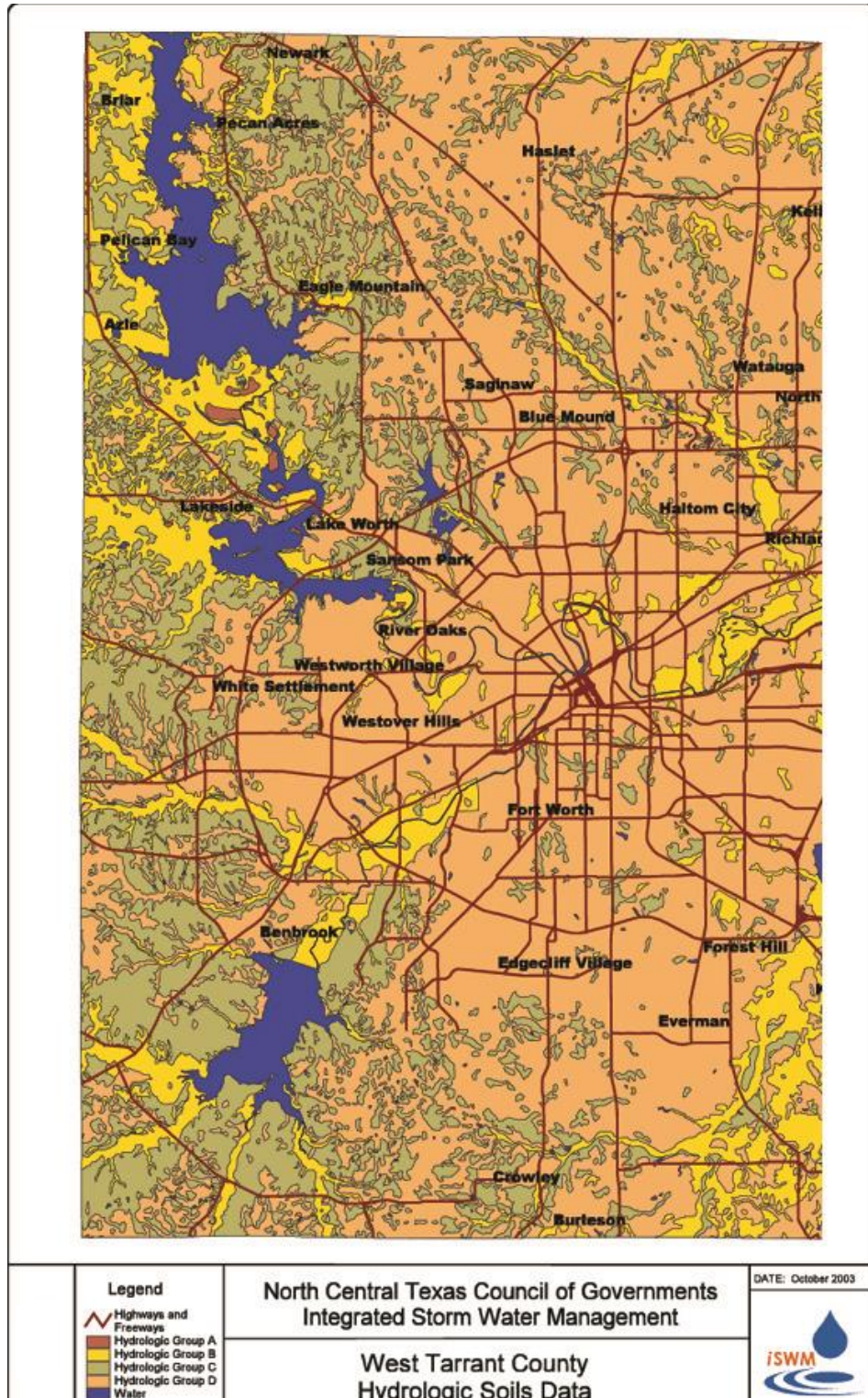


Figure 6.1 Example SSURGO Soil Map – West Tarrant County