CHAPTER 2: HYDROLOGY

2. HYDROLOGY

2.1 DRAINAGE DATA

Identifying drainage data needs should be a part of the early design phase of a project, best accomplished at the same time that you select appropriate procedures for performing hydrologic and hydraulic calculations. Several categories of data may be relevant to a particular project:

- Published data on precipitation, soils, land use, topography, streamflow, and flood history
- Field investigations and surveys to:
    - determine drainage areas
    - identify pertinent features
    - obtain high water information
    - survey lateral ditch alignments
    - survey bridge and culvert crossings

Information on types of data available and the sources of that data are presented in Appendix A of this document.

2.2 PROCEDURE SELECTION

Occasionally, streamflow measurements for determining peak runoff rates for pre-project conditions are available from agencies such as water management districts and/or the USGS. Where measurements are available, the Florida Department of Transportation (Department) usually relies upon agencies such as the United States Geological Survey (USGS) to perform the statistical analysis of streamflow data; however, guidelines for determining flood flow frequencies from observed streamflow data may be obtained from Bulletin 17C of the U.S. Geological Survey (May 2019, ver. 1.1).

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, while also documenting the reasons for using that method.

It is generally adequate to consider peak runoff rates for design conditions 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 must typically be created. Computer programs are available to help develop a runoff hydrograph.

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

Table 2.2-1 shows guidelines for selecting peak runoff rate and flood hydrograph procedures.

TABLE 2.2-1
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 NRCS 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 National Oceanic and Atmospheric Administration's (NOAA's) Hydrometeorological Design Studies Center developed historical point precipitation frequency estimates for all areas of Florida. These estimates, commonly referred to as NOAA Atlas 14 Rainfall Data, provide a reasonable basis for design. Under the FLOODS Act (Public Law No. 117-316, Dec. 2022) NOAA is authorized to update these precipitation frequency estimates no less than once every 10-years. NOAA's Atlas 14 interactive map is available on NOAA Precipitation Frequency Data Server (PFDS):

http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=fl

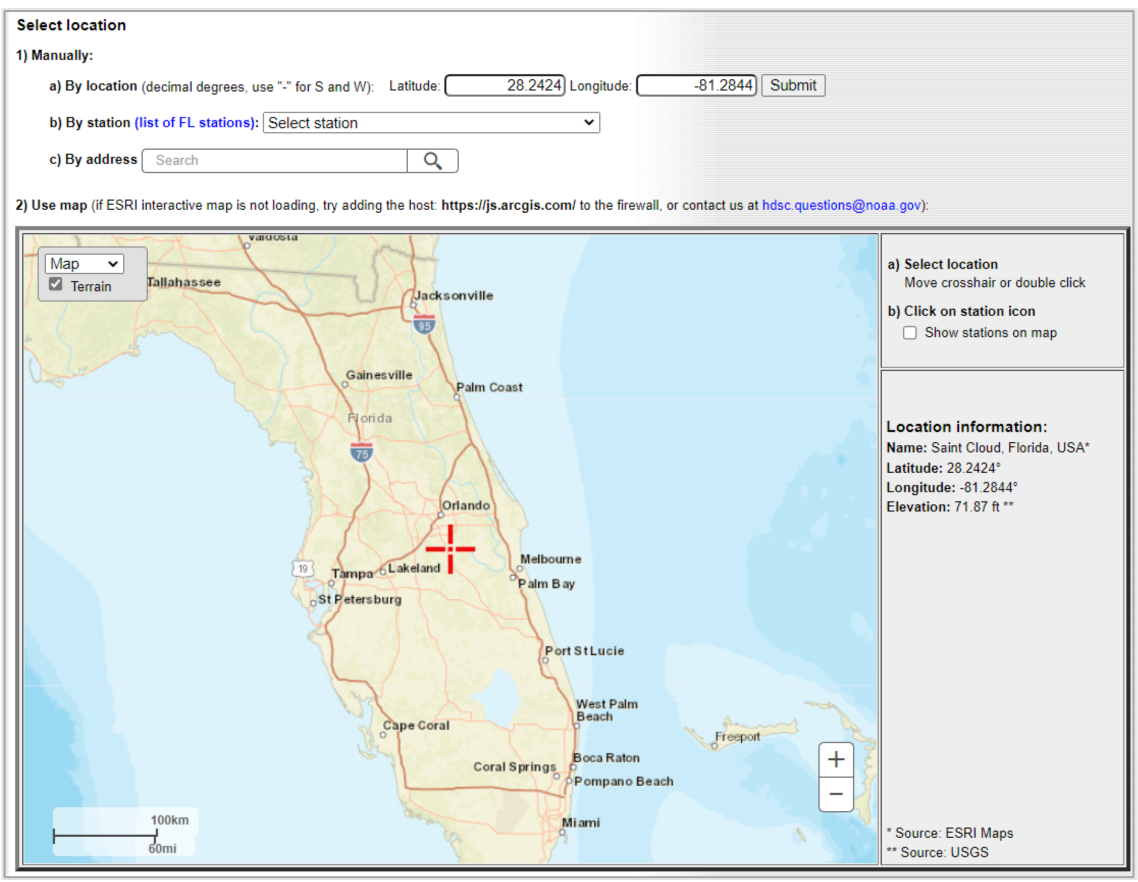
Frequency can be defined either in terms of an annual exceedance probability or a return period. The annual exceedance probability is the probability that an event having a specified volume and duration will be exceeded in a year. The inverse of the annual exceedance probability is known as the return period, which 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 50 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 annual exceedance probability (p) and return period (T) are related as follows:

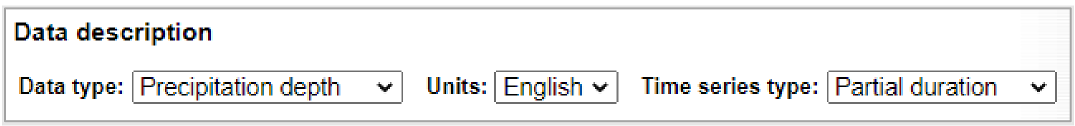
$$p = \frac{1}{T} \qquad \textbf{(2.2-1)}$$

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

Rainfall depths or intensities are required for many types of design problems. A designer must estimate this for a selected location, duration, and return period. The rainfall associated with a storm frequency and duration at a particular location can be determined from the NOAA Atlas 14 webpage mentioned above. Once the location is entered, it is displayed on an interactive map. As an example, if the project is located at latitude 28.2424 degrees N and longitude 81.2844 W, you can enter this location into the NOAA Atlas 14 webpage:



Choose the rainfall data type as either precipitation depth (inches) or precipitation intensity (inches per hour) from the menu. Two different time series options are available, Partial Duration Series (PDS) and Annual Maximum Series (AMS). The differences between these series are notable for smaller storm events (<15-years) but are negligible for larger storm events (i.e. both have similar rainfall and intensity results for larger storm events). PDS includes all rainfall amounts for specified durations above a pre-defined threshold, thus it can include data for more than one event in any particular year, whereas AMS includes only the largest precipitation amounts in a continuous calendar or water year for the specified durations. This difference in analysis results in PDS having higher rainfall and intensity results, especially for the smaller storm return frequencies. PDS is considered more reliable for designs based on frequent events (***NOAA Atlas 14 Volume 9 Version 2.0, Section 4.6.1***). Therefore, PDS is the recommended time series option for design.



For our example, the precipitation depth is shown below:

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.473** (0.379-0.593)	**0.540** (0.432-0.677)	**0.645** (0.514-0.811)	**0.728** (0.577-0.921)	**0.838** (0.640-1.09)	**0.918** (0.688-1.22)	**0.994** (0.722-1.36)	**1.07** (0.746-1.52)	**1.16** (0.781-1.71)	**1.22** (0.808-1.85)
10-min	**0.693** (0.555-0.868)	**0.791** (0.632-0.991)	**0.944** (0.752-1.19)	**1.07** (0.845-1.35)	**1.23** (0.937-1.60)	**1.34** (1.01-1.79)	**1.46** (1.06-2.00)	**1.56** (1.09-2.22)	**1.70** (1.14-2.50)	**1.79** (1.18-2.71)
15-min	**0.845** (0.676-1.06)	**0.964** (0.771-1.21)	**1.15** (0.918-1.45)	**1.30** (1.03-1.65)	**1.50** (1.14-1.95)	**1.64** (1.23-2.18)	**1.77** (1.29-2.44)	**1.91** (1.33-2.71)	**2.07** (1.40-3.05)	**2.18** (1.44-3.30)
30-min	**1.36** (1.09-1.70)	**1.55** (1.24-1.94)	**1.84** (1.47-2.32)	**2.08** (1.65-2.63)	**2.39** (1.83-3.11)	**2.61** (1.96-3.48)	**2.83** (2.06-3.89)	**3.04** (2.13-4.33)	**3.31** (2.23-4.88)	**3.50** (2.31-5.29)
60-min	**1.82** (1.46-2.28)	**2.08** (1.66-2.60)	**2.48** (1.98-3.12)	**2.80** (2.22-3.54)	**3.22** (2.46-4.20)	**3.53** (2.64-4.69)	**3.82** (2.77-5.24)	**4.10** (2.87-5.82)	**4.45** (3.00-6.55)	**4.69** (3.10-7.10)
2-hr	**2.28** (1.84-2.84)	**2.61** (2.10-3.25)	**3.12** (2.50-3.90)	**3.53** (2.81-4.43)	**4.05** (3.12-5.25)	**4.44** (3.35-5.86)	**4.80** (3.51-6.54)	**5.15** (3.62-7.27)	**5.58** (3.79-8.17)	**5.89** (3.91-8.84)
3-hr	**2.52** (2.03-3.12)	**2.88** (2.33-3.58)	**3.46** (2.79-4.31)	**3.93** (3.14-4.92)	**4.54** (3.51-5.87)	**5.00** (3.79-6.59)	**5.44** (4.00-7.40)	**5.87** (4.15-8.27)	**6.42** (4.38-9.37)	**6.81** (4.55-10.2)
6-hr	**2.93** (2.38-3.61)	**3.35** (2.73-4.13)	**4.07** (3.29-5.03)	**4.67** (3.76-5.81)	**5.53** (4.33-7.17)	**6.21** (4.76-8.20)	**6.91** (5.12-9.41)	**7.63** (5.45-10.8)	**8.61** (5.93-12.6)	**9.37** (6.30-13.9)
12-hr	**3.39** (2.77-4.14)	**3.87** (3.16-4.73)	**4.74** (3.86-5.82)	**5.54** (4.49-6.85)	**6.77** (5.39-8.85)	**7.82** (6.06-10.4)	**8.95** (6.72-12.2)	**10.2** (7.35-14.4)	**11.9** (8.32-17.4)	**13.4** (9.04-19.8)
24-hr	**3.88** (3.20-4.71)	**4.44** (3.66-5.41)	**5.52** (4.53-6.74)	**6.56** (5.35-8.05)	**8.20** (6.60-10.7)	**9.64** (7.55-12.8)	**11.2** (8.50-15.3)	**13.0** (9.46-18.3)	**15.5** (10.9-22.6)	**17.6** (12.0-25.9)
2-day	**4.44** (3.68-5.36)	**5.10** (4.22-6.16)	**6.36** (5.26-7.72)	**7.59** (6.23-9.25)	**9.53** (7.72-12.4)	**11.2** (8.85-14.8)	**13.1** (10.0-17.8)	**15.2** (11.2-21.3)	**18.2** (12.9-26.4)	**20.7** (14.2-30.3)
3-day	**4.88** (4.07-5.87)	**5.52** (4.59-6.65)	**6.77** (5.61-8.18)	**8.00** (6.60-9.72)	**9.98** (8.13-13.0)	**11.7** (9.30-15.4)	**13.7** (10.5-18.5)	**15.9** (11.7-22.2)	**19.0** (13.5-27.5)	**21.7** (14.9-31.5)
4-day	**5.27** (4.40-6.32)	**5.88** (4.91-7.06)	**7.10** (5.90-8.55)	**8.31** (6.87-10.1)	**10.3** (8.40-13.3)	**12.0** (9.56-15.8)	**14.0** (10.8-18.9)	**16.2** (12.0-22.6)	**19.4** (13.8-28.0)	**22.1** (15.3-32.0)
7-day	**6.26** (5.26-7.47)	**6.88** (5.76-8.21)	**8.09** (6.76-9.69)	**9.29** (7.72-11.2)	**11.2** (9.22-14.4)	**13.0** (10.4-16.9)	**14.9** (11.5-20.0)	**17.1** (12.7-23.6)	**20.3** (14.5-29.0)	**22.9** (15.9-33.0)
10-day	**7.14** (6.01-8.49)	**7.81** (6.57-9.29)	**9.09** (7.62-10.8)	**10.3** (8.61-12.4)	**12.3** (10.1-15.6)	**14.0** (11.2-18.1)	**15.9** (12.3-21.2)	**18.0** (13.4-24.8)	**21.1** (15.1-30.0)	**23.6** (16.4-33.9)
20-day	**9.70** (8.22-11.5)	**10.6** (9.01-12.6)	**12.3** (10.3-14.5)	**13.7** (11.5-16.3)	**15.8** (12.9-19.7)	**17.5** (14.0-22.3)	**19.4** (15.0-25.3)	**21.3** (15.9-28.9)	**24.0** (17.3-33.7)	**26.2** (18.4-37.4)
30-day	**12.0** (10.2-14.1)	**13.2** (11.2-15.5)	**15.2** (12.8-17.9)	**16.8** (14.2-20.0)	**19.2** (15.7-23.6)	**21.0** (16.8-26.4)	**22.8** (17.7-29.6)	**24.7** (18.5-33.2)	**27.2** (19.6-37.8)	**29.1** (20.5-41.4)
45-day	**14.9** (12.8-17.5)	**16.5** (14.1-19.4)	**19.1** (16.2-22.4)	**21.1** (17.8-24.9)	**23.8** (19.5-29.0)	**25.8** (20.7-32.1)	**27.7** (21.5-35.6)	**29.6** (22.1-39.4)	**31.9** (23.1-44.1)	**33.7** (23.8-47.6)
60-day	**17.6** (15.1-20.5)	**19.5** (16.7-22.8)	**22.5** (19.2-26.4)	**24.9** (21.1-29.4)	**28.0** (22.9-33.9)	**30.1** (24.2-37.3)	**32.2** (25.1-41.1)	**34.1** (25.6-45.2)	**36.5** (26.4-50.0)	**38.1** (27.0-53.7)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Estimates from the table in CSV format: Precipitation frequency estimates | Submit

Rainfall intensity is calculated from the precipitation depth and storm duration of a particular return period. For example, designing for a 25year, 30-minute storm, the intensity is (2.39 inches / 0.5 hour =) 4.78 inches per hour. Or, you can obtain intensities directly from the NOAA Atlas 14 data by selecting the precipitation intensity option:

Data description

Data type: Precipitation intensity **Units:** English **Time series type:** Partial duration

This provides the rainfall curves and tabulated data:

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**5.68** (4.55-7.12)	**6.48** (5.18-8.12)	**7.74** (6.17-9.73)	**8.74** (6.92-11.1)	**10.1** (7.68-13.1)	**11.0** (8.26-14.6)	**11.9** (8.66-16.4)	**12.8** (8.95-18.2)	**13.9** (9.37-20.5)	**14.7** (9.70-22.2)
10-min	**4.16** (3.33-5.21)	**4.75** (3.79-5.95)	**5.66** (4.51-7.13)	**6.40** (5.07-8.09)	**7.36** (5.62-9.59)	**8.06** (6.04-10.7)	**8.73** (6.34-12.0)	**9.38** (6.55-13.3)	**10.2** (6.86-15.0)	**10.7** (7.10-16.2)
15-min	**3.38** (2.70-4.23)	**3.86** (3.08-4.83)	**4.61** (3.67-5.79)	**5.20** (4.12-6.58)	**5.98** (4.57-7.80)	**6.55** (4.91-8.72)	**7.10** (5.16-9.74)	**7.62** (5.33-10.8)	**8.28** (5.58-12.2)	**8.74** (5.77-13.2)
30-min	**2.72** (2.17-3.40)	**3.09** (2.47-3.87)	**3.68** (2.93-4.63)	**4.15** (3.29-5.25)	**4.77** (3.65-6.22)	**5.23** (3.92-6.96)	**5.67** (4.12-7.78)	**6.09** (4.26-8.66)	**6.62** (4.47-9.75)	**6.99** (4.62-10.6)
60-min	**1.82** (1.46-2.28)	**2.08** (1.66-2.60)	**2.48** (1.98-3.12)	**2.80** (2.22-3.54)	**3.22** (2.46-4.20)	**3.53** (2.64-4.69)	**3.82** (2.77-5.24)	**4.10** (2.87-5.82)	**4.45** (3.00-6.55)	**4.69** (3.10-7.10)
2-hr	**1.14** (0.920-1.42)	**1.30** (1.05-1.62)	**1.56** (1.25-1.95)	**1.76** (1.41-2.22)	**2.03** (1.56-2.62)	**2.22** (1.67-2.93)	**2.40** (1.76-3.27)	**2.58** (1.81-3.64)	**2.79** (1.89-4.08)	**2.94** (1.96-4.42)
3-hr	**0.838** (0.677-1.04)	**0.959** (0.775-1.19)	**1.15** (0.928-1.44)	**1.31** (1.05-1.64)	**1.51** (1.17-1.96)	**1.67** (1.26-2.19)	**1.81** (1.33-2.47)	**1.96** (1.38-2.76)	**2.14** (1.46-3.12)	**2.27** (1.51-3.40)
6-hr	**0.489** (0.398-0.602)	**0.560** (0.455-0.690)	**0.679** (0.550-0.840)	**0.780** (0.629-0.970)	**0.923** (0.723-1.20)	**1.04** (0.794-1.37)	**1.15** (0.856-1.57)	**1.27** (0.910-1.80)	**1.44** (0.991-2.10)	**1.57** (1.05-2.33)
12-hr	**0.281** (0.230-0.344)	**0.321** (0.262-0.393)	**0.393** (0.320-0.483)	**0.460** (0.373-0.568)	**0.562** (0.447-0.735)	**0.649** (0.503-0.861)	**0.743** (0.558-1.01)	**0.845** (0.610-1.19)	**0.991** (0.690-1.45)	**1.11** (0.751-1.64)
24-hr	**0.162** (0.133-0.196)	**0.185** (0.152-0.225)	**0.230** (0.189-0.281)	**0.273** (0.223-0.335)	**0.342** (0.275-0.448)	**0.401** (0.314-0.533)	**0.467** (0.354-0.639)	**0.541** (0.394-0.763)	**0.647** (0.455-0.943)	**0.735** (0.501-1.08)
2-day	**0.093** (0.077-0.112)	**0.106** (0.088-0.128)	**0.133** (0.109-0.161)	**0.158** (0.130-0.193)	**0.198** (0.161-0.259)	**0.234** (0.184-0.308)	**0.273** (0.208-0.371)	**0.316** (0.232-0.444)	**0.380** (0.269-0.550)	**0.432** (0.296-0.630)
3-day	**0.068** (0.056-0.082)	**0.077** (0.064-0.092)	**0.094** (0.078-0.114)	**0.111** (0.092-0.135)	**0.139** (0.113-0.180)	**0.163** (0.129-0.214)	**0.190** (0.146-0.257)	**0.220** (0.162-0.308)	**0.264** (0.188-0.382)	**0.301** (0.207-0.437)
4-day	**0.055** (0.046-0.066)	**0.061** (0.051-0.074)	**0.074** (0.061-0.089)	**0.087** (0.072-0.105)	**0.107** (0.088-0.139)	**0.125** (0.100-0.164)	**0.146** (0.112-0.197)	**0.169** (0.125-0.235)	**0.202** (0.144-0.291)	**0.230** (0.159-0.334)
7-day	**0.037** (0.031-0.044)	**0.041** (0.034-0.049)	**0.048** (0.040-0.058)	**0.055** (0.046-0.067)	**0.067** (0.055-0.086)	**0.077** (0.062-0.100)	**0.089** (0.069-0.119)	**0.102** (0.076-0.141)	**0.121** (0.086-0.172)	**0.136** (0.095-0.196)
10-day	**0.030** (0.025-0.035)	**0.033** (0.027-0.039)	**0.038** (0.032-0.045)	**0.043** (0.036-0.052)	**0.051** (0.042-0.065)	**0.058** (0.047-0.075)	**0.066** (0.051-0.088)	**0.075** (0.056-0.103)	**0.088** (0.063-0.125)	**0.098** (0.068-0.141)
20-day	**0.020** (0.017-0.024)	**0.022** (0.019-0.026)	**0.026** (0.022-0.030)	**0.029** (0.024-0.034)	**0.033** (0.027-0.041)	**0.037** (0.029-0.046)	**0.040** (0.031-0.053)	**0.044** (0.033-0.060)	**0.050** (0.036-0.070)	**0.055** (0.038-0.078)
30-day	**0.017** (0.014-0.020)	**0.018** (0.016-0.022)	**0.021** (0.018-0.025)	**0.023** (0.020-0.028)	**0.027** (0.022-0.033)	**0.029** (0.023-0.037)	**0.032** (0.025-0.041)	**0.034** (0.026-0.046)	**0.038** (0.027-0.053)	**0.040** (0.028-0.057)
45-day	**0.014** (0.012-0.016)	**0.015** (0.013-0.018)	**0.018** (0.015-0.021)	**0.020** (0.017-0.023)	**0.022** (0.018-0.027)	**0.024** (0.019-0.030)	**0.026** (0.020-0.033)	**0.027** (0.020-0.036)	**0.030** (0.021-0.041)	**0.031** (0.022-0.044)
60-day	**0.012** (0.010-0.014)	**0.014** (0.012-0.016)	**0.016** (0.013-0.018)	**0.017** (0.015-0.020)	**0.019** (0.016-0.024)	**0.021** (0.017-0.026)	**0.022** (0.017-0.029)	**0.024** (0.018-0.031)	**0.025** (0.018-0.035)	**0.026** (0.019-0.037)

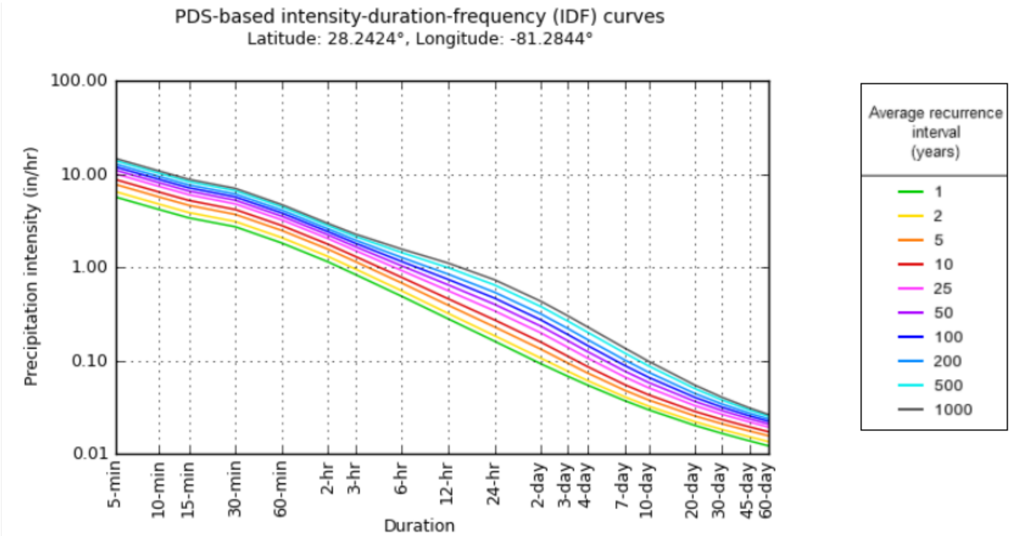
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Estimates from the table in CSV format: Precipitation frequency estimates | Submit

By clicking the graphical tab, you can display the intensity-duration-frequency (IDF) graph:



To import the NOAA Atlas 14 IDF curves into OpenRoads Designer (ORD), refer to ***FDOTConnect Drainage Design & 3D Modeling with Plans Development Training Guide*** for the step-by-step procedure.
https://www.fdot.gov/cadd/main/FDOTCaddTraining.shtm

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. Refer to ***Drainage Manual*** for the minimum allowable time of concentration, as applicable.

You can use either of the following methods for calculating the time of concentration:

2.2.2.1 Velocity Method

The Velocity Method is a segmental approach, which you can use 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, you can calculate a travel time using the equation:

$$t_i = \frac{L_i}{60\, v_i} \qquad \textbf{(2.2-2)}$$

where:

t_i = Travel time for velocity in segment i, in minutes

L_i =	Length of the flow path for segment i, in feet
v_i =	Average velocity for segment i, in feet/second

The time of concentration is calculated as:

$$t_c = t_1 + t_2 + t_3 + \ldots t_i \tag{2.2-3}$$

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. Determining travel time for overland flow, shallow channel flow, and main channel flow are discussed below.

(A) Overland Flow (t_1)

If you know the average slope and the land use, you can determine the time of concentration for overland flow using Figure B-2 in Appendix B (Hydrology Design Aids). This chart gives reasonable values and is used by district drainage staff around the state.

The Federal Highway Administration (FHWA) prefers the Kinematic Wave Equation developed by Ragan (1971) for calculating the travel time for overland conditions. Figure B-1 in Appendix B (Hydrology Design Aids) presents a nomograph that you can use to solve this equation, as follows:

$$t_1 = \frac{0.93\, L^{0.6}\, n^{0.6}}{i^{0.4}\, s^{0.3}} \tag{2.2-4}$$

where:

t_1 =	Overland flow travel time, in minutes
L =	Overland flow length, in feet (maximum 100 feet recommended by Natural Resources Conservation Service [NRCS])
n =	Manning roughness coefficient for overland flow (See Table B-1 in Appendix B, Hydrology Design Aids)
i =	Rainfall intensity, in inches/hour
S =	Average slope of overland flow path, in feet/feet

Manning's n values reported in Table B-1 in Appendix B were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations. Equation 2.2-4 generally involves 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 B-1 (Appendix B).
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)

Knowing the slope of the flow segment, average velocities for shallow channel flow (shallow concentrated flow) are obtained from Figure B-3 in Appendix B (Hydrology Design Aids).

Calculate the velocity using this equation:

$$V = kS^{0.5} \tag{2.2-5}$$

where:
V = Velocity (feet per second)
S = Longitudinal slope in feet / feet
k = Constant for different flow types. (Refer to table below Figure B-3 in Appendix B)

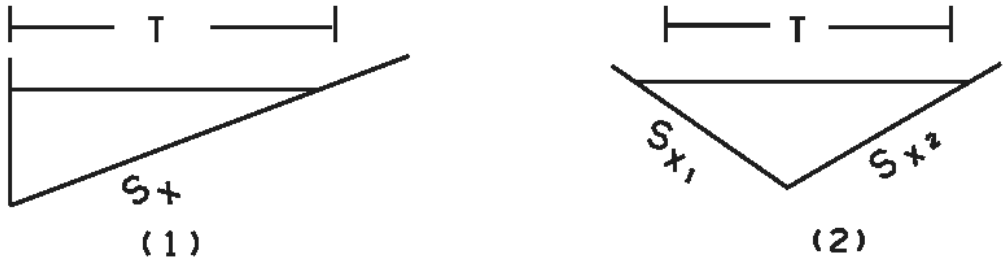
You also can calculate gutter flow velocities using the following equation:

$$V = \frac{1.12}{n} S^{0.5} S_X^{0.67} T^{0.67} \tag{2.2-6}$$

where:

S = Longitudinal slope
n = Manning’s n for street and pavement gutters (Appendix B, Table B-2)

S_X and T are as shown on (1) below.



For a triangular gutter, S_x and T are as shown in (2) above.

$$S_X = \frac{S_{X1} S_{X2}}{S_{X1} + S_{X2}} \qquad \textbf{(2.2-7)}$$

Use the conventional form of Manning's Equation to evaluate shallow channel flow.

(C) Main Channel Flow (t_3)

Flow in rills, gullies, and/or gutters empties into channels or pipes. Assume that open channels begin where either a blue line stream shows on USGS quad maps or where the channel is visible on aerial photos.

Evaluate average velocities for main channel flow using Manning's Equation.

$$V = \frac{1.486}{n} R^{0.67} S^{0.5} \qquad \textbf{(2.2-8)}$$

where:

V = Velocity in feet per second
n = Manning's n value from Table B-3 (Appendix B)
R = Hydraulic Radius (A/P)
S = Longitudinal Slope in feet/feet
P = Wetted perimeter of channel, in feet
A = Cross-sectional area of the open channel, in square feet

More discussion on using Manning's Equation is provided in Chapter 3, Section 3.1.2.1.

2.2.2.2 Kirpich (1940) Equation

You can use the Kirpich Equation 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 from zero percent to 56 percent, and watershed areas ranged from 1.2 acres to 112 acres. Although these data appear to be limited and site-specific, the Kirpich Equation has given good results in Florida applications. The Kirpich Equation is expressed as:

$$t_c = 0.0078 \frac{L^{0.77}}{S^{0.385}} F_s \qquad \textbf{(2.2-9)}$$

where:

t_c = Time of concentration, in minutes
L = Length of travel, in feet
S = Slope, in feet/feet
F_s = 1.0 for natural basins with well-defined channels, overland flow on bare earth, and mowed grass roadside channels
= 2.0 for overland flow on grassed surfaces
= 0.4 for overland flow on concrete or asphaltic surfaces
= 0.2 for concrete channels

Separate the flow path into different reaches if there are breaks in the slope and changes in the topography. Add together the times of travel in each reach to obtain the time of concentration (see Equation 2.2-3).

2.2.3 Peak Runoff Rates—Ungaged Sites

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

2.2.3.1 Rational Equation

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

$$Q = C\,i\,A \qquad \textbf{(2.2-10)}$$

where:

Q = Peak flow rate (cubic feet per second)
C = Runoff coefficient

i = Rainfall intensity (inches per hour)
A = Area (acres)

(A) Runoff Coefficient

The runoff coefficient is a dimensionless number that represents the percent of rainfall that runs off a site. Table B-4 in Appendix B (Hydrology Design Aids) presents runoff coefficient ranges for various land uses, soil types, and watershed slopes. Perform a site review and use your best engineering judgment to select the coefficient within these ranges. Table B-5 in Appendix B 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 2.2-1.) For sites with several land uses, the weighted average of the runoff coefficient is expressed as:

$$Weighted\ C = \frac{\Sigma\, C_i\, A_i}{A_{Total}} \qquad \textbf{(2.2-11)}$$

(B) Rainfall Intensity

The rainfall intensity is determined using NOAA Atlas 14 data based on the time of concentration and the storm frequency (recurrence interval).

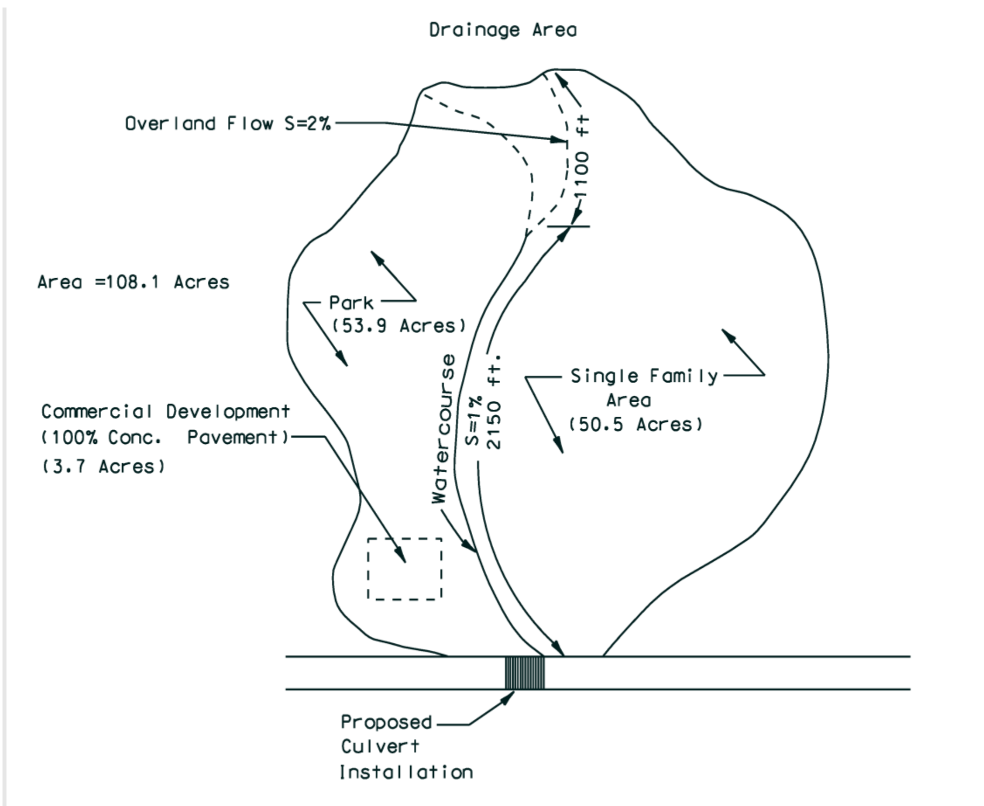
(C) Assumptions and Limitations

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 the Drainage Manual.

Example 2.2-1: Use of the Rational Method

A flooding problem exists along a farm road near Somewhere Springs, Florida (sandy soil). A low water crossing is to be replaced by a culvert to improve the road safety during rainstorms. The drainage area is shown 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 B-4 and B-5 in Appendix B, 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 (1,100 ft) – "Residential" at 2-percent slope.

      From Figure B-2 (Appendix B)
      Velocity = 57 ft/min

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

   b. Channelized flow (2,150 ft) – "Grassed Waterway" at 1-percent slope.

      From Figure B-3 (Appendix B)
      Velocity = 1.6 ft/sec

$$t_2 = \Sigma \frac{Distance_2}{Velocity_2} = \frac{2150\ ft}{1.6\ ft/\sec\ x\ 60 \sec/\min} = 22.4 \min.$$

   c. Time of concentration is estimated as:

$$t_c = t_1 + t_2 = 19.3 + 22.4 = 41.7 \text{ min.}$$

   d. Intensity is obtained from the NOAA Atlas 14data using a duration equal to the time of concentration (t_c). For this project location, the following NOAA Atlas 14 data is generated:

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	6.31 (5.21-7.49)	7.10 (5.86-8.42)	8.33 (6.84-9.92)	9.29 (7.58-11.1)	10.5 (8.26-13.0)	11.4 (8.76-14.5)	12.3 (9.07-16.1)	13.1 (9.24-17.8)	14.1 (9.53-19.9)	14.7 (9.74-21.4)
10-min	4.63 (3.82-5.48)	5.20 (4.28-6.17)	6.10 (5.00-7.27)	6.80 (5.55-8.15)	7.72 (6.04-9.54)	8.38 (6.41-10.6)	9.00 (6.64-11.8)	9.59 (6.77-13.0)	10.3 (6.98-14.5)	10.8 (7.13-15.7)
15-min	3.76 (3.10-4.46)	4.23 (3.48-5.02)	4.96 (4.07-5.90)	5.53 (4.52-6.63)	6.28 (4.91-7.76)	6.81 (5.22-8.61)	7.32 (5.40-9.56)	7.80 (5.50-10.6)	8.38 (5.67-11.8)	8.78 (5.80-12.8)
30-min	2.91 (2.40-3.45)	3.28 (2.70-3.89)	3.85 (3.16-4.59)	4.30 (3.51-5.15)	**4.88** (3.82-6.03)	5.30 (4.06-6.70)	5.69 (4.20-7.44)	6.06 (4.28-8.23)	6.52 (4.41-9.19)	6.83 (4.51-9.92)
60-min	1.92 (1.58-2.27)	2.14 (1.76-2.54)	2.50 (2.06-2.98)	2.80 (2.29-3.36)	**3.20** (2.51-3.97)	3.50 (2.69-4.44)	3.79 (2.81-4.98)	4.09 (2.89-5.57)	4.47 (3.03-6.33)	4.75 (3.14-6.89)
2-hr	1.19 (0.986-1.40)	1.32 (1.10-1.56)	1.54 (1.27-1.82)	1.73 (1.42-2.05)	1.98 (1.57-2.45)	2.17 (1.68-2.75)	2.37 (1.77-3.10)	2.57 (1.83-3.49)	2.84 (1.94-4.00)	3.04 (2.02-4.39)
3-hr	0.861 (0.718-1.01)	0.957 (0.797-1.13)	1.12 (0.929-1.32)	1.26 (1.04-1.50)	1.46 (1.17-1.81)	1.62 (1.26-2.05)	1.79 (1.34-2.34)	1.96 (1.40-2.66)	2.19 (1.51-3.10)	2.38 (1.59-3.42)

Although IDF curves are available from the NOAA Atlas 14 website, it can be difficult to accurately pick data from the IDF curve, so using the tabular data just above and below the time-of-concentration should be used to interpolate for the desired value The resulting two data points are:

$i_{25yr,30min}$ = 4.88 in/hr

$i_{25yr,60min}$ = 3.20 in/hr

Using a rainfall duration equal to the time of concentration of 42 minutes (or 0.7 hours), a linear interpolation* is performed, and the resulting rainfall intensity:

i_{25} = 4.21 in/hr

*IDF Curves are plotted on log-log distributions. Linear interpolation skews towards the lower duration value, resulting in slightly higher intensities and slightly lower rainfall depths than a log-log interpolation. For rational methods, which use intensities, linear interpolation provides some additional conservatism to the design. Analysis of linear interpolation of rainfall depths shows the difference is negligible as compared to log-log interpolation, and results are typically within the 90% confidence intervals. Therefore, linear interpolation is an acceptable practice, with verification the results are within the 90% confidence interval bounding values.

3. Calculate the peak flow.

$Q_{25} = C \times i_{25} \times A = 0.35 \times 4.21 \times 108.1 = 159.29$ cubic feet per second

2.2.3.2 Regression Equations

(A) Urban Conditions

You can use regression equations developed by the USGS (Verdi, 2006) 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, you can use the method presented in WSP No. 1674 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 B-10 through B-13 in Appendix B (Hydrology 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:

$$Q_T = C\,A^a\,(ST + 1.0)^b \tag{2.2-12}$$

where:

Q_T =	Peak runoff rate for return period T, in ft^3/sec.
C =	Regression constant (See Appendix B, B-10 through B-13)
A =	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 Appendix B, B-10 through B-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 B-10 through B-13 (Appendix B). 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 B-14 through B-16 (Appendix B). 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 B-17 (Appendix B). The standard error of model is a measure of how well the regression equation model estimates flood flows.

When applying the regression equations, you should consider the following limitations:

1. The relationship of the regression equations for areas with basin characteristics outside the ranges given above. Do not use the equations for watershed conditions

outside the range of applicability shown in Tables B-10 through B-13 in Appendix B (Hydrology Design Aids).

2. In areas of karst 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 you determine the drainage area from 7.5-minute topographic maps, subtract any area containing sinkholes or depressions (non-contributing areas) 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, you should take the following steps:

1. Locate the appropriate region on Figure B-4 (Appendix B).
2. Select the appropriate equations (from Appendix B, Tables B-10 through B-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.

(B) Urban Conditions

You can use regression equations developed by the USGS as part of a nationwide project to estimate peak runoff for urban watershed conditions. Regionalized regression equations for the Tampa Bay area, Leon County, and West-Central Florida also are 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 Appendix B, Table B-14) 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:

$$UQ_T = C\ A^{B_1}\ SL^{B_2}(i_2 + 3)^{B_3}(ST + 8\ )^{B_4}(13 - BDF\ )^{B_5}\ IA^{B_6}(\ RQ_T\ )^{B_7} \qquad \textbf{(2.2-13)}$$

where:

UQ_T = Peak discharge, in cubic feet per second, for the urban watershed for

	recurrence interval T
C =	Regression constant (See Appendix B, Table B-14)
A =	Contributing drainage area in square miles
SL =	Channel slope (feet/mile) between points 10 percent and 85 percent of the distance from the design point to the watershed boundary
i_2 =	Rainfall intensity, in inches, for the two-hour, two-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 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 cubic feet per second, 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 to B_7 =	Regression exponents (See Appendix B, Table B-14)

Basin Development Factor—Determine the BDF from drainage maps and by field inspection of the watershed. First, divide the basin into three sections so that each sub-area contains approximately one-third of the drainage area. Mark distances along main streams and tributaries so that, within each third, the travel distances of two or more streams are about equal. Generally, you can draw the lines on the drainage map by visual estimate without the need for measurements. Complex basin shapes and drainage patterns require more judgment when subdividing.

You will examine four drainage aspects for each subsection, assigning a code of zero or one to each aspect for each subsection. The BDF, therefore, can range from zero for an undeveloped watershed to 12 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 percent or more of the main channels and principal tributaries (those that drain directly into the main channel) have been improved from natural conditions, assign a code of one; otherwise, assign a code of zero. Improvements include straightening, enlarging, deepening, and clearing.
2. Channel Linings—Assign a code of one if more than 50 percent of the length of the main channels and principal tributaries have impervious linings, such as concrete; otherwise, assign a code of zero. Lined channels are an indication of a more

developed drainage system in which channels probably have 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) that 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 percent of the secondary tributaries within a sub-basin consist of storm drains, assign a code of one to this aspect; otherwise, assign a code of zero. Note that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, you also would assign the aspects of channel improvements and channel linings a code of one.
4. Curb and Gutter Streets—If more than 50 percent of a sub-basin is urbanized (covered by residential, commercial, or industrial development), and if more than 50 percent of the streets and highways in the sub-basin are constructed with curbs and gutters, then assign a code of one to this aspect; otherwise, assign a code of zero. 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 you should perform field checks to obtain the best estimate.

Example 2.2-2: 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 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
460/2,500 < 50% Code = 0
Middle third: 2,020 ft have been straightened and deepened
2,020/3,800 > 50% Code = 1
Lower third: 1,720 ft have been straightened and deepened
1,720/3,000 > 50% Code = 1
Channel Linings
Upper third: 0 ft have been lined
0/2,500 < 50% Code = 0
Middle third: 1,770 ft have been lined
1,770/3,800 < 50% Code = 0
Lower third: 1,570 ft have been lined
1,570/3,000 > 50% Code = 1
Storm Drains on Secondary Tributaries
Upper third: 1,345 ft have been converted to storm drains
1,345/5,180 < 50% Code = 0
Middle third: 2,330 ft have been converted to storm drains
2,330/3,940 > 50% Code = 1
Lower third: 1,510 ft have been converted to storm drains
1,510/2,160 > 50% Code = 1
Curb and Gutter Streets
Upper third: 690 ft of curb and gutter street
690/2,850 < 50% Code = 0
Middle third: 3,020 ft of curb and gutter street
3,020/4,700 > 50% Code = 1
Lower third: 3,180 ft of curb and gutter street
3,180/5,610 > 50% Code = 1
Total BDF = 7

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

You can use regression equations developed as part of a nationwide project by the USGS (Sauer et al., 1983) 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 B-15, B-16, and B-17 in Appendix B show the USGS Regionalized Regression Equations for the Tampa Bay area, Leon County, and West-Central Florida respectively.

(a) 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:

$$Q_T = C\, A^{B_1} BDF^{B_2}\; SL^{B_3} (DTENA + 0.01)^{B_4} \qquad \textbf{(2.2-14)}$$

For 25-, 50-, and 100-year frequencies:

$$Q_T = C\, A^{B_1} (13 - BDF)^{B_2}\; SL^{B_3} \qquad \textbf{(2.2-15)}$$

where:

Q_T =	Peak runoff rate for return period T, in cubic feet per second
C =	Regression constant (See Appendix B, Table B-15)
A =	Drainage area in square miles
BDF =	Basin development factor (dimensionless)
SL =	Channel slope (feet/mile) between points 10 percent 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 drainage area.
B_1, B_2, etc. =	Regression exponents (See Appendix B, Table B-15)

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

1. Determine input parameters, including drainage area, basin development factor (see Example 2.2-2), channel slope, and the surface area of lakes,

ponds, etc.

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

(b) 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:

$$Q_T = C \ A^{B_1} IA^{B_2} \tag{2.2-16}$$

where:

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

These equations must not be used for watershed conditions outside the range of applicability shown in Table B-16 (Appendix B). 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 B-16 (Appendix B), 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 B-16 (Appendix B).

(c) 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:

$$Q_T = C \ A^{B_1} (LK + 0.6)^{B_2} \tag{2.2-17}$$

For Regions 2 through 4:

$$Q_T = C \; A^{B_1} (LK + 3.0)^{B_2} SL^{B_3} \qquad \textbf{(2.2-18)}$$

where:

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

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

1) Locate the appropriate region on Figure B-5 (Appendix B).
2) Select the appropriate equations (from Appendix B, Table B-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.

(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 also may 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.

2.2.4 Flood Hydrographs

When observed data for deriving unit hydrograph parameters is not available, use either the Modified Rational Method or the NRCS unit hydrograph procedures. Both procedures utilize the precipitation frequency data from NOAA Atlas 14 described above in Section 2.2.1. The Department's rainfall distributions are available from the Department's Internet site. Each Water Management District specifies rainfall distributions appropriate for their respective regions.

2.2.4.1 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 5.4.2.)

Example: Using a drainage area of 0.981 acres, t_c of 10 minutes, Rational runoff coefficient (C) of 0.82, and NOAA Atlas 14 data, calculate an inflow hydrograph for the 100-year,2-hour rainfall.

From the NOAA Atlas 14 data, the 100-year, 2-hour precipitation frequency estimate (P_{total}) can be found. For this case, assume that the webpage provides P_{total} = 5.4 inches

(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 data table
Column 3 is Column 2 times P_{total}
Column 4 is Column 3 times CA (0.82 for this example)

2.2.4.2 NRCS Hydrograph

Techniques developed by the NRCS, 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 NRCS approach also considers the time distribution of the rainfall, initial losses to interception and depression storage, and infiltration that decreases during the storm. Since NRCS hydrographs are calculated using computers, the discussion in this guide will address the basic concepts rather than computation methods.

(A) Time of Concentration

Calculate the time of concentration using any of the methods in Section 2.2.2.

(B) Curve Number

The NRCS 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 NRCS relationship. Antecedent Moisture Condition I (AMC-I) is the lower limit of antecedent rainfall or the upper limit of the potential 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 generally is selected for design purposes. The curve number values in the tables in the Appendix B (Hydrology 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 Appendix B, Table B-6.) In general:
   - A = deep sand, deep loess, aggregated silts, high infiltration
   - B = shallow loess, sandy loam, moderate infiltration
   - C = clay loams, shallow sandy loam, soils low in organic content, soils usually high in clay, slow infiltration,
   - D = soils that swell significantly, heavy plastic clays, some saline soils, very slow infiltration
3. Identify drainage areas with uniform soil type and land use conditions.
4. Use tables B-7 through B-9 (Appendix B) 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:

$$CN_C = \frac{\Sigma\, CN_i\, A_i}{A_T} \qquad \textbf{(2.2-19)}$$

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 U.S. Department of Agriculture (USDA) 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, April 1996.

The resulting equation for estimating the CN is:

$$CN = 58.38 - 8.2716 \ln(A) + 0.50274\ HCIA + 6.22971 \ln(L) + 0.68079 \ln(L_c) - 0.14986\ S \quad \textbf{(2.2-20)}$$

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)

(C) Rainfall-Runoff Relationship

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

$$S = \frac{1000}{CN} - 10 \quad \textbf{(2.2-21)}$$

where:

S = Potential maximum soil storage, in inches
CN = Watershed curve number, dimensionless

Knowing the potential maximum soil storage, calculate the rainfall excess using the following NRCS relationship:

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \textbf{(2.2-22)}$$

where:

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

Additional information on the NRCS relationship is available in USDA, NRCS publications TP-149 (1973), NEH-4 (1972), and TR-55 (1986).

(D) Shape Factor

The hydrograph shape factor (B) generally is considered to be a constant characteristic of a watershed. The NRCS dimensionless unit hydrographs are based on a B value of 484. However, since the value of B generally ranges from 600 in steep terrain to 300 or less in flat swampy areas, you may need to make adjustments to the unit hydrograph shape. You can make these adjustments 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 NRCS 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 NRCS for watersheds with average slope of 0.5 percent 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 to 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 to 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 NRCS shape factor is:

$$B = \exp\left[390 - 0.01396A - 0.00473HCIA + 0.00064L - 0.00053L_C + 0.00567S\right]$$

(2.2-23)

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.