Chapter 4

Reconciliation of Hydrologic Models to Coastal Flatland Watersheds

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Runoff hydrographs from coastal flatwood watersheds in Southwest Florida have been found to exhibit prolonged recession limbs. Commonly-used hydrologic models with default parameters appear to have difficulty simulating these runoff hydrograph shapes. While adjustments to timing parameters seem to be effective in reducing and/or shifting the time of the peak runoff rate, they may not accurately account for the elongated shape of the runoff hydrograph. Stream flow and rainfall data collected by the U S Geological Survey and the Palmer Ranch Developer were used in an attempt to reconcile the SCS Unit Hydrograph model and Runoff Block of EPA's Stormwater Management Model (SWMM).

4.1 Introduction

Synthetic unit hydrograph methods are popular and play an important role in urban stormwater drainage design. These methods are simple, requiring only an easy determination of watershed and land use characteristics. Therefore, these methods serve as useful tools to simulate runoff for ungaged watersheds, design, rainfall events and watersheds undergoing land use change.

Several of the most popular computer simulation models such as HEC-1 (USACE, 1985), SWMM (Huber and Dickinson, 1988), and TR-20 (SCS, 1973)

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are available for developing peak discharge rate, volume or a runoff hydrograph. Because the parameters used in the equations are empirical, the model is limited to the same physiographic, climatic, and land use conditions. Therefore, the model should be evaluated against local data.

Descriptions of runoff models have been published in the ASCE Journal of the Hydraulic Division/Engineering. These models represent a wide variety of simulation techniques in the mathematical description of the urban runoff processes, ranging from relatively simple to relatively complex.

The scarcity of quantitative comparisons of urban runoff models for flat terrain such as Southwest Florida prompted the initiation of the study.

In this chapter an attempt is made to compare two hydrologic models with regard to their ability to estimate runoff hydrograph peak rates and time as well as shape.

4.2 Basis for Study

4.2.1 Selection of Models

For the purpose of this study, two models were selected which are well documented, easily accessible, and extensively tested on different watersheds. These models also represent a range of techniques and complexity in the mathematical description of the runoff process. These are:

1. SCS hydrograph, and
2. EPA Stormwater Management Model.

These runoff simulation models, though obviously not complete, are fairly representative of those used in Southwest Florida. These models were examined by comparing the calculated runoff hydrographs with measured watersheds hydrographs.

The following additional aspects were taken into consideration.

- The modeled basins were described to an identical or similar degree of detail in both models.
- The models were applied for a reasonably large number of storms which varied in intensity and duration and which occurred on watersheds of various physical characteristics.
- The comparison of the computed hydrographs with those measured was done for the peak discharge points as well as for the entire hydrographs.

4.2.2 SCS Hydrograph

Several techniques are available to develop a synthetic unit hydrograph. The most popular method used is the Soil Conservation Service (SCS) curvilinear unit hydrograph. This method is based on the assumption that the same unit hydrograph shape applies to all watersheds, only the scale differs. The unit hydrograph is dimensionless with axis of q/q_p and t/t_p, in which q equals the discharge rate at any time t, and q_p equal the peak discharge at time t_p; thus, the peak of the dimensionless unit hydrograph is dimensionalized by multiplying the time values by the estimated time-to-peak, and the discharge ordinates by the peak discharge which is given by:

$$q_p = \frac{DAR}{t_p} \tag{4.1}$$

where:

q_p = the peak discharge in cfs;
A = the drainage area in square miles;
R = runoff depth in inches;
t_p = the time to peak; and
D = peak rate factor, which has units of hours-cubic feet per second per square mile per inch.

The peak rate factor $D = 484$ results from the conversion of square miles to acres and the assumption that 37.5% of the runoff volume occurs under the rising limb of the hydrograph. The SCS indicates that D can vary from 300-600 with a value of 300 in very flat, swampy country and a value of 600 in steep terrain. However, no specific value of D for this watershed has been proposed. A study by Woodward et al. (1980) suggested a value of 284 for the Delmarva Peninsula. The University of Florida (1986) has found that a peak rate factor of 75-100 is appropriate for flatwoods watersheds.

Because the peak discharge computed with Equation 4.1 is linearly related to D, Southwest Florida has widely accepted D values of from 256 to 323. This created some questions on the applicability of the above values for low flat sloped areas in the Southwest Florida watersheds. This concern emphasizes the need for an accurate, systematic method for determining the peak rate factor and dimensionless unit hydrograph for ungaged watersheds, where the above peak rate factors in the SCS dimensionless unit hydrograph are considered to be inappropriate.

4.2.3 Procedure for Estimating Peak Rate Factor

Recent research has demonstrated that the hydrologic and geomorphologic approaches to defining watershed hydrologic response functions are convergent and may be expressed through the gamma function (Rosso 1984). The parallel between the geomorphologic and hydrologic approaches has led to the development of a geomorphologic based method for estimating unit hydrograph shapes and scale parameters and is represented by the Nash model (1959).

$$Q(t) = \frac{1}{\Gamma(n)K^n} \cdot t^{n-1} \cdot e^{-t/k} \qquad (4.2)$$

where $Q(t)$ is the unit hydrograph ordinate at time t; K and n are the scale and shape parameters, respectively; and $\Gamma(n)$ is the gamma function.

Wu (1963) related K to n and t_p by equating the first derivative of Equation 4.2 to zero:

$$K = \frac{t_p}{n-1} \qquad (4.3)$$

After substituting Equation 4.3 into Equation 4.2, to remove K, Wu (1963) suggested the following equation:

$$Q = Qp[\frac{t}{t_p} . e^{(1-t/tp)}]^{n-1} \qquad (4.4)$$

where:

Q = the unit hydrograph ordinate in cfs,
Q_p and t_p = the peak discharge rate in cfs, and time to peak in hours, respectively, and
n = a shape parameter. This is the form of the two-parameter gamma function used in this chapter.

Meadows and Blandford (1983) developed a relationship between Q_p and n using a peak rate factor (*PRF*) concept. They developed the following equation by substituting Equations 4.3 into Equation 4.2:

$$Q_p = \frac{(n-1)^n}{(n-1)!e^{n-1}} \cdot \frac{1}{t_p} \qquad (4.5)$$

$$Q_p = \frac{B}{t_p} \qquad (4.6)$$

where B is a peak rate factor and is a function of the shape parameter n.

For Qp in cfs and t_p in hours, Equation 4.6 can be expressed as:

$$Q_p = \frac{(645.33B)A}{t_p} \qquad (4.7)$$

or

$$Q_p = (PRF)\cdot\frac{A}{t_p} \qquad (4.8)$$

where A is the watershed area in square miles and *PRF* is the unit hydrograph peak rate factor.

From Equations 4.5 to 4.8, it appears that the *PRF* is a function of n. Table 4.1 shows the gamma function shape parameter relationships, while Figure 4.1 shows unit hydrograph shape with different *PRF* values. Figure 4.1 was constructed by plotting the product of the hydrograph ordinates and tp against dimensionless time. Neidrauer (undated) has also developed a general equation for curvilinear dimensionless unit hydrograph shape with a different *PRF*.

Table 4.1 Gamma shape parameter function.

Shape Parameter (n)	Peak Factor (PRF)	Rate Factor (B)
1.50	156	0.2420
2.00	237	0.3679
2.50	298	0.4625
3.00	349	0.5413
3.50	393	0.6102
4.00	433	0.6721
4.50	470	0.7288
5.00	504	0.7815

(After Meadows and Ramsey 1991)

4.2.4 Analysis

By combining Equations 4.4 and 4.8 and the optimizing n value (when the estimated hydrograph ordinates were best fitted with the observed hydrograph ordinates by using a method of successive approximation (trial and error method)), *PRF*s and t_p were computed for various storm events. The computed grouped data of *PRF*s were used to perform a non-linear regression analysis to establish a relationship between the *PRF* and the watershed parameters. Two main parameters, basin area and imperviousness, were considered in the analysis. Since the effect of slope on the hydrograph shape is accounted for in calculating

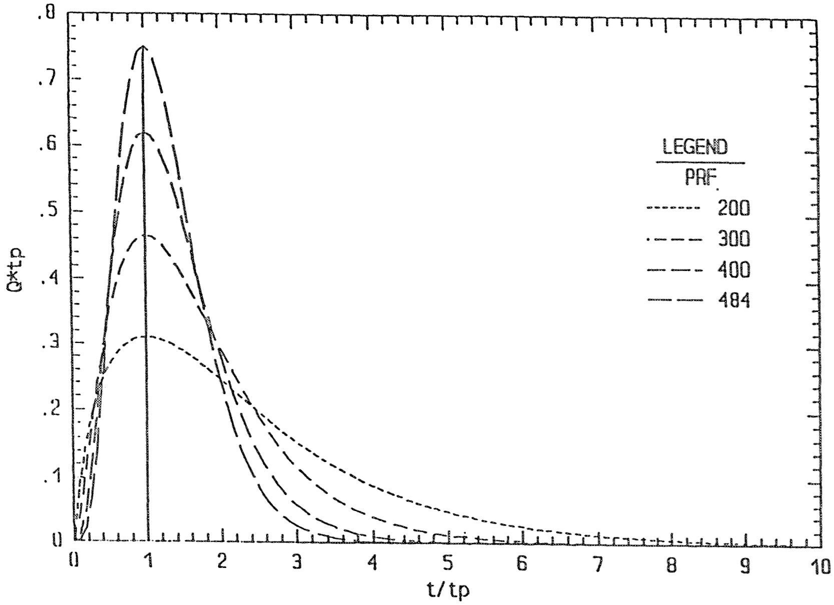


Figure 4.1 Normalized gamma function unit hydrograph (after Meadows and Ramsey, 1991).

time of concentration, t_c, the effect of slope was not considered in the regression analysis. It was assumed that (1) a minimum of 5% impervious area was considered in the analysis, and (2) the storm event which produced a runoff volume between 0.94 inches (24 mm) and 1.07 inches (27 mm) was adopted as a unit hydrograph for the watershed under consideration. Based on the above technique, the following equation was obtained for Southwest Florida:

$$PRF = 60.\frac{Imp^{0.28}}{A^{0.15}} \tag{4.9}$$

where:

A = the watershed area is square miles, and
Imp = the imperviousness of the watershed.

Meadows and Ramsey (1991) developed a relationship similar to Equation 4.9.

To facilitate the designer, Table 4.2 summarizes the recommended design values of the *PRF* for the various imperviousnesses. Equation 4.9 predicts a more accurate value of the *PRF* for given watershed characteristics.

The application of Equation 4.9 is limited to watersheds with physical measures contained within the statistical data base summarized in Table 4.3.

Table 4.2 Recommended PRF.

Imperviousness (%)	PRF
Undeveloped	
for Slope 0.0% to 0.5%	75
for Slope 0.5% to 1.0%	100
10	115
20	130
30	140
40	150
50	170
60	190
70	205
80	225

4.2.5 EPA Stormwater Management Model (SWMM)

SWMM has been developed under the sponsorship of the Environmental Protection Agency (EPA). The computer program for this model was obtained from the U.S. EPA.

SWMM is one of the most comprehensive computer models and is capable of simulating runoff quantity and quality, as well as dry weather flow, treatment facilities and receiving water quality. In this study only the quantity sections were considered. The Runoff Block was used to compute runoff from each subbasin area and the Extran Block was used to route the runoff through the storm conveyance system.

4.3 Verification

The proposed procedure (Equation 4.9) for estimating a value for the peak rate factor for an ungaged watershed was tested using measured rainfall and runoff data. The data was provided by the U.S. Geological Survey and the Palmer Ranch Developer in Sarasota County. It included rainfall and stream flow data for between two and seventeen events at fifteen urban watersheds in Southwest Florida. The watersheds range in size from 0.14 square miles (0.36 km^2) to 15.22 square miles (40 km^2), imperviousness from 0-85%, and slope from 0.03 - 0.89%.

Table 4.3 shows the watershed characteristics. Table 4.4 and Figure 4.2 show the comparison between observed and estimated peak discharge rates. The results shown in Table 4.4 and Figure 4.2 indicate that the mean *PRF* computed for watersheds with physical measures that fall within the watersheds contained in the statistical data showed good agreement with the values of *PRF* obtained

Table 4.3. Watershed characteristics.

Map No.	Identification No.	Watershed Name	DA	SL	IA	Wet	Res	Com	Ag	Pas	For	Open
						Land use, in percentage of total area						
1	02306002	Artic Street storm drain	0.34	12.3	40.0	0	50.0	50.0	0	0	0	0
2	02306006	Kirby Street drainage ditch	1.15	8.1	5.5	3.5	72.0	11.0	0	0	0	13.5
3	02306021	St.Louis St. drainage ditch	0.51	10.2	9.0	0	68.0	16.0	0	0	0	16.0
4	02306071	Gandy Blvd drainage ditch	1.29	4.6	20.0	0.9	37.0	29.0	0	0	0	33.1
5	02307731	Allen Creek	1.79	23.4	20.0	0.9	63.0	20.0	0	0	0	16.1
6	274215082072000	IMC Creek	0.17	47.0	0	0	0	0	0	67.0	33.0	0
7	274141082051300	Grace Creek	0.66	26.0	0	0	0	0	33.0	33.0	34.0	0
8	273806081535000	CFI-3 Creek	0.14	36.0	0	0	0	0	67.0	0	33.0	0
9	02299861	Walker Creek	4.78	6.3	40.0	1.0	52.0	16.0	0	0	16.0	16.0
10	02299742	Clower Creek	0.35	3.7	85.0	0.1	15.0	85.0	0	0	0	0
11	02299741	Catfish Creek	4.77	3.5	10.0	0.5	25.0	10.0	0	10.0	30.0	25.0
12	02299737	South Creek	15.22	2.9	0	31.0	10.0	0	0	35.0	24.0	0
13	02299684	Forked Creek	2.72	2.8	0	15.0	0	0	30.0	55.0	0	0
14	02299681	Gottfried Creek	2.00	1.4	10.0	15.0	0	0	30.0	55.0	0	0
15	02299680	Rock Creek	2.63	2.9	0	25.0	0	0	0	50.0	25.0	0

[DA: drainage area (mi^2); SL: slope (ft/mi); IA: impervious area; Wet: wetlands; Res: residential; Com: commercial; Ag: agricultural; Pas: pasture or rangeland; For: forest or woodland; Open: open space]

Table 4.4 Comparison of peak discharge rates.

Watershed	Peak Discharge Rate (cfs) Date of Storm	Observed	Estimated
Artic Street Storm Drain	08/03/76	120	86
	08/04/76	133	110
	09/26/77	137	66
	05/20/78	142	147
Kirby Street Drainage Ditch	07/19/75	57	41
	08/30/75	95	105
	08/15/78	96	106
St. Louis Street Drainage Ditch	05/15/76	357	474
	06/18/76	226	210
	06/29/77	326	237
Gandy Boulevard Drainage Ditch	06/18/75	223	152
	07/11/75	301	362
	08/07/75	207	216
	05/15/76	692	748
	05/17/76	410	265
Allan Creek	07/28/76	341	359
	07/01/77E	379	312
	07/01/77L	819	853
	07/03/77	335	265
	12/02/77	89	97
	02/18/78	286	369
IMC Creek	11/23/88	11	11
	07/12/89	5	3
	02/23/90	4	6
	07/21/90	9	8
Grace Creek	08/07/88	59	60
	08/23/88	40	43
	07/12/90	16	13
	07/14/90	25	32
CFI-3 Creek	07/05/89	19	16
	02/23/90	7	11
	06/02/90	6	4
Walker Creek	06/92	971	-
	07/23/92	438	360
	08/07/92	398	384
	09/04/92	334	308
	09/05/92	278	164
	09/25/92	199	224
	09/26/92	292	319
	01/15/93	235	288
	04/01/93	319	376
	07/01/93	237	184

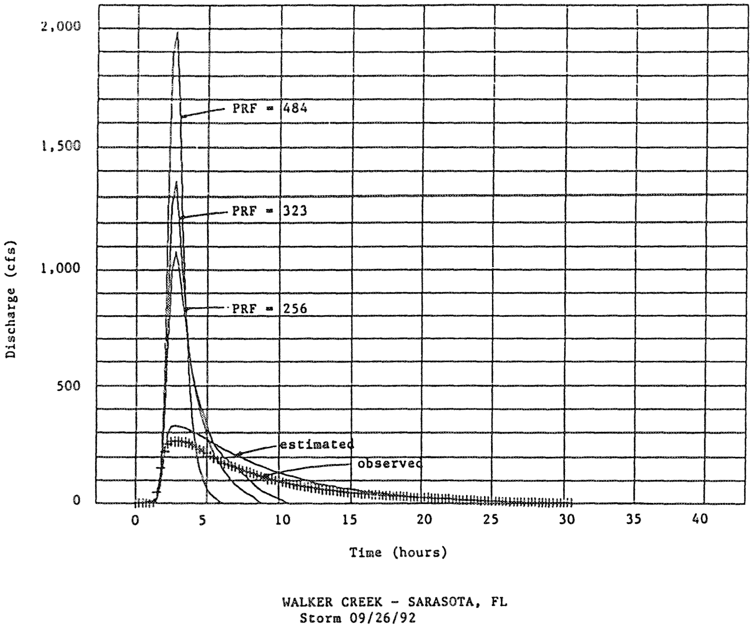
Table 4.4 continued Comparison of peak discharge rates.

Watershed	Peak Discharge Rate (cfs) Date of Storm	Observed	Estimated
Clower Creek	02/05/92	77	73
	06/92	205	-
	09/02/92	66	54
	09/13/92	110	116
	01/14/93	42	34
	03/13/93	60	69
	04/01/93	116	145
Catfish Creek	01/14/93	70	51
	01/15/93	76	79
	03/13/93	140	117
	04/01/93	300	309
South Creek	06/92	442	654
	09/06/92	143	-
	09/13/92	96	60
	03/13/93	94	105
	04/01/93	168	193
Forked Creek	06/92	287	357
	08/09/92	45	34
Gottfried Creek	06/92	119	202
	08/11/92	21	10
	10/92	18	15
Rock Creek	06/92	109	182
	09/25/92	24	28
	10/92	25	25

from the analysis. Table 4.4 shows the comparison between observed and estimated peak discharge rates. Table 4.5 compares observed peak discharge rate for various storm events with those estimated by SWMM.

4.4 Discussion

The SCS methods are some of the most widely used hydrologic design methods. The standard SCS peak rate factor of 484; the peak rate factor of 284, resulted from the Delmarva peninsula study; the peak rate factors of 256 and 323 commonly used in the Southwest Florida may not be applicable for the flat terrains. Specific values can be derived for the watersheds within Southwest



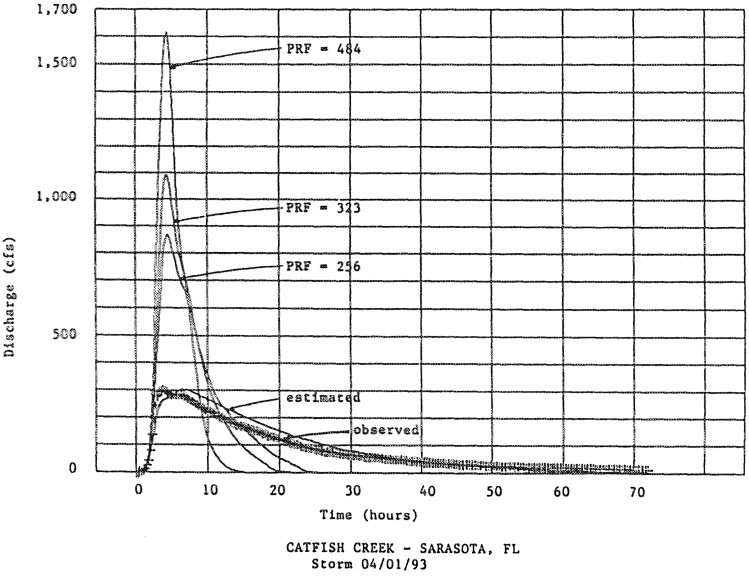
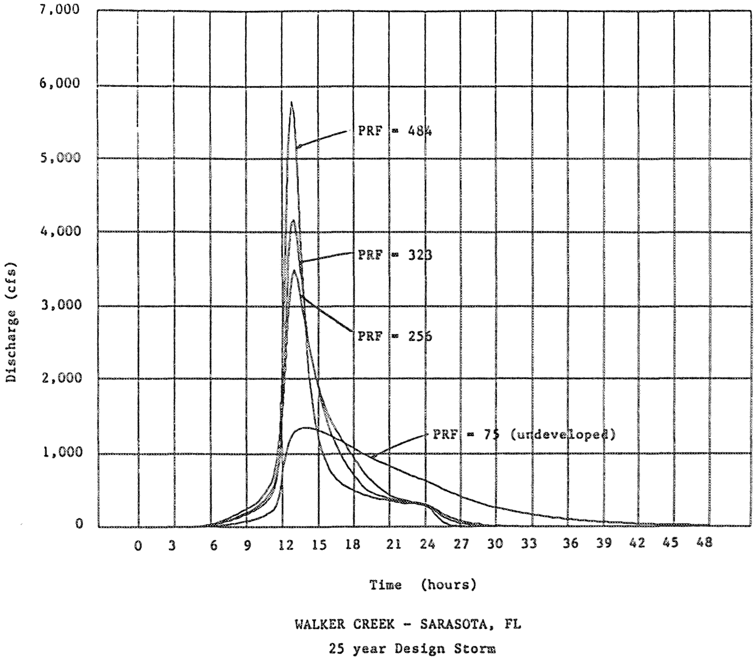


Figure 4.2 Comparison of observed and estimated runoff hydrographs.



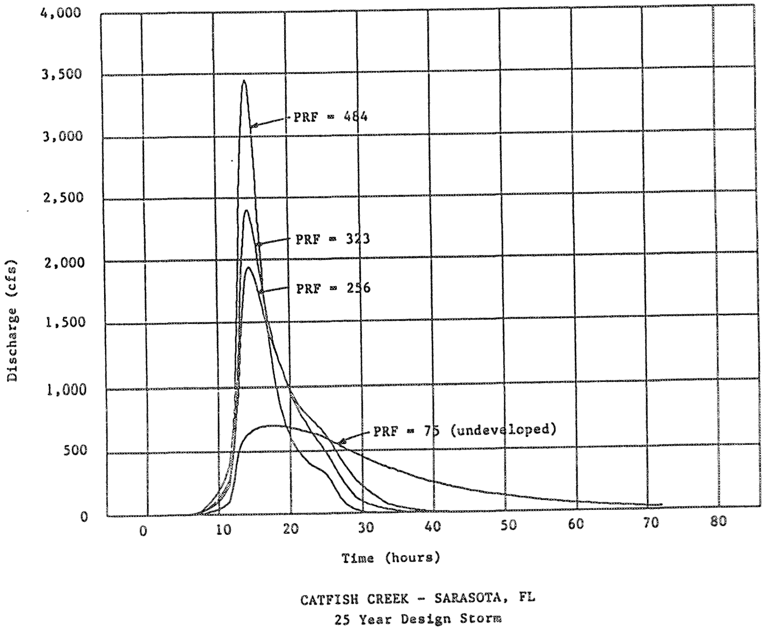


Figure 4.3 Runoff hydrographs (25 year - 24 hour storm).

Table 4.5 Comparison of peak discharge rates (EPA).

Watershed	Peak Discharge Rate (cfs) Date of Storm	Observed	Estimated
Walker Creek	07/23/92	438	468
	08/07/92	398	322
	09/04/92	334	430
	09/05/92	278	383
	09/26/92	292	241
	04/01/93	319	355
Clower Creek	09/02/92	66	54
	09/13/92	110	102
Catfish Creek	01/14/93	70	72
	03/13/93	140	169
	04/01/93	300	339
Gottfried Creek	06/92	119	-
	08/11/92	21	31

Florida using the procedure outlined in this chapter. For watersheds with significant amounts of storage, the *PRF* should be even less than the value computed from Equation 4.9. Figure 4.3 compares runoff hydrographs using the peak rate factors of 484, 323, 256 and 75 for a 25 year 24 hours storm event.

The procedure outlined in this chapter for deriving the dimensionless unit hydrograph from topographic and land use data is a very reasonable method for estimating the SCS peak rate factor when the measured rainfall and runoff data are not available. The designer should make the following additional assumptions, if the SCS method is used for design:

1. the gamma distribution or Neidrauer method can be used to represent the shape and proportion of the design unit hydrograph;
2. the shape and scale parameters of the design unit hydrograph are related to the time-to-peak;
3. the shape parameter can be determined from the proportion of the area under rising limb; and
4. the SCS relationship between time-to-peak and time of concentration is valid (i.e. $t_c = 1.5\ t_p$).

With respect to the SWMM analyses, comparison of the observed hydrograph peaks with those estimated by SWMM, reveals that eight storm events are overestimated and four storm events are underestimated. From Table 4.5, it can be seen that the positive and negative error in the peak estimates by SWMM are +37.8% and -20%.

4.5 Conclusions

1. The two-parameter gamma distribution can be used to estimate the peak rate factor (*PRF*) and time-to-peak (t_p) parameters.
2. The relationship derived in this study is intended for screening level analysis and is not intended to replace more complex analysis. Equation 4.9 estimates runoff hydrographs more accurately than the peak rate factors of 256, 323 and 484 for peak flow rates when compared with observed data (Figure 4.3).
3. The *PRF* is a function of drainage area and imperviousness.
4. This study is based on the assumption that a single storm produces both peak flow rate and hydrograph volume of equivalent frequency.
5. The proposed method did not perform well in simulating multi peak hydrographs.
6. Further investigations may be warranted to: (1) develop direct runoff hydrographs (alternative methods for the subtraction of surficial groundwater from the runoff hydrograph); (2) fine tune a best fit peak rate factor; (3) verify findings on other watersheds in Southwest Florida where continuous rainfall and discharge information is available; and (4) develop improved techniques for determining basin times of concentration to assist in the generation of design hydrographs for ungaged watersheds.
7. Overall, SWMM yielded fairly good agreement between the observed and estimated runoff events on urban as well as rural watersheds.
8. Antecedent moisture conditions assumptions as reflected in the Horton infiltration parameters, play a major role in determining the peak flow.

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