PDHonline Course H119 (2 PDH)

Estimating Storm Water Runoff

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CHAPTER 3
ESTIMATING STORMWATER RUNOFF

CHAPTER NOTE

Estimating stormwater runoff is a basic initial step in the design of the stormwater management system as well as the erosion control plan. This chapter presents several commonly used methods and procedures used in this process.

3.1 INTRODUCTION

To determine the volume of stormwater runoff from precipitation, hydrologic calculations are used to quantify precipitation losses which occur as part of the hydrologic cycle. Typically, stormwater management calculations only consider infiltration, interception and surface storage losses, since short time scales will render losses from evaporation and transpiration insignificant.

A wide variety of procedures have been developed to estimate runoff volume and peak discharge rate; and to route the runoff through stormwater management systems. This section discusses only a few methods which are acceptable for estimating the runoff treatment volume required to meet the water quality objectives of the Stormwater Rule. For anyone wishing to obtain a greater understanding of hydrologic methods, especially those used in designing stormwater systems to achieve flood protection purposes, the following documents are recommended:

1. "Urban Hydrology for Small Watersheds", Technical Release 55 (TR55), USDA-Soil Conservation Service, 1986.

2. Drainage Manual, Florida Department of Transportation, 1987.

3. National Engineering Handbook, Section 4-Hydrology, USDA-Soil Conservation Service, 1985.

3.2 DESIGN STORMS

To estimate runoff, the amount of rainfall contributing to the runoff of a given area must be known. The designer must estimate the runoff from predevelopment and postdevelopment conditions and design a stormwater management system to retain the excess quantity and treat the reduced quality of the water. Regulations will dictate a minimum "design storm" for use in stormwater calculations.

A design storm is a theoretical storm event based on rainfall intensities associated with frequency of occurrence and having a set duration. For example, a 50 year - 24 hour storm event is one that theoretically occurs once every fifty years and lasts for 24 hours. A stormwater management system designed for such a storm would theoretically fail every fifty years. The amount of rainfall for a design storm is based on the historical rain data of the geographical location in question. For a 100 year period, the probability of any particular design storm occurring in any given year is the storm frequency divided by 100. The following is a list of average rainfall amounts for different design storms for Orlando, Florida (these were interpolated from the U.S. Weather Service 24 hour duration maps found in Technical Publication 40) and the probability of occurrence in any given year:

Design Storm	Avg. Rainfall	Probability
2 year - 24 hour	4.5"	50 %
5 year - 24 hour	6.5"	20 %
10 year - 24 hour	7.5"	10 %
25 year - 24 hour	8.5"	4 %
50 year - 24 hour	9.5"	2 %
100 year - 24 hour	10.5"	1 %

3.3 HYDROGRAPHS

A hydrograph is a graph displaying some property of water flow, such as stage (i.e. water level), discharge, velocity, etc., versus time. For displaying runoff characteristics of a watershed, the hydrograph is one of discharge (cubic feet per second) versus time (hours). It represents watershed runoff at a certain point in the flow and includes only the rainfall upstream of the point in question. Any rainfall downstream of this point is not represented.

A typical hydrograph is illustrated in Figure 3.3a. There are three basic parts to the hydrograph: (1) the rising limb or concentration curve, (2) the crest segment, and (3) the recession curve or falling limb. Analytical properties of the hydrograph are: (1) Lag time (L) which is the time interval from the center of mass of the rainfall excess to the peak of the hydrograph; (2) Time to peak (T_p) which is the time interval from the start of rainfall excess (direct runoff) to the peak of the hydrograph; (3) Time of concentration (T_c) which is the time interval from the end of the rainfall excess to the point on the falling limb of the hydrograph where the recession curve begins (the point of inflection). Time of concentration is the travel time between the furthest point on the watershed to the point represented by the hydrograph or point of interest. This will be discussed further in the Rational Method section.

In Figure 3.3a the rectangle above the hydrograph, which in hydrologic terminology is called the hyetograph, consists of two separate parts - the losses (upper shaded portion) due to infiltration, evaporation, etc. and the rainfall excess (lower white portion) which is the runoff that produces the hydrograph. The duration (D) of the rainfall excess is shown. The volume of rainfall excess is the rainfall intensity (inches per hour) × duration (hours) × the watershed area. The volume of runoff can also be determined by calculating the area under the hydrograph.

Hydrographs are an excellent way to compare predevelopment versus postdevelopment conditions. As seen in Figure 3.3b, peak runoff for postdevelopment is considerably greater than that of predevelopment. Also, the time of concentration for postdevelopment conditions is shorter; therefore, the runoff is traveling at a greater velocity which can contribute to increased erosion rates. The hydrograph for postdevelopment with peak discharge control shows how proper stormwater management can reduce peak runoff and lengthen time of concentration.

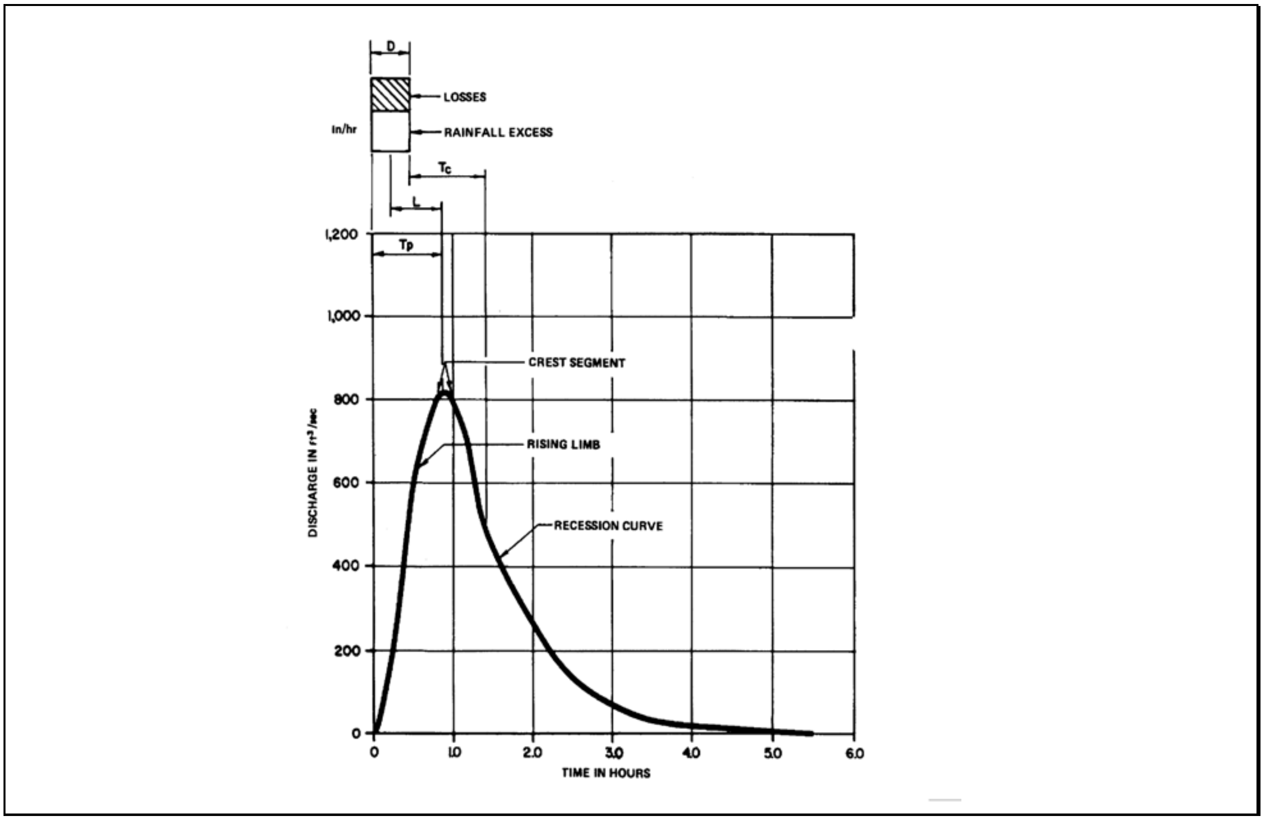

Plate 3.3a Hydrograph Properties
Source: Florida Development Manual

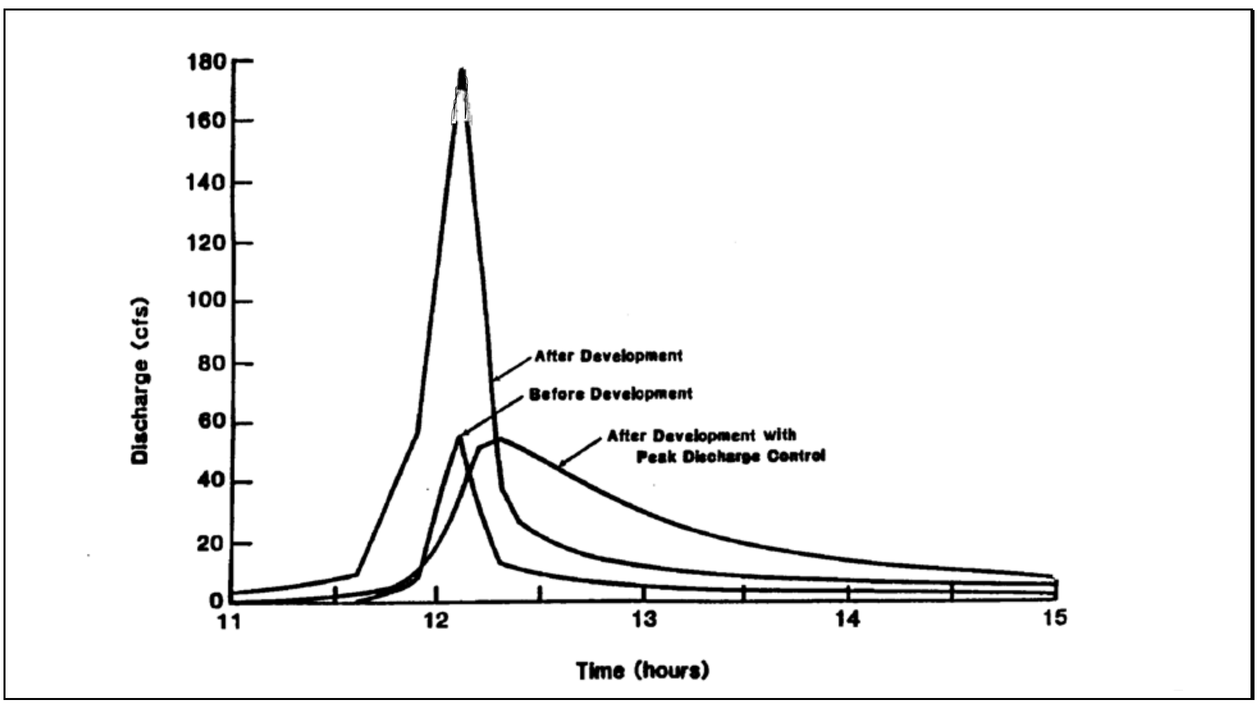

Plate 3.3b Comparison of Hydrographs
Source: Florida Development Manual

3.4 GENERAL PROCEDURE

To meet the water quality objectives of the Stormwater Rule, it is vital that the first flush of pollutants be captured and treated. Many of the methods used to estimate runoff will under estimate runoff volumes because of various factors (e.g., abstraction losses). Therefore, to assure that the first flush is captured and treated, the easiest method to determine the stormwater treatment volume is simply to multiply the project size or contributing drainage area times the treatment volume.

EXAMPLE 3-1: **What is the treatment volume for a 50 acre subdivision with a desired retention of 0.5 inches of runoff and a detention of 1.0 inches of runoff?**

a. Retention treatment

$$\frac{(\,50\ acres\,)(\,0.5\ inches\ runoff\,)}{12\ in\,/\,ft} = 2.08\ acft$$

b. Detention treatment

$$\frac{(\,50\ acres\,)(\,1.0\ inches\ runoff\,)}{12\ in\,/\,ft} = 4.17\ ac\text{-}ft$$

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3.5 RATIONAL METHOD

The Rational Formula is the most commonly used method of determining peak discharges from small drainage areas. This method is traditionally used to size storm sewers, channels and other stormwater structures which handle runoff from drainage areas less than 200 acres.

The Rational Formula is expressed as

$$Q=(C\ )(i\ )(A\ ) \qquad \textbf{[Eq 3-1]}$$

where:

Q = peak rate of runoff in cubic feet per second *(cfs)*
C = runoff coefficient, a dimensionless unit
i = average intensity of rainfall in inches per hour *(in/hr)*
A = the watershed area in acres *(ac)*.

COMPONENTS OF THE RATIONAL FORMULA

A - The area

The area, A, draining to any point under consideration in a stormwater management system must be determined accurately. Drainage area information should include:

a. Land use - present and predicted future - as it affects degree of protection to be provided and percentage of imperviousness.

b. Character of soil and ground cover as they may affect the runoff coefficient.

c. General magnitude of ground slopes which, with previous items above and shape of drainage area, will affect the time of concentration. This includes information about individual lot grading and the flow pattern of runoff along swales, streets and gutters.

C - The runoff coefficient

The runoff coefficient, *C*, is expressed as a dimensionless decimal that represents the ratio of runoff to rainfall. Except for precipitation, which is accounted for in the formula by using the average rainfall intensity over some time period, all other portions of the hydrologic cycle are contained in the runoff coefficient. Therefore, *C* includes interception, infiltration, evaporation, depression storage and groundwater flow. The variables needed to estimate *C* should include soil type, land use, degree of imperviousness, watershed slope, surface roughness, antecedent moisture condition, duration and intensity of rainfall, recurrence interval of the rainfall, interception and surface storage. The fewer of these variables used to estimate *C*, the less accurately the rational formula will reflect the actual hydrologic cycle.

The use of average runoff coefficients for various surface types is common. In addition, *C* is assumed to be constant although the coefficient will increase gradually during a storm as the soil becomes saturated and depressions become filled. A suggested range of runoff coefficients is shown in Table 3-1. These coefficients are only applicable for storms of 5 to 10 year return frequencies and they were originally developed when many streets were uncurbed and drainage was conveyed in roadside swales (grassed waterways). For recurrence intervals longer than 10 years, the indicated runoff coefficients should be increased since nearly all of the rainfall in excess of that expected from the 10 year storm will become runoff.

i - Rainfall Intensity

The determination of rainfall intensity, *i*, for use in the Rational Formula involves consideration of three factors:

a. Average frequency of occurrence.

b. Intensity-duration characteristics for a selected rainfall frequency.

c. The rainfall intensity averaging time, T_C.

The critical storm duration that will produce the peak discharge of runoff is the duration equal to the rainfall intensity averaging time. The average frequency of rainfall occurrence used in the design of the stormwater system theoretically determines how often the structure will fail to serve the protective purpose for which it was designed.

The rainfall intensity averaging time, T_C, is usually referred to as the time of concentration. However, rainfall intensity averaging time more accurately defines the reason for and the use of this variable. T_C is not the total duration of a storm, but is a period of time within some total storm duration during which the maximum average rainfall intensity occurs.

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. The rainfall intensity averaging time (T_C) is computed by summing all the travel time for consecutive components of the stormwater conveyance system. Several factors will affect the time of concentration and the travel time. These include:

SURFACE ROUGHNESS - One of the most important effects of urbanization on stormwater runoff is increased flow velocity. Undeveloped areas have very slow and shallow overland flow through vegetation which becomes modified by development. The flow is then delivered to streets, gutters and storm sewers that transport runoff downstream more rapidly due to the decreased resistance of the ground cover. Thus, reducing travel time through the watershed.

CHANNEL SHAPE AND FLOW PATTERNS - In small rural watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying stormwater into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

SLOPE	LAND USE	SANDY SOILS		CLAYEY SOILS	
		MIN	MAX	MIN	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
	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, etc.	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
	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, etc.	0.50	0.70	0.60	0.80
	Commercial and Industrial	0.50	0.95	0.60	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
	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, etc.	0.60	0.75	0.65	0.85
	Commercial and Industrial	0.60	0.95	0.65	0.95

Source: FDOT (1987)

[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

For recurrence intervals longer than ten years, the indicated runoff coefficients should be increased, assuming that nearly all of the rainfall in excess of that expected from the ten year recurrence interval rainfall will become runoff and should be accommodated by an increased runoff coefficient.

The runoff coefficients indicated for different soil conditions reflect runoff behavior shortly after initial construction. With the passage of time, the runoff behavior shortly after initial construction. With the passage of time, the runoff behavior in sandy areas will tend to approach that in heavy soil areas. If the designer's interest is long term, the reduced response indicated for sandy soils should be disregarded.

DESIGN STORM FREQUENCY - For recurrence intervals longer than ten years, the indicated runoff coefficients should be increased. This assumes that nearly all of the rainfall in excess of that expected from the ten year recurrence interval rainfall will become runoff. Therefore, it should be accommodated by an increased runoff coefficient.

FUTURE CONSIDERATION - The runoff coefficients indicated for different soil conditions reflect runoff behavior shortly after initial construction. With the passage of time, the runoff behavior in sandy areas will tend to approach that in heavy soil areas. If the designer's interest is long-term, the reduced response indicated for sandy soil areas should be disregarded.

SLOPE - Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which swales and storm sewers are used in the stormwater management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers or street gutters.

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

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

$$T_t = \frac{L}{3600\,V} \qquad \textbf{[Eq 3-2]}$$

where:

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

Time of concentration is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \ldots + T_{tm} \qquad \textbf{[Eq 3-3]}$$

where:

T_C = time of concentration (hr)
m = number of flow segments

SHEET FLOW is flow over plane surfaces which usually occurs in the headwaters of streams. With sheet flow, the friction value (Manning's *n*) is an effective roughness coefficient that includes the effect of raindrops impact; drag over the plane surface; obstacles such as litter, crop ridges and rocks; and erosion and transportation of sediment. Table 3-2 gives Manning's *n* values for sheet flow (depths of about 0.1 foot) for various surface conditions.

For sheet flow of less than 300 feet, use Manning's kinematic solution to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad \textbf{[Eq 3-4]}$$

where:

T_t = travel time (hr)
n = Manning's roughness coefficient (Table 3-2)
L = flow length (ft)
P_2 = 2 year, 24-hour rainfall (in)
S = slope of hydraulic grade line (ft/ft)

This simplified form of the Manning's kinematic solution is based on the following:

1. Shallow steady uniform flow.
2. Constant intensity of rainfall excess.
3. Rainfall duration of 24-hours.
4. Minor effect of infiltration on travel time.

Table 3-2
ROUGHNESS COEFFICIENTS (MANNING'S n) FOR SHEET FLOW

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

Source: SCS (1986)

[1]Includes species such as weeping lovegrass, bluegrass, buffalograss and native grass mixtures.

[2]When selecting n, consider cover to a height of about 0.1 ft. This is only part of the plant cover that will obstruct sheet flow.

After a maximum of 300 feet, sheet flow usually becomes ***SHALLOW CONCENTRATED FLOW.*** The average velocity for this flow can be determined from Figure 3.5a, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, the average velocity can be calculated from the following equations:

$$\text{UNPAVED} \quad V = 16.1345\,(S\,)^{0.5}$$
$$\text{PAVED} \quad V = 20.3282\,(S\,)^{0.5} \qquad \textbf{[Eq 3-5]}$$

These two equations are based on the solution of Manning's equation with different assumptions for *n* and *r*. For unpaved areas, *n* is 0.05 and *r* is 0.4; for paved areas *n* is 0.025 and *r* is 0.2.

After determining the average velocity in Figure 3.5a or Equation 3-5, use Equation 3-2 to estimate travel time for the shallow concentrated flow segment.

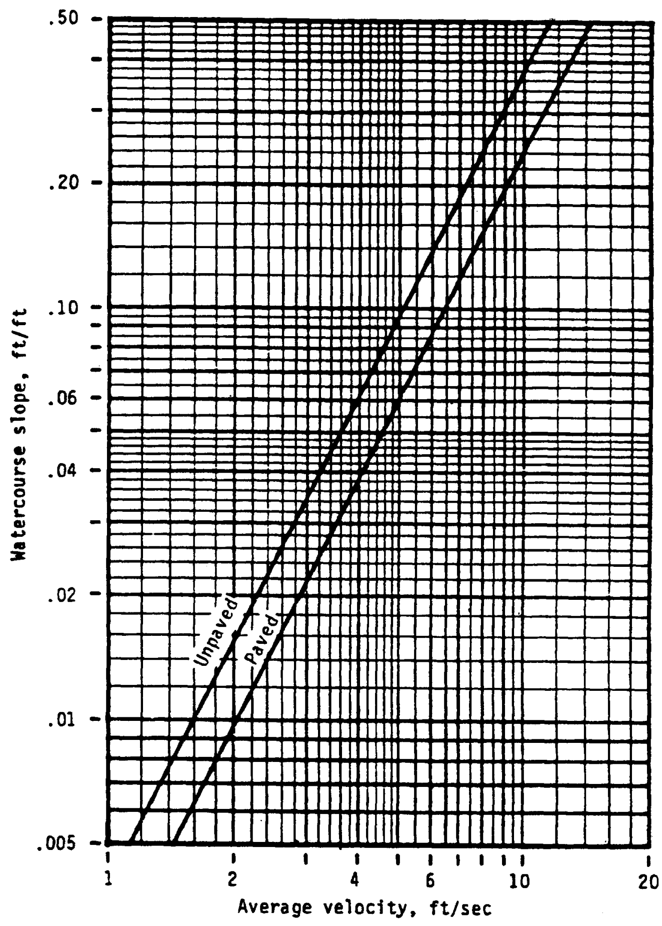


Plate 3.5a Average velocities for estimating travel time for shallow concentrated flow
Source: Florida Development Manual

OPEN CHANNELS are assumed to begin where surveyed cross-section information has been obtained, where channels are visible on aerial photographs or where blue lines (indicating streams) appear on USGS quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is:

$$V = \frac{1.49\, r^{2/3}\, s^{1/2}}{n} \qquad \textbf{[Eq 3-6]}$$

where:

V = average velocity (ft/sec)
r = hydraulic radius (ft) and is equal to a/P_w
a = cross sectional flow area (ft^2)
P_w = wetted perimeter (ft); this is the length of the portion of the cross sectional area in contact with the open channel
s = slope of the hydraulic grade line (ft/ft)
n = Manning's roughness coefficient for open channel flow

Manning's *n* values for open channel flow can be obtained from Table 3-2. Standard textbooks such as Chow (1959) or Linsley et.al (1982) also may be consulted to obtain Manning's *n* values for open channel flow. Manning's *n* values for other conditions can be found in Tables 3-3 through 3-5. After average velocity is computed using Equation 3-6, T_t for the channel segment can be estimated using Equation 3-2.

Table 3-3
RECOMMENDED MANNING'S *n* VALUES FOR ARTIFICIAL CHANNELS WITH BARE SOIL AND VEGETATIVE LININGS

CHANNEL LINING	DESCRIPTION	n
Bare earth, fairly uniform	Clean, recently completed	0.022
	Short grass and some weeds	0.028
Dragline excavated	No vegetation	0.030
	Light brush	0.040
Channels not maintained	Clear bottom, brush sides	0.08
	Dense weeds to flow depth	0.10
Maintained grass or sodded ditches	Good strand, well maintained 2"-6"	0.06*
	Fair strand, length 12"-24"	0.20*

Source FDOT (1987)

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

Table 3-4
RECOMMENDED MANNING'S N VALUES FOR ARTIFICIAL CHANNELS WITH RIGID LININGS

CHANNEL LINING	FINISH DESCRIPTION	*n*
Concrete paved	Broomed	0.016
	"Roughened" - Standard	0.020
	Gunite	0.020
	Over ruble	0.023
Asphalt concrete paved	Smooth	0.013
	Rough	0.016

Source: FDOT (1987)

Table 3-5
RECOMMENDED MANNING'S *n* VALUES FOR CULVERT DESIGN

CULVERT TYPE	*n*
Concrete pipe	0.012
Concrete box culvert precast or cast in place	0.012
Corrugated metal pipe (non-spiral flow - all corrugations):	
Round 15" - 24"	0.020
Round 30" - 54"	0.022
Round 60" - 120"	0.024
Corrugated metal pipe (spiral flow - all corrugations):	
Round 15" - 24"	0.017
Round 30" - 54"	0.021
Round 60" - 120"+	0.024
Corrugated metal pipe-arch - all sizes:	
2-2/3 x 1/2	0.024
3 x 1	0.027
5 x 1	0.027
Corrugated structural plate pipe and pipe-arch - all sizes:	
6 x 1	0.030
6 x 2	0.033
9 x 2-1/2	0.034

Source: FDOT (1987)

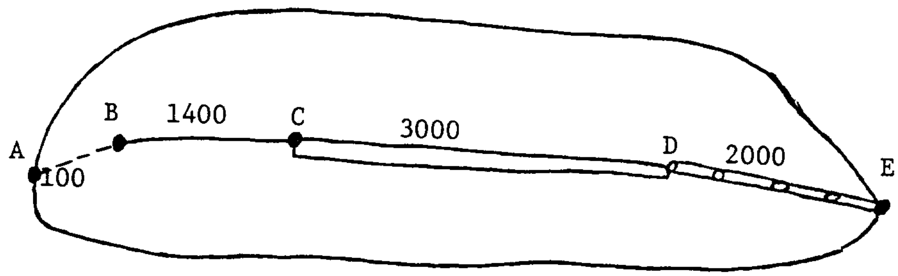
PIPE FLOW is simply water flowing in a pipe. The flow velocity can be calculated using the same equation as for open channel flow.

When the pipe in question is at full flow a modification of Equation 4-6 can be used for the pipe diameter (*d*) instead of using hydraulic radius (*r*)

$$V = \frac{0.59\,(d\,)^{2/3}\,(s\,)^{1/2}}{n} \qquad \textbf{[Eq 3-7]}$$

EXAMPLE 3-2

The sketch below shows an urbanized watershed in Leon County, Florida. The problem is to compute T_C at the outlet of the watershed (point E). The 2 year 24-hour rainfall depth is 4.8 inches (Figure 3.5b). Four types of flow occur from the hydraulically most distant point (A) to the point of interest (E). To compute T_C, first determine T_t for each segment based on the following data:



REACH	DESCRIPTION	SLOPE %	LENGTH (FT)
A to B	Sheet flow; dense grass	1.0	100
B to C	Shallow concentrated; unpaved	1.0	1400
C to D	Channel flow (Manning's n = 0.05 a = 27 ft^2, P_w = 0.015)	0.5	3000
D to E	Storm sewer (Manning's n = 0.015 diameter = 3 ft)	1.5	2000

1. Calculate sheet flow travel time (segment A to B):

   Given in this segment are length (L = 100 ft.) and slope (s = 0.01 ft/ft). The runoff in this segment is sheet flow; therefore, Table 4-2 is used for determining the n value. The n value for dense grass is 0.24. The 2 year - 24 hour rainfall P_2 for Leon County can be estimated from Figure 3.5b to be 4.8 inches.

   Solving Equation 3-4 with the above variables we get:

$$T_t = \frac{0.007\,(0.24x100\,)^{0.8}}{(4.8\,)^{0.5}(0.01\,)^{0.4}} = 0.256\ hr$$

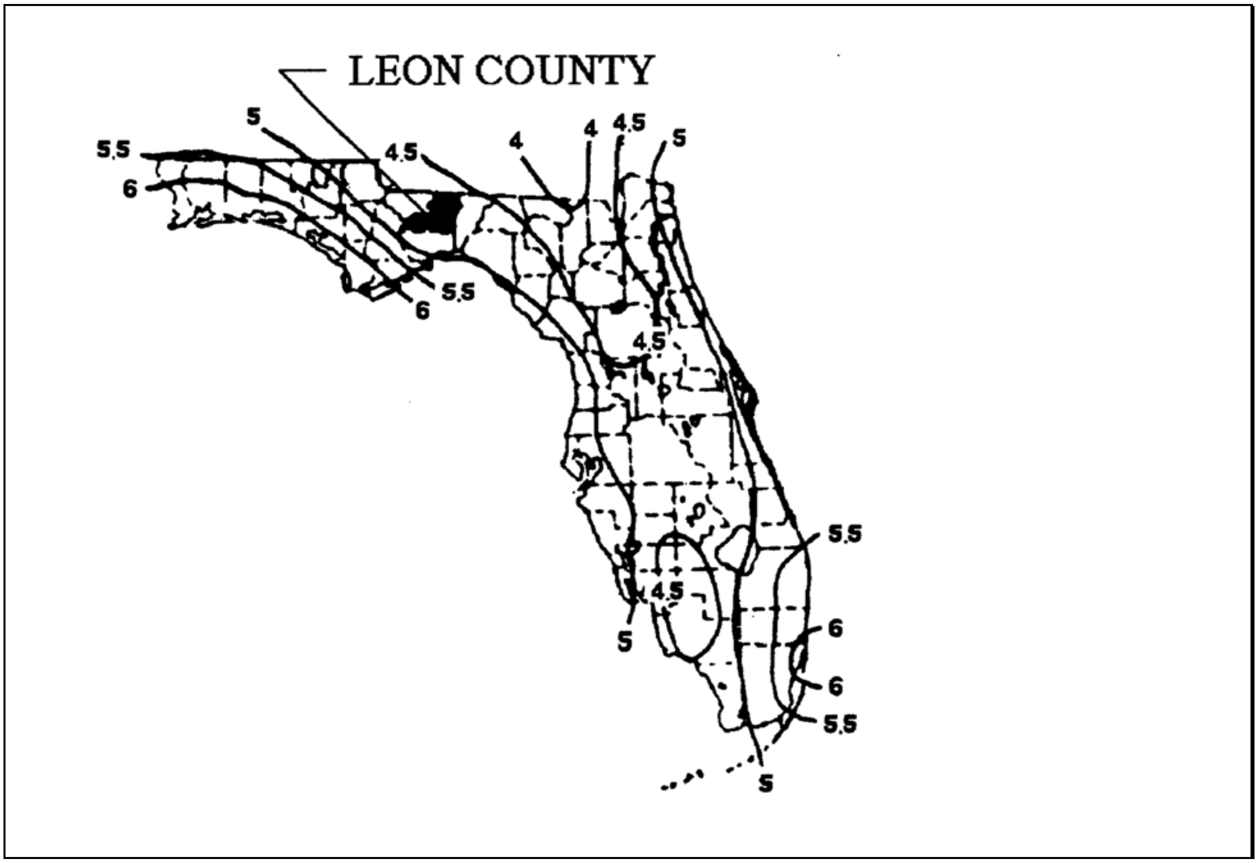


Plate 3.5b 2 Year - 24 Hour Rainfall (inches)
Source: FDOT

2. Calculate shallow concentrated flow (segment B to C).

For this segment length (*L* = 1400 ft) and slope (*s* = 0.01 ft/ft) are given. Use Figure 3-3 to find the average velocity for an unpaved watercourse with 0.01 ft/ft slope. The given slope (*s*) intersects the line representing "unpaved" at a velocity (*V*) of 1.6 ft/sec.

Solving Equation 3-2 with the above variables we get:

$$T_t = \frac{1400}{(3600)(1.6)} = 0.24\ hr$$

3. Calculate channel flow.

The runoff in segment C to D is now channel flow. We are given cross sectional flow area (*a* = 27 ft^2) and the wetted perimeter (P_w = 28.2 ft). With this information we can calculate the hydraulic radius (*r*)

$$r = \frac{27}{28.2} = 0.957\ ft$$

Also given in this segment are channel slope (s = 0.005 ft/ft) and Manning's roughness coefficient (n = 0.05). From these variables and the hydraulic radius calculated above we can calculate the velocity (V) of the runoff using Equation 3-6

$$V = \frac{1.49\,(0.957\,)^{2/3}\,(0.005\,)^{1/2}}{0.05} = 2.05\ ft/\sec$$

Now that we have the velocity and the given distance of this segment (L = 3000 ft) we can determine the travel time (T_t) by using Equation 3-2

$$T_t = \frac{3000}{(3600)(2.05)} = 0.406\ hr$$

4. Calculate storm sewer travel time (assume the pipe is flowing full).

In this segment (D to E) we are given the sewer pipe diameter (d = 3 ft) and Manning's roughness coefficient (n = 0.015). We can determine velocity by plugging the above variables into Equation 3-7

$$V = \frac{0.59\,(3\,)^{2/3}\,(0.015\,)^{1/2}}{0.015} = 10\ ft/\sec$$

Again using Equation 3-2 we can calculate travel time with the velocity determined above and the given length of sewer pipe (L = 2000 ft)

$$T_t = \frac{2000}{(3600)\,(10)} = 0.056\ hr$$

5. Calculate time of concentration for the watershed.

This is simply the addition of the travel times of the four flow segments (Eq 3-3)

$$T_c = 0.256 + 0.240 + 0.406 + 0.056 = 0.958\ hr = 57.5\ \min$$

How to Use the Rational Formula

The general procedure for determining peak discharge with the Rational Formula is:

Step 1) Determine the drainage area (in acres).

Step 2) Determine the runoff coefficient, C, for the type of soil/ cover in the drainage area (Table 3-1). If land use and soil cover are homogeneous over the drainage area, a C value can be determined directly from Table 3-1. If there are multiple soil cover conditions, a weighted average must be performed (see Example 3-3).

Step 3) Determine the rainfall intensity averaging time, T_C, in minutes for the drainage area (time required for water to flow from the hydraulically most distant point of that tributary watershed which produces the greatest discharge to the point of design). Example 3-2 illustrates how to calculate the time of concentration, T_C.

Step 4) Determine the Rainfall Intensity Factor, *i*, for the selected design storm. This is done by using the Rainfall Intensity - Frequency Duration chart (Figure 3.5d). Enter the "Duration" axis of the chart with the calculated time of concentration, T_C. Move vertically until you intersect the curve of the appropriate design storm, then move horizontally to read the Rainfall Intensity Factor, *i*, in inches per hour.

Step 5) Determine the peak discharge (*Q* - in cubic feet per second) by inserting the previously determined factors into the rational formula (Equation 3-1).

Example 3-3

Given: Drainage Area: 80 acres
30% - Rooftops (24 acres)
10% - Streets & Driveways (8 acres)
20% - Lawns @ 5% slope (16 acres) on sandy soil
40% - Woodland (32 acres)
Time of Concentration (T_C) = 15 min.
Location: Tallahassee, Florida (Leon County)

Find: Peak runoff rate from 10-year frequency storm.

Solution: 1. Drainage Area = 80 acres (given)
2. Determine runoff coefficient (c)

Plate 3.5c Zones for Precipitation Intensity - Duration - Frequency
Source: FDOT

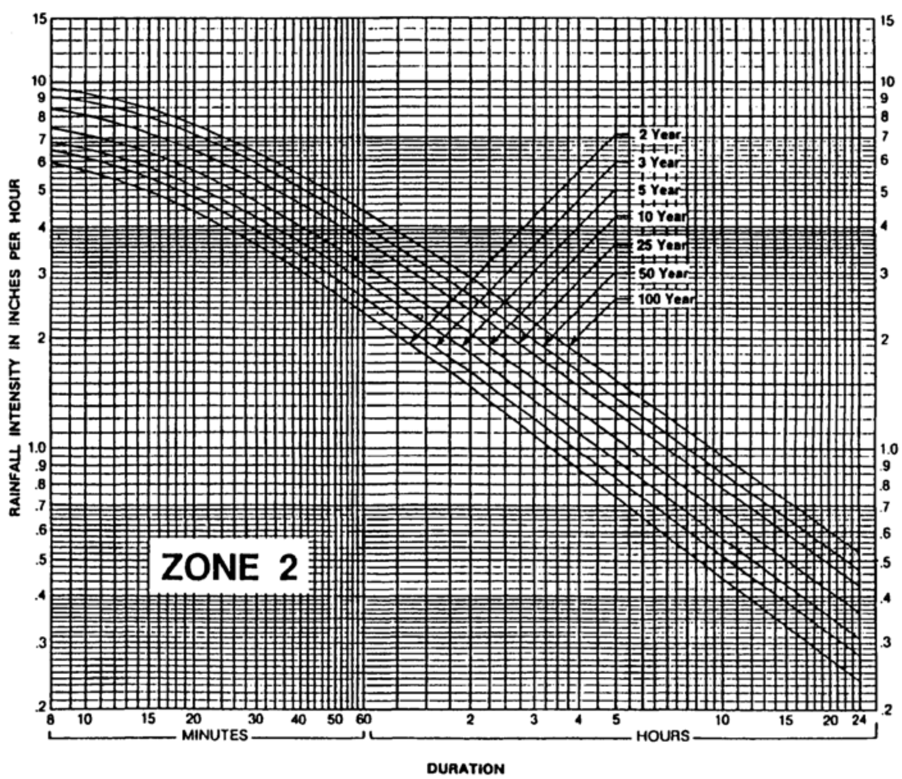


Plate 3.5d Rainfall Intensity - Duration - Frequency Curves for Zone 2
Source: FDOT

Perform Weighted Average

Ground Cover	Area (acres)	C from Table 4-1	Area × C
Rooftops	24	0.90	21.6
Streets	8	0.90	7.2
Lawns	16	0.15	2.4
Woodland	32	0.10	3.2
Total	80	Total	34.4

Weighted average of C is the total of the "C × Area" column divided by the total of the "Area" column

$$C = \frac{34.4}{80} = 0.43$$

3. Time of concentration (T_c) = 15 min. (given)

4. Determine Rainfall Intensity Factor (i)
   (i) = 6.2 in./hr. (from Figure 4-6)

5. Plug the above variables into Equation 3-1

$$Q = C\,(i\,)\,(A)$$
$$= 0.43\,(6.2)\,(80)$$
$$= 213.3\ cfs$$

Assumptions and Misconceptions

Assumptions and misconceptions are grouped together because an assumption used in the Rational Formula might in itself be a misconception. Several assumptions are listed below with each followed by a brief discussion.

1. The peak rate of runoff at any point is a direct function of the tributary drainage area and the average rainfall intensity during the time of concentration to that point. This is the rational formula stated in words.

2. The return period of the peak discharge rate is the same as the return period of the average rainfall intensity or rainfall event. While watershed-related variations may cause this relationship to break down, this assumption is widely used in methodologies for estimating peak flows or hydrographs.

3. The rainfall is uniformly distributed over the watershed. Whether this assumption is true depends upon the size of the watershed and the rainfall event.

4. The rainfall intensity remains constant during the time period equal to T_C. Based on rainfall records, this assumption is true for short periods of time (a few minutes), but becomes less true as time increases. In turn, this assumption has led to a common misconception that the duration of the storm is equal to T_C. This is theoretically possible but it is much more common for the total storm duration to be considerably longer than T_C.

   Of equal importance is the concept that *Tc* (the rainfall intensity averaging time) can occur during any segment of the total storm duration--at the beginning; before, during or after the middle portion; or near the end. This concept has important implications for the runoff coefficient *C* and how well the Rational Formula mirrors the hydrologic cycle. If T_C occurs at the beginning of the storm, then the antecedent moisture conditions become important. If T_C occurs near the end of a long storm, then the ground may be saturated and depression storage already filled when T_C begins.

5. The relationship between rainfall and runoff is linear. If rainfall is doubled then runoff is doubled. This is not accurate because of all the variables which interact and determine runoff. In fact, one of the major misconceptions in the use of the formula is that each of the variables (*C, i, A*) is independent and estimated separately. In reality, there is some interdependency among variables; however, the aids used in estimating the variables do not recognize such a relationship.

6. The runoff coefficient, *C*, is constant for storms of any duration or frequency on the watershed. This is a major misconception of many who use the Rational Formula. *C* is a variable and during the design of a stormwater system, especially a storm sewer, it should take on several different values for the various segments even though the land use remains the same.

Limitations of the Formula

The major limitation is that the Rational Formula only produces one point on the runoff hydrograph--the peak discharge rate. When basins become complex, and where sub-basins combine, the Rational Formula will tend to over estimate the actual flow. The over estimation will result in the oversizing of stormwater management systems.

When the formula is used for larger developments as a basis for establishing predevelopment flow rates which are to define the restrictions needed for peak rate control, higher flow rates are likely to be obtained than actually occur. The implication of this is that greater flow rates will then be allowed after development, resulting in less on-site flow reduction being required and higher post-development flow rates. This condition can adversely affect downstream property owners.

The average rainfall intensities used in the method bear no time sequence relation to the actual rainfall pattern during a storm. The intensity-duration-frequency curves prepared by the Weather Bureau are not true sequence curves of precipitation but are developed from data on peak rainfall intensities of various duration. For example, an intensity of one inch per hour may occur for various durations at various frequencies (e.g. 25 minute duration for a 5 year return period; 45 minute duration for a 25 year return period). In neither case does this analysis deal with any part of the total storm other than the peak, nor does the formula differentiate between an intense summer thunderstorm or a winter frontal storm. This weakness becomes especially glaring in the design of stormwater systems since the design storms specified by local governments are usually large, long duration events.

The method assumes that the rainfall intensity is uniform over the entire watershed. This assumption is true only for small watersheds and time periods, thus limiting the use of the formula to small watersheds. Whether "small" means 20 acres or 200 acres is still being debated.

Finally, one of the most important limitations is that the results are usually not replicable from user to user. There are considerable variations in interpretation and methodology in the use of the formula. The simplistic approach of the formula permits, and in fact, requires, a wide latitude of subjective judgement in its application. Each firm or agency has its favorite T_C formula, its favorite table for determining C, and its own method for determining which recurrence interval is to be used in certain situations.

3.6 OTHER METHODS

The intent of the Stormwater Rule is to obtain 80 to 90% pollutant removal from stormwater discharges. The pollutant removal efficiency of any system is difficult to accurately predict since the treatment ability of any BMP is highly variable among storm events, depending upon many factors which fluctuate independently with time and location. However, the average annual removal of various pollutants can be estimated based on two of the more often studied properties of storm events and runoff waters. These properties are the frequency distribution of rainstorm volumes and the first flush of pollutants.

The requirements in the Stormwater Rule are based on a statistical analysis of Florida rainfall data and field investigations of the first flush undertaken in Florida. Nearly 90% of all storm events that occur in any region of Florida during a given year will provide one inch of rainfall or less (Table 3-1). These storms account for over 75% of the total annual volume of rain. If the concentrations of pollutants are relatively constant with time during the course of a storm, as has been observed in large watersheds, then the percentage mass of pollutants removed would be proportional to the fraction of yearly runoff waters which are treated. However, first flush effects, in which the amount of pollutants are greater during the early part of a storm, occur in small urbanized drainage areas. Thus, the infiltration of the first one-half inch of runoff from watersheds less than 100 acres in size will result in the capture of at least 80% of the annual average stormwater pollutant load.

It is important that the first flush treatment volume be estimated accurately and conservatively in order to achieve water quality objectives. Designers are not limited to the Rational Formula or the General Procedure for capturing the first 1/2" or 1" of runoff. These are relatively simplistic methods in comparison to other methods available, which we won't discuss in detail due to their increased compexity. Table 3-6 presents a summary of the treatment volumes estimated by various hydrologic methods. As seen, the Rational Formula, despite its deficiencies and over simplicity which may render it undesirable for stormwater quantity calculations, will work to estimate the first flush treatment volume.

The SCS equations will tend to under predict runoff volume from most small storms when compared to the rational method due to the basic presumptions in the "curve number" procedure. The SCS method assumes that runoff will occur only when the storage (S) capacity of the watershed is exceeded. It is postulated that the storage is directly related to the curve number (CN), which is determined by soil type and ground cover. As a result, for low volumes of rainfall (e.g. 6 inches or less, typical of storms with a return period of five years or less) the use of the SCS curve number procedures will underestimate runoff for most urbanizing situations. This will also reduce the amount of the first flush pollutants that are captured and reduce the overall treatment efficiency of a stormwater system. The Wanielista design equations were developed as a result of stormwater research conducted by Dr. Martin P. Wanielista and his colleagues at the University of Central Florida. These equations can be used to determine the stormwater treatment volume for off line retention systems. A complete discussion of these equations can be found in RETENTION BASINS (SW BMP 3.07).

Table 3-6
Cumulative Probability Values (%) for 15 Florida Locations

Location	Volume (in) / Probability (%)				
	0 - 1/2	1/2 - 1	1 - 2	2 - 3	3 - 4
Niceville	68.2	84.5	93.8	97.7	98.4
Tallahassee	70.3	83.7	94.2	98.0	99.6
Jacksonville	77.1	91.7	97.7	99.1	99.6
Appalachicola	75.3	87.9	97.4	99.3	99.7
Gainesville	76.9	90.0	97.0	98.9	99.8
Daytona	75.9	89.3	96.2	98.7	99.8
Inglis	71.1	85.1	96.6	99.2	99.8
Orlando	80.1	90.0	98.0	99.6	99.9
Tampa	76.4	89.7	97.9	99.5	99.9
Vero Beach	77.5	89.9	98.7	99.3	99.5
Clewiston	74.3	87.3	97.0	98.9	99.6
West Palm Beach	80.6	90.8	97.0	98.7	99.1
Fort Myers	70.5	86.4	95.6	98.4	99.6
Miami	82.7	93.3	98.5	99.4	99.6
Key West	84.9	94.0	98.4	99.3	99.6
Florida	76.4	89.0	97.0	99.0	99.6

Table 3-7
Comparison of Different Methods for Calculating Runoff Volume (Ac-Ft) to Satisfy Section 17-25.035(2)(b) Criteria

		% Impervious				
Method	**Acres**	**10**	**40**	**60**	**80**	**90**
Santa Barbra Urban Hydrograph	10	0.04	0.16	0.35	0.48	0.62
	50	0.21	0.79	1.75	2.38	3.08
	100	0.42	1.58	3.50	4.75	6.17
	200	0.83	3.17	7.00	9.50	12.33
SCS Weighted Q	10	0.07	0.27	0.39	0.53	0.59
	50	0.33	1.33	1.96	2.63	2.96
	100	0.67	2.67	3.92	5.25	5.92
	200	1.33	5.33	7.83	10.50	11.83
Rational	10	0.16	0.38	0.53	0.67	0.74
	50	0.79	1.88	2.63	3.33	3.71
	100	1.58	3.75	5.25	6.67	7.42
	200	3.17	7.50	10.50	13.33	14.83
1/2" Volume	10	0.42	0.42	0.42	0.42	0.42
	50	2.08	2.08	2.08	2.08	2.08
	100	4.17	4.17	4.17	4.17	4.17
	200	8.33	8.33	8.33	8.33	8.33
Wanielista Equation (80%) Efficiency	10	0.23	0.25	0.26	0.28	0.28
	50	1.81	1.97	2.07	2.17	2.23
	100	4.40	4.78	5.02	5.27	5.40
	200	10.67	11.61	12.19	12.81	13.12

REFERENCES

Florida Department of Environmental Regulation, 1988, The Florida Development Manual A Guide to Sound Land and Water Management (Chapter 5). Tallahassee, Florida

Wanielista, Martin P., 1978. Stormwater Management: Quantity and Quality, Ann Arbor Science, Ann Arbor, Michigan.