STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK
HYDROLOGY

OFFICE OF DESIGN, DRAINAGE SECTION FEBRUARY 2012
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TABLE OF CONTENTS

Chapter 1

Introduction

1.1 Background

The 1987 Florida Department of Transportation Drainage Manual was published as a three volume set: Volume 1 - Policy; Volumes 2A and 2B - Procedures; Volume 3 - Theory. On October 1, 1992, Volume 1 - Policy was revised to Volume 1 - Standards. With that revision, Volumes 2A, 2B, and 3 were designated as general reference documents. The Volume 1 - Standards was revised in January 1997 as was renamed to simply the "Drainage Manual". No revisions have been nor will be made to Volumes 2A, 2B, and 3 of the 1987 Drainage Manual.

This handbook is one of several the Central Office Drainage section is developing to replace Volumes 2A, 2B, and 3 of the 1987 Drainage Manual. In this form, the current Drainage Manual will be maintained as a "standards" document, while the handbooks will cover general guidance on FDOT drainage design practice, analysis and computational methods, design aids, and other reference material.

1.2 Purpose

This handbook is intended to be a reference for designers of FDOT projects, and to provide guidelines for the hydrologic analysis of storm drains, cross drains, and stormwater management. Pertinent sections of the 1987 Drainage Manual have been incorporated into this handbook.

The guidance and values provided in this handbook are suggested or preferred approaches and values, not requirements nor standards. The values provided in the Drainage Manual are the minimum standards. In cases of discrepancy, the Drainage Manual standards shall apply. As the Drainage Manual states about the standards contained in it, situations exist where the guidance provided in this handbook will not apply. The inappropriate use of and adherence to the guidelines contained herein does not exempt the engineer from the professional responsibility of developing an appropriate design.

1.3 Distribution

This handbook is available for downloading from the Drainage Internet site.

1.4 Revisions

Any comments or suggestions concerning this handbook may be made by e-mailing the State Hydraulics Engineer.

1.5 Definitions of Terms and Acronyms

Abstraction	Hydrologic processes that remove water from precipitation before it becomes surface runoff; types include evaporation, infiltration, transpiration, interception, depression storage, and detention storage.
Curve number	A dimensionless site-specific runoff parameter developed by the (former) Soil Conservation Service (now National Resource Conservation Service) to empirically estimate rainfall excess; it accounts for infiltration losses and initial abstractions.
Duration	The time of a rainfall hyetograph used to perform runoff calculations.
Flood hydrograph	A continuous plot of the surface runoff flow rate versus time. The volume is equal to the volume of water contained in the rainfall excess hyetograph.
Hydrology	The science dealing with the disposition of water on the earth.
Infiltration	Abstraction process in which water flows or is absorbed into the ground.
Intensity	The rate of precipitation, usually in inches/hour.
Overland flow	That water which travels over the ground surface to the stream channel, usually limited to a maximum length of 100 feet.
Regression equation	A statistical method that correlates peak discharge with physical features such as watershed area and stream slope.
Runoff	Precipitation remaining after appropriate hydrologic abstractions have been accounted for.
Runoff coefficient	Empirical parameter used to calculate rainfall excess as a fixed percentage of precipitation; it accounts for interception, surface

storage, and infiltration.

Time of concentration	The time of travel from the hydraulically most distant point of a watershed to the design point.
Watershed	An area bounded peripherally by a drainage divide that concentrates runoff to a particular water course or body; the catchment's area or drainage basin from which the waters of a stream are drawn.
Watershed lag time	Time from the center of mass of the rainfall excess to the runoff hydrograph peak.

Chapter 2

Hydrology

2.1 Drainage Data

Identification of drainage data needs should be a part of the early design phase of a project, when appropriate procedures for performing hydrologic and hydraulic calculations are selected. Several categories of data may be relevant to a particular project, including published data such as precipitation, soils, land use, topography, streamflow, and flood history. Published mapping is usually inadequate, so field investigations and surveys are necessary to determine drainage areas, identify pertinent features, obtain high water information, survey lateral ditch alignments, and survey bridge and culvert crossings. Information on types of data available and the sources of that data are presented in Appendix A of this handbook.

2.2 Procedure Selection

Streamflow measurements for determining peak runoff rates for pre-project conditions are often unavailable. Where measurements are available, the Department usually relies upon agencies such as the USGS to perform the statistical analysis of streamflow data; however, guidelines for determining floodflow frequencies from observed streamflow data may be obtained from Bulletin 17B of the U.S. Water Resources Council (revised 1981).

Where streamflow measurements are not available, it is accepted practice to estimate peak runoff using the rational method or one of the regression equations developed for Florida. In general, the method that best reflects project conditions should be used, with the reasons for using the method documented.

Consideration of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm drains or open channels. However, if the design must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required. The development of a runoff hydrograph is usually accomplished using computer programs.

In general, procedures using streamflow analysis and unit hydrograph theory are applicable to all watershed categories.

Guidelines for selecting peak runoff rate and flood hydrograph procedures are presented in Table 1.

TABLE 1
GUIDELINES FOR SELECTING PEAK RUNOFF RATE AND FLOOD HYDROGRAPHS

		Peak Runoff Rates						Flood Hydrographs
Application	Watershed Category	Streamflow Analysis	Rational Method	Natural Flow USGS Equations	Developed USGS Equations	Developed Tampa Equations	Developed Leon County Equations	[a] Modified Rational Method or SCS Unit Hydrograph
Storm Drains	0 to 600 acres	X	X					
Cross Drains Side Drains	0 to 600 acres	X	X		X	X	X	
	600+ acres	X		X	X			
Stormwater Management	None	X						X

[a] The modified rational method is not recommended for drainage basins with t_c greater than 15 minutes.

2.2.1 Rainfall Data

The Department has developed Intensity-Duration-Frequency (IDF) curves for 11 zones in Florida using depth-duration-frequency data from HYDR0-35 and TP-40. The curves are available on the Drainage Internet Site. The IDF curves developed by the Department are intended to provide a reasonable basis for design, and in areas where intensities would vary, reflect values near locations of higher development.

Depth-frequency data for durations of 1, 2, 4, 7, and 10 days, which depict maps of Florida with contours of precipitation depth for return period frequencies of 2, 5, 10, 25, 50, and 100 years are also available on the internet.

Frequency can be defined either in terms of an exceedance probability or a return period. Exceedance probability is the probability that an event having a specified volume and duration will be exceeded in a specified time period (usually one year). Return period is the average length of time between events having the same volume and duration. The problem with using return period is that it can be misinterpreted. If a 50-year flood occurs one year, some people believe that it will be fifty years before another flood of that magnitude occurs. Instead, because floods occur randomly, there is a finite probability that the 50-year flood could occur in two consecutive years. The exceedance probability (p) and return period (T) are related as follows:

$$p = \frac{1}{T} \quad \textbf{(1)}$$

A 25-year storm has a 0.04 or 4% exceedance probability (probability of occurrence in any given year), a 50-year storm is 0.02 or 2%, etc.

2.2.2 Time of Concentration

The time of concentration is defined as the time it takes runoff to travel from the most remote point in the watershed to the point of interest. Either of the following methods are suggested for calculating the time of concentration:

1. Velocity Method

The Velocity Method is a segmental approach, which can be used to account for overland flow, shallow channel flow (rills or gutters), and main channel flow. By considering the average velocity in each segment being evaluated, a travel time can be calculated using the equation:

$$t_i = \frac{L_i}{60\ v_i} \qquad \textbf{(2)}$$

where:
t_i = Travel time for velocity in segment I, in minutes
L_i = Length of the flow path for segment I, in ft
v_i = Average velocity for segment I, in ft/sec

The time of concentration can then be calculated as:

$$t_c = t_1 + t_2 + t_3 + \dots t_i \qquad \textbf{(3)}$$

where:
t_c = Time of concentration, in minutes
t_1, t_2, t_3, t_i = Travel time in minutes for segments 1, 2, 3, i, respectively

The segments should have uniform characteristics and velocities. Determination of travel time for overland flow, shallow channel flow, and main channel flow are discussed below:

A. Overland Flow (t_1)

The time of concentration for overland flow may be developed using Figure F-2 in the Design Aids, if the average slope and the land use are known. This chart gives reasonable values and is used by district drainage staff around the state.

The Kinematic Wave Equation developed by Ragan (1971) is preferred by FHWA for calculating the travel time for overland conditions. Figure F-1 in the Design Aids presents a nomograph that can be used to solve this equation, which is expressed as:

$$t_1 = \frac{0.93 L^{0.6} n^{0.6}}{i^{0.4} s^{0.3}} \qquad \textbf{(4)}$$

where:
t_1 = Overland flow travel time, in minutes
L = Overland flow length, in ft (maximum 100 feet)
n = Manning roughness coefficient for overland flow (See Table T-1 in the Design Aids)
i = Rainfall intensity, in inches/hr
S = Average slope of overland flow path, in ft/ft

Manning's n values reported in Table T-1 in the Design Aids were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations. Equation 4 generally entails a trial and error process using the following steps:

1. Assume a trial value of rainfall intensity (i).
2. Find the overland travel time (t_1), using Figure F-1.
3. Find the actual rainfall intensity for a storm duration of t_1, using the appropriate IDF curve.
4. Compare the trial and actual rainfall intensities. If they are not similar, select a new trial rainfall intensity and repeat the process.

B. Shallow Channel Flow (t_2)

Average velocities for shallow channel flow in rills and gutters can be obtained directly from Figure F-3 in the Design Aids, if the slope of the flow segment in percent is known. The velocity can be calculated by:

$$V = kS^{0.5} \qquad \textbf{(5)}$$

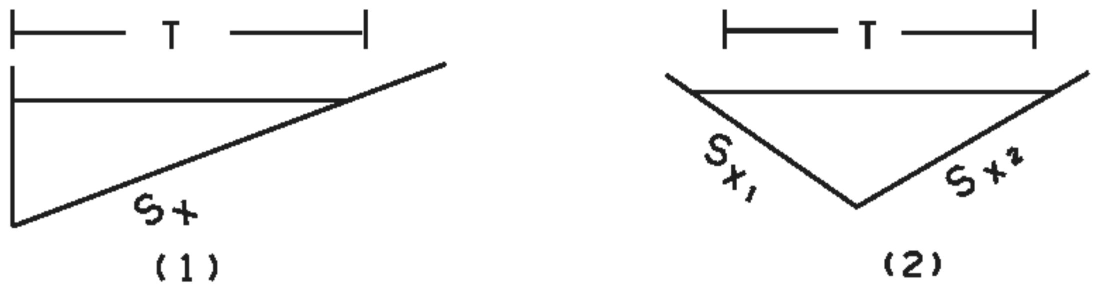
Where:
V = velocity (feet per second)
S = longitudinal slope in percent
k = 1.61 for rills (shallow concentrated flow, unpaved)
2.03 for gutters (shallow concentrated flow, paved area)

Gutter flow velocities can also be calculated using the following equation:

$$V = \frac{1.12}{n} S^{0.5} S_X^{0.67} T^{0.67} \qquad \textbf{(6)}$$

where:

S = longitudinal slope
n = Manning's n for street and pavement gutters (Table T-2)
S_X and T are as shown on (1) below.



For a triangular gutter as shown in (2),

$$S_X = \frac{S_{X1} S_{X2}}{S_{X1} + S_{X2}} \qquad \textbf{(7)}$$

The conventional form of Manning's equation can also be used to evaluate shallow channel flow.

C. Main Channel Flow (t_3)

Flow in rills, gullies and/or gutters empties into channels or pipes. Open channels can be assumed to begin where either a blue line stream shows on USGS quad maps or where the channel is visible on aerial photos. Average velocities for main channel flow should be evaluated using Manning's Equation.

$$V = \frac{1.486}{n} R^{0.67} S^{0.5} \qquad \textbf{(8)}$$

Where:

V = Velocity in feet per second
x = Manning's n value from Table T-3
R = Hydraulic Radius (A/P)
S = slope

2. Kirpich (1940) Equation

The Kirpich equation can be used for rural areas to estimate the watershed t_c directly. The Kirpich equation is based on data reported by Ramser (1927) for six small agricultural watersheds near Jackson, Tennessee. The slope of these watersheds was steep, the soils well drained, the timber cover ranged zero to 56 percent, and watershed areas ranged from 1.2 to 112 acres. Although this data appears to be limited and site-specific, the Kirpich equation has given good results in Florida applications. The Kirpich equation is expressed as:

$$t_c = 0.0078 \frac{L^{0.77}}{S^{0.385}} F_s \tag{9}$$

where:
t_c = Time of concentration, in minutes
L = Length of travel, in ft
S = Slope, in ft/ft
F_s = 1.0 for natural basins with well-defined channels, overland flow on bare earth, and mowed grass roadside channels
= 2 for overland flow on grassed surfaces
= 0.4 for overland flow on concrete or asphaltic surfaces
= 0.2 for concrete channels

The flow path should be separated into different reaches if there are breaks in the slope and changes in the topography. The time of travel in each reach are added together to obtain the time of concentration (See Equation 3).

2.2.3 Peak Runoff Rates-Ungaged Sites

Synthetic procedures recommended for developing peak flow rates include the rational equation and USGS regression equations.

Rational Equation

The rational equation is an easy method for calculating peak flow rates. The equation is expressed as:

$$Q = C\,i\,A \tag{10}$$

Where: Q = Peak Flow Rate (cfs)
C = Runoff Coefficient
i = Rainfall Intensity (inches/hour)
A = Area (acres)

A. Runoff Coefficient

The runoff coefficient is a dimensionless number, which represents the percent of rainfall that runs off a site. Table T-4 in the Design Aids presents runoff coefficient ranges for various land uses, soil types, and watershed slopes. A site review and engineering judgment should be used to select the coefficient within these ranges. Table T-5 presents adjustment factors for pervious area runoff coefficients for design storm frequencies greater than 10 years. (Note: The adjusted runoff coefficient should not be greater than 1. See Example 1.) For sites with several land uses, the runoff coefficient should be a weighted average expressed as:

$$Weighted\ C = \frac{\Sigma\, C_i\, A_i}{A_{Total}} \qquad \textbf{(11)}$$

B. Rainfall Intensity

The rainfall intensity is determined from the appropriate Intensity-Duration-Frequency (IDF) curve based on the time of concentration and the storm frequency (recurrence interval).

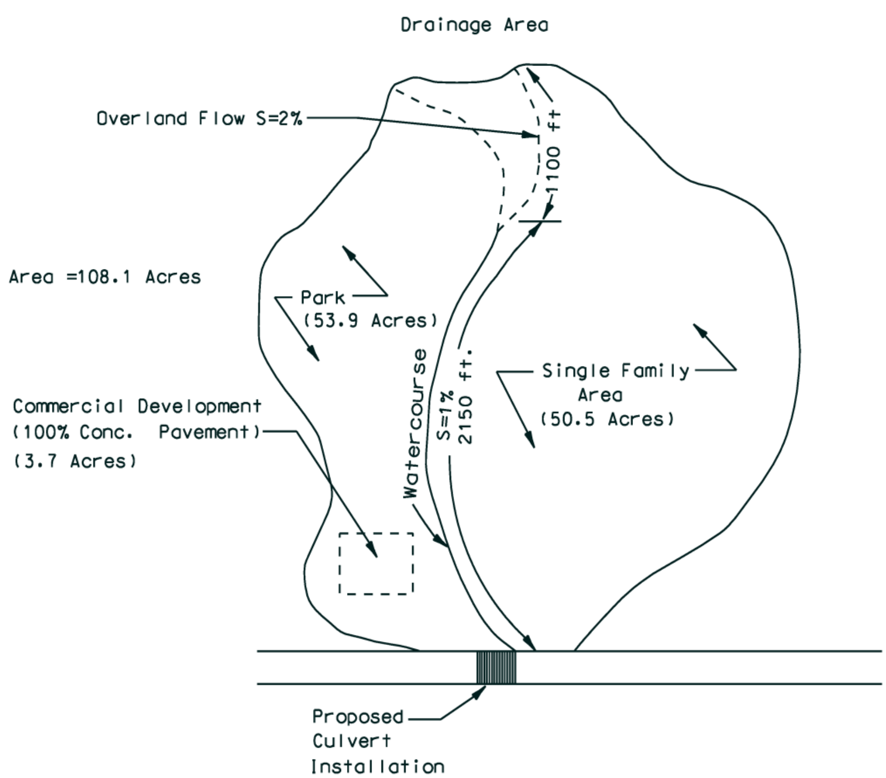
C. Assumptions and Limitations

1. Rainfall is constant for the duration of the time of concentration.

2. Peak flow occurs when the entire watershed is contributing.

3. Drainage area is limited to those given in FDOT Drainage Manual.

Example 1: Use of the Rational Method



A flooding problem exists along a farm road near Zolfo Springs, Hardee County (sandy soil, zone 8). A low water crossing is to be replaced by a culvert to improve the road safety during rainstorms. The drainage area is as sketched above and has an area of 108.1 acres. Determine the maximum flow the culvert must pass for a 25-year storm.

1. Determine the weighted "C" assuming sandy soil. From the sketch and Tables T-4 and T-5, develop a summary of "C" values, adjusted for design storm frequency.

Description	"C" Value	Adjustment	Adjusted C	Area	C_i A_i
Park	0.20	1.1	0.22	53.9	11.9
Commercial Development	0.95	N/A	0.95	3.7	3.5
Single Family	0.40	1.1	0.44	50.5	22.2
	TOTALS			108.1	37.6

$$Weighted\ C = \frac{\Sigma C_i A_i}{A} = \frac{37.60}{108.1} = 0.35$$

2. Determine Intensity. To determine the intensity, the time of concentration (t_c) must first be determined.

   a. Overland flow (1100 ft) - "Residential" at 2 percent slope. From Figure F-2, Velocity = 57 ft/min

   $$t_1 = \Sigma \frac{Distance_1}{Velocity_1} = \frac{1100\ ft}{57\ ft/\min} = 19.3\ \min.$$

   b. Channelized flow (2150 ft) - "Rills" at 1 percent slope. From Figure F-3, Velocity = 5.75 ft/sec

   $$t_2 = \Sigma \frac{Distance_2}{Velocity_2} = \frac{2150\ ft}{5.75\ ft/\sec\ x\ 60\ \sec/\min} = 6.2\ \min.$$

   c. Time of Concentration is estimated as:

   $t_c = t_1 + t_2 = 19.3 + 6.2 = 25.5$ min.

   d. Intensity is obtained from the Department's IDF Curves using a duration equal to the time of concentration (t_c). IDF Curves are available on the Drainage Internet Site.

   $i_{25} = 6$ in/hr

3. Calculate the peak flow.

   $Q_{25} = C \text{ x } i_{25} \text{ x } A$

   $Q_{25} = 0.35 \text{ x } 6 \text{ x } 108.1 = 227$ cfs

Regression Equations

Natural Conditions

Regression equations developed by the USGS (Verdi, 2006) can be used to estimate peak runoff for natural flow conditions.

The USGS Equations in “Magnitude and Frequency of Floods for Rural Streams in Florida, 2006” by Verdi (2006) supersede the information presented by Bridges (1982) and in the USGS Water Supply Paper (WSP) No. 1674 by Pride (1958). Although not recommended as a design procedure, the method presented in WSP No. 1674 can be used as an independent check for evaluating natural flow estimates for watershed areas between 100 and 10,000 square miles.

The Statistical Analysis System (SAS) was used to perform multiple regression analyses of flood peak data from 275 gagging stations in Florida and 30 in the adjacent states of Georgia and Alabama. Tables T-10 through T-13 in the Design Aids show the USGS Regression Equations for each designated region in the State of Florida.

The natural flow regression equations for Regions 1 through 4 take the following general form:

$$Q_T = C A^a (ST + 1.0)^b \quad \textbf{(12)}$$

Where:

Q_T = Peak runoff rate for return period T, in ft^3/sec.
C = Regression constant (See T-10 through T-13).
A = The drainage area in square miles.
ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST.
a, b = Regression exponents (See T-10 through T-13).

The standard error of prediction, in percent, is reported for each natural flow regression equation for each of the Regions 1 through 4, tables T-10 through T-13. The standard error of prediction is a measure of how well the regression equation estimates flood flows when applied to ungaged basins.

The square of the multiple regression coefficient (R^2), unit less, and the standard error, in percent, are reported for each regression equation for the urban, and Tampa Bay area and Leon County, tables T-14 through T-16. The R^2 value provides a measure of the equation's ability to account for variation in the dependent variable. The standard error is the standard deviation of the distribution of residuals about the regression line.

The standard error of model, in percent, is reported for each West-Central Florida regression equation, table T-17. The standard error of model is a measure of how well the regression equation model estimates flood flows.

When applying the regression equations, the following limitations should be considered:

1. The relationship of the regression equations for areas with basin characteristics outside the ranges given above. The equations are not to be used for watershed conditions outside the range of applicability shown in Tables T-10 through T-13 in the Design Aids.

2. In areas of karsts topography for the Tampa Area and Leon County regression equations, some basins may contain closed depressions and sinkholes, which do not contribute to direct runoff. When the drainage area is determined from 7.5-minute topographic maps, any area containing sinkholes or depressions (non-contributing areas) should be subtracted from the total drainage area.

3. Regression equations are not applicable where manmade changes have a significant effect on the runoff. These changes may include construction of dams, reservoirs, levees and diversion canals, strip mines, and areas with significant urban development.

To apply the USGS regression equations, the following steps should be taken:

1. Locate the appropriate region on Figure F-4.
2. Select the appropriate table (from Tables T-10 through T-13) for the region in which your site is located.
3. Determine the input parameters for your selected regression equation.
4. Calculate peak runoff rates for the desired return periods.

Urban Conditions

Regression equations developed by the USGS as part of a nationwide project can be used to estimate peak runoff for urban watershed conditions. Regionalized regression equations for the Tampa Bay area, Leon County, and West-Central Florida are also available.

1. Nationwide Equations

Sauer et al. (1983) provide two seven parameter equations and a third set based on three parameters. The seven-parameter equations based on lake and reservoir (presented in Table T-14 of the Design Aids) are recommended. The equations account for regional runoff variations through the use of the equivalent rural peak runoff rate (RQ). The equations adjust RQ to an urban condition using the basin development factor (BDF), the percentage of impervious area (IA), and other variables. These equations have the following general form:

$$UQ_T = C\ A^{B_1}\ SL^{B_2} (i_2 + 3)^{B_3} (ST + 8)^{B_4} (13 - BDF)^{B_5}\ IA^{B_6} (RQ_T)^{B_7} \qquad \textbf{(14)}$$

where:

UQ_T = Peak discharge, in ft^3/sec, for the urban watershed for recurrence interval T.

C = Regression constant (See Table T-14).

A = Contributing drainage area in $miles^2$.

SL = Channel slope (ft/mi) between points 10 and 85 percent of the distance from the design point to the watershed boundary.

i_2 = Rainfall intensity, in inches, for the 2-hour, 2-year occurrence.

ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST.

BDF = The basin development factor is an index of the prevalence of (1) channel improvements, (2) impervious channel linings, (3) storm drains, and (4) curb and gutter streets and ranges from 0 to 12. More discussion and an example follow these definitions.

IA = Impervious area is the percentage of the drainage basin occupied by impervious surfaces, such as buildings, parking lots, and streets.

RQ_T = Peak discharge, in cfs, for an equivalent rural drainage basin in the same hydrologic area as the urban basin for recurrence interval T. This value is developed using the USGS regression equations for natural flow conditions for the appropriate region.

B_1 through B_7 = Regression exponents (See Table T-14).

Basin Development Factor - The BDF should be determined from drainage maps and by field inspection of the watershed. The basin is first divided into three sections so that each sub-area contains approximately one third of the drainage area. Distances along main streams and tributaries should be marked so that within each third the travel distances of two or more streams are about equal. The lines can generally be drawn on the drainage map by visual estimate without the need for measurements. Complex basin shapes and drainage patterns require more judgment when subdividing.

Four drainage aspects are examined for each subsection, with a code of zero or one being assigned to each aspect for each subsection. The BDF, therefore, can range from zero for an undeveloped watershed to twelve for a completely urbanized watershed. A code of zero does not mean that the watershed is completely unaffected by urbanization. A basin could have some impervious area, some improved channels and some curb and gutter streets and still have a BDF of zero. The four drainage aspects are:

(1) Channel Improvements - If 50% or more of the main channels and principal tributaries (those that drain directly into the main channel) have been improved from natural conditions, a code of one is assigned; otherwise a code of zero is assigned. Improvements include straightening, enlarging, deepening and clearing.

(2) Channel Linings - A code of one is assigned if more than 50% of the length of the main channels and principal tributaries have impervious linings, such as concrete; otherwise a code of zero is assigned. Lined channels are an indication of a more developed drainage system in which channels have probably been improved.

(3) Storm Drains - Storm drains are enclosed drainage structures (usually pipes) frequently used on the secondary tributaries (those that drain into principal tributaries) which receive drainage directly from streets or parking lots. Many of these drains empty into open channels; in some basins, however, they empty into channels enclosed as box or pipe culverts. When more than 50% of the secondary tributaries within a sub-basin consist of storm drains, a code of one is assigned to this aspect; otherwise a code of zero is assigned. Note that if 50% or more of the main drainage channels and principal tributaries is enclosed, the aspects of channel improvements and channel linings would also be assigned a code of one.

(4) Curb and Gutter Streets - If more than 50% of a sub-basin is urbanized (covered by residential, commercial, or industrial development), and if more than 50% of

the streets and highways in the sub-basin are constructed with curbs and gutters, than a code of one will be assigned to this aspect; otherwise a code of zero will be assigned. Drainage from curb and gutter streets frequently empties into storm drains.

These guidelines are not intended to be precise measurements. A certain amount of subjectivity will be involved, and field checking should be performed to obtain the best estimate.

Example 2 - Estimating the BDF

A watershed is divided into three sub-areas based on homogeneity of hydrologic conditions. Information for the watershed is collected from topographic maps and from field reviews and is tabulated below:

Subarea	Main channel length (ft)	Length of secondary tributaries (ft)	Road length (ft)	Length of channel improved (ft)	Length of channel lined (ft)	Length of storm drains (ft)	Length of curb & gutter (ft)
Upper	2500	5180	2850	460	0	1345	690
Middle	3800	3940	4700	2020	1770	2330	3020
Lower	3000	2160	5610	1720	1570	1510	3180

The BDF is determined as follows:

Channel Improvements
Upper third - 460 ft have been straightened and deepened
460 / 2500 < 50% Code = 0
Middle third - 2020 ft have been straightened and deepened
2020 / 3800 > 50% 1
Lower third - 1720 ft have been straightened and deepened
1720 / 3000 > 50% 1
Channel Linings
Upper third - 0 ft have been lined
0 / 2500 < 50% 0
Middle third - 1770 ft have been lined
1770 / 3800 < 50% 0
Lower third - 1570 ft have been lined
1570 / 3000 > 50% 1
Storm Drains on Secondary Tributaries
Upper third - 1345 ft have been converted to storm drains
1345 / 5180 < 50% 0
Middle third - 2330 ft have been converted to storm drains
2330 / 3940 > 50% 1
Lower third - 1510 ft have been converted to storm drains
1510 / 2160 > 50% 1
Curb and Gutter Streets
Upper third - 690 ft of curb and gutter street
690 / 2850 < 50% 0
Middle third - 3020 ft of curb and gutter street
3020 / 4700 > 50% 1
Lower third - 3180 ft of curb and gutter street
3180 / 5610 > 50% 1

Total BDF = 7

2. Tampa Bay Area, Leon County, West-Central Florida:

Regression equations developed by the USGS (Sauer et al., 1983) as part of a nationwide project can be used to estimate peak runoff for urban watershed conditions. Regionalized regression equations for urban watersheds in the Tampa Bay area and for Leon County are presented by Lopez and Woodham (1983), Franklin and Losey (1984), and Hammett and DelCharco (2001) respectively. Tables T-15, T-16, and T-17 in the Design Aids show the USGS Regionalized Regression Equations for the Tampa Bay area, Leon County, and West-Central Florida respectively.

Tampa Bay Area

For urban drainage areas of less than 10 square miles in the Tampa Bay area, the general form of the regression equations are:

for 2, 5, and 10 year frequencies:

$$Q_T = C\,A^{B_1} BDF^{B_2}\ SL^{B_3} (DTENA + 0.01)^{B_4} \qquad \textbf{(14)}$$

for 25, 50, and 100 year frequencies:

$$Q_T = C\,A^{B_1} (13 - BDF)^{B_2}\ SL^{B_3} \qquad \textbf{(15)}$$

where:

Q_T	= Peak runoff rate for return period T, in cfs.
C	= Regression constant (See Table T-15).
A	= Drainage area in square miles.
BDF	= Basin development factor (dimensionless).
SL	= Channel slope (ft/mi) between points 10 and 85 percent of the distance from the design point to the watershed boundary.
DTENA	= Surface area of lakes, ponds, and detention and retention basins expressed as a percent of the drainage area.
B_1, B_2, B_3, B_4	= Regression exponents (See Table T-15)

The equations are not to be used for watershed conditions outside the range of applicability shown in Table T-15. To apply the Tampa Bay regression equations:

1. Determine input parameters, including drainage area, basin development factor (see example 2), channel slope, and the surface area of lakes, ponds, etc.

2. Calculate peak runoff rates for the desired return periods.

Leon County

For urban drainage areas of less than 16 square miles in Leon County, Franklin and Losey (1984) developed regression equations for areas inside and outside the Lake Lafayette Basin. The general form of both sets of equations is:

$$Q_T = C \ A^{B_1} IA^{B_2} \tag{16}$$

where:

Q_T = Peak runoff rate for return period T, in cfs.
C = Regression constant (See T-16).
A = Drainage area in square miles.
IA = Impervious area, in percent of drainage area.
B_1, B_2 = Regression exponents (See Table T-16).

These equations must not be used for watershed conditions outside the range of applicability shown in Table T-16 of the Design Aids. The following steps are used to apply the Leon County regression equations:

1. Determine input parameters, including drainage area and impervious area.

2. Select the appropriate equations from Table T-16, depending on whether the area is inside or outside the Lake Lafayette Basin.

3. Calculate peak runoff rates for the desired return periods using the equations in Table T-16.

3. Water Management District and Local Drainage District Procedures

Some Water Management Districts (WMDs) in Florida set allowable discharge or removal rates for specific watershed areas. WMDs may also have computer programs for surface hydrology calculations available. Consult the appropriate WMD handbook and, if needed, appropriate WMD or FDOT District drainage personnel for guidance. There are also local drainage districts that control runoff amounts to particular streams or water bodies.

West-Central Florida

For drainage areas in West-Central Florida, Hammett and DelCharco (2001) developed regression equations for areas inside and outside the Southwest Florida Water Management District. The general form of the regression equations are:

for Region 1:

$$Q_T = C\ A^{B_1} (LK + 0.6)^{B_2} \tag{17}$$

for Regions 2 through 4:

$$Q_T = C\ A^{B_1} (LK + 3.0)^{B_2}\, SL^{B_3} \tag{18}$$

where:

- Q_T = Peak runoff rate for return period T, in cfs.
- C = Regression constant (See T-17)
- A = Drainage area in square miles
- LK = Drainage area covered by lakes, in percent of drainage area.
- SL = Channel slope (ft/mi) between points 10 and 85 percent of the distance from the design point to the watershed boundary.
- B_1, B_2, B_3 = Regression exponents (See T-17)

These equations must not be used for watershed conditions outside the range of applicability shown in Table T-18 of the Design Aids. The following steps are used to apply the West-Central Florida regression equations:

1. Locate the appropriate region on Figure F-5.
2. Select the appropriate table (from Table T-17) for the region in which your site is located.
3. Determine the input parameters for your selected regression equation.
4. Calculate peak runoff rates for the desired return periods.

2.2.4 Flood Hydrographs

Because observed data are not available for deriving unit hydrograph parameters in most cases, synthetic procedures are often required. The two flood hydrograph procedures which can be performed are the modified rational method and the SCS unit hydrograph. The rainfall distributions to be used with either of these methods are the Suwannee River Water Management District distribution curves shown in the Drainage Manual and copied in Figures F-5 through F-12 of the Design Aids.

Modified Rational Method

Because of the assumptions and limitations of the rational method (see section 2.2.3), use of the modified rational method for flood hydrograph procedures is limited to small basins having a time of concentration of 15 minutes or less. (See the Drainage Manual Section 5.3.2.)

Example - Using a drainage area of 0.981 acres, t_c of 10 min, CA of 0.82, IDF Zone 5, calculate an inflow hydrograph for the 100 year 2-hour rainfall.

From the zone 5 IDF curves, the 2-hr 100-yr i = 2.7 in/hr, therefore P_{total} = 5.4 in.

(1) Time (hours)	(2) i / P_{total}	(3) i (in / hr)	(4) Q (cfs)
0.2	0.50	2.70	2.21
0.4	0.75	4.05	3.31
0.6	1.00	5.40	4.41
0.8	1.25	6.75	5.51
1.0	0.50	2.70	2.21
1.2	0.30	1.62	1.32
1.4	0.25	1.35	1.10
1.6	0.20	1.08	0.88
1.8	0.15	0.81	0.66
2.0	0.00	0.00	0

Columns 1 & 2 are from the rainfall distribution curves
Column 3 = Column 2 times P_{total}
Column 4 = Column 3 times CA (0.82 for this example)

SCS Hydrograph

Techniques developed by the U.S. Natural Resource Conservation Service (formerly the Soil Conservation Service - SCS) for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach also considers the time distribution of the rainfall, initial losses to interception and depression storage, and infiltration that decreases during the storm. Since SCS hydrographs are calculated using computers, the discussion in this handbook will address the basic concepts rather than computation methods.

Time of Concentration
The time of concentration can be calculated using any of the methods in Section 2.2.2 of this handbook.

Curve number:
The SCS developed an empirical relationship for estimating rainfall excess that accounts for infiltration losses and initial abstractions by using a site-specific runoff parameter called the curve number (CN). The watershed CN is a dimensionless coefficient that reflects watershed cover conditions, hydrologic soil group, land uses, and antecedent moisture conditions.

Three levels of antecedent moisture conditions are considered by the SCS relationship. Antecedent Moisture Condition I (AMC-I) is the lower limit of antecedent rainfall or the upper limit of the maximum soil storage (S). Antecedent Moisture Condition II (AMC-II) represents average antecedent rainfall conditions, and Antecedent Moisture Condition III (AMC-III) is the upper limit of antecedent rainfall or the lower limit of S. Only AMC-II is generally selected for design purposes. The curve number values in the tables in the Design Aids are based on AMC II.

To determine the curve number:
1. Identify soil types using the appropriate county soil survey report.
2. Assign a hydrologic group (A, B, C, or D) to each soil type. (See Table T-6 in the Design Aids.) In general,
   A = deep sand, deep loess, aggregated silts;
   B = shallow loess, sandy loam;
   C = clay loams, shallow sandy loam, soils low in organic content, soils usually high in clay; and
   D = soils that swell significantly, heavy plastic clays, some saline soils.

3. Identify drainage areas with uniform soil type and land use conditions.
4. Use tables T-7 - T-9 in the Design Aids or other references to select curve number values for each uniform drainage area identified in step 3.
5. Calculate a composite curve number using the equation:

$$CN_C = \frac{\Sigma\, CN_i\, A_i}{A_T} \qquad \textbf{(19)}$$

Where: CN_C = Composite curve number
CN_i = Curve number for sub-area I
A_i = Area for sub-area I
A_T = Total area of watershed

The curve number tables developed by the US Department of Agriculture are based on the assumption that all impervious areas have a CN of 98 and are hydraulically connected. If the rain on the roof of a house runs off onto the lawn, that roof area is not hydraulically connected. If the roof drains into a gutter, which in turn flows onto the driveway, then on to the street, that area is hydraulically connected.

If these assumptions don't fit the project area, there is an alternate method of predicting curve number from Department-sponsored research on estimating coefficients for hydrologic methods used for the design of hydraulic structures. The results were reported in "Techniques for Estimating Hydrologic Parameters for Small Basins in Florida", by Scott Kenner, et al, FDOT Project Number 99700-3542 and April 1996. The resulting equation for estimating the CN is:

$$CN = 58.38 - 8.2716 \ln(A) + 0.50274\ HCIA + 6.22971 \ln(L) + 0.68079 \ln(L_c) - 0.14986\ S \qquad \textbf{(20)}$$

where: A = drainage area (acres)
HCIA = hydraulically connected impervious area (percent of A)
L = length of main flow channel (feet)
L_c = length to centroid (feet)
S = main channel slope (feet/mile)

<u>Rainfall - Runoff Relationship</u>

The maximum soil storage and a CN value for a watershed can be related by the following expression:

$$S = \frac{1000}{CN} - 10 \qquad \textbf{(21)}$$

Where: S = Maximum soil storage, in inches.
CN = Watershed curve number, dimensionless

When the maximum soil storage is known, the rainfall excess can be calculated using the following SCS relationship:

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \textbf{(22)}$$

Where: R = Accumulated rainfall excess (or runoff), in inches
P = Accumulated rainfall, in inches
S = Maximum soil storage, in inches

Additional information on the SCS relationship can be found in USDA, SCS publications TP-149 (1973) and NEH-4 (1972).

Shape Factor

The hydrograph shape factor (B) is generally considered to be a constant characteristic of a watershed. The SCS dimensionless unit hydrographs are based on a B value of 484. However, since the value of B can be expected to range from 600 in steep terrain to 300 or less in flat swampy areas, adjustments to the unit hydrograph shape may be warranted. These adjustments are accomplished by changing the percent of area under the rising and recession limbs of the unit hydrograph to reflect the corresponding change in the hydrograph shape factor. The B value of 484 reflects a hydrograph that has ⅜ of its area under the rising limb. For mountainous terrain, a larger percentage of the area would probably be under the rising limb, represented by a larger B value.

The South Florida Water Management District has a memorandum dated June 25, 1993 concerning hydrograph shape (peak rate) factors. For slopes less than 5 feet per mile, a factor of 100 is recommended, and for slopes in south Florida greater than 5 feet per mile, a factor of 256 is recommended.

Hal Wilkening of the St. Johns River Water Management District prepared a memorandum for a "Procedure for Selection of SCS Peak Rate Factors for Use in MSSE Permit Applications", dated April 25, 1990. The memorandum provides a summary of the SCS unit hydrograph methodology and information on research on, as well as recommendations for the selection of, hydrograph shape (peak rate) factors. His recommendations are outlined in the following table.

Site Conditions	Shape Factor
Represents watersheds with very mild slopes, recommended by SCS for watersheds with average slope of 0.5% or less. Significant surface storage throughout the watershed. Limited onsite drainage ditches. Typical ecological communities include: North Florida flat woods, South Florida flat woods, freshwater marsh and ponds, swamp hardwoods, cabbage palm flatlands, cypress swamp, and similar vegetative communities.	256 - 284
Intermediate peak rate factor representing watersheds with moderate surface storage in some locations due to depression areas, mild slopes and/or lack of existing drainage features. Typical ecological communities include: oak hammock, upland hardwood hammock, mixed hardwood and pine, and similar vegetative communities.	323 - 384
Standard peak rate factor developed for watersheds with little or no storage. Represents watersheds with moderate to steep slopes and/or significant drainage works. Typical ecological communities include: long leaf pine - turkey oak hills, and similar vegetative communities.	484

The Department sponsored research on estimating coefficients for hydrologic methods used for the design of hydraulic structures. The results were reported in "Techniques for Estimating Hydrologic Parameters for Small Basins in Florida", by Scott Kenner, et al, FDOT Project Number 99700-3542, April 1996. The resulting equation for estimating the SCS shape factor is:

$$B = \exp\left[390 - 0.01396\ A - 0.00473\ HCIA + 0.00064\ L - 0.00053\ L_C + 0.00567\ S\right] \qquad \textbf{(23)}$$

where:
- A = drainage area (acres)
- HCIA = hydraulically connected impervious area (percent)
- L = length of main flow channel (feet)
- L_c = length to centroid (feet)
- S = main channel slope (feet/mile)

The designer should consult with district drainage personnel and, if necessary, WMD personnel before using a shape (peak rate) factor other than the standard factor of 484.

Appendix A

Data Collection/Published Data

TABLE OF CONTENTS

A.1 Data Collection

Table A-1 on page A-4 lists examples of data, along with typical sources and uses, for the following three data categories:

- Completed or ongoing studies
- Natural resource base
- Manmade features

Not all the information presented in this section may be required to address the needs of each project.

There are numerous potential sources of the data that are typically required for drainage projects. Identifying these sources can be difficult, and making the subsequent necessary contacts can be time-consuming. To assist with identification, typical data sources are included in Table A-1. In many cases, the local community or Water Management District in which the drainage project is being conducted is either the best source of data or the most logical starting point.

The principal use of drainage data is to quantify the hydrologic-hydraulic characteristics of the watershed in order to evaluate stormwater runoff discharge and volume. This quantifying of watershed characteristics must be done for both existing and future conditions. Examples of data uses are presented in Table A-1.

Drainage data should be collected before calculations are initiated, under the following general guidelines:

1. Identify data needs, sources, and uses, using Table A-1 as a checklist. Much of this information will have to be provided in the environmental document and supporting files.

2. Collect published data, based on sources identified in Step 1 and information presented in Section A.2.

3. Compile and document the results of Step 2, and compare data needs and uses with published data availability. Identify any additional field data needs.

4. Collect field data based on needs identified in Steps 1 and 3, using information presented in Section A.3.

5. Compile and document the results of Step 4.

Table A-1

DATA NEEDS, SOURCES, AND USES

Data Needs	Examples	Typical Sources	Examples of Uses
1. Completed or Ongoing Studies	Storm Master Plan	o County, City or Water Management District	o Establish type and configuration of future stormwater control facilities
	208 Plan	o U.S. Environmental Protection Agency o Regional Planning Agency	o Delineate watersheds and subbasins
	SCS Pl 566 Plan	o U.S. Environmental Protection Agency	o Establish floodflows, stages, and area of inundation on principle streams
	Flood Plain Information	o U.S. Army Corps of Engineers	o Establish floodflows, stages, and area of inundation on principle streams
	Special Studies	o City or County o U.S. Geological Survey o Regional Planning Agency	o Varies with Study
	Flood Insurance Study	o U.S. Federal Emergency Management Agency/Department of Housing and Urban Development o City or County	o Establish Floodflows, stages, and area of inundation of principle streams
	Topographic Map	o U.S. Geological Survey o Regional Planning Agency o Water Management District o Field Survey	o Delineate watersheds and subbasins o Identity potential detention sites o Determine land slope
2. Natural Resource Base	Soils	o U.S. Natural Resource Conservation Service o Construction Logs	o Determine the runoff coefficients, curve numbers, and other runoff factors o Evaluate erosion potential o Project construction condition
	Historic Inundation Areas and High Waters	o U.S. Geological Survey o City or County o Water Management District o Regional Planning Agency o News Media – Newspapers, Radio, T.V. o Museums, Historical Societies o Residents o Field Survey	o Document location and severity of historic inundation and other problems

Data Needs	Examples	Typical Sources	Examples of Use
2. Natural Resource Base (Continued)	Precipitation Intensity-Soils Duration-Frequency Data	o National Weather Service o Water Management District	o Develop design storms
	Historic Stage and Discharge	o National Weather Service o Water Management District	o Assess severity of historic floods
3. Manmade Features	Stream Stage and Discharge	o U.S. Geological Survey o Water Management District	o Develop discharge-probability relationships o Asses severity of historic floods
	Existing Land Use Areas and High Waters	o Regional Planning Agency o Field Survey	o Determine runoff coefficients, curve numbers, and other factors
	Land Use Plan	o Regional Planning Agency o City or County	o Determine runoff coefficients, curve numbers, and other factors
	Zoning Map and Ordinance	o City or County	o Project future land use
	Subdivision Plats	o City or County	o Project future land use o Established type and configuration of future stormwater control facilities
	Agricultural and Other Land Management Measures	o U.S. Natural Resources Conversation Services o Regional Planning Agency o Field Survey	o Determine runoff coefficients, curve numbers, and other runoff factors
	Transportation, Sewage and Other Public Facility-Systems and Plans	o Regional Planning Agency o City or County o Department of Transportation	o Establish future watershed and sub-basin divides o Project future land use
	Stormwater Systems Maps, Plans, Profiles; As-Builts	o Regional Planning Agency o City or County	o Delineate existing/future watershed and sub-basin divides o Develop hydraulic characteristics
	Bridge, Culverts, Channels, and Other Hydraulic Structure As-Bulits or Plans Subdivisions Plats	o Water Management District o Department of Transportation o Field Survey	o Delineate existing/future watershed and sub-basin divides o Develop hydraulic characteristics
	Land Ownership—Public vs. Private	o City or County	o Identify potential sites for detention and other facilities

A.2 Published Data

Published data includes soils, land use, precipitation, topography and contour, streamflow and flood history, and groundwater. A good basic reference for water resources data in Florida is the Water Resources Atlas of Florida (Florida State University, 1984). Of particular relevance to drainage projects are data on weather and climate, surface water, groundwater, water quality, drainage, flood control, navigation, and ecosystems.

A.2.1 Soils

Published soils data should be collected by the following procedure:

1. Identify soils data needed to evaluate runoff, soil erosion, slope and foundation stabilities, and hydraulic conductivity.

2. Obtain soils data from the U.S. Natural Resource Conservation Service (formerly SCS) detailed soils reports for the county area being considered. Old plans, construction logs, and soil boring results can provide additional site-specific data. Specific project information is usually available during the final design stage.

When a project involves a channel in which storm tide surge conditions may be expected to result in resizing the channel by erosion, the nature of the geology in the area of the channel is important to an analysis of the nature of the enlargement. More detailed and extensive borings may be desirable than would be the case where channel stability is reasonably assured. Preliminary assessment of the potential for enlargement may be needed to specify the extent of the geotechnical study required.

A.2.2 Land Use

Published land use data should be collected by the following procedure:

1. Determine historical land use from older land use maps or aerials.

2. Determine current land use from sources such as land use maps, aerial photographs, and field reconnaissance. Contact appropriate county and municipal governments. Regional Planning Councils and Water Management Districts may also have existing land use data. A comparison of historical (from Step 1) and current land use can be used to identify areas undergoing rapid growth and an approximate rate of change. Establishing land use at the time of design can be crucial to project success.

3. Determine future land use based on projections of existing land use, land use plans and site-specific layouts of proposed development, zoning maps, and discussions with public officials. County and municipal governments as well as Regional Planning Councils and Water Management Districts may also be a good source of future land use data.

4. Ascertain the existence of master drainage plans, stormwater management plans, and similar plans that may designate or restrict land use.

A.2.3 Precipitation

Published precipitation data should be collected by the following procedure:

1. Select an appropriate procedure for hydrologic calculations using information presented in this handbook.

2. Determine the type of precipitation data that are needed. Generally, either intensity-duration-frequency (IDF) curves or hyetographs for historic or design storm conditions are used.

3. Collect published precipitation data. The primary source is the National Weather Service. Additional data may be available from Water Management Districts. Sources of published precipitation data are briefly discussed below.

A series of publications by the National Weather Service (formerly the U.S. Weather Bureau) presents precipitation depth-duration-frequency data developed from observed precipitation data across the United States. HYDRO-35 by Frederick et al. (1977) is particularly useful for small drainage areas, since rainfall depths for durations of 5, 10, 15, 30, and 60 minutes are presented for return periods of 2, 5, 10, 25, 50, and 100 years, The publication by Hershfield (1961), commonly known as TP-40, is a standard reference for obtaining hydrologic design rainfall depths for durations of 30 minutes and one, 2, 3, 6, 12, or 24 hours, and for return periods of one, 2, 5, 10, 25, 50, and 100 years. The publication by Miller (1964) extends the depth-duration-frequency data presented by Hershfield (1961) to include rainfall depths for durations of 2, 4, 7, and 10 days at return periods of 2, 5, 10, 25, 50, and 100 years.

The Department has developed rainfall curves based on these references. They are presented in the FDOT Drainage Manual and on the internet.

A.2.4 Topography and Contour Information

Topographic data should be collected by the following procedure:

1. Obtain published topographic data. The principal source of published topographic maps is the U.S. Geological Survey (USGS). In Florida, USGS maps have 5-foot or 10-foot contour intervals, which normally are not detailed enough for design. Additional sources include Water Management Districts and municipal or county government agencies.

2. Contours may be developed by the Department from aerial photographs for large-scale projects or by survey for small areas, if published data are either unavailable or inadequate for project needs.

A.2.5 Streamflow and Flood History

Streamflow and flood history data should be collected by the following procedure:

1. Obtain published data. The principal source of published streamflow data is the USGS. Additional sources include Water Management Districts and municipal or county government agencies.

2. Because published streamflow data may not be available for a specific project site, an evaluation of flood history may require researching news media sources, making field survey observations, and interviewing local residents and other knowledgeable persons.

A.2.6 Groundwater

Data on groundwater levels and movements could be obtained from information on existing detention ponds and other ponds in the area; existing non-pumping wells or wells that could be temporarily shut off to determine the static groundwater level; observations made by inspectors and others during construction of sanitary sewers, storm drains, and major buildings; and regional or area wide reports prepared by the USGS or similar state agencies. If existing data sources are not sufficient to define the position of the groundwater table, it may be necessary to construct special observation wells, particularly at potential sites of detention facilities. These wells could be installed in the boreholes used to take soil samples during a site-specific subsurface exploration.

A.3 Field Investigations and Surveys

A.3.1 Drainage Areas

If there is sufficient topographic information for a project site from readily available sources, a field determination of drainage area may not be necessary, but spot-checking selected control elevations is always advised. For those project sites for which detailed information is not available, field survey work should be performed. In all cases, a site visit is highly recommended to confirm drainage area conditions.

Depending on District preference, drainage areas may be outlined by field survey or drainage personnel on county maps, aerial photographs, USGS contour maps, or specially prepared maps. Drainage area boundaries should connect with the job centerline, typically at high points in grade or at other locations where there is a definite division in the direction of storm runoff flow. After the overall areas are plotted, the Drainage Engineer should subdivide the drainage area to show how the various sections contribute to the structures in the proposed drainage or storm drain system.

All drainage area boundaries should be followed from project centerline around the area being covered and closed again to the centerline. Ridges that do not establish an area draining to the project should not be shown unless pertinent to determination of runoff concentration points or flow path segments. Exceptions to the rule for closing all drainage area boundaries to centerline are to be indicated clearly on the map by notation. These notations should show location and elevation of break over or diversion to or from the drainage area.

Typically, a drainage area should close to each existing culvert along the project and for each probable cross drain location. As an exception, where two or more structures operate together to drain a single area, flow distribution information should be noted.

For municipal-type construction surveys, appropriate city maps or specially prepared maps should be marked to show the boundaries of total areas contributing to the project. Streets or other drainage facilities in these areas should be marked with flow arrows. In many instances, elevations may have to be determined to accurately delineate direction of flow in gutters.

All areas contributing to existing storm drains, which drain to or across the project, should be shown. In very flat terrain, such as in South Florida, it is often necessary to develop profiles for cross streets and parallel streets to make a definite determination of drainage areas. In flat terrain, agricultural ditches may require the collection of additional field data to confirm flow patterns.

Specially flown aerial photography is available for most new construction projects. Ridge lines usually can be indicated on the photographs. When photographs are used, the field survey party should verify questionable points and supplement the information with structure sizes, elevations, and high waters as required. Drainage areas can also be determined by stereo interpretation with spot field survey work as appropriate.

A.3.2 High Water Information

Reliable high water information is necessary to evaluate flood elevations and establish roadway grades. High water elevations should be shown upstream of the proposed project, upstream of significant existing structures, and at some point along or at the end of outfall ditch surveys. The location at which a high water elevation is taken should be clearly recorded in the field notes, along with the date and time if available.

At many locations, it is not possible to obtain documented information on high water. In such cases, elevation may be estimated by observation of natural growth or by other means; the survey crew should provide complete information on the methods used. The crew chief should attempt to obtain information from local residents, maintenance personnel (both state and county), and rural mail carriers, school bus drivers, police officers, and school board officials.

The soils crew usually supplies water table information within the right-of-way; however, the survey crew should note information pertaining to standing water, areas of heavy seepage, or springs within the basin area.

Appendix B

Design Aids

TABLE OF CONTENTS

Tables

Figures

Table T-1
Overland Flow Manning's n Values

	Value	Recommended Range of Values
Concrete	0.011	0.010 - 0.013
Asphalt	0.012	0.010 - 0.015
Bare sand [a]	0.010	0.010 - 0.016
Graveled surface [a]	0.012	0.012 - 0.030
Bare clay-loam (eroded) [a]	0.012	0.012 - 0.033
Fallow (no residue) [b]	0.05	0.006 - 0.16
Chisel plow (<1/4 tons/acre residue)	0.07	0.006 - 0.17
Chisel plow (1/4 - 1 tons/acre residue)	0.18	0.070 - 0.34
Chisel plow (1 - 3 tons/acre residue)	0.30	0.190 - 0.47
Chisel plow (>3 tons/acre residue)	0.40	0.340 - 0.46
Disk/Harrow (<1/4 tons/acre residue)	0.08	0.008 - 0.41
Disk/Harrow (1/4 - 1 tons/acre residue)	0.16	0.100 - 0.25
Disk/Harrow (1 - 3 tons/acre residue)	0.25	0.140 - 0.53
Disk/Harrow (>3 tons/acre residue)	0.30	-- --
No till (</4 tons/acre residue)	0.04	0.030 - 0.07
No till (1/4 - 1 tons/acre residue)	0.07	0.010 - 0.13
No till (1 - 3 tons/acre residue)	0.30	0.160 - 0.47
Plow (Fall)	0.06	0.020 - 0.10
Coulter	0.10	0.050 - 0.13
Range (natural)	0.13	0.010 - 0.32
Range (clipped)	0.08	0.020 - 0.24
Grass (bluegrass sod)	0.45	0.390 - 0.63
Short grass prairie [a]	0.15	0.100 - 0.20
Dense grass [c]	0.24	0.170 - 0.30
Bermuda grass [c]	0.41	0.300 - 0.48
Woods	0.45	-- --

All values are from Engman (1983), unless noted otherwise.

[a]Woolhiser (1975).

[b]Fallow has been idle for one year and is fairly smooth.

[c]Palmer (1946). Weeping love grass, bluegrass, buffalo grass, blue gamma grass, native grass mix (OK), alfalfa, lespedeza.

Note: These values were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations. See the open channel handbook for open channel flow procedures.

Table T-2
Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slopes, where sediment may accumulate increase above values of n by	0.002

Note: Estimates are by the Federal Highway Administration.

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

Table T-3
Recommended Manning's n Values for Artificial Channels with Various Linings

Channel Lining	Lining Description	Design Manning's n Value
Bare Earth or Vegetative Linings		
Bare earth, fairly uniform	Clean, recently completed	0.022
Bare earth, fairly uniform	Short grass and some weeds	0.028
Dragline excavated	No vegetation	0.030
Dragline excavated	Light brush	0.040
Channels not maintained	Dense weeds to flow depth	0.100
Channels not maintained	Clear bottom, brush sides	0.080
Maintained grass or sodded ditches	Good stand, well maintained 2" - 6"	0.060*
Maintained grass or sodded ditches	Fair stand, length 12" - 24"	0.200*
Rigid Linings		
Concrete paved	Broomed**	0.016
Concrete paved	"Roughened" - standard	0.020
Concrete paved	Gunite	0.020
Concrete paved	Over rubble	0.023
Asphalt concrete	Smooth	0.013
Asphalt concrete	Rough	0.016

* Decrease 30% for flows > 0.7' (maximum flow depth 1.5').

** Because this is not the standard finish, it must be specified.

Table T-4
Runoff Coefficients for a Design Storm Return Period of 10 Years or Less[a]

Slope	Land Use	Sandy Soils Min.	Sandy Soils Max.	Clay Soils Min.	Clay Soils Max.
Flat (0-2%)	Woodlands	0.10	0.15	0.15	0.20
	Pasture, grass, and farmland [b]	0.15	0.20	0.20	0.25
	Bare Earth	0.30	0.50	0.50	0.60
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavements [c]	0.75	0.95	0.90	0.95
	SFR: 1/2-acre lots and larger	0.30	0.35	0.35	0.45
	Smaller lots	0.35	0.45	0.40	0.50
	Duplexes	0.35	0.45	0.40	0.50
	MFR: Apartments, townhouses, and condominiums	0.45	0.60	0.50	0.70
	Commercial and Industrial	0.50	0.95	0.50	0.95
Rolling (2-7%)	Woodlands	0.15	0.20	0.20	0.25
	Pasture, grass, and farmland [b]	0.20	0.25	0.25	0.30
	Bare Earth	0.40	0.60	0.60	0.70
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavements [c]	0.80	0.95	0.90	0.95
	SFR: 1/2-acre lots and larger	0.35	0.50	0.40	0.55
	Smaller lots	0.40	0.55	0.45	0.60
	Duplexes	0.40	0.55	0.45	0.60
	MFR: Apartments, townhouses, and condominiums	0.50	0.70	0.60	0.80
	Commercial and Industrial	0.50	0.95	0.50	0.95
Steep (7%+)	Woodlands	0.20	0.25	0.25	0.30
	Pasture, grass, and farmland [b]	0.25	0.35	0.30	0.40
	Bare Earth	0.50	0.70	0.70	0.80
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavements [c]	0.85	0.95	0.90	0.95
	SFR: 1/2-acre lots and larger	0.40	0.55	0.50	0.65
	Smaller lots	0.45	0.60	0.55	0.70
	Duplexes	0.45	0.60	0.55	0.70
	MFR: Apartments, townhouses, and condominiums	0.60	0.75	0.65	0.85
	Commercial and Industrial	0.60	0.95	0.65	0.95

[a] Weighted coefficient based on percentage of impervious surfaces and green areas must be selected for each site.

[b] Coefficients assume good ground cover and conservation treatment.

[c] Depends on depth and degree of permeability of underlying strata.

Note: SFR = Single Family Residential
MFR = Multi-Family Residential

Table T-5
Design Storm Frequency Factors for Pervious Area Runoff Coefficients *

Return Period (years)	Design Storm Frequency Factor, X_T
2 to 10	1.0
25	1.1
50	1.2
100	1.25

Reference: Wright-McLaughlin Engineers (1969).

* DUE TO THE INCREASE IN THE DURATION TIME THAT THE PEAK OR NEAR PEAK DISCHARGE RATE IS RELEASED FROM STORMWATER MANAGEMENT SYSTEMS, THE USE OF THESE SHORT DURATION PEAK RATE DISCHARGE ADJUSTMENT FACTORS IS NOT APPROPRIATE FOR FLOOD ROUTING COMPUTATIONS.

Table T-6
Definitions of Four SCS Hydrologic Soil Groups

Hydrologic Soil Group	Definition
A	Low Runoff Potential Soils having high infiltration rates even when thoroughly wetted, consisting chiefly of deep, well-to-excessively-drained sands or gravels. These soils have a high rate of water transmission.
B	Moderately Low Runoff Potential Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep, to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Moderately High Runoff Potential Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, soils with moderate fine to fine texture, or soils with moderate water tables. These soils have a slow rate of water transmission.
D	High Runoff Potential Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Reference: USDA, SCS, NEH-4 (1972).

Table T-7
SCS Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Land Use

Land Use Description		Hydrologic Soil Group A	B	C	D
Cultivated Land[a]:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or Forest Land:					
Thin stand, poor cover, no mulch		45	66	77	83
Good cover [b]		25	55	70	77
Open Spaces, Lawns, Parks, Golf Courses, Cemeteries:					
Good condition: grass cover on 75% or more of the area		39	61	74	80
Fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Poor condition: grass cover on 50% or less of the area		68	79	86	89
Commercial and Business Areas (85% impervious)		89	92	94	95
Industrial Districts (72% impervious)		81	88	91	93
Residential [c]					
Average lot size	Average % Impervious [d]				
1/8 acre or less	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
Paved Parking Lots, Roofs, Driveways [e]:		98	98	98	98
Streets and Roads:					
Paved with curbs and storm sewers [e]		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89
Paved with open ditches		83	89	92	93
Newly graded area (no vegetation established)[f]		77	86	91	94

[a] For a more detailed description of agricultural land use curve numbers, refer to Table T-8. .
[b] Good cover is protected from grazing and litter and brush cover soil.
[c] Curve numbers are computed assuming the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur, which depends on the depth and degree of the permeability of the underlying strata.
[d] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
[e] In some warmer climates of the country, a curve number of 96 may be used.
[f] Use for temporary conditions during grading and construction.
Note: These values are for Antecedent Moisture Condition II, and $I_a = 0.2S$.

Reference: USDA, SCS, TR-55 (1984).

Table T-8
SCS Runoff Curve Numbers for Agricultural Use

Land Use	Treatment or Practice	Hydrologic Condition	Hydrologic Soil Group A	B	C	D
Fallow	Straight row	-----	77	86	91	94
Row Crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	and terraced	Poor	66	74	80	82
	and terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured	Good	55	69	78	83
	and terraced	Poor	61	72	79	82
	and terraced	Good	59	70	78	81
Close seeded legumes[a]	Straight row	Poor	66	77	85	89
or rotation meadow	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	and terraced	Poor	63	73	80	83
	and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		-----	59	74	82	86
Road (dirt)[b]		-----	72	82	87	89
(hard surface)[b]		-----	74	84	90	92

[a] Closed-drilled or broadcast.
[b] Including right-of-way.
Note: These values are for Antecedent Moisture Condition II, and $I_a = 0.2S$.

Reference: USDA, SCS, NEH-4 (1972).

Table T-9
SCS Classifications of Vegetative Covers by Their Hydrologic Properties

Vegetative Cover	Hydrologic Condition	
Crop rotation	Poor:	Contains a high proportion of row crops, small grain, and fallow.
	Good:	Contains a high proportion of alfalfa and grasses.
Native pasture or range	Poor:	Heavily grazed or having plant cover on less range than 50% of the area.
	Fair:	Moderately grazed; 50 - 75% plant cover.
	Good:	Lightly grazed; more than 75% plant cover.
	Permanent Meadow: 100% plant cover.	
Woodlands	Poor:	Heavily grazed or regularly burned so that litter, small trees, and brush are destroyed.
	Fair:	Grazed but not burned; there may be some litter.
	Good:	Protected from grazing so that litter and shrubs cover the soil.

Reference: USDA, SCS, NEH-4 (1972).

Table T-10
USGS Regression Equations for Natural Flow Conditions in Florida - Region 1

Peak Runoff Equation	Standard Error of Prediction (%)
$Q_2 = 127\ A^{0.656}\ (ST+1)^{-0.098}$	43
$Q_5 = 248\ A^{0.662}\ (ST+1)^{-0.189}$	40
$Q_{10} = 357\ A^{0.666}\ (ST+1)^{-0.239}$	42
$Q_{25} = 528\ A^{0.671}\ (ST+1)^{-0.293}$	47
$Q_{50} = 684\ A^{0.675}\ (ST+1)^{-0.328}$	52
$Q_{100} = 864\ A^{0.679}\ (ST+1)^{-0.362}$	57
$Q_{200} = 1072\ A^{0.683}\ (ST+1)^{-0.392}$	62
$Q_{500} = 1395\ A^{0.688}\ (ST+1)^{-0.430}$	70

Q_T = Peak runoff rate for return period of T-years, in cfs

A = Drainage area, in $miles^2$

ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST

Basin Characteristic	Range of Applicability
Drainage Area (A)	0.14 $miles^2$ (89.6 acres) to 4,385 $miles^2$
Storage Area (ST)	0% to 44.29%

Reference: Verdi (2006)

See Figure F-4 for zone delineation.

Table T-11
USGS Regression Equations for Natural Flow Conditions in Florida - Region 2

Peak Runoff Equation	Standard Error of Prediction (%)
$Q_2 = 101\ A^{0.617}\ (ST+1)^{-0.211}$	58
$Q_5 = 184\ A^{0.620}\ (ST+1)^{-0.212}$	53
$Q_{10} = 253\ A^{0.621}\ (ST+1)^{-0.215}$	52
$Q_{25} = 353\ A^{0.621}\ (ST+1)^{-0.221}$	53
$Q_{50} = 435\ A^{0.621}\ (ST+1)^{-0.226}$	54
$Q_{100} = 525\ A^{0.621}\ (ST+1)^{-0.231}$	56
$Q_{200} = 622\ A^{0.621}\ (ST+1)^{-0.236}$	59
$Q_{500} = 764\ A^{0.620}\ (ST+1)^{-0.244}$	63

Q_T = Peak runoff rate for return period of T-years, in cfs

A = Drainage area, in miles^2

ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST

Basin Characteristic	Range of Applicability
Drainage Area (A)	0.06 miles^2 (38.4 acres) to 2,647 miles^2
Storage Area (ST)	0% to 74.33%

Reference: Verdi (2006)

See Figure F-4 for zone delineation.

Table T-12
USGS Regression Equations for Natural Flow Conditions in Florida - Region 3

Peak Runoff Equation	Standard Error of Prediction (%)
$Q_2 = 72.7\ A^{0.741}\ (ST+1)^{-0.589}$	87
$Q_5 = 164\ A^{0.704}\ (ST+1)^{-0.587}$	62
$Q_{10} = 250\ A^{0.686}\ (ST+1)^{-0.592}$	56
$Q_{25} = 390\ A^{0.668}\ (ST+1)^{-0.601}$	53
$Q_{50} = 517\ A^{0.656}\ (ST+1)^{-0.608}$	53
$Q_{100} = 664\ A^{0.646}\ (ST+1)^{-0.616}$	54
$Q_{200} = 833\ A^{0.638}\ (ST+1)^{-0.625}$	56
$Q_{500} = 1094\ A^{0.629}\ (ST+1)^{-0.638}$	59

Q_T = Peak runoff rate for return period of T-years, in cfs

A = Drainage area, in $miles^2$

ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST

Basin Characteristic	Range of Applicability
Drainage Area (A)	0.41 $miles^2$ (262.4 acres) to 3,244 $miles^2$
Storage Area (ST)	0.18% to 48.04%

Reference: Verdi (2006)

See Figure F-4 for zone delineation.

Table T-13
USGS Regression Equations for Natural Flow Conditions in Florida - Region 4

Peak Runoff Equation	Standard Error of Prediction (%)
$Q_2 = 171\ A^{0.628}\ (ST+1)^{-0.401}$	36
$Q_5 = 321\ A^{0.618}\ (ST+1)^{-0.395}$	39
$Q_{10} = 447\ A^{0.614}\ (ST+1)^{-0.396}$	43
$Q_{25} = 636\ A^{0.610}\ (ST+1)^{-0.401}$	48
$Q_{50} = 797\ A^{0.609}\ (ST+1)^{-0.406}$	53
$Q_{100} = 975\ A^{0.608}\ (ST+1)^{-0.411}$	57
$Q_{200} = 1171\ A^{0.608}\ (ST+1)^{-0.416}$	62
$Q_{500} = 1461\ A^{0.609}\ (ST+1)^{-0.424}$	69

Q_T = Peak runoff rate for return period of T-years, in cfs
A = Drainage area, in miles^2
ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST

Basin Characteristic	Range of Applicability
Drainage Area (A)	0.20 miles^2 (120 acres) to 2,833 miles^2
Storage Area (ST)	0% to 34.12%

Reference: Verdi (2006)

See Figure F-4 for zone delineation.

Table T-14
USGS Nationwide Regression Equations for Urban Conditions

Peak Runoff Equation	R^2	Standard Error (%)
$UQ_2 = 2.35A^{0.41}\ SL^{0.17}\ (i_2 + 3)^{2.04}\ (ST + 8)^{-0.65}\ (13 - BDF)^{-0.32}\ IA^{0.15}\ RQ_2^{0.47}$	0.93	38
$UQ_5 = 2.70A^{0.35}\ SL^{0.16}\ (i_2 + 3)^{1.86}\ (ST + 8)^{-0.59}\ (13 - BDF)^{-0.31}\ IA^{0.11}\ RQ_5^{0.54}$	0.93	37
$UQ_{10} = 2.99A^{0.32}\ SL^{0.15}\ (i_2 + 3)^{1.75}\ (ST + 8)^{-0.57}\ (13 - BDF)^{-0.30}\ IA^{0.09}\ RQ_{10}^{0.58}$	0.93	38
$UQ_{25} = 2.78A^{0.31}\ SL^{0.15}\ (i_2 + 3)^{1.76}\ (ST + 8)^{-0.55}\ (13 - BDF)^{-0.29}\ IA^{0.07}\ RQ_{25}^{0.60}$	0.93	40
$UQ_{50} = 2.67A^{0.29}\ SL^{0.15}\ (i_2 + 3)^{1.74}\ (ST + 8)^{-0.53}\ (13 - BDF)^{-0.28}\ IA^{0.06}\ RQ_{50}^{0.62}$	0.92	42
$UQ_{100} = 2.50A^{0.29}\ SL^{0.15}\ (i_2 + 3)^{1.76}\ (ST + 8)^{-0.52}\ (13 - BDF)^{-0.28}\ IA^{0.06}\ RQ_{100}^{0.63}$	0.92	44
$UQ_{500} = 2.27A^{0.29}\ SL^{0.16}\ (i_2 + 3)^{1.86}\ (ST + 8)^{-0.54}\ (13 - BDF)^{-0.27}\ IA^{0.05}\ RQ_{500}^{0.63}$	0.90	49

UQ_T = Peak discharge, in cfs, for the urban watershed for recurrence interval T.

SL = Main channel slope, in ft/mile, measured between points which are 10 and 85 percent of the main channel length upstream from the study site. For sites where SL is greater than 70 ft/mile, 70 ft/mile is used in the equations.

A = Contributing drainage area, in miles^2.

i_2 = Rainfall intensity, in inches, for the 2-hour 2-year occurrence.

ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetland. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST.

BDF = Basin development factor, an index of the prevalence of the drainage aspects of (a) storm sewers, (b) channel improvements, (c) impervious channel linings, and (d) curb and gutter streets. The range of BDF is 0-12. A value of zero for BDF indicates the above drainage aspects are not prevalent, but does not necessarily mean the basin is non-urban. A value of 12 indicates full development of the drainage aspects throughout aspects throughout the basin. See section 2.2.3 & Example 2 of the handbook for details of computing BDF.

IA = Percentage of the drainage basin occupied by impervious surfaces, such as houses, buildings, streets, and parking lots.

RQ_T = Peak discharge, in cfs, for an equivalent rural drainage basin in the same hydrologic area as the urban basin, and for recurrence interval T.

Reference: Sauer et al. (1983).

Table T-15
Urban Watershed Regression Equations for the Tampa Bay, Florida Area

Peak Runoff Equation	R^2	Standard Error in %
$Q_2 = 3.72\ A^{1.07}\ BDF^{1.05}\ SL^{0.77}(DTENA + 0.01)^{-0.11}$	0.92	33
$Q_5 = 7.94\ A^{1.03}\ BDF^{0.87}\ SL^{0.81}\ (DTENA + 0.01)^{-0.10}$	0.90	32
$Q_{10} = 12.9\ A^{1.04}\ BDF^{0.75}\ SL^{0.83}\ (DTENA + 0.01)^{-0.10}$	0.88	35
$Q_{25} = 214\ A^{1.13}\ (13 - BDF)^{-0.59}\ SL^{0.73}$	0.85	37
$Q_{50} = 245\ A^{1.14}\ (13 - BDF)^{-055}\ SL^{0.74}$	0.83	39
$Q_{100} = 282\ A^{0.918}\ (13- BDF)^{-0.51}\ SL^{0.76}$	0.83	42

Q_T = Peak runoff rate for return period of T-years, in cfs
A = Drainage area, in miles^2
BDF = Basin development factor, dimensionless; see Example 2 and the discussion on Nationwide Regression Equations in section 2.2.3 of this handbook.
SL = Channel slope, in ft/mile, measured between points at 10 and 85 percent of the distance from the design point to the watershed boundary.
DTENA = Surface area of lakes, ponds, and detention and retention basins, expressed as a percentage of drainage area.

Watershed Characteristic	Range of Applicability
Drainage Area	0.34 miles^2 (220 acres) to 3.45 miles^2
Noncontributing internal drainage	0 to 0.3 percent of watershed area
Soil-infiltration index	2.05 to 3.89 inches
Total impervious area	19 to 61 percent of watershed area
Hydraulically connected impervious area	5.5 to 53 percent of watershed area
Effective impervious area	5.5 to 40 percent of watershed area
Channel slope	4.6 to 23.6 ft/mile
Lake and detention basin area	0 to 3.5 percent of watershed area
Basin development factor	3 to 12 (dimensionless)

Reference: Lopez and Woodham (1983).

Table T-16
Urban Watershed Regression Equations for Leon County, Florida

Peak Runoff Equation: Outside Lake Lafayette Basin	Peak Runoff Equation: Inside Lake Lafayette Basin	R^2	Standard Error in %
$Q_2 = 10.7\ A^{0.766}\ IA^{1.07}$	$Q_2\ (LL) = 1.71\ A^{0.766}\ IA^{1.07}$	0.99	18
$Q_5 = 24.5\ A^{0.770}\ IA^{0.943}$	$Q_5\ (LL) = 4.51\ A^{0.770}\ IA^{0.943}$	0.98	18
$Q_{10} = 39.1\ A^{0.776}\ IA^{0.867}$	$Q_{10}\ (LL) = 7.98\ A^{0.776}\ IA^{0.867}$	0.98	20
$Q_{25} = 63.2\ A^{0.787}\ IA^{0.791}$	$Q_{25}\ (LL) = 14.6\ A^{0.787}\ IA^{0.791}$	0.98	22
$Q_{50} = 88.0\ A^{0.797}\ IA^{0.736}$	$Q_{50}\ (LL) = 22.1\ A^{0.797}\ IA^{0.736}$	0.97	24
$Q_{100} = 118\ A^{0.808}\ IA^{0.687}$	$Q_{100}\ (LL) = 32.4\ A^{0.808}\ IA^{0.687}$	0.97	25
$Q_{500} = 218\ A^{0.834}\ IA^{0.589}$	$Q_{500}\ (LL) = 71.7\ A^{0.834}\ IA^{0.589}$	0.97	30

Q_T = Peak runoff rate outside Lake Lafayette Basin for return period T, in cfs.
A = Drainage area, in $miles^2$
IA = Impervious area, in percentage of drainage area.
Q_T (LL) = Peak runoff rate inside Lake Lafayette Basin for return period T, in cfs.

Watershed Characteristic	Range of Applicability
Drainage Area	0.26 $miles^2$ (166 acres) to 15.9 $miles^2$
Impervious area	5.8 to 54 %
Channel slope	11.9 to 128 ft/mile
Basin development factor	0 to 8 (dimensionless)
Main Channel Length	0.58 to 6.50 miles
Storage (area of ponds, lakes, swamps)	0 to 4.26 percent

Reference: Franklin and Losey (1984).

Table T-17
USGS Watershed Regression Equations for West-Central Florida

Regression equations	Standard error of model, SE_m (percent)	SE_m (+ percent)	SE_m (– percent)	Average standard error of prediction (ASEP) (percent)	Equivalent length of record (years)
Region 1					
$Q_2 = 132\ (DA)^{0.528}\ (LK+0.6)^{-0.542}$	57.9	71.2	-41.6	69	1.35
$Q_5 = 267\ (DA)^{0.510}\ (LK+0.6)^{-0.534}$	50.3	60.8	-37.8	60	2.29
$Q_{10} = 389\ (DA)^{0.500}\ (LK+0.6)^{-0.535}$	48.3	58.0	-36.7	58	3.27
$Q_{25} = 583\ (DA)^{0.489}\ (LK+0.6)^{-0.540}$	47.1	56.5	-36.1	57	4.64
$Q_{50} = 760\ (DA)^{0.481}\ (LK+0.6)^{-0.545}$	46.9	56.2	-36.0	58	5.64
$Q_{100} = 965\ (DA)^{0.474}\ (LK+0.6)^{-0.550}$	47.0	56.4	-36.1	58	6.58
$Q_{200} = 1{,}200\ (DA)^{0.467}\ (LK+0.6)^{-0.557}$	47.4	56.9	-36.3	59	7.43
$Q_{500} = 1{,}562\ (DA)^{0.460}\ (LK+0.6)^{-0.566}$	48.4	58.2	-36.8	61	8.41
Region 2					
$Q_2 = 2.03\ (DA)^{1.065}\ (LK+3.0)^{-0.259}\ (SL)^{-0.017}$	57.3	70.3	-41.3	68	1.98
$Q_5 = 5.82\ (DA)^{1.023}\ (LK+3.0)^{-0.339}\ (SL)^{0.149}$	54.9	67.1	-40.1	65	2.58
$Q_{10} = 9.84\ (DA)^{0.999}\ (LK+3.0)^{-0.371}\ (SL)^{0.226}$	54.7	66.7	-40.0	65	3.34
$Q_{25} = 17.0\ (DA)^{0.972}\ (LK+3.0)^{-0.398}\ (SL)^{0.298}$	54.3	66.3	-39.9	66	4.48
$Q_{50} = 24.1\ (DA)^{0.953}\ (LK+3.0)^{-0.412}\ (SL)^{0.339}$	54.0	65.8	-39.7	66	5.39
$Q_{100} = 32.7\ (DA)^{0.936}\ (LK+3.0)^{-0.423}\ (SL)^{0.372}$	53.5	65.2	-39.5	66	6.34
$Q_{200} = 42.8\ (DA)^{0.921}\ (LK+3.0)^{-0.432}\ (SL)^{0.400}$	52.9	64.4	-39.2	66	7.31
$Q_{500} = 58.7\ (DA)^{0.903}\ (LK+3.0)^{-0.440}\ (SL)^{0.428}$	52.3	63.5	-38.8	66	8.59
Region 3					
$Q_2 = 21.0\ (DA)^{0.890}\ (LK+3.0)^{-0.601}\ (SL)^{0.452}$	54.6	66.7	-40.0	60	1.99
$Q_5 = 54.0\ (DA)^{0.841}\ (LK+3.0)^{-0.593}(SL)^{0.374}$	49.1	59.1	-37.2	54	3.02
$Q_{10} = 87.2\ (DA)^{0.819}\ (LK+3.0)^{-0.594}\ (SL)^{0.338}$	49.9	60.2	-37.6	56	3.85
$Q_{25} = 140\ (DA)^{0.799}\ (LK+3.0)^{-0.593}\ (SL)^{0.308}$	52.6	63.9	-39.0	59	4.74
$Q_{50} = 186\ (DA)^{0.789}\ (LK+3.0)^{-0.591}\ (SL)^{0.294}$	55.2	67.4	-40.3	62	5.27
$Q_{100} = 236\ (DA)^{0.782}\ (LK+3.0)^{-0.588}(SL)^{0.284}$	57.9	71.3	-41.6	66	5.69
$Q_{200} = 289\ (DA)^{0.776}\ (LK+3.0)^{-0.584}\ (SL)^{0.278}$	60.9	75.4	-43.0	69	6.02
$Q_{500} = 364\ (DA)^{0.771}\ (LK+3.0)^{-0.578}\ (SL)^{0.274}$	64.9	80.9	-44.7	74	6.36
Region 4					
$Q_2 = 62.3\ (DA)^{0.661}\ (LK+3.0)^{-0.367}\ (SL)^{0.497}$	36.3	42.1	-29.6	40	3.86
$Q_5 = 127\ (DA)^{0.669}\ (LK+3.0)^{-0.435}\ (SL)^{0.493}$	35.9	41.7	-29.4	40	5.44
$Q_{10} = 182\ (DA)^{0.678}\ (LK+3.0)^{-0.474}\ (SL)^{0.495}$	37.0	43.1	-30.1	42	6.91
$Q_{25} = 262\ (DA)^{0.691}\ (LK+3.0)^{-0.514}\ (SL)^{0.502}$	38.9	45.6	-31.3	45	8.65
$Q_{50} = 326\ (DA)^{0.701}\ (LK+3.0)^{-0.538}\ (SL)^{0.508}$	40.7	47.9	-32.4	47	9.74
$Q_{100} = 394\ (DA)^{0.712}\ (LK+3.0)^{-0.559}\ (SL)^{0.513}$	42.6	50.5	-33.5	50	10.62
$Q_{200} = 465\ (DA)^{0.722}\ (LK+3.0)^{-0.576}\ (SL)^{0.519}$	44.7	53.2	-34.7	52	11.33
$Q_{500} = 562\ (DA)^{0.736}\ (LK+3.0)^{-0.595}\ (SL)^{0.527}$	47.6	57.1	-36.4	56	12.04

Reference: Hammett and DelCharco (2001).

Table T-18
USGS Watershed Regression Equations' Range of Applicability for West-Central Florida

Basin characteristic	Region 1	Region 2	Region 3	Region 4
Drainage area (square miles)	18.5 – 9,640	28.6 – 2,100	4.43 – 390	0.94 – 330
Slope (feet per mile)	0.51 – 23.5	0.09 – 3.6	0.41 – 9.8	1.02 – 7.52
Lake area (percent)	0.03 – 8.67	0 – 26.35	0 – 27.5	0 – 19.3

Reference: Hammett and DelCharco (2001).

Table T-19
Department Intensity-Duration-Frequency (IDF) Regression Equation Constants and Coefficients
(Page 1 of 3)

Rainfall Zone	Storm Frequency in Years	Polynomial Coefficients for a Third Degree Polynomial			
		A	B	C	D
1	2	11.0983	-2.47240	0.00711	0.01886
1	3	11.97845	-2.67930	0.02444	0.01812
1	5	11.82413	-2.28931	-0.07735	0.02535
1	10	12.01819	-1.91394	-0.20146	0.03519
1	25	13.48736	-1.84775	-0.32753	0.04818
1	50	13.12334	-1.04283	-0.52846	0.06176
2	2	10.57745	-2.10106	-0.08181	0.02557
2	3	10.89437	-1.83103	-0.19244	0.03537
2	5	10.85901	-1.50267	-0.27902	0.04121
2	10	12.30743	-1.94991	-0.22855	0.03903
2	25	12.81040	-1.40033	-0.43207	0.05602
2	50	14.17099	-1.56750	-0.47317	0.06168
3	2	11.87566	-2.78202	0.02345	0.02058
3	3	11.40436	-2.01001	-0.18000	0.03550
3	5	11.42451	-1.65788	-0.29070	0.04438
3	10	11.51866	-1.25713	-0.41757	0.05430
3	25	11.30909	-0.30052	-0.70475	0.07704
3	50	12.16856	-0.12834	-0.82217	0.08822
4	2	12.75884	-3.55763	0.21171	0.00678
4	3	12.36825	-2.82718	0.00820	0.02248
4	5	11.81456	-2.18321	-0.14397	0.03283
4	10	12.54028	-2.13586	-0.20440	0.03866
4	25	12.76532	-1.45996	-0.42819	0.05666
4	50	14.56743	-2.19263	-0.30685	0.04897
5	2	12.89666	-3.55805	0.21227	0.00619
5	3	12.49905	-2.90429	0.04609	0.01794
5	5	12.28117	-2.34803	-0.11099	0.02995
5	10	13.68290	-2.93192	-0.00385	0.02241
5	25	12.69696	-1.22300	-0.49561	0.06173
5	50	13.36862	-0.83912	-0.66880	0.07724

(Page 2 of 3)

Rainfall Zone	Storm Frequency in Years	Polynomial Coefficients for a Third Degree Polynomial A	B	C	D
6	2	14.09519	-4.17207	0.31773	0.00029
6	3	14.98331	-4.44963	0.35683	-0.00224
6	5	14.54762	-3.89935	0.22564	0.00674
6	10	14.35386	-3.10140	-0.01003	0.02525
6	25	16.15961	-3.48135	-0.00160	0.02677
6	50	15.67671	-2.52635	-0.26055	0.04609
7	2	12.10821	-2.79255	0.02002	0.02053
7	3	12.43560	-2.56458	-0.06903	0.02787
7	5	12.51872	-2.17764	-0.19805	0.03849
7	10	12.49556	-1.67116	-0.34901	0.05017
7	25	12.92209	-1.11084	-0.55019	0.06666
7	50	13.29550	-0.70432	-0.70152	0.07933
8	2	11.51282	-2.10568	-0.16578	0.03515
8	3	11.13440	-1.44999	-0.34027	0.04808
8	5	11.41155	-1.34465	-0.38409	0.05149
8	10	11.54908	-0.89694	-0.53000	0.06319
8	25	10.92111	0.51710	-0.93480	0.09473
8	50	11.58787	0.73605	-1.04111	0.10384
9	2	11.08062	-1.66022	-0.28464	0.04453
9	3	11.54667	-1.49353	-0.35960	0.05071
9	5	11.76664	-1.38391	-0.39880	0.05352
9	10	12.08400	-1.00328	-0.53661	0.06491
9	25	12.38592	-0.27352	-0.77352	0.08370
9	50	14.16172	-0.73486	-0.75377	0.08518
10	2	11.33384	-1.86569	-0.22813	0.04005
10	3	11.32916	-1.38557	-0.36672	0.05012
10	5	11.19083	-0.93165	-0.48526	0.05836
10	10	10.84265	-0.18976	-0.69575	0.07495
10	25	11.83969	0.09353	-0.84451	0.08783
10	50	11.59208	1.00204	-1.10384	0.10762

(Page 3 of 3)

Rainfall Zone	Storm Frequency in Years	Polynomial Coefficients for a Third Degree Polynomial A	B	C	D
11	2	10.09256	-2.25031	0.01661	0.01544
11	3	9.30810	-1.21537	-0.25504	0.03590
11	5	9.02699	-0.47796	-0.46784	0.05263
11	10	10.23814	-1.23242	-0.27724	0.03685
11	25	11.68811	-1.61200	-0.25239	0.03706
11	50	9.94772	0.31312	-0.73271	0.07222

$I = A + BX + CX^2 + DX^3$ $X = \log_e$ (time in minutes)

These equations were derived from the rainfall curves and are not exact representations thereof. Appropriate values for X are 8 to 180 minutes.

Table T-20
Example Application of Department IDF Regression Equations

EXAMPLE

Zone 6 - 50 years

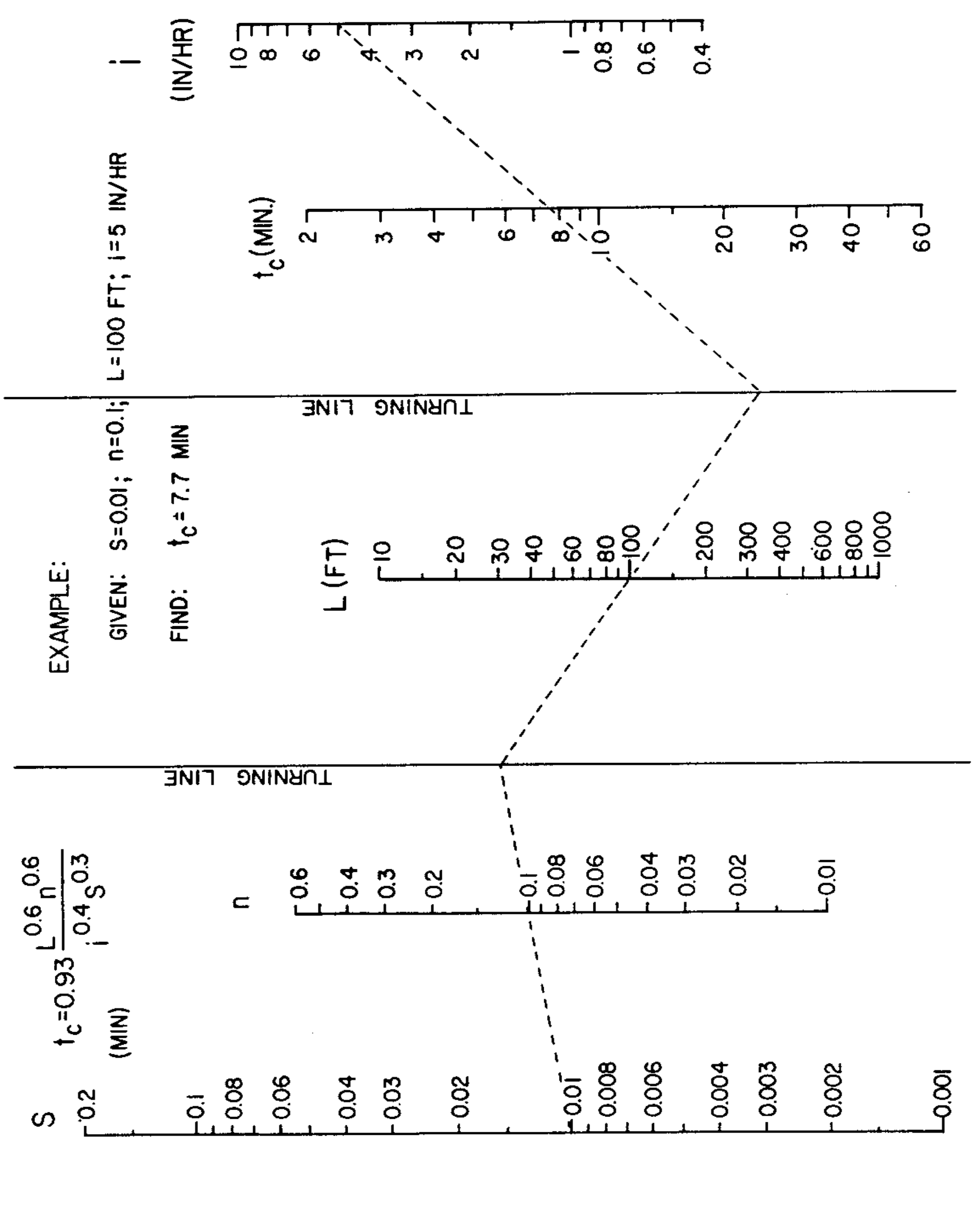
$I = A + BX + CX^2 + DX^3$ $X = \log_e$ (time in minutes)

$I = 15.67671 - 2.52635X - 0.26055X^2 + 0.04609X^3$

Time	I (curve)	I (calculated)
8 min	9.4	9.7
10 min	8.9	9.0
20 min	7.2	7.0
30 min	5.9	5.9
40 min	5.1	5.1
50 min	4.5	4.6
60 min	4.1	4.1
2 hr	2.67	2.7
3 hr	2.02	2.0
4 hr	1.65	1.59*
5 hr	1.40	1.34*
10 hr	0.87	0.92*
15 hr	0.65	0.94*
20 hr	0.54	1.09*
24 hr	0.47	1.25*

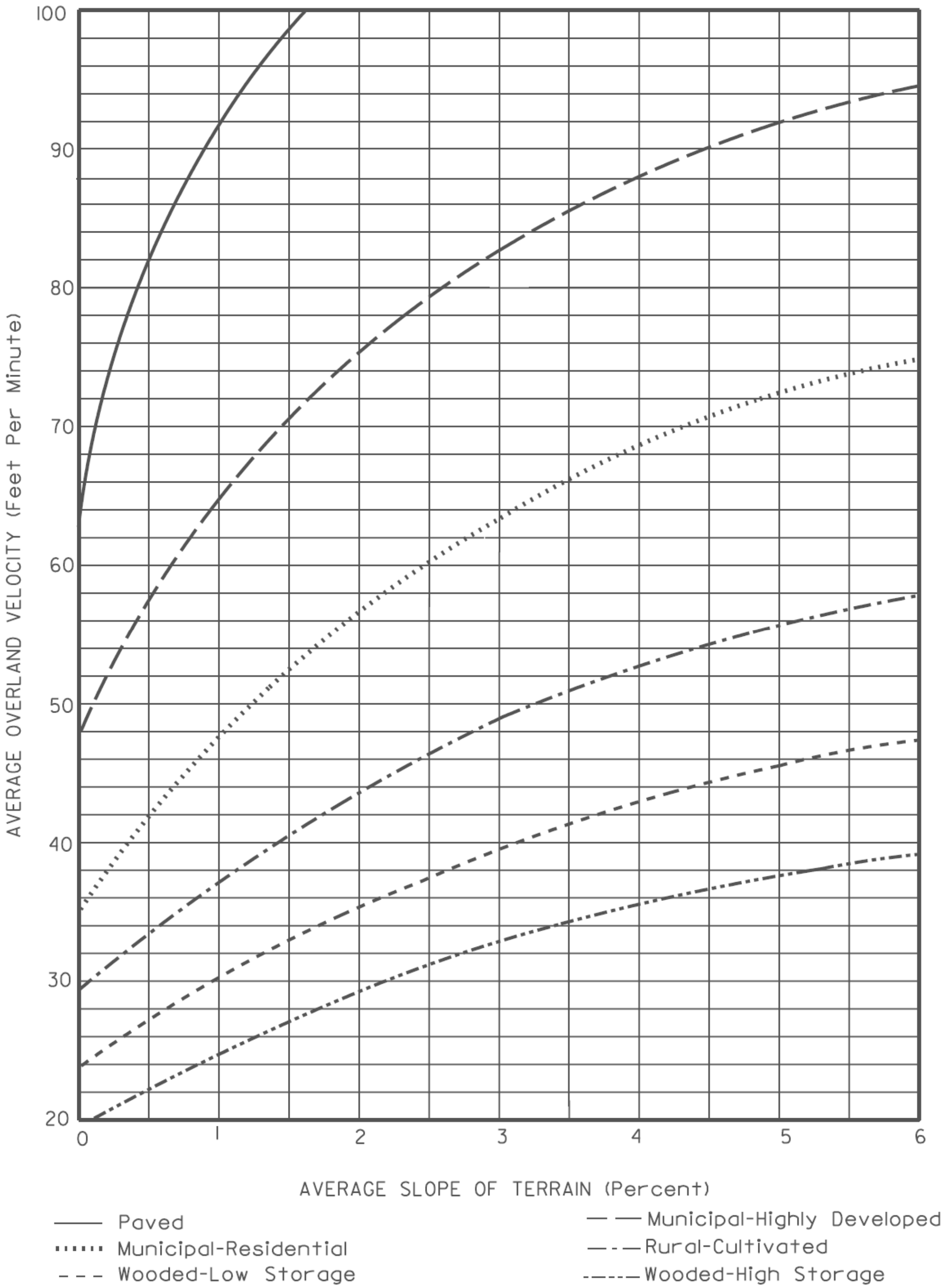
* These values are provided for comparison purposes only, since the regression equations are not valid beyond a 3-hour period.

F-1 Kinematic Wave Formula for Determining Overland Flow Travel Time

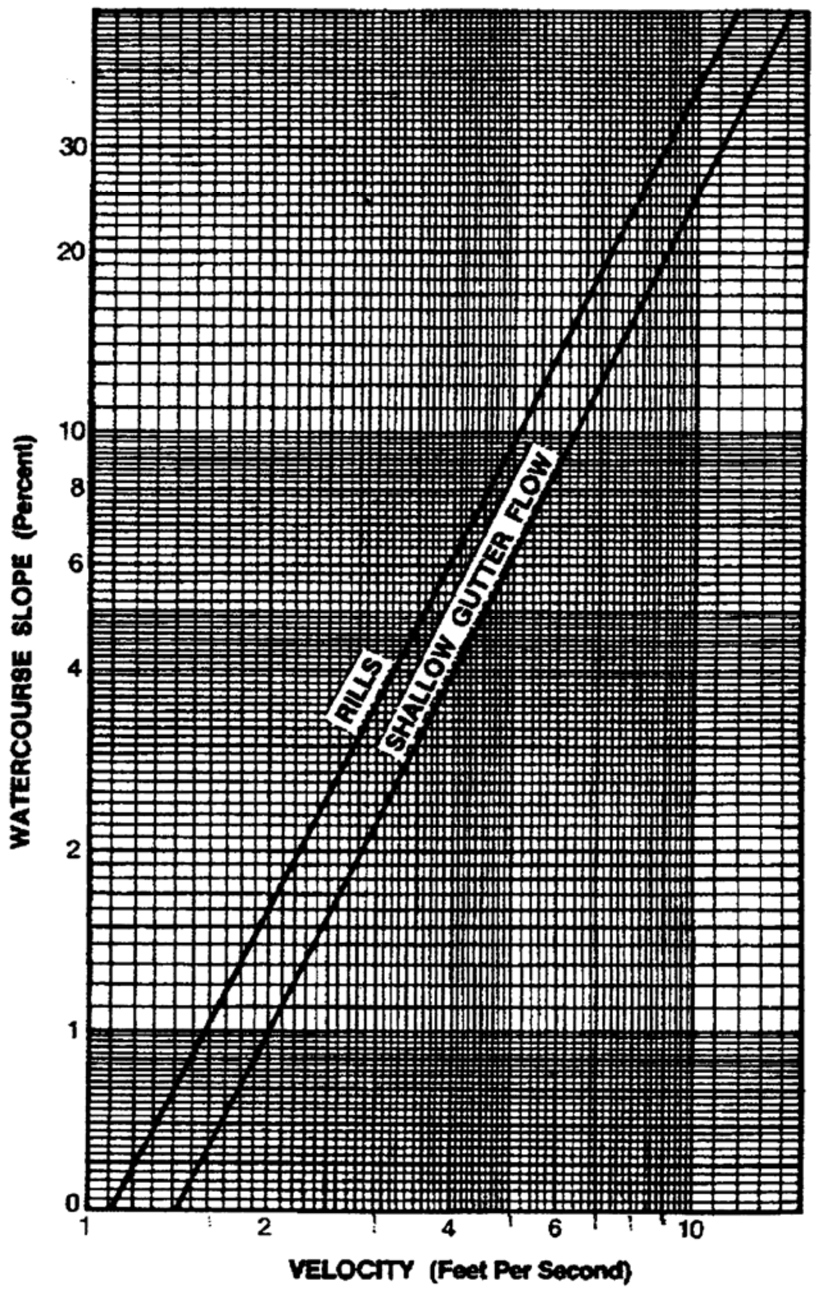


Reference: USDOT, FHWA, HEC-12 (1984).

F-2 Overland Flow Velocities for Various Land Use Types



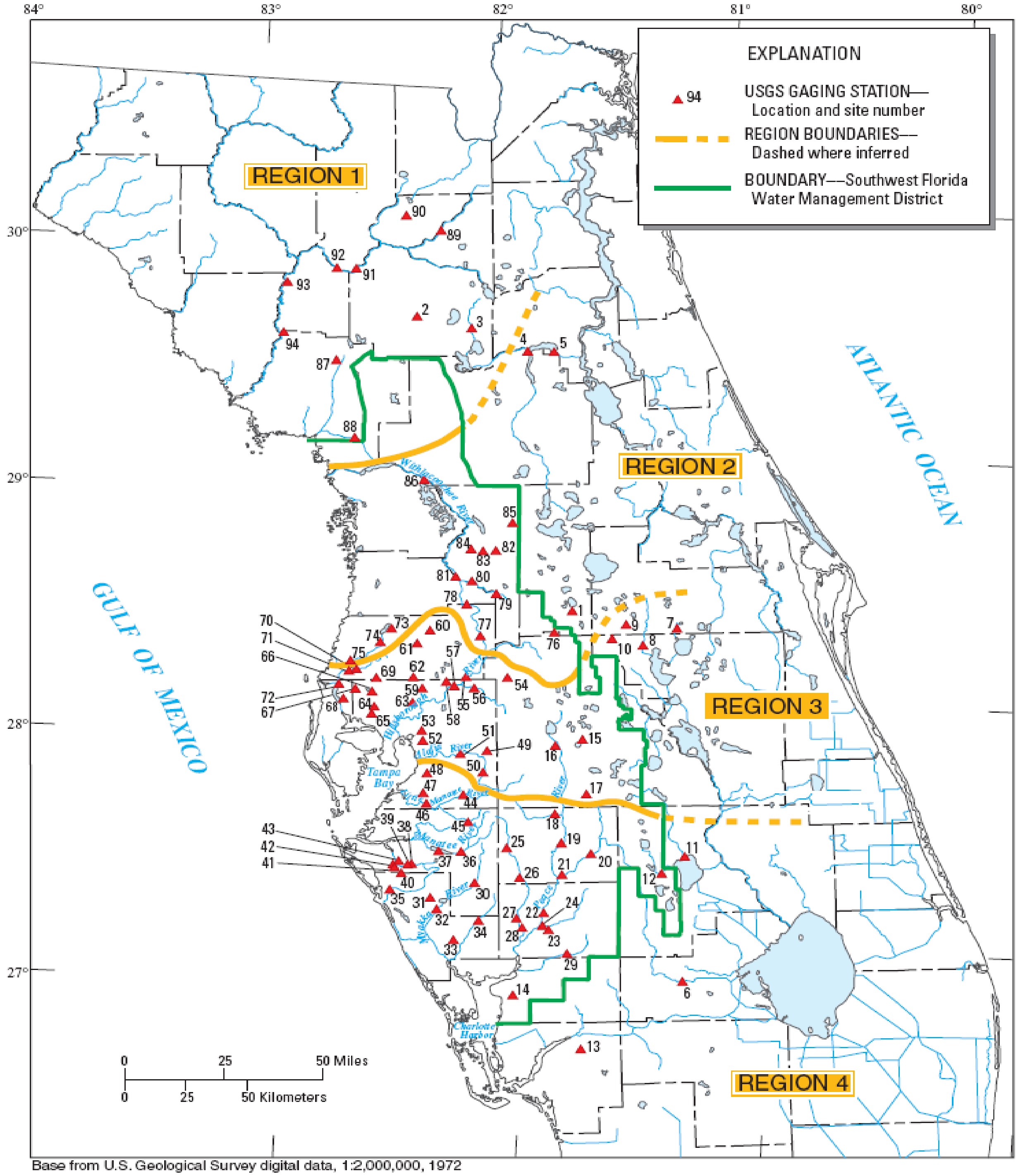
F-3 Average Velocities for Estimating Travel Time for Small Channel Flows



Reference: USDA, SCS, TR-55 Draft (1984).

F-4 Regions for USGS Regression Equations for Natural Flow Conditions in Florida

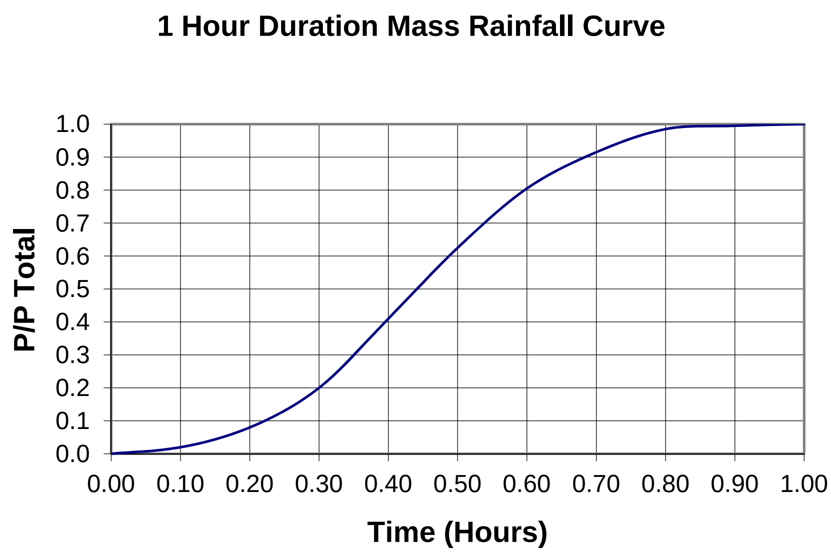
F-5 Regions for USGS Regression Equations for Natural Flow Conditions in West-Central Florida



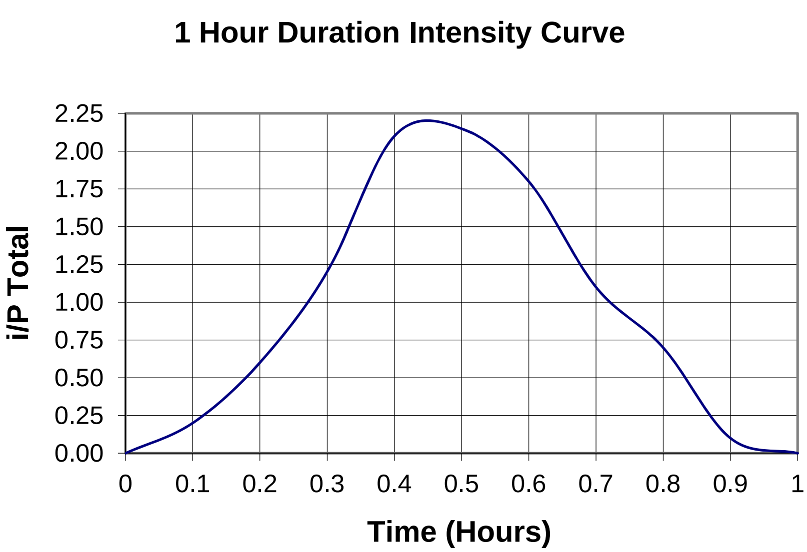
Base from U.S. Geological Survey digital data, 1:2,000,000, 1972
Albers Equal-Area Conic projection
Standard Parallels 29°30′ and 45°30′, central meridian –83°00′

Reference: Hammett and DelCharco (2001).

F-5 Rainfall Distribution Curves 1 Hour Duration

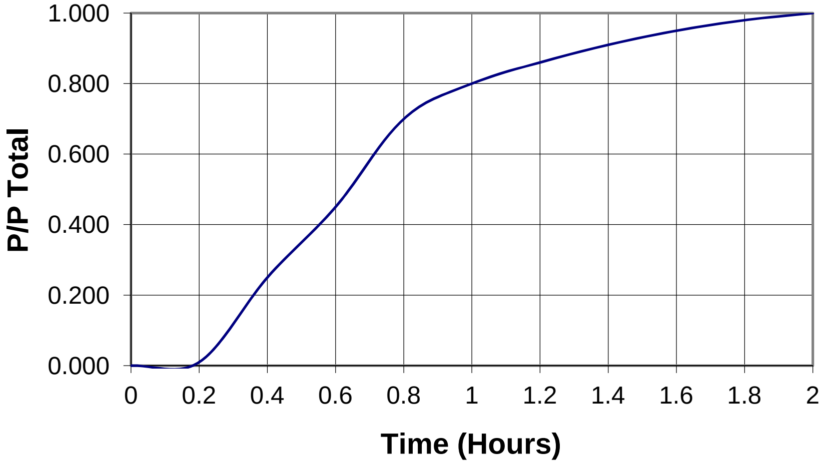


T(hrs)	P/P tot	i/P tot
0.0	0.000	0.000
0.1	0.020	0.200
0.2	0.080	0.600
0.3	0.200	1.200
0.4	0.410	2.100
0.5	0.625	2.150
0.6	0.805	1.800
0.7	0.915	1.100
0.8	0.985	0.700
0.9	0.995	0.100
1.0	1.000	0.000



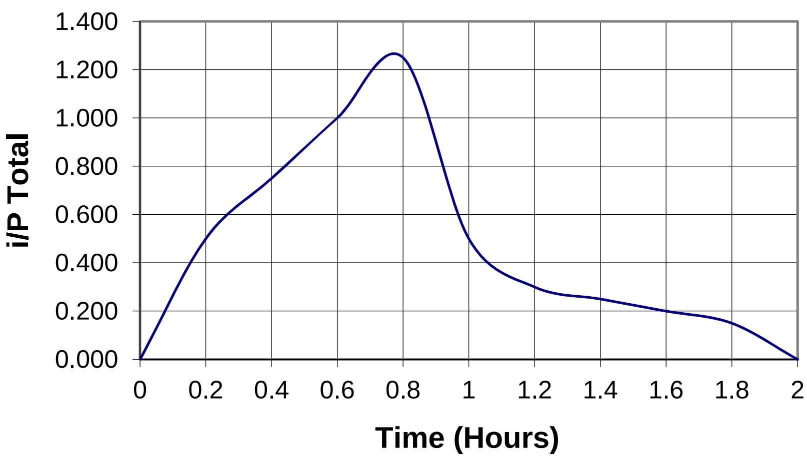
F-6 Rainfall Distribution Curves 2 Hour Duration

2 Hour Duration Mass Rainfall Curve

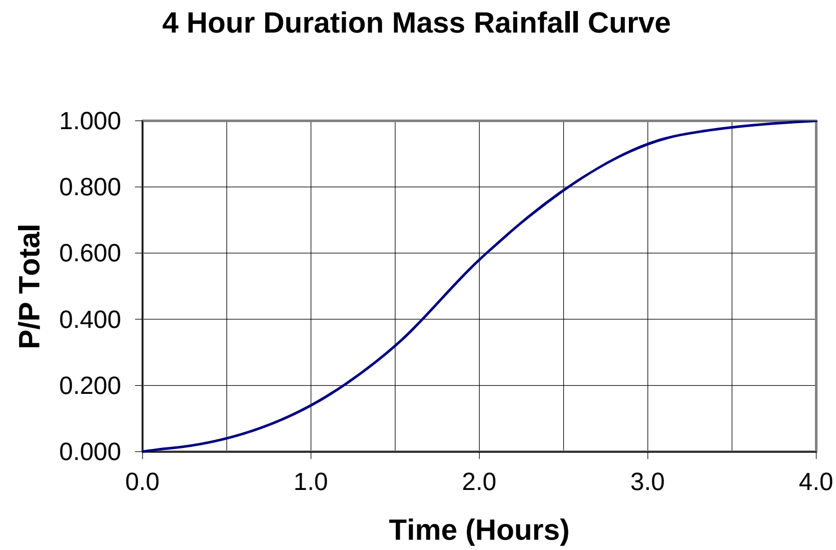


T(hrs)	P/P tot	i/P tot
0.0	0.000	0.000
0.2	0.010	0.500
0.4	0.250	0.750
0.6	0.450	1.000
0.8	0.700	1.250
1.0	0.800	0.500
1.2	0.860	0.300
1.4	0.910	0.250
1.6	0.950	0.200
1.8	0.980	0.150
2.0	1.000	0.000

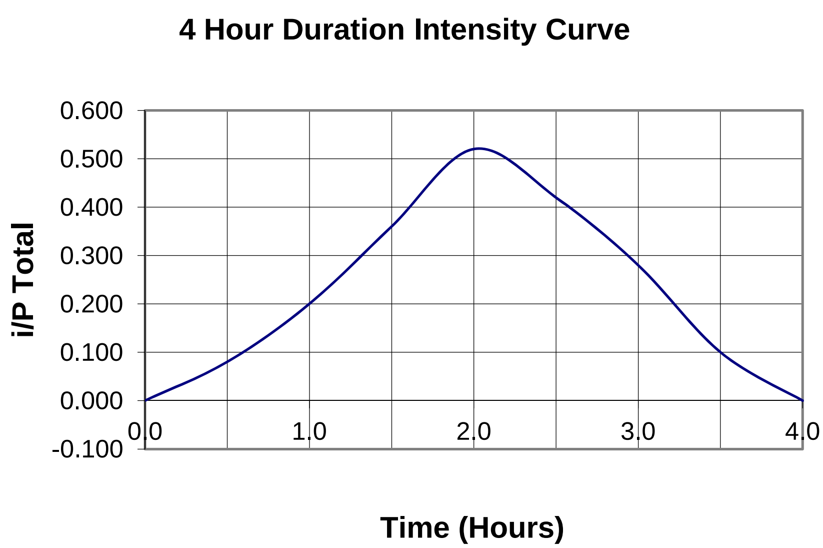
2 Hour Duration Intensity Curve



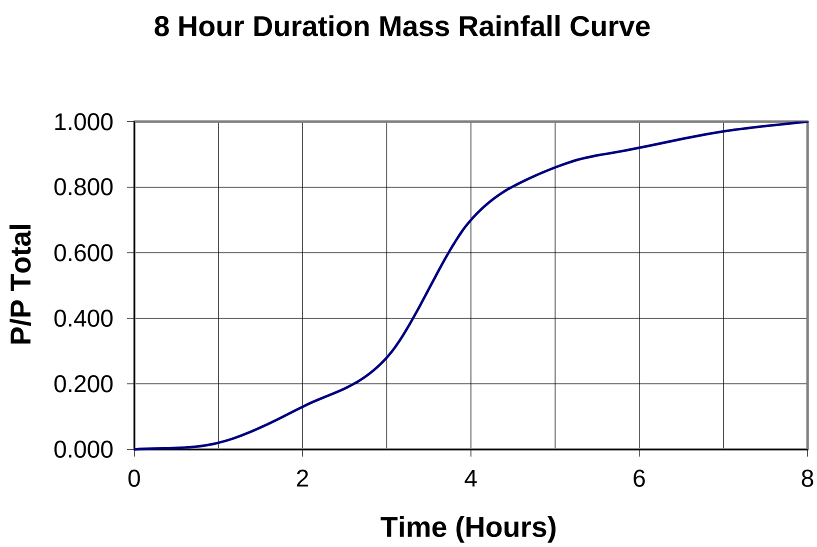
F-7 Rainfall Distribution Curves 4 Hour Duration



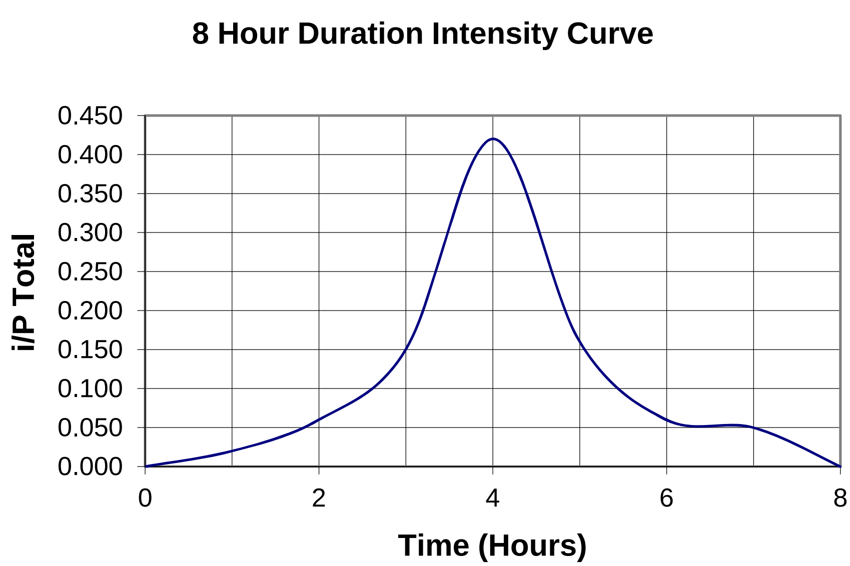
T (hrs)	P/P tot	i/P tot
0.0	0.000	0.000
0.5	0.040	0.080
1.0	0.140	0.200
1.5	0.320	0.360
2.0	0.580	0.520
2.5	0.790	0.420
3.0	0.930	0.280
3.5	0.980	0.100
4.0	1.000	0.000



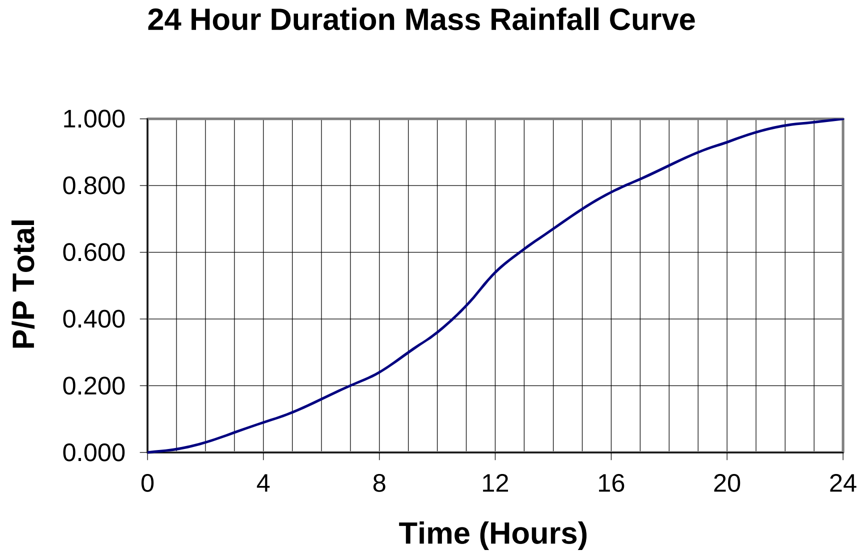
F-8 Rainfall Distribution Curves 8 Hour Duration

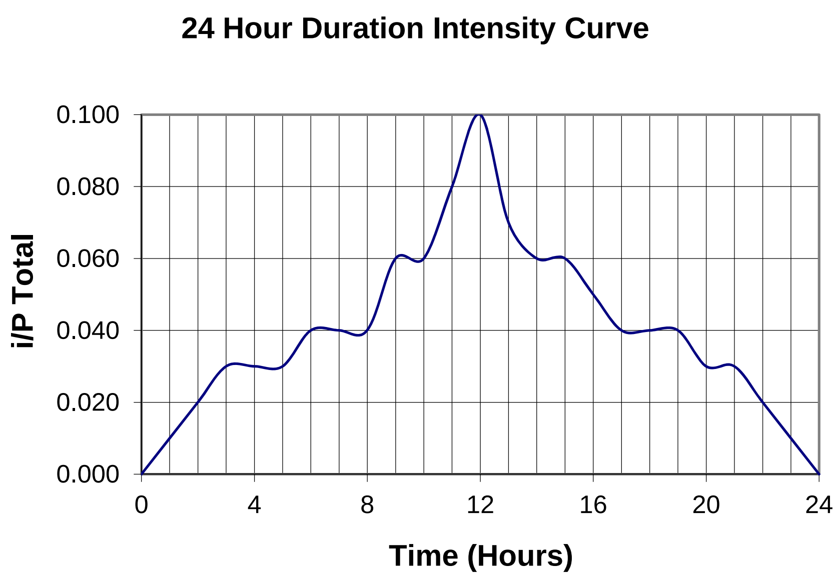


T (hrs)	P/P tot	i/P tot
0	0.000	0.000
1	0.020	0.020
2	0.130	0.060
3	0.280	0.150
4	0.700	0.420
5	0.860	0.160
6	0.920	0.060
7	0.970	0.050
8	1.000	0.000



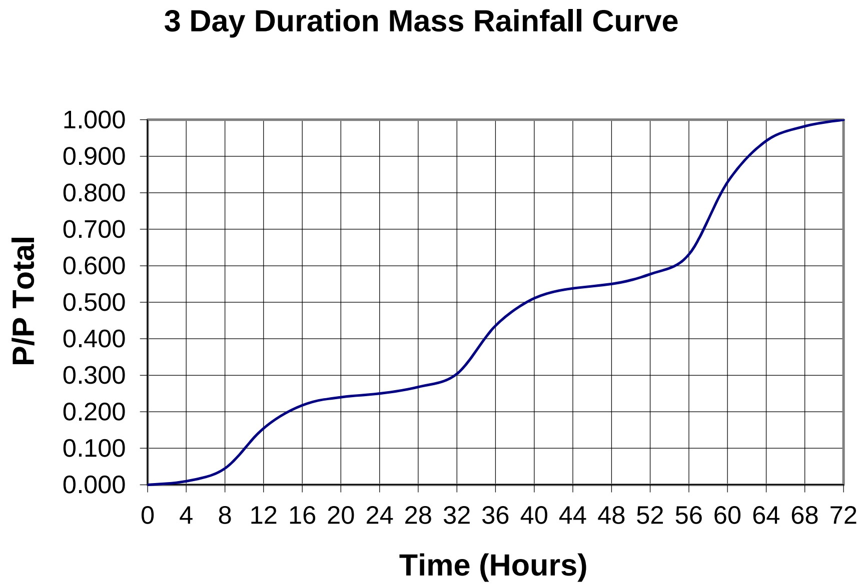
F-9 Rainfall Distribution Curves 24 Hour Duration

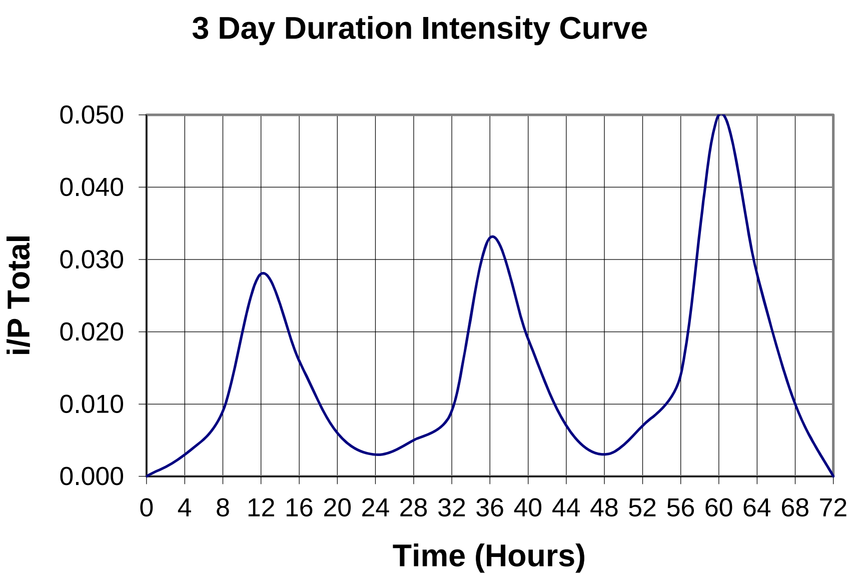




T (hrs)	P/P tot	i/P tot
0	0.000	0.000
1	0.010	0.010
2	0.030	0.020
3	0.060	0.030
4	0.090	0.030
5	0.120	0.030
6	0.160	0.040
7	0.200	0.040
8	0.240	0.040
9	0.300	0.060
10	0.360	0.060
11	0.440	0.080
12	0.540	0.100
13	0.610	0.070
14	0.670	0.060
15	0.730	0.060
16	0.780	0.050
17	0.820	0.040
18	0.860	0.040
19	0.900	0.040
20	0.930	0.030
21	0.960	0.030
22	0.980	0.020
23	0.990	0.010
24	1.000	0.000

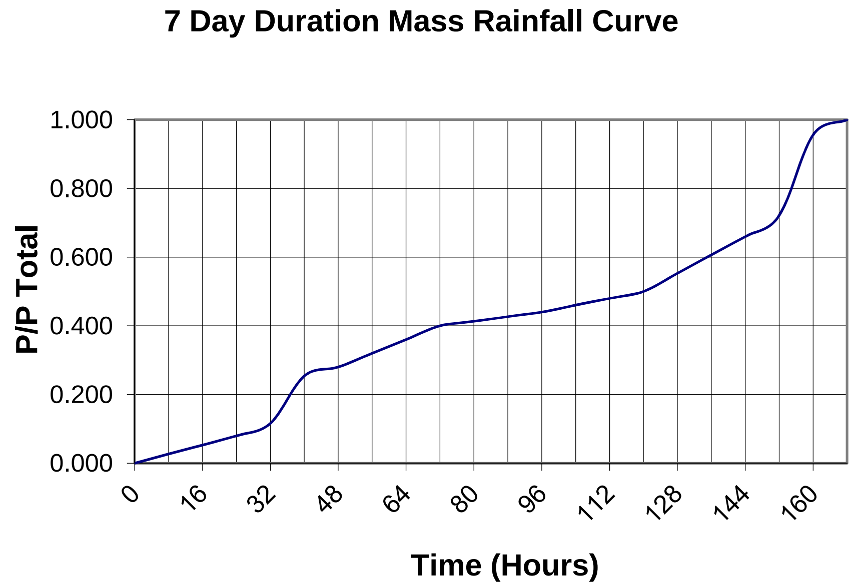
F-10 Rainfall Distribution Curves 3 Day Duration

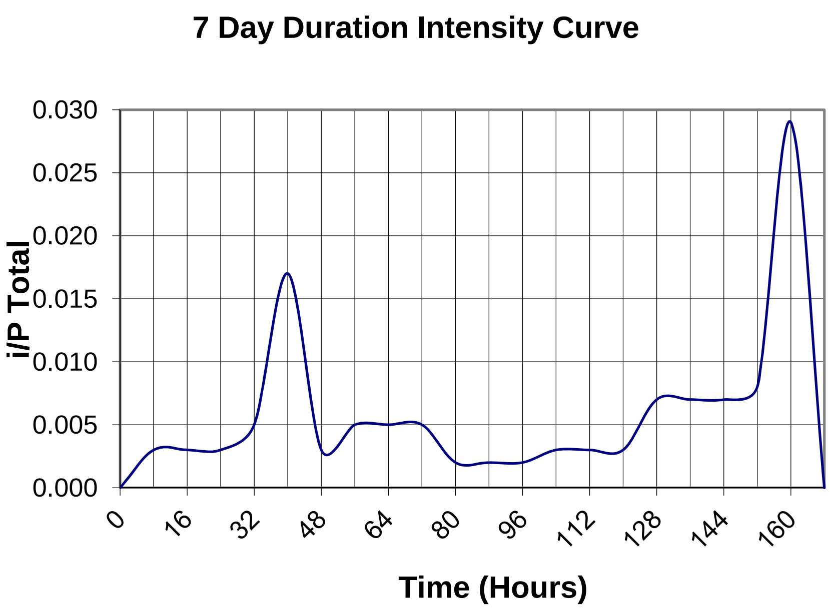




T (hrs)	P/P tot	i/P tot
0	0	0
4	0.01	0.003
8	0.045	0.009
12	0.155	0.028
16	0.218	0.016
20	0.24	0.006
24	0.25	0.003
28	0.268	0.005
32	0.304	0.009
36	0.436	0.033
40	0.511	0.019
44	0.538	0.007
48	0.55	0.003
52	0.577	0.007
56	0.631	0.014
60	0.829	0.05
64	0.942	0.028
68	0.982	0.01
72	1	0

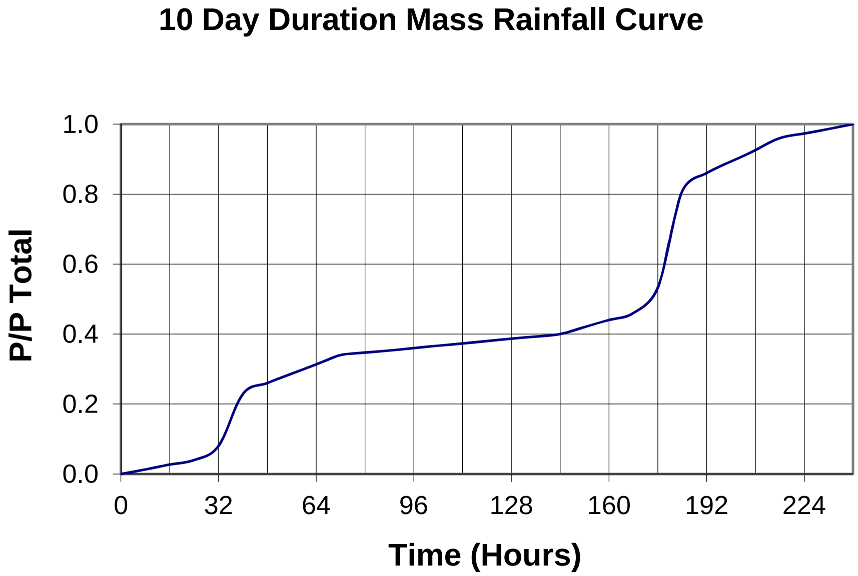
F-11 Rainfall Distribution Curves 7 Day Duration

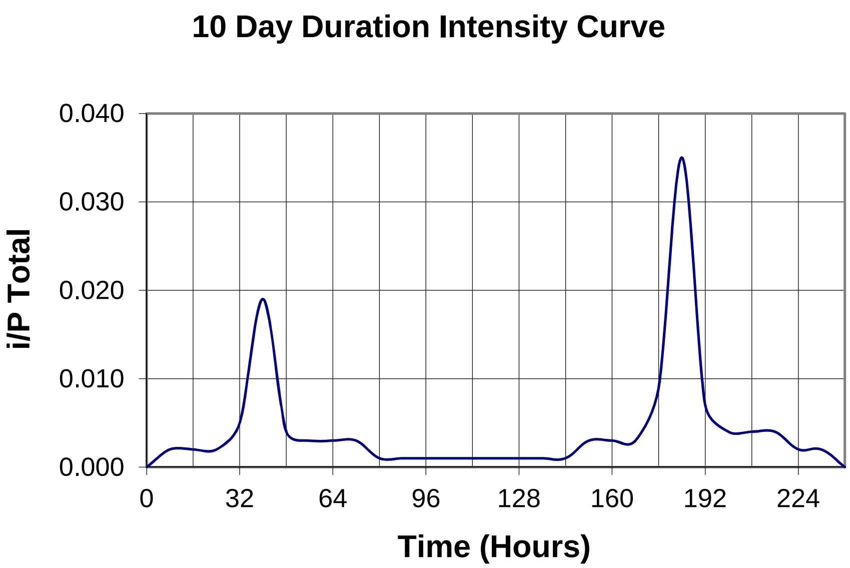




T (hrs)	P/P tot	i/P tot
0	0.000	0.000
8	0.027	0.003
16	0.053	0.003
24	0.080	0.003
32	0.116	0.005
40	0.254	0.017
48	0.280	0.003
56	0.320	0.005
64	0.360	0.005
72	0.400	0.005
80	0.413	0.002
88	0.427	0.002
96	0.440	0.002
104	0.460	0.003
112	0.480	0.003
120	0.500	0.003
128	0.553	0.007
136	0.607	0.007
144	0.660	0.007
152	0.721	0.008
160	0.956	0.029
168	1.000	0.000

F-12 Rainfall Distribution Curves 10 Day Duration





T (hrs)	P/P tot	i/P tot
0	0.000	0.000
8	0.013	0.002
16	0.027	0.002
24	0.040	0.002
32	0.080	0.005
40	0.229	0.019
48	0.260	0.004
56	0.287	0.003
64	0.313	0.003
72	0.340	0.003
80	0.347	0.001
88	0.353	0.001
96	0.360	0.001
104	0.367	0.001
112	0.373	0.001
120	0.380	0.001
128	0.387	0.001
136	0.393	0.001
144	0.400	0.001
152	0.420	0.003
160	0.440	0.003
168	0.460	0.003
176	0.532	0.009
184	0.808	0.035
192	0.860	0.007
200	0.893	0.004
208	0.926	0.004
216	0.960	0.004
224	0.973	0.002
232	0.986	0.002
240	1.000	0.000