A 24-HOUR RAINFALL DISTRIBUTION AND PEAK RATE FACTORS FOR USE IN SOUTHWEST FLORIDA

BY

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THESIS

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ABSTRACT

The objectives of this research were to derive a design 24-hour duration rainfall distribution for use in southwest Florida, and peak rate factors for use in the Soil Conservation Service (SCS) unit hydrograph method for two watersheds, also in the southwest Florida area.

The rainfall distribution is derived by applying a least squares polynomial curve fitting technique to National Weather Service hourly rainfall data collected in the study area. The screening criteria for data included in the curve fitting procedure are: storm duration of 18 to 26 hours, at least three inches of rainfall volume, and peak intensity period falling near the center of the storm. The analysis technique includes converting the raw data to dimensionless form which allows the flexibility of applying the 24-hour distribution with any volume of rainfall, and so simulating any return frequency of 24-hour storm.

Peak rate or attenuation factors are determined for the Hickory Creek (2400 acres) and the Gallagher Ditch (300 acres) watersheds. Stream gage data collected over a three-year term by the U.S. Geological Survey is used for the analysis. The screening criteria for the hydrographs produced from this data

include a stable baseflow condition and a single hydrograph peak. The resulting hydrographs provide input to the Soil Conservation Service triangular unit hydrograph. An average of the peak rate factors calculated from the screened hydrographs is taken as the suitable factor for each watershed.

The project results are then compared to the currently used rainfall distributions and the SCS peak rate default value of 484. The comparisons are accomplished by modeling a hypothetical watershed using both the SCS unit hydrograph and the Santa Barbara methods. The model is run on a microcomputer using seven distributions, three return frequency volumes of rainfall, differing watershed sizes, times of concentrations and antecedent moisture conditions.

The conclusion recommends a rainfall distribution and peak rate factor best suited to estimate hydrographs.

DEDICATION

To my wife, Jean, who's patience and sacrifice will always be remembered.

ACKNOWLEDGEMENTS

This study was suggested as a thesis topic by Dr. Martin P. Wanielista eighteen months ago. His constant support and genuine interest in each phase led to its completion. Thank you. Greiner Engineering Sciences, Inc. also deserves a big thank you for the support they have shown throughout the project. Also, I wish to thank Dr. Paul Hartman and Dr. Shiou-San Kuo for their comments and review of the final product.

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CHAPTER I

INTRODUCTION

Objective Statement

This work was undertaken to produce a peak rate factor and rainfall distribution combination that will more accurately simulate hydrographs for the southwest Florida area. It will provide engineers and hydrologists with more regionally specific stormwater modeling tools.

Qualifiers

This work is most applicable to watersheds having a long time of concentration (a few hours to near 24 hours). Short duration, high intensity rainfalls were not used to construct the time distribution. For watersheds with shorter times of concentration, the recommended 24-hour rainfall distribution will underestimate the hydrograph peak flow rate.

Using a 24-hour distribution for all projects is suitable for determining volumes of runoff, i.e., comparing post-development to pre-development runoff volumes. However, for most "interior" design projects, peak runoff rates must be determined to size the open channels and pipes. Projects located in watersheds with a time of concentration under 24 hours should use a shorter duration rainfall distribution because for a given return frequency as storm duration shortens, the peak rainfall intensity increases. To accurately size a pipe, for the 25-year storm, for example, one must use the combination of rainfall intensity and duration which results in the largest flow rate for a 25-year storm. That flow rate will be produced by using a storm duration close to the time of concentration of the watershed being modeled. Rainfall distributions with durations less than the watershed time of concentration will contain higher peak intensities, but not all of the watershed will contribute runoff to the peak. This situation will not accurately estimate the peak runoff rate of a watershed. On the other hand, durations longer than the time of concentration will contain peak intensities too low to properly reflect the return frequency desired.

Background

Computers generate most of the hydrographs used in stormwater management. The computer programs producing the hydrographs require as input a series of basic parameters. These input parameters can vary among programs, but usually include:

- a rainfall distribution (hyetograph)
- total volume of rainfall

- watershed area

- a translation reflecting the percentage of rainfall resulting in runoff

- a variable that sets the hydrograph shape

A rainfall distribution must be input because rainfall rarely, if ever, occurs uniformly with respect to time. The rainfall distribution can vary regionally and so the modeler must be careful to select a distribution appropriate for the region to be modeled.

Because rainfall gauge data and the variation of rainfall with time are lacking for most small watersheds, it is desirable that variations in rainfall with respect to time be standardized for a region for the design of stormwater control structures.

Until now, the standard rainfall distributions applied to watersheds in southwest Florida were derived for extremely large sections of the United States. But, because rainfall patterns vary so widely, a more regionally specific distribution is needed to produce accurate estimates of hydrographs.

The TR-20 computer program is a popular hydrograph estimator which uses the Soil Conservation Service unit hydrograph method. This program has an internal peak rate factor (K = 484) used to determine hydrograph shape (SCS 1982).

This default value of 484 does not apply to all topographies, however.

This value was derived by the Soil Conservation Service from a large number of natural unit hydrographs from watersheds varying widely in size and geographical locations (USDA-SCS 1972). But, the flat terrain of southwest Florida in many areas does not support using the 484 factor. The TR-20 program allows the hydrologist to override the default value when necessary. In this case, the hydrologist must select a peak rate factor believed to be more applicable. Although general guidelines are available, several variables enter into the decision. This study analyzes two small watersheds and derives the "K" factors best suited for them.

The study also develops a rainfall distribution and tests it against the currently used distributions. Suggestions and guidelines for using these factors are included in the conclusion.

CHAPTER II

LITERATURE REVIEW

Factors Influencing Hydrograph Shape

In modeling a watershed to estimate hydrographs resulting from large volume storms, the first step is to determine what elements of the system must be included in the model. A review of the literature indicates that the following are most likely very important factors:

- Time of Concentration
- Watershed Shape
- Watershed Area
- Topography
- Surface Storage
- Antecedent Moisture Condition
- Rainfall Volume
- Rainfall Distribution

Each one of these factors will be discussed in this chapter. The following chapter on hydrograph estimating methods shows how selected factors in the above list are included in some models while omitted in others.

Time of Concentration

The definition of time of concentration is the longest travel time of stormwater runoff flowing from a most distant point in a watershed to the outlet. This travel time can be calculated by breaking the flow path into overland (sheet flow) and conduit segments.

The overland flow travel time can be calculated using an equation such as the kinematic wave formula (FDOT 1987).

$$T = 0.93 \frac{L^{0.6} n^{0.6}}{i^{0.4} S^{0.3}}$$

where:

T = overland flow travel time (min)

L = flow length (feet)

S = slope of flow path (ft/ft)

i = rainfall intensity (in/hr)

n = Manning's roughness coefficient for overland flow

L and S are measured from the topographic map or survey data, n is determined from the type of flow surface.

However, instead of using an equation, often a chart is used to determine overland flow velocity (see Figure 1). As before, the slope and surface material must be known, then referring to Figure 1, a corresponding velocity is obtained. Dividing the flow length by the velocity yields overland travel time.

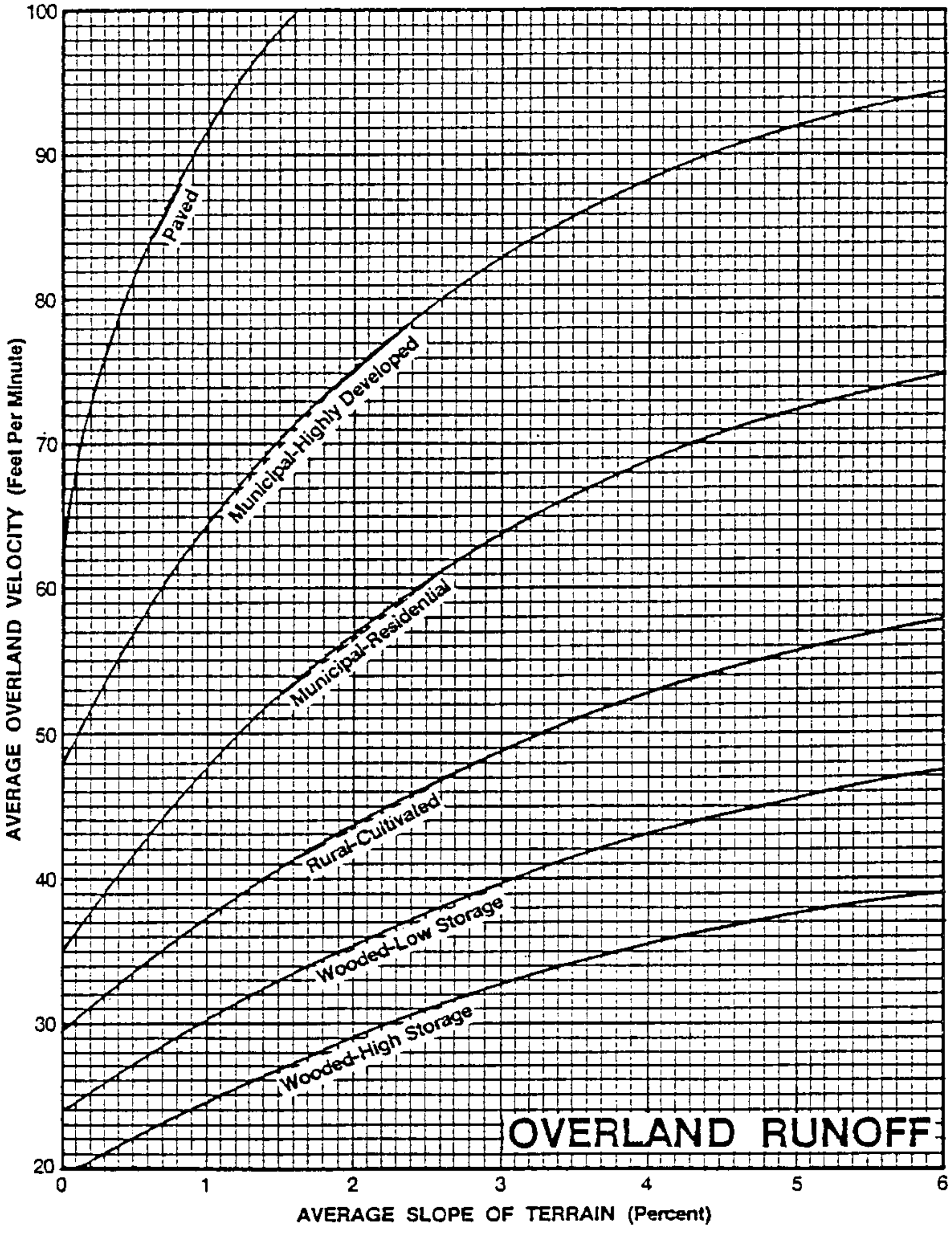


Figure 1. Average velocities for estimating travel time for overland flow (USDA-SCS 1972).

The travel time for the channelized segment of the flow path is calculated by using the Manning or similar equation. The travel times for each segment of the flow path are then added to estimate the watershed time of concentration.

The time of concentration affects the duration of the hydrograph's rising limb. At the time of concentration, all portions of a watershed are contributing runoff to the outlet, if precipitation was continuous during this time. For uniform intensity storms, this means the maximum runoff rate will occur at the time of concentration.

Watershed Shape

Watershed shape affects hydrograph shape by influencing time of concentration and by influencing when the various portions of the watershed contribute runoff.

As covered previously, before the time of concentration is reached, only a portion of the watershed is contributing runoff at the outlet. So, at any time less than the time of concentration, it may be estimated that the amount of water reaching the outlet is proportional to the area contributing runoff (Rogers 1968). The amount of flow should be modified depending on the shape of the area, however (see Figure 2), for a comparison of four watershed shapes.

The rectangle provides a uniform increase in contributing land area over time up to 100% at time of concentration. The

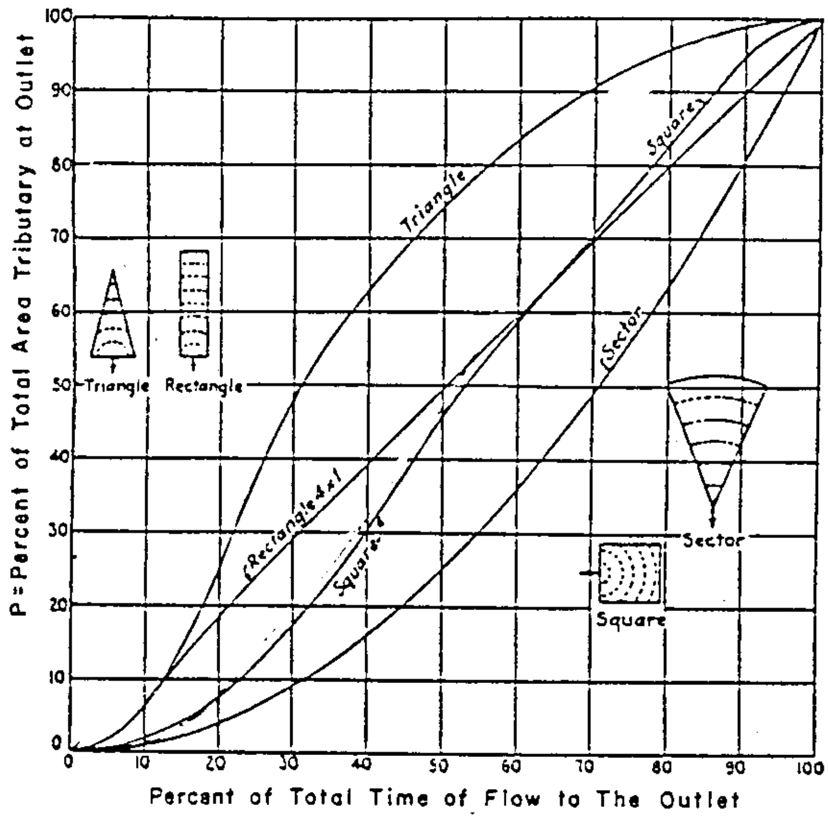
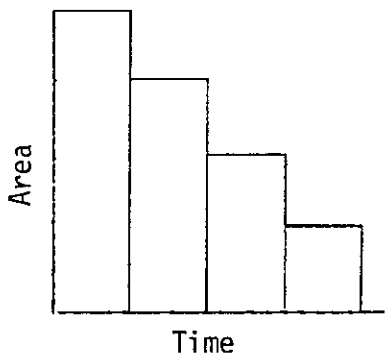
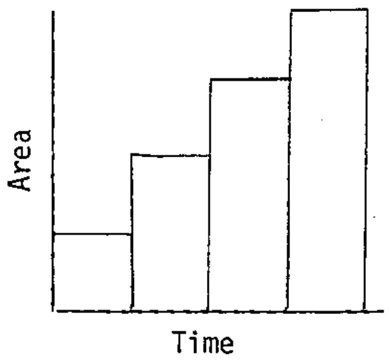


Figure 2. Time contour analysis (Rogers 1968).

other three shapes have non-uniform contributing area-time relationships. The largest difference lies between the triangle which has the majority of its area contributing runoff early compared to the sector which contributes the majority of its runoff near to the time of concentration. This area-time relationship is reflected in hydrograph shape by influencing the rising limb slope, or rate of flow increase under uniform rainfall conditions. The triangular watershed will produce the area-time curve shown below.



Whereas, the sector shaped watershed will produce this type hydrograph:



Irregular shaped watersheds of sufficient size should be subdivided into more regular shapes and calculate each sub-basin time of concentration separately. Then, develop a hydrograph for each sub-basin and add them to develop the system hydrograph at the outlet, as shown in Figure 3 (Mockus 1964).

Watershed Area

The watershed size or area directly affects the volume of runoff and so the area under the hydrograph. Larger watersheds generally have longer times of concentration than small ones. Because of the longer flow lengths, both the rising and recession limbs of the hydrograph are longer for large watersheds. One additional factor that enters into modeling of watersheds over 200 square miles is the uniformity of rainfall over the entire watershed. As watershed area increases, the total rainfall volume should be decreased for a given storm return frequency (U.S. Army Corps of Engineers 1965).

Topography

Watershed topography will affect runoff flow rate which effects the time of concentration. Also, mild slopes allow greater opportunity for infiltration, thus reducing runoff volume than do steep slopes. Topography also affects the amount of surface storage (Snyder 1938).

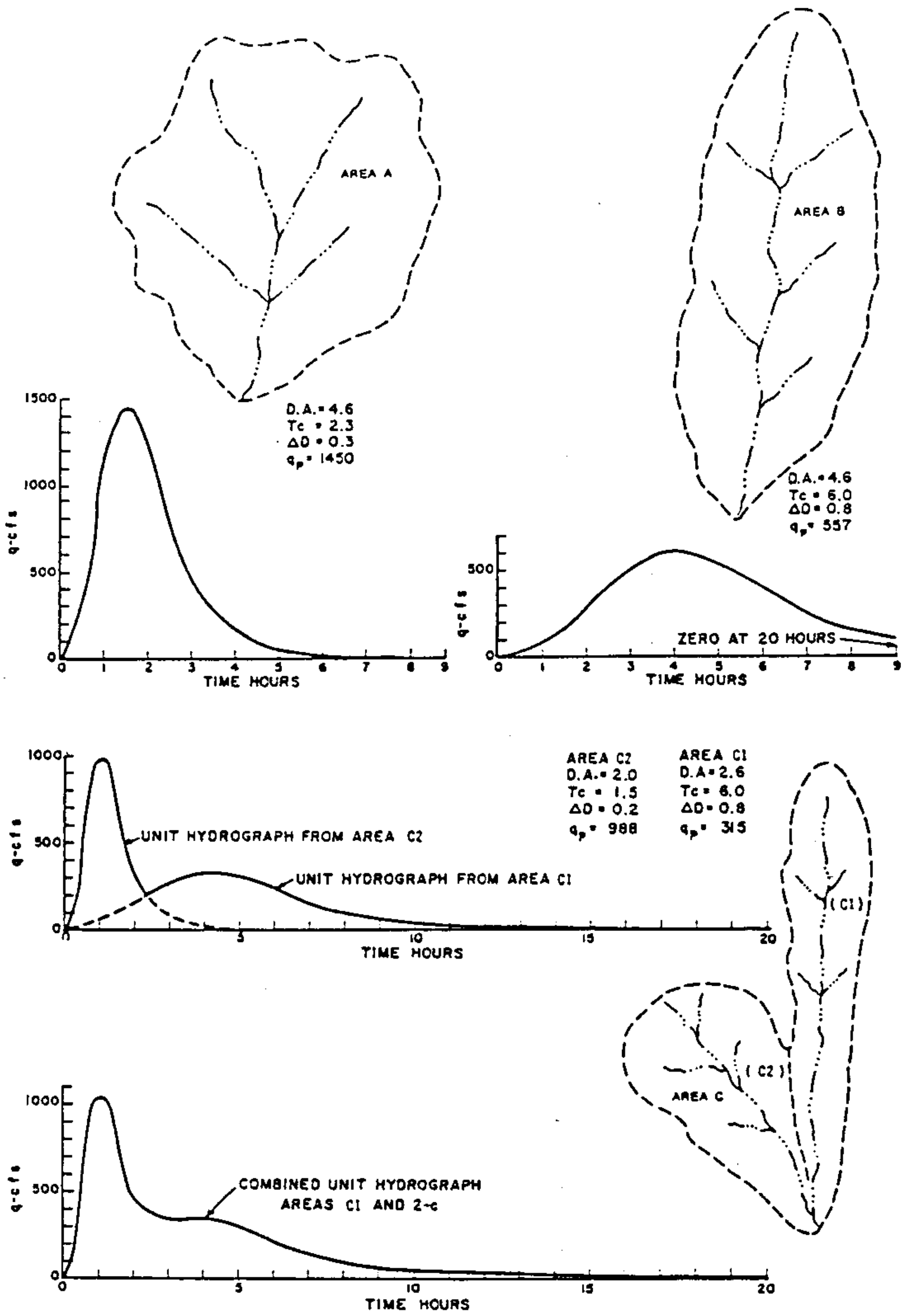


Figure 3. The effect of watershed shape on the peaks of unit hydrographs (Mockus 1964).

Surface Storage

Surface storage affects the area under the hydrograph by reducing runoff volume and by reducing the peak discharge. The stage-storage, stage-discharge relationship of the watershed storage areas reflect factors which alter the rate of increase of the runoff in complex ways an requires careful modeling to accurately estimate their effect.

Antecedent Moisture Condition

The storage potential of a soil depends on the immediate rainfall history for the area. Soils still saturated from a previous storm will allow a larger volume of rainfall to become runoff than soils which have had time to recover their storage capacity (Keifer and Chu 1957).

A wet antecedent moisture condition is reflected in the hydrograph by a steep sloping rising limb that extends to a greater peak flow rate than does the hydrograph for the same watershed under dry antecedent conditions.

Rainfall Volume

Intuitively, the more rain that falls, the more runoff can be expected. Hydrograph shape responds to the volume of rainfall by following the factors discussed above. The early portions of the rising limb will normally reflect reductions in the runoff that satisfies initial abstraction, soil and surface storages. Once these are satisfied, the remaining rainfall results completely in runoff (Keifer and Chu 1957). The greater the volume of rainfall, the higher the peak runoff flow rate. By using the kinematic wave formula, as rainfall intensity increases, travel time (time of concentration) decreases, thus the runoff peak would occur sooner as rainfall volume increased.

Rainfall Distribution

How the rainfall volume is distributed over time influences hydrograph shape and the runoff peak rate. Consider a long duration storm where most of the rain falls early in the storm. The greater infiltration capacity at the beginning and the surface storage would absorb much of the peak rainfall, resulting in a lower runoff rate. However, if the bulk of rainfall occurred later in the storm, most of the previously mentioned losses would already be satisfied before the time of peak rainfall intensity, and a higher peak rate would result (Keifer and Chu 1957).

Factors Determined by the Engineer

The engineer or hydrologist modeling a watershed can calculate the various flow lengths and slopes from a topographic map and development stormwater plans to determine an estimate for the time of concentration. The watershed shape and area can be determined once the drainage boundary is fixed. On the other hand, topography and surface storage are more difficult to estimate for a natural system in terms that can be inserted into a hydrograph estimation model. This problem is addressed for small watersheds in southwest Florida through a selection of the appropriate peak rate factor, discussed in Chapter VI.

Moving down the list of factors influencing hydrograph shape, the permitting agencies usually give the antecedent moisture condition to be used when modeling in their areas. The rainfall volume to be used will depend on the type of project. Again, the permitting agencies determine what severity of storm should be used for various project types. This is promulgated by assigning storm return frequencies to the project types. For example, Orange County requires suburban streets to be designed for the 10-year storm, while bridges must be designed for the 50-year storm (Orange County 1985). The volumes of rainfall associated with these return frequencies have been published by the National Weather Service and the Southwest Florida Water Management District in the form of intensity-duration-frequency charts, and isohytes for various durations and return frequencies (SWFWMD 1987; U.S. Weather Bureau 1961). The final factors, rainfall distribution within the storm, and hydrograph peak factor (K) are addressed in detail in the following chapters. The commonly used distributions are presented along with a brief explanation of their derivation, then a distribution developed for the southwest Florida area is presented along with the logic and methodology.

CHAPTER III

RAINFALL DISTRIBUTIONS CURRENTLY IN USE

The Keifer-Chu Method

In 1957, Clint Keifer and Henry Chu developed a method of creating rainfall distributions. The method is based on intensity-duration-frequency curves and was created to more accurately determine the peak runoff rate for urban sewer design.

This method estimates the major factors affecting peak runoff rates in an urban area to be:

1. Volume of water falling within the maximum period
2. Amount of antecedent rainfall
3. Location of the peak rainfall intensity

They reasoned that a rainfall distribution built around the average of these three factors would be adequate for use in hydrograph estimation methods. A function is derived with time as the dependent variable and rainfall intensity as the independent variable. The derivation of this function follows.

The volume of water falling within the maximum period can be taken from the intensity duration curve of a given frequency. The equation for this curve may take the form:

$$i_{av} = \frac{a}{t_d^b + c} \tag{1}$$

where:

i_{av} = average intensity (in/hour)

t_d = duration of maximum period (min)

a, b, c = constants

The rainfall volume in inches is:

$$P = i_{av} \frac{t_d}{60} \tag{2}$$

Substituting

$$P = \frac{a}{t_d^b + c} \frac{t_d}{60} \tag{3}$$

where P is the volume of rainfall. The area under a hyetograph curve can be expressed by

$$P = \frac{1}{60} \int_0^{t_d} i_d \, t_d \tag{4}$$

where i is the hyetograph ordinate in inches per hour. Differentiating equation (4):

$$\frac{dP}{dt_d} = \frac{i}{60} \tag{5}$$

But from equations (1) and (2):

$$P = \frac{a}{60} \frac{t_d}{t_d^b + c} \tag{6}$$

Differentiating equation (6):

$$\frac{dP}{dt_d} = \frac{a}{60} \frac{[(1 - b)t_d^b + c]}{(t_d^b + c)^2} \tag{7}$$

Combining equations (5) and (7)

$$i = \frac{a[(1 - b)t_d^b + c]}{(t_d^b + c)^2} \tag{8}$$

This equation represents a completely advanced type storm, one whose peak rainfall period falls at the beginning. The following modifications make equation (8) applicable to mid-peaking distributions.

Within the maximum period of any rainfall, the duration t_d can be split up into the part occurring before the most intense moment and the part after the most intense moment. Let (r) represent the portion of any duration occurring before the most intense moment, expressed as a ratio of the entire duration:

$$t_b = r\ t_d \tag{9a}$$

$$t_a = (1 - r)\ t_d \tag{9b}$$

where:

t_b = the time before the peak in minutes measured from the peak to the left

t_a = the time after the peak in minutes measured to the right of the peak

So (r) is a measure of how advanced the distribution is. Equation (8) covers the condition r = 0.

If r = 1, the storm is completely delayed, it peaks at the end of each duration and has considerable antecedent rainfall before every maximum period.

Solving equations (9a) and (9b) for t_d and substituting into equation (8), one obtains:

$$i = \frac{a[(1 - b)(\frac{t_b}{r})^b + c]}{[(\frac{t_b}{r})^b + c]^2} \qquad (10)$$

and

$$i = \frac{a[(1 - b)(\frac{t_a}{1-r})^b + c]}{[(\frac{t_a}{1-r})^b + c]^2} \qquad (11)$$

A distribution plotted from equations (10) and (11) will have, for all durations taken during the most intense period, the same average intensity as the intensity-duration curve from which the constants a, b, and c are derived.

Equations (10) and (11) will be used to estimate factor 1, the volume of rainfall for a given intensity-duration curve. The next step is to assign a value to the constant (r) so that the other two factors of the distribution will conform to the statistical data. This is accomplished by listing out the area's severe storms for the period of record. Create a table (see Table 1) which includes the durations of peaks of interest in the study. The times of concentration of the projects under consideration will determine these durations.

Find the amount of rainfall which occurred during the period of maximum intensity for each storm, remember multiple durations can be calculated by adding more columns to the table. In the example given in Table 1, four durations were considered. Next, calculate the volume of rainfall antecedent to the maximum period. Notice that as the duration around the peak intensity increases 15 minutes to 120 minutes, the maximum volume increases and the antecedent volume decreases. Then, determine when during the maximum duration did the actual peak occur. This requires detailed rainfall data. Keifer and Chu used 5-minute readings and were able to obtain a high degree of detail. The third column of each duration contains the particular peak 5-minute reading within that duration. Of course, as the durations lengthen to the right in the table, more variation in peak location occurs.

The mean values of antecedent rainfall (r) and the location of the peak (t_b, t_a) for each of the given durations is next

TABLE 1

RAINFALL RECORDS FROM FOUR STATIONS IN THE CHICAGO AREA

Sta.	Date	15 MIN. DURATION			30 MIN. DURATION			60 MIN. DURATION			120 MIN. DURATION		
		Max. Mass	Ante-cedent Mass	Peak Five Min.	Max. Mass	Ante-cedent Mass	Peak Five Min.	Max. Mass	Ante-cedent Mass	Peak Five Min.	Max. Mass	Ante-cedent Mass	Peak Five Min.
14	May 11-12 1935	.39	1.40	2	.67	.67	6	1.14	.44	11	1.82	.05	20
7		.44	1.01	3	.76	.65	6	1.26	.60	7	1.76	.10	19
15		.49	.46	2	.83	.46	2	1.44	.31	4	1.76	.20	6
10	Sept. 12 1936	.70	.54	2	.97	.36	3	1.05	.29	4			
14		.56	.06	2	.71	.01	3						
7		.83	.14	1	1.07	0	2	1.22	0	2			
15		.84	.01	2	1.14	.01	2	1.21	0	3			
10	June 20-21 1937	.81	.05	3	.90	.05	3	1.02	.03	7			
14		.92	.21	2	1.22	.04	3	1.35	0	4			
7		.51	.23	2	.95	.23	2	1.23	.01	6	1.45	.01	6
15		1.26	1.57	1	2.19	.50	4	2.90	.08	9	2.99	0	16
10	July 5-6 1939	.51	0	2	.56	0	2						
14		.40	0	2				.81	0	2			
7		.47	.25	2	.87	.32	1	1.19	.05	5			
15		.91	.19	2	1.22	.01	3	1.84	.01	3	1.88	0	9
10	Oct. 3-4 1941				.50	.16	3						
14		.62	.28	2	.78	.12	5	.96	.06	6	1.40	.06	6
15		.46	.07	2	.51	.07	2						
14	August 7 1942	.42	.01	2	.66	.48	5	1.06	.28	10	1.61	0	3
15		.56	.77	3	.99	.34	6	1.29	.16	10	1.54	.04	15

SOURCE: Keifer and Chu (1957)

calculated. Then, substitute these values (r, t_b, t_a) back into equations (10) and (11). Keep in mind that the constants a, b and c will depend on the shape of the study area's intensity-duration-frequency curve. A design rainfall distribution can be obtained by using equations (10) and (11) that will satisfy the three characteristics outlined in the beginning of this section.

This method was not used to determine the southwest Florida distribution because it is most applicable to short duration storms whose peak intensity period match the modeled watershed's time of concentration. Because this study uses a 24-hour storm, the rainfall data set contains only a limited number of sufficiently long duration storms. To attempt to determine the average antecedent rainfall volume occurring before a 24-hour duration rainfall is impractical, and due to the natural variations in rainfall intensity within a long duration storm, it is also unnecessary. The storage volumes filled by the antecedent rainfall prior to the short storm's peak duration are also filled by the early stage of a long duration storm.

The Pilgrim Cordery Method

This method produces a rainfall distribution using average intensity variations within the design rainfall, and the most likely sequence of these varying intensities.

First, a brief description of the Pilgrim Cordery Method is presented, followed by a description of how it was applied in this study. The method requires as input the most intense rainfall events of a selected duration recorded in the study area. Begin the analysis by selecting the rainfall duration to be distributed. Then, divide the storm duration into a number of equal periods. Select the number of periods based on the minimum time period of the unit hydrograph (or equivalent) to be used with the rainfall distribution and the adequacy of definition of the pattern. The more time periods, the better defined the pattern of rainfall intensities will be.

Next, determine the total rainfall volume for each storm included in the analysis. Then, determine the rainfall volume for each time period in each storm. Create a table to aid in performing the process (see Table 2). In columns 1 through 3, list the date, total rainfall volume, and relative ranking of the storm compared to the others included in the study. In columns 4 through 7, list the rainfall volumes by period in chronological order (note that this example only uses four time periods). Then, rank the periods in columns 8 through 11 by amount of rainfall volume with (1) assigned to the period within a storm containing the greatest volume and so on. Where ties occur in rainfall volumes between periods, list the average of the rank in these columns. An example of this is the first row in Figure 1. Next, add the values down each column for columns 8 through 11 and

TABLE 2

SAMPLE TABLE OF VALUES FOR PILGRIM-CORDERY METHOD

DATE (1)	TOTAL RAIN, IN INCHES (milli-meters) (2)	RANK (3)	RAIN IN EACH PERIOD, IN INCHES (millimeters)				RANK OF EACH PERIOD'S RAINFALL				RAIN IN PERIOD OF EACH RANK, AS A PERCENTAGE			
			PERIOD 1 (4)	PERIOD 2 (5)	PERIOD 3 (6)	PERIOD 4 (7)	PERIOD 1 (8)	PERIOD 2 (9)	PERIOD 3 (10)	PERIOD 4 (11)	RANK 1 (12)	RANK 2 (13)	RANK 3 (14)	RANK 4 (15)
November 20, 1932	1.76 (44.6)	1	0.32 (8.0)	0.48 (12.2)	0.48 (12.2)	0.48 (12.2)	4	2		2	27	27	27	19
March 20, 1914	1.68 (42.7)	2	0.30 (7.6)	0.44 (11.2)	0.44 (11.2)	0.50 (12.7)	4	2.5	2.5	1	30	26	26	18
September 29, 1943	1.66 (42.2)	3	0.48 (12.2)	0.46 (11.7)	0.31 (7.9)	0.41 (10.4)	1	2	4	3	29	27	25	19
October 26, 1922	1.57 (39.9)	4	0.42 (10.7)	0.65 (16.5)	0.35 (8.9)	0.15 (3.8)	2	1	3	4	41	27	22	10
March 9, 1913	1.53 (38.9)	5	0.18 (4.6)	0.50 (12.7)	0.45 (11.4)	0.40 (10.2)	4	1	2	3	33	29	26	12
October 25, 1919	1.50 (38.2)	6	0.40 (10.2)	0.27 (6.9)	0.41 (10.4)	0.42 (10.7)	3	4	2	1	28	27	27	18
November 20, 1961	1.40 (35.6)	7	0.35 (8.9)	0.35 (8.9)	0.35 (8.9)	0.35 (8.9)	2.5	2.5	2.5	2.5	25	25	25	25
January 19, 1926	1.39 (35.3)	8	0.36 (9.1)	0.48 (12.2)	0.40 (10.2)	0.15 (3.8)	3	1	2	4	35	29	26	10
September 25, 1951	1.37 (34.8)	9	0.44 (11.2)	0.20 (5.1)	0.37 (9.4)	0.36 (9.1)	1	4	2	3	32	27	26	15
June 15, 1949	1.33 (33.7)	10	0.42 (10.7)	0.40 (10.2)	0.35 (8.9)	0.16 (3.9)	1	2	3	4	32	30	26	12
Average							2.55	2.20	2.50	2.75	31	27	26	16
Std. Deviation							1.25	1.11	0.66	1.13	4.6	1.5	1.4	4.8
Assigned Rank							3	1	2	4				
Period							1	2	3	4				
Final Pattern (total rainfall as a percentage)							26	31	27	16				

SOURCE: Pilgrim and Cordery (1975)

divide by the number of storms to get the average of the values in each column.

Now, rank the average values for the columns. Use a (1) for the column with the lowest average and so on. Write these assigned ranks under the columns, as done in the figure. This step is used to find the most likely chronological order of the average heaviest period, second heaviest period, and so on.

In columns 12 through 15, the percentages of rainfall in the periods are listed in order of magnitude. Average the values of columns 12 through 15, as was done for columns 8 through 11. These average percentages of rainfall are an estimate of the percentage that would occur in the periods of rainfall of average variability. Now, arrange the average percentages in the most likely chronological order, as determined previously. These values should be listed at the bottom of the table below columns 8 through 11, entitled "Final Pattern" so a distribution has been created that can be used for any return frequency design storm by multiplying the Final Pattern percentages by the design storm total rainfall volume.

The Pilgrim Cordery Method Applied to Southwest Florida

For this application of the Pilgrim Cordery Method, a 24-hour storm duration is chosen. Eight storms are selected from the five-station data set for use in the procedure. The same screening criteria is used here as is used for the polynomial curve fitting technique which is covered in detail in Chapter IV. Each storm has approximately a 24-hour duration and rainfall intensities peaking close to the mid-point of the 24-hour event. Mid-peaking storms were chosen because their use coincides with the antecedent moisture condition II. This is the condition normally used for design, as will be explained in the next chapter.

Each storm is divided into 24 equal periods for the analysis. The calculations required were accomplished using a program run on an IBM microcomputer. The results are presented in Table 3. The Pilgrim Cordery Method enabled the production of a dimensionless distribution of cumulative rainfall amount P_{total}. The cumulative rainfall amount for any period in the distribution is found by multiplying that period's ratio given in Table 3 by the total rainfall volume for the design storm. In this study, a distribution was required for each of three total rainfall depths corresponding to the return frequencies of 10, 25 and 100 years for 24-hour duration storms. The total volumes are 7.9 inches for the 10-year, 9.0 inches for the 25-year and 11 inches for the 100-year 24-hour storms occurring in the southwest Florida region. To produce these distributions, each total rainfall volume was multiplied by the hourly ratio values. The results are included in Appendix I, Table 16.

TABLE 3

RAINFALL DISTRIBUTION BY THE PILGRIM-CORDERY METHOD

HOUR	% OF P_{TOTAL}	RATIO
1	0.1	0.001
2	0.5	0.006
3	0.6	0.012
4	1.0	0.022
5	1.0	0.032
6	1.0	0.042
7	1.2	0.054
8	4.0	0.094
9	4.9	0.143
10	6.2	0.205
11	11.5	0.320
12	14.2	0.462
13	24.0	0.702
14	10.6	0.808
15	3.6	0.844
16	2.8	0.872
17	2.8	0.900
18	2.8	0.928
19	2.6	0.954
20	2.6	0.980
21	1.8	0.998
22	0.1	0.999
23	0.08	0.9998
24	0.02	1.000
	100.00	

Soil Conservation Service Rainfall Distributions

Stream gauge measurements are rarely available, especially for small watersheds. Generalized rainfall data, however, are available nationally. The Weather Bureau's Rainfall Frequency Atlas covering the United States provides rainfall frequency data for areas less than 400 square miles, many durations, and frequencies from 1 to 100 years (U.S. Weather Bureau 1961).

Unlike the Army Corps of Engineers method to be discussed next, adjustment of rainfall with respect to watershed size is not necessary because the drainage areas for this method are small.

Two major regions of the United States were identified as having markedly different distributions. The time-intensity distributions for each are shown graphically in Figure 4.

SCS Type I and Type II

The Soil Conservation Service Type I distribution applies to regions along the West Coast and Alaska. The Type II distribution applies to regions where peak runoff rates for small watersheds result from summer thunderstorms. This covers the majority of the United States, including part of Florida (see Figure 5). Both Type I and Type II distributions are based on generalized rainfall depth-duration frequency relationships obtained from Weather Bureau Technical Paper No. 40. The accumulative graphs in Figure 6, which are the basis for the distributions, were established by

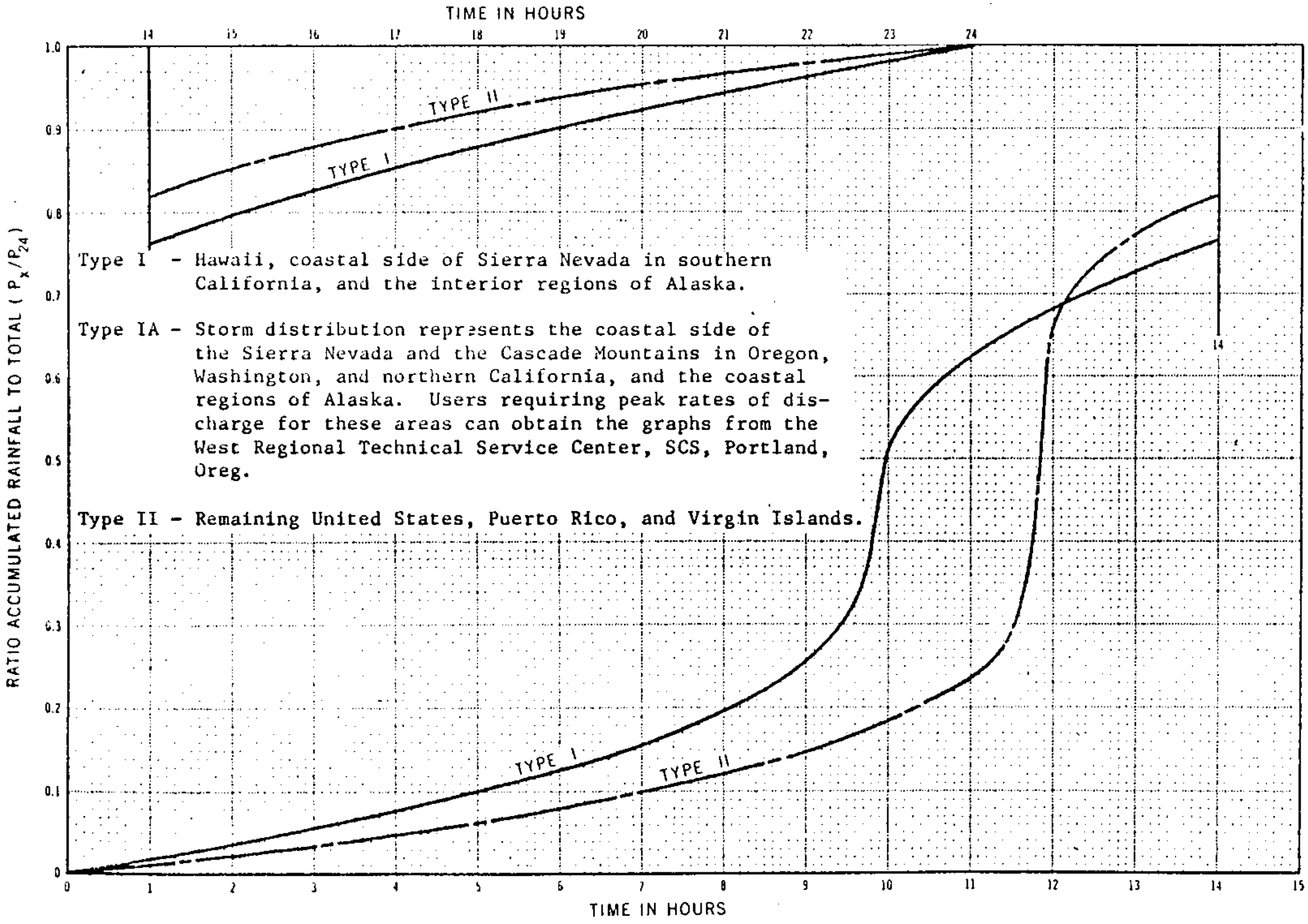


Figure 4. SCS Type I and Type II rainfall distributions (Kent 1973).

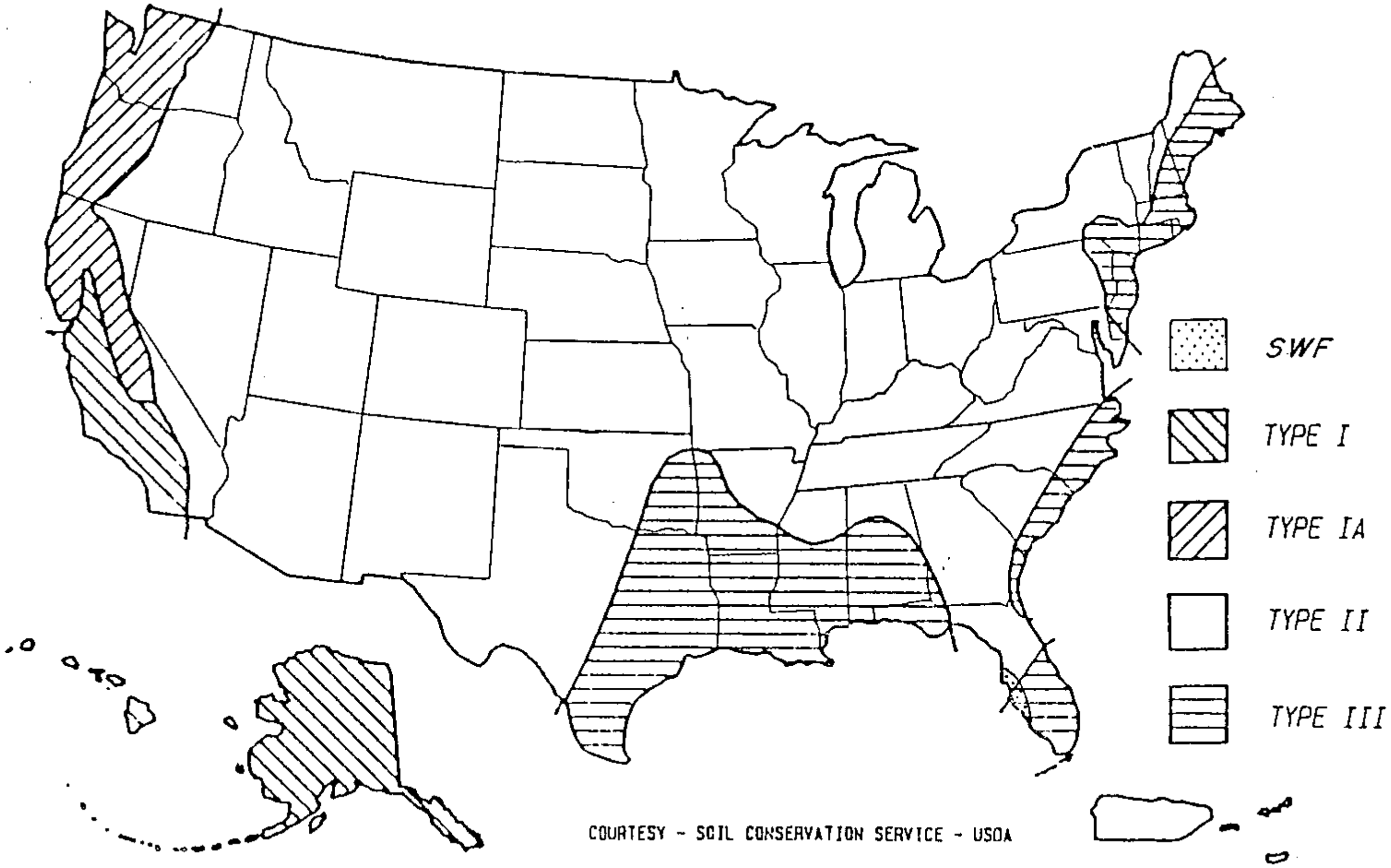


Figure 5. Approximate areas for rainfall distributions (USDA-SCS 1986).

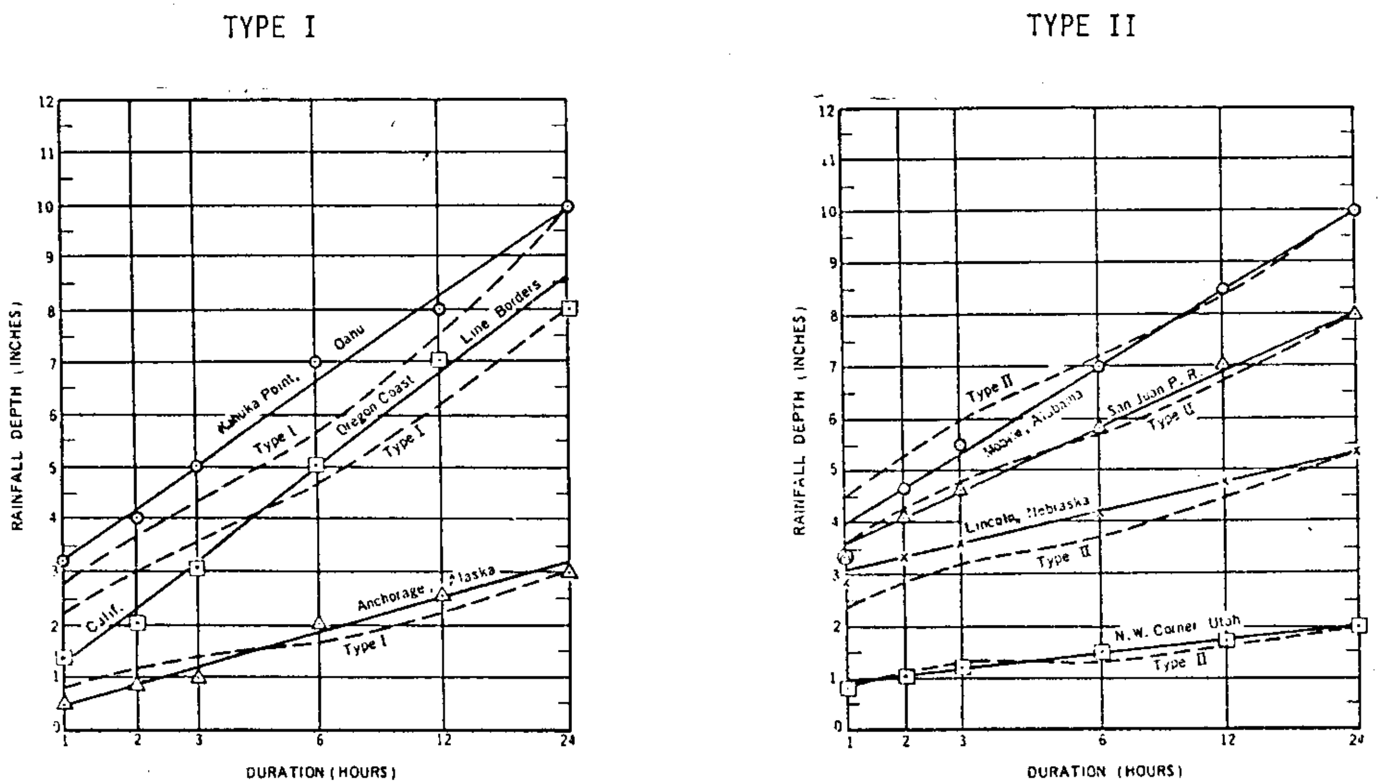


Figure 6. Generalized 25-year frequency rainfall depth-duration relationships (U.S. Weather Bureau 1961).

(1) plotting a ratio of rainfall amount for any duration over the 24-hour amount against duration for a number of locations and (2) selecting a curve of best fit as determined from a graphical presentation of the data (Kent 1973). The result of this operation is presented in Figure 4. The actual curve of best fit is not used directly, however. The average intensity-duration values used to develop the dashed lines in Figure 6 are rearranged to form the Type I and Type II distributions in Figure 4. The Type I distribution is arranged so that the greatest 30-minute volume occurs at about the 10-hour point of the 24-hour period, the second largest in the next 30 minutes, and the third largest in the preceding 30 minutes. This alternation continues with each decreasing order of magnitude until the smallest increments fall at the beginning and end of the 24-hour rainfall (Figure 7). The Type II distribution is arranged in a similar manner, but the greatest 30-minute depth occurs near the middle of the 24-hour period. The selection of the period of maximum intensity for both distributions was based on design considerations rather than on meteorological factors (Kent 1973).

SCS Type II Florida Modified

The Soil Conservation Service developed the Type II Florida Modified distribution using the same methodology used to obtain the Type II distribution. Data from Hydro-35 rather than Weather Bureau Technical Paper No. 40 was used, however (St. Johns River

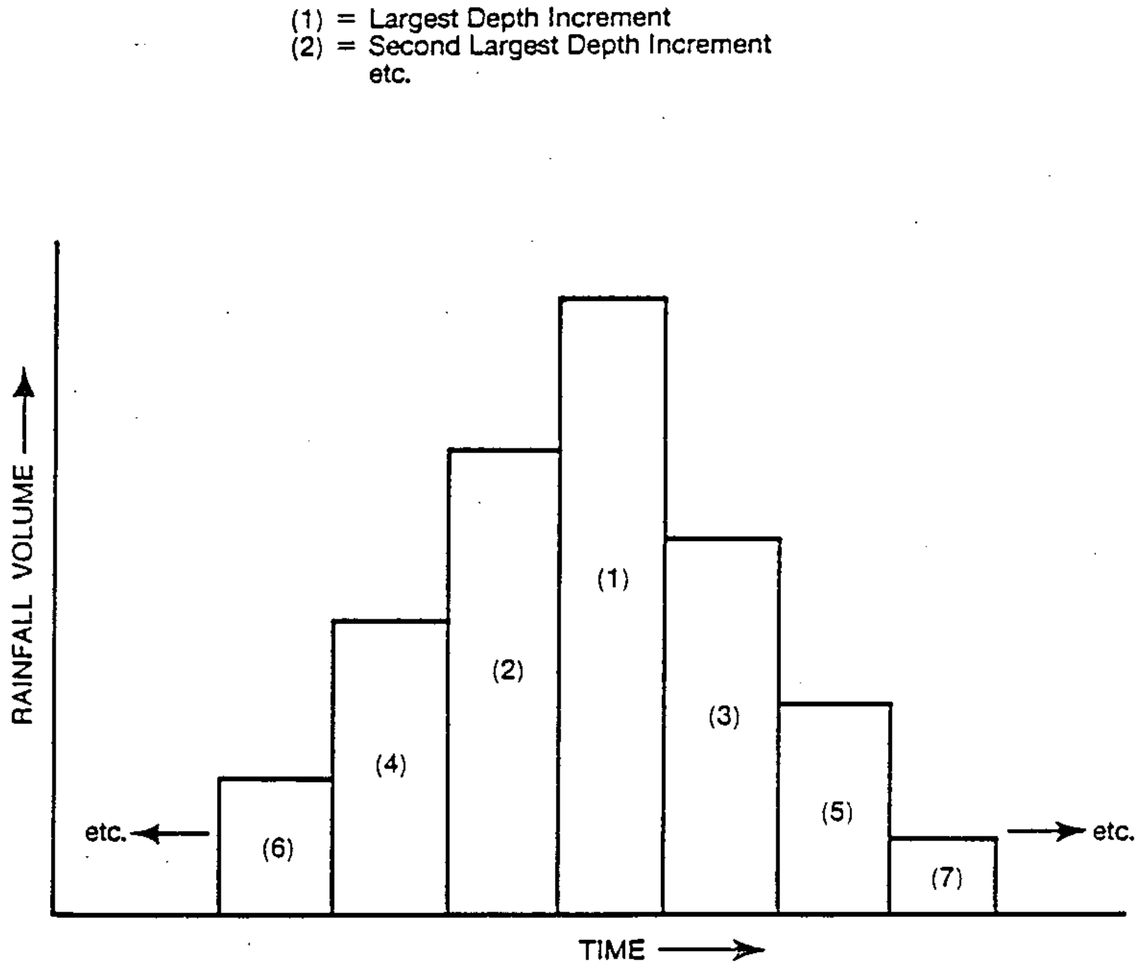


Figure 7. Schematic of SCS design rainfall distributions (St. Johns River Water Management District 1984).

Water Management District 1984). The third distribution was created due to the pronounced difference between the hyetograph shapes found along the eastern seaboard and the Gulf regions compared to the hyetograph shapes of the majority of the continental U.S. The time-intensity data for the areas covered by the Type II Florida Modified distribution contain tropical storm rainfalls.

The distribution was developed for 24-hour storm durations, then ratios were calculated to scale the 24-hour values down for application in shorter storms. Table 4 shows the 24-hour SCS Type II Florida Modified distribution in half-hour increments. The Type II is included for comparison. Figure 8 is a comparison of the type II and Type II Florida Modified plotted together. Notice that they generally follow the same pattern, but that the Florida Modified distribution allows a greater volume early in the storm and then is more restrictive in the latter portion. This produces a less significant peak period at the mid-point, but a well-defined peak still is provided.

U.S. Army Corps of Engineers Distribution

An in-depth discussion of this distribution method is presented here to explain the rainfall distribution required to be created for the region under study as part of the comparison portion of this work.

TABLE 4

SCS TYPE II AND TYPE II-FLORIDA
MODIFIED 24-HOUR RAINFALL DISTRIBUTION

TIME (hrs)	RAINFALL RATIO (ACCUMULATED TOTAL/24-HOUR TOTAL)	
	TYPE II	TYPE II-FLORIDA MODIFIED
0.0	0.000	0.000
0.5	0.005	0.006
1.0	0.011	0.012
1.5	0.017	0.018
2.0	0.022	0.025
2.5	0.029	0.032
3.0	0.035	0.039
3.5	0.042	0.046
4.0	0.048	0.054
4.5	0.056	0.062
5.0	0.064	0.071
5.5	0.072	0.080
6.0	0.080	0.089
6.5	0.090	0.099
7.0	0.100	0.110
7.5	0.110	0.122
8.0	0.120	0.135
8.5	0.134	0.149
9.0	0.147	0.164
9.5	0.163	0.181
10.0	0.181	0.201
10.5	0.204	0.226
11.0	0.235	0.258
11.5	0.283	0.307
12.0	0.663	0.606
12.5	0.735	0.718
13.0	0.772	0.757
13.5	0.799	0.785
14.0	0.820	0.807
14.5	0.835	0.826
15.0	0.850	0.842
15.5	0.865	0.857
16.0	0.880	0.870
16.5	0.889	0.882
17.0	0.898	0.893
17.5	0.907	0.903
18.0	0.916	0.913
18.5	0.925	0.922
19.0	0.934	0.931
19.5	0.943	0.939
20.0	0.952	0.947
20.5	0.958	0.955
21.0	0.964	0.962
21.5	0.970	0.969
22.0	0.976	0.976
22.5	0.982	0.983
23.0	0.988	0.989
23.5	0.994	0.995
24.0	1.000	1.000

SOURCE: St. Johns River Water Management District (1985)

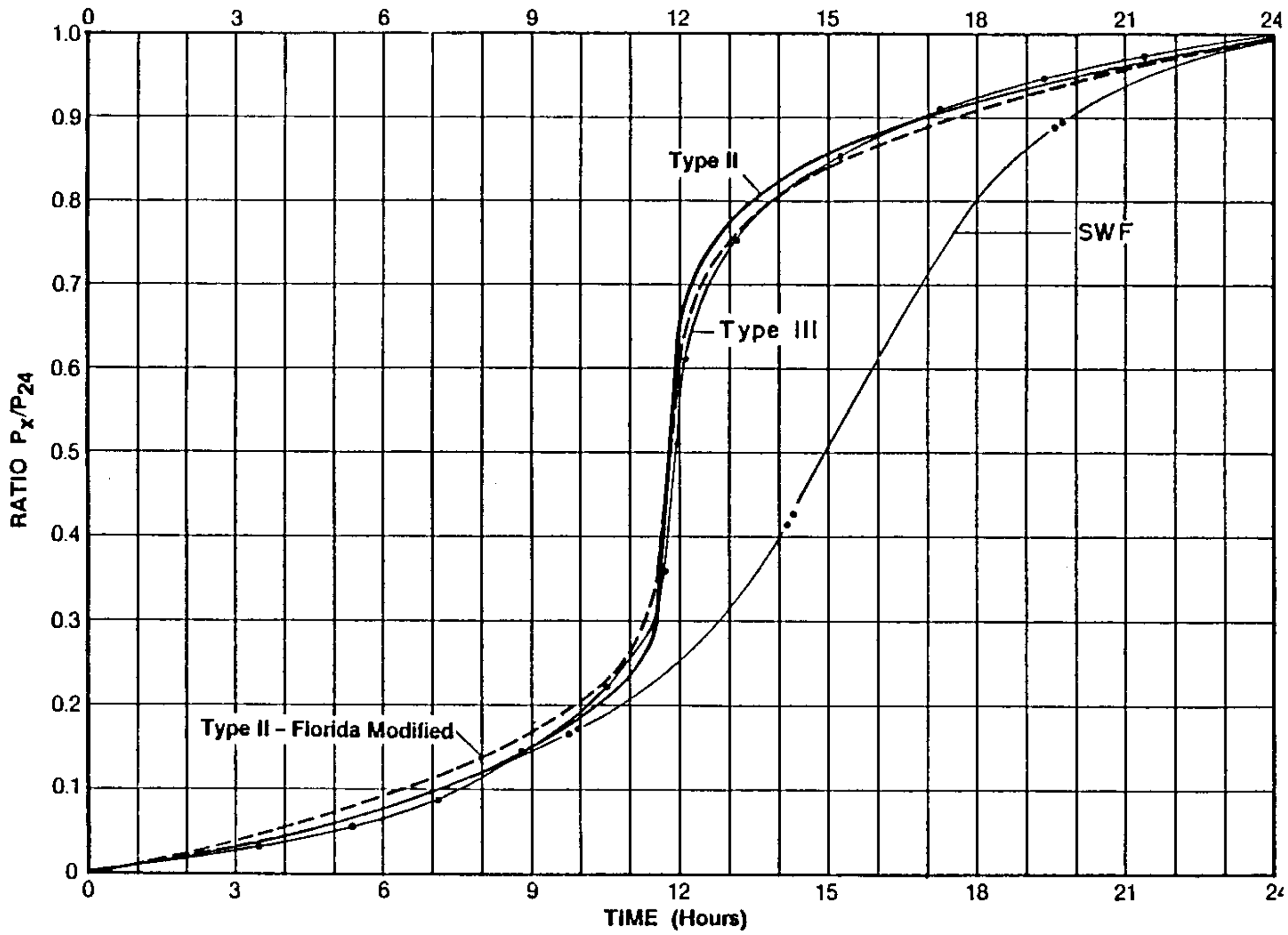


Figure 8. SCS Type II, Type II-Florida Modified, Type III, and Southwest Florida cumulative rainfall distributions (St. Johns River Water Management District 1984).

The U.S. Army Corps of Engineers (COE) use a design rainfall distribution called the Standard Project Storm in their flood studies. This distribution and the methodology behind it are covered in COE Civil Engineering Bulletin No. 52-8. This Standard Project Storm (SPS), when applied to a particular drainage area, is defined as an estimate which represents the most severe flood producing rainfall depth-area-duration relationship and isohyetal pattern of any storm that is considered reasonably characteristic for the region. A general comparison of a region's recorded maximum storms, supplemented by meteorological research, serve as a base in selecting rainfall criteria outlining the most severe storm considered reasonably characteristic of a region. Certain storms of extraordinary severity may be eliminated as too extreme to rate being Standard Project Storms. Approximately ten percent of the storms studied have equalled or exceeded the SPS. This demonstrates that the SPS is not of unprecedented magnitude, but it is definitely a major storm.

The Standard Project Storm criteria described here apply to drainage areas east of longitude 105^{o} and to basins under 1000 square miles in area. The rainfall criteria are primarily based on major storms of record that occurred in the spring, summer and fall seasons when convective activity is prominent. Figure 9 shows the Standard Project Storm Index Rainfall Isohyets. The isohyets show the maximum average rainfall depth in 24-hours over a 200 square mile basin during the SPS. The Army Corps of

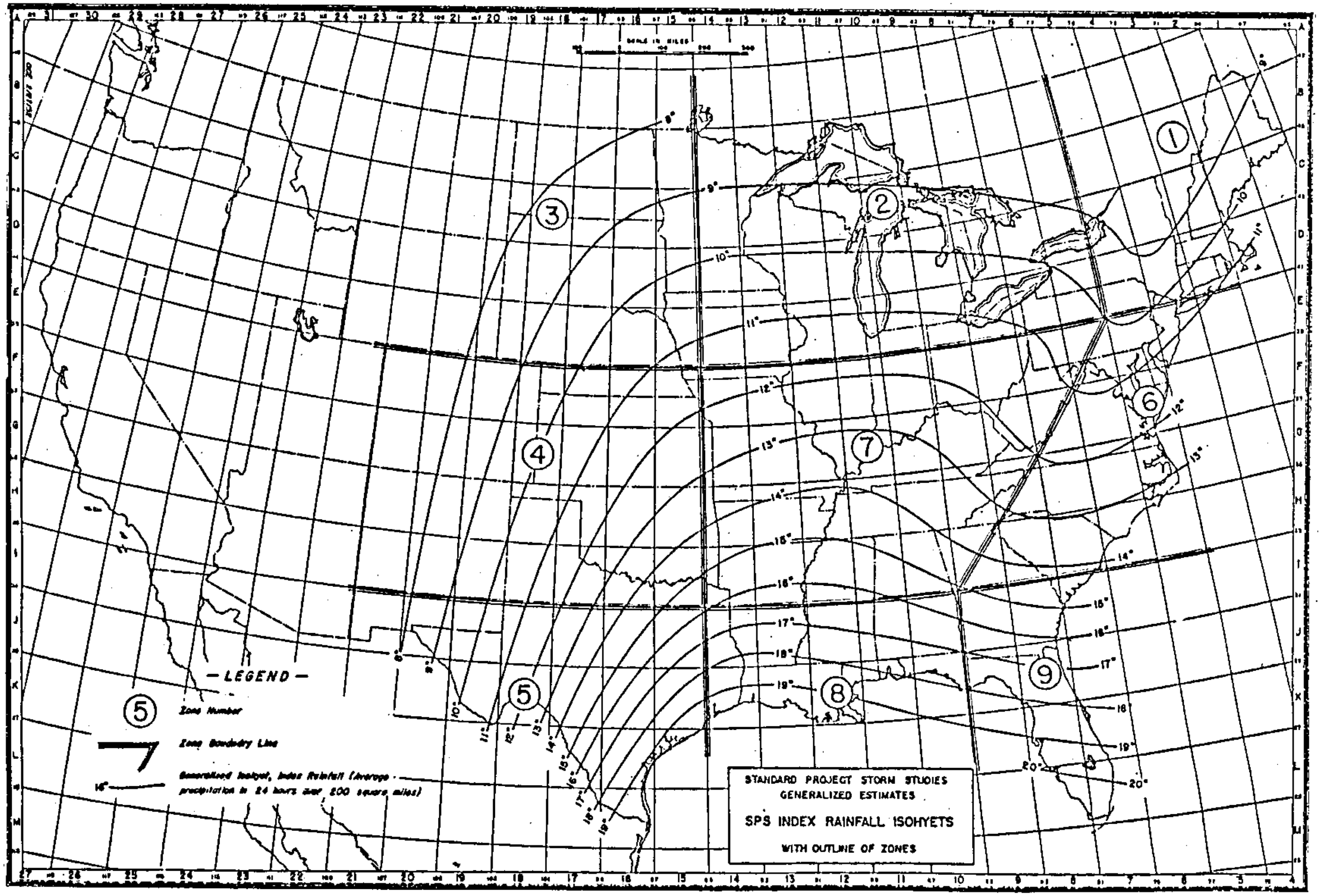


Figure 9. U.S. Army Corps of Engineers standard project storm index rainfall isohyets (U.S. Army Corps of Engineers 1965).

Engineers varies the volume of rainfall for Standard Project Storms applied to different sized basins within a region. Figure 10 presents the SPS depth-area curve for 24-hour rainfall events. This chart gives the volume of rainfall for a basin size as a percentage of the 200 mile SPS. Notice that rainfall volumes increase as drainage basin size decreases. The percentage values given in the chart are used as a multiplication factor to adjust the SPS volume given in Figure 9 for the basin size under study.

Once the rainfall volume has been determined, a time-intensity distribution is made. The Army Corps of Engineers has found, through relatively extensive study of actual storm hyetographs, that the maximum 6-hour rainfall may occur near the beginning, middle or end of the maximum 24-hour rainfall period of a storm. They break up a 24-hour duration storm into four segments of 6 hours each. The segment of most intense rainfall is placed somewhat arbitrarily after two less intense segments on the basis that this sequence will produce critical runoff from most basins. The first 12 hours of rainfall will fill to capacity the surface and sub-surface storage areas in the basin. Therefore, the 13th through 18th hours of rainfall will result completely in runoff. Figure 11b shows this typical arrangement of 6-hour rainfall segments in the Standard Project Storm (notice the similarity with the SCS pattern in Figure 7). Next, the percentages of total rainfall volume allotted to each 6-hour segment is adjusted on the basis of total SPS Index volume. A

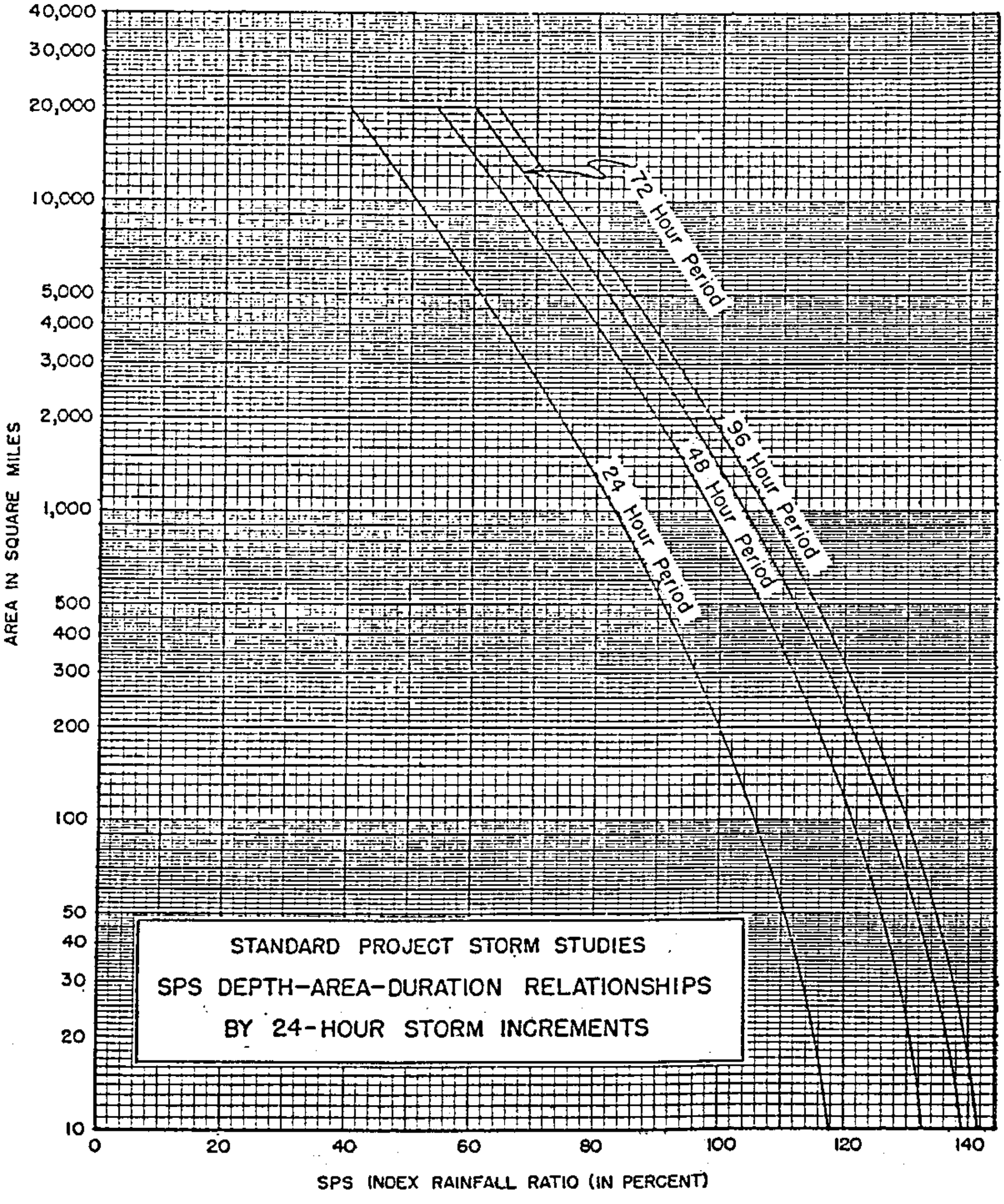


Figure 10. U.S. Army Corps of Engineers SPS depth area curve for 24-hour rainfall events (U.S. Army Corps of Engineers 1965).

table of 6-hour segment allotments in percent of total index volume appears in Figure 11c. The table values show that as Index Rainfall volume increases, less volume is allotted to the peak segment. The author believe this is due to the rainfall record not including many severe storms whose peak 6-hour intensities sustain values of over two inches per hour. To present the information contained in Figure 11c graphically, Figure 11a is provided. It shows the maximum 6-hour precipitation segment percentage sliding downward as SPS Index Rainfall Volume increases. And, of course, as the maximum segment decreases, the remaining three segments increase their proportional percentage of the total volume.

Analysis of major storms approaching Standard Project Storm intensities over areas of a few hundred square miles show that the rate of rainfall is fairly uniform during the maximum 6-hour segment of the storm. Rainfall rates during less intense 6-hour segments are generally more erratic, and may follow many different sequences and rate changes in different storms. However, studies indicate that the assumption of uniform rainfall intensities during successive 6-hour segments of the SPS, with the exception of the maximum 6-hour segment for estimates applicable to small drainage basins, give satisfactory flood discharge estimates. For drainage areas under 300 square miles, the maximum 6-hour rainfall can be subdivided further to provide greater detail to the design rainfall distribution.

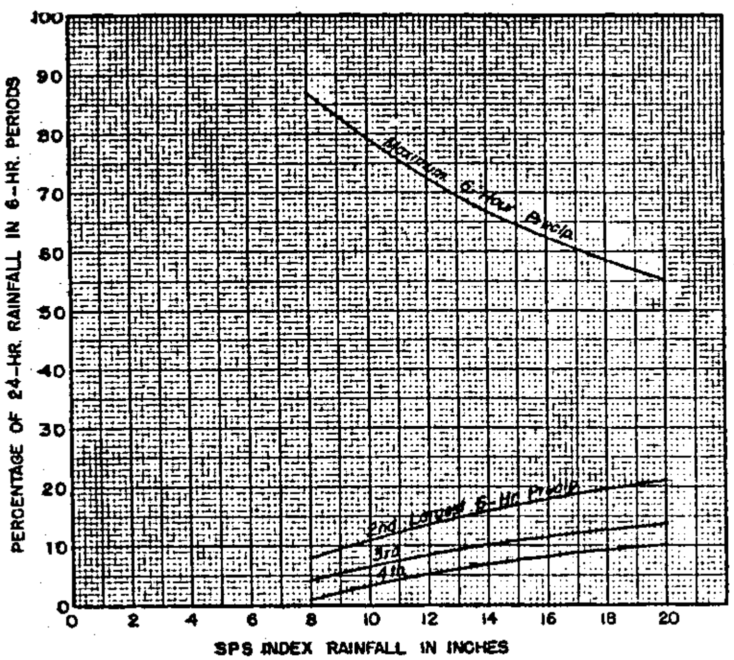


FIG. (a) SPS 24-HOUR PRECIPITATION OVER 200 SQ. MI. vs. PERCENT IN 6-HOUR PERIODS

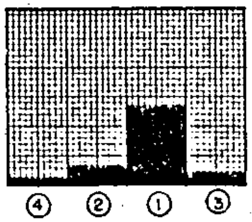


FIG. (b) TYPICAL ARRANGEMENT OF 6-HOUR RAINFALL QUANTITIES IN SPS

INDEX RAINFALL IN INCHES	PERCENTAGE OF 24-HOUR SPS RAINFALL IN DESIGNATED 6-HR. PERIOD			
	④	②	①	③
1	2	3	4	5
8	1.0	8.0	87.0	4.0
9	2.1	9.5	83.0	5.4
10	3.2	11.0	79.2	6.6
11	4.3	12.3	75.9	7.5
12	5.3	13.8	72.5	8.4
13	6.1	14.9	69.6	9.4
14	7.0	16.0	66.9	10.1
15	7.6	17.0	64.5	10.9
16	8.1	17.9	62.4	11.6
17	8.8	18.9	60.3	12.0
18	9.1	19.7	58.5	12.7
19	9.8	20.3	56.8	13.1
20	10.1	21.0	55.1	13.8

FIG. (c) TABULATION OF DATA FROM FIG. (a)

Figure 11. Time distribution of 24-hour SPS rainfall.

Table 5 provides the suggested distribution of the maximum 6-hour SPS rainfall segment. The table is divided by "selected unit rainfall duration" (t_r), to allow greater distribution detail for smaller watersheds. Table 1 describes the procedure for determining t_r. Column #2 of the table is for larger watersheds whose t_r = 6. For this case, use the maximum 6-hour segment without further subdivisions. Column #3 is for watersheds with t_r = 3. Here, the maximum 6-hour segment can be divided into two parts with the first 3-hour sub-segment having 33% of the 6-hour segment's volume and the second 3-hour sub-segment having 67% of the 6-hour segment's volume. Column #4 is for watersheds with t_r = 2. Now the maximum 6-hour segment can be divided into three sub-segments, with each 2-hour sub-segment having the listed percentage of the 6-hour segment volume. Lastly, for the smaller watersheds where t_r = 1, the maximum 6-hour segment can be divided into six sub-segments of one hour each. Column #5 gives the percentages to be assigned each sub-segment.

Therefore, by following the above procedure, a rainfall distribution can be developed. One criticism of this type of method was written by Pilgrim and Cordery (1975). It states: "In several design patterns, the sequence of intensity blocks is arranged arbitrarily to give a maximum value of peak discharge. This gives the joint occurrence of a rainfall intensity of low probability and a pattern of low probability. The frequency of exceedance of the resulting flood estimate would then be lower than that of the rainfall causing it."

The qualifiers section of the introduction also stressed the importance of selecting a storm distribution that matches the volume of rainfall in order to produce a specified return frequency storm. The Army Corps method can be "tuned" to the proper balance by using good engineering judgement in the selection of the Index Rainfall volume. Remember, Figure 9 gives values for only the Standard Project Storm.

Two Army Corps of Engineers SPS Distributions for Southwest Florida

This study includes the Army Corps of Engineers' Standard Project Storm rainfall distribution as one of the existing, commonly used distributions. To compare these commonly used distributions in Chapter IX, two SPS distributions are developed here.

Starting the first with the Standard Project Storm Index Rainfall corresponding to southwest Florida, using Figure 9, the SPS Index is 20 inches. Then, using an SPS Index of 11 inches corresponding to the area's estimated 100-year 24-hour storm volume (U.S. Weather Bureau Technical Paper No. 40). These two rainfall volumes are routed through the procedure described previously to derive the two distributions. The maximum 6-hour segment was broken into hourly sub-units to obtain as much detail as this method allows in describing time-intensity distributions.

Using Figure 11 for 11 inch SPS rainfall yields this breakdown:

	6-Hour Periods			
	4th	2nd	1st	3rd
hours	1-6	7-12	13-18	19-24
%	4.3	12.3	75.9	7.5

Using Table 5 to distribute the maximum 6-hour segment further:

hour	13	14	15	16	17	18
%	10	12	15	38	14	11

Then multiply the maximum 6-hour segment percentage (75.9) by the values obtained from Table 5:

7.59	9.11	11.38	28.84	10.63	8.35

By dividing the remaining three 6-hour segment percentages by 6, the hourly rainfall portion for each of the 24-hours in the design storm can be obtained (see Table 6).

Using Figure 11 for 20 inch SPS rainfall yields this breakdown:

TABLE 5

TIME DISTRIBUTION OF MAXIMUM 6-HOUR SPS RAINFALL

RAINFALL PERIOD (SUBDIVISION OF 6-HOUR PERIOD)	TIME DISTRIBUTION OF MAXIMUM 6-HOUR SPS RAINFALL, EXPRESSED IN PERCENT OF TOTAL 6-HOUR RAINFALL			
	SELECTED UNIT RAINFALL DURATION, t_R			
	6-HOURS	3-HOURS	2-HOURS	1-HOUR
# 1	# 2	# 3	# 4	# 5
1st	100	33	26	10
2nd		67	53	12
3rd			21	15
4th				38
5th				14
6th				11
TOTAL	100	100	100	100

* NOTE: The "selected unit rainfall duration," t_R, is determined approximately from the synthetic unit hydrograph equation, $t_r = t_p/5.5$ in which "t_p" is the lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph, in hours. The following rounded-off values are to be used in the above table:

If t_p exceeds 16, use $t_R = 6$

If t_p is between 12 and 16, use $t_R = 3$

If t_p is between 6 and 12, use $t_R = 2$

If t_p is between 4 and 6, use $t_R = 1$

SOURCE: U.S. Army Corps of Engineers (1965)

TABLE 6

DISTRIBUTION OF THE 11- AND 20-INCH
ARMY CORPS OF ENGINEERS
STANDARD PROJECT STORM

HOUR	11-INCH % OF TOTAL VOLUME	20-INCH % OF TOTAL VOLUME
1	0.71	1.68
2	0.71	1.68
3	0.71	1.68
4	0.71	1.68
5	0.71	1.68
6	0.71	1.68
7	2.05	3.5
8	2.05	3.5
9	2.05	3.5
10	2.05	3.5
11	2.05	3.5
12	2.05	3.5
13	7.59	5.61
14	9.11	6.73
15	11.38	8.41
16	28.84	21.32
17	10.63	7.85
18	8.35	6.17
19	1.25	2.3
20	1.25	2.3
21	1.25	2.3
22	1.25	2.3
23	1.25	2.3
24	1.25	2.3

	6-Hour Periods			
	4th	2nd	1st	3rd
hours	1-6	7-12	13-18	19-24
%	10.1	21.0	56.1	13.8

Using Table 5 to distribute the maximum 6-hour segment further:

hour	13	14	15	16	17	18
%	10	12	15	38	14	11

Then multiply the maximum 6-hour segment percentage (56.1) by the values obtained from Table 5:

5.61	6.73	8.41	23.32	7.85	6.17

By dividing the remaining three 6-hour segments by six, the hourly rainfall volume for each of the 24-hours in the design storm can be obtained (see Table 6).

The distributions derived for the 11 and 20 inch SPS are presented in Table 18 in Appendix I, using the volumes from the 10-, 25- and 100-year 24-hour storm.

Local Government Distributions

Seminole County: Recognizing that the majority of hydrograph approximation models used within the County will be applied to small watersheds, Seminole County has developed both a 6- and 3-hour time-intensity rainfall distribution. Following the recommendation of the Agricultural Research Service for Florida, the two rainfalls were distributed in accordance with the Soil Conservation Service Type II distribution. A 25-year return frequency volume corresponding to the 6-hour duration was obtained from the U.S. Weather Bureau's Technical Paper No. 40. For this area, it is 6.00 inches. Table 7 presents the results in 15-minute intervals.

TABLE 7

SEMINOLE COUNTY DESIGN RAINFALL 6-HOUR DURATION
25-YEAR FREQUENCY IN 15-MINUTE INCREMENTS

Time Minutes	Time Hours	ΣP Inches	ΔP Inches
0	0	0	0
15	.25	.10	.10
30	.50	.21	.11
45	.75	.33	.12
60	1.00	.48	.15
75	1.25	.64	.16
90	1.50	.81	.17
105	1.75	1.08	.27
120	2.00	1.38	.30
135	2.25	2.46	1.08
150	2.50	3.60	1.14
165	2.75	3.90	0.30
180	3.00	4.20	0.30
195	3.25	4.44	0.24
210	3.50	4.68	0.24
225	3.75	4.86	0.18
240	4.00	5.01	0.15
255	4.25	5.16	0.15
270	4.50	5.28	0.12
285	4.75	5.40	0.12
300	5.00	5.52	0.12
315	5.25	5.64	0.12
330	5.50	5.76	0.12
345	5.75	5.88	0.12
360	6.00	6.00	0.12

SOURCE: Seminole County

CHAPTER IV

DERIVING THE SOUTHWEST FLORIDA RAINFALL DISTRIBUTION

Data Sources and Forms

To create a design rainfall distribution for the southwest Florida area, a thorough data collection effort was necessary. All of the potential sources of rainfall data were contacted. These include:

- The Florida State Climotologist
- The United States Geological Survey
- National Weather Service
- The Southwest Florida Water Management District
- Local airports within study area

To create a 24-hour rainfall distribution for the region, detailed rainfall records are required. Simple daily volumes collected at most recording stations are not able to help in determining the shape of the region's intense storm events. The State Climatologist could supply daily readings from fire watch towers, the local airports also keep daily records. These sources were not useful to the study.

What was needed for this work were hourly or more frequent rainfall readings. Only automatic recording gauges are set up to provide this level of detail. The U.S. Geological Survey had a few recording rain gauges set up within the study area for a project lasting three years, but these data could not be used due to the short period of record obtained. No high volume storm events were recorded in the brief time the gauges were in operation.

The National Weather Service, however, has at least five recording rain gauge stations in the southwest Florida area. See Figure 12 for their locations. They are: Tampa, Brooksville, Venice, St. Leo and St. Petersburg. These stations have varying periods of record. One began operation in the 1920s and the remainder started in the 1950s.

The National Weather Service archives in Asheville, North Carolina, provided the hourly rainfall readings for each station for the period of record. Rainfall values to 0.01 inch are recorded for each hour, with daily and monthly totals included as well.

Storm Selection Criteria

The first step in utilizing these data was to locate all the storms lasting approximately 24 hours. A design storm of 24-hour duration was selected due to the common use of this length storm. The 24-hour period was originally chosen for design storms

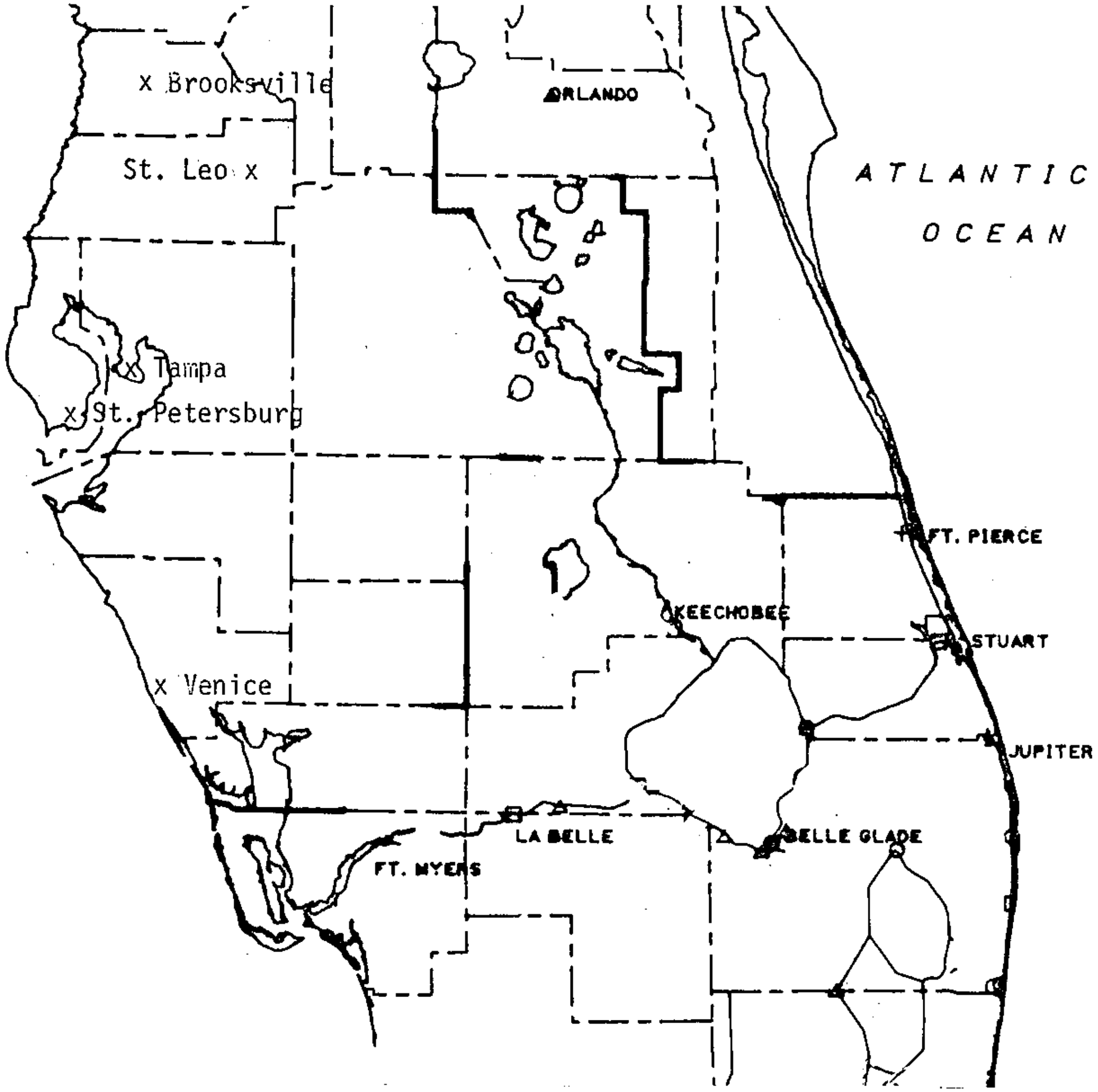


Figure 12. National Weather Service recording rain gauge locations in southwest Florida (National Weather Service 1961).

because of the general availability of daily rainfall data. The commonly used national distributions have 24-hour durations, and this time frame spans most of the applications of TR 55 as well.

Because very few storms have exactly 24-hour durations, an envelope was allowed. Storms lasting between 19 and 26 hours were identified. If a period of four hours or longer occurred with no rainfall in the course of the storm, then the rainfall was considered to consist of two separate events. Because long duration storms can result in low total precipitation volumes, the storms next were screened for volume. No total rainfall under three inches was considered. The justification for these criteria being that a four-hour break in a storm event would cause an unnatural hyetograph shape factor in the analysis that followed. Because this work is to develop a storm distribution for design purposes, only storms producing significant volumes were considered. The storms meeting these criteria were then plotted to study their shapes (see figures 13 through 16). The plots are dimensionless to eliminate differences in volume and duration. In this form, the storm shapes can more easily be compared. The Weather Service data was made dimensionless by dividing the storm's incremental rainfall volume by the total storm volume, and then each time increment was divided by the total storm duration.

These dimensionless hyetograph data were next treated to a series of statistical tests to determine an equation that would best represent all of their shapes.

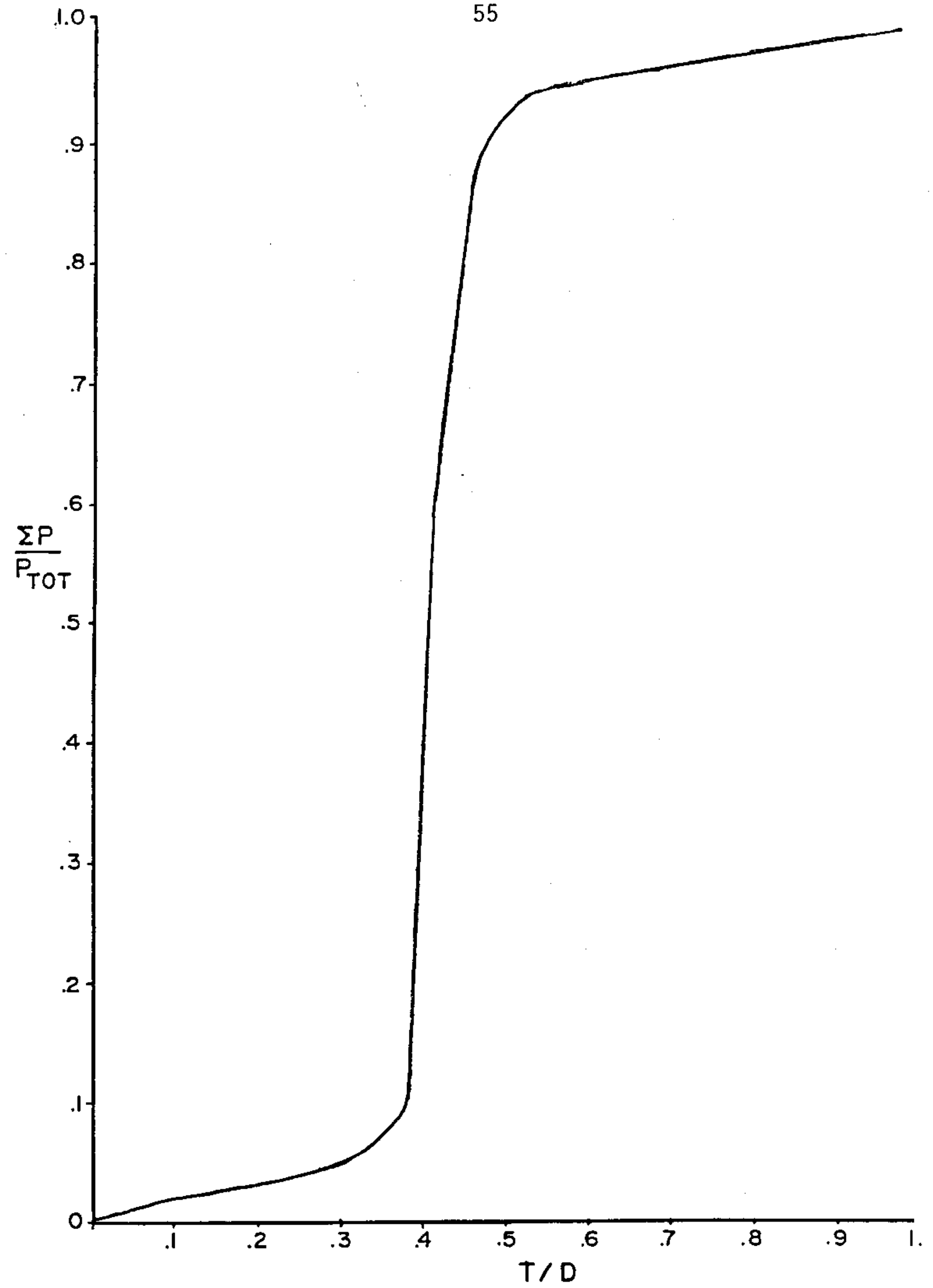


Figure 13. Hyetograph of 5/15/76 Tampa storm used in the SWF rainfall distribution.

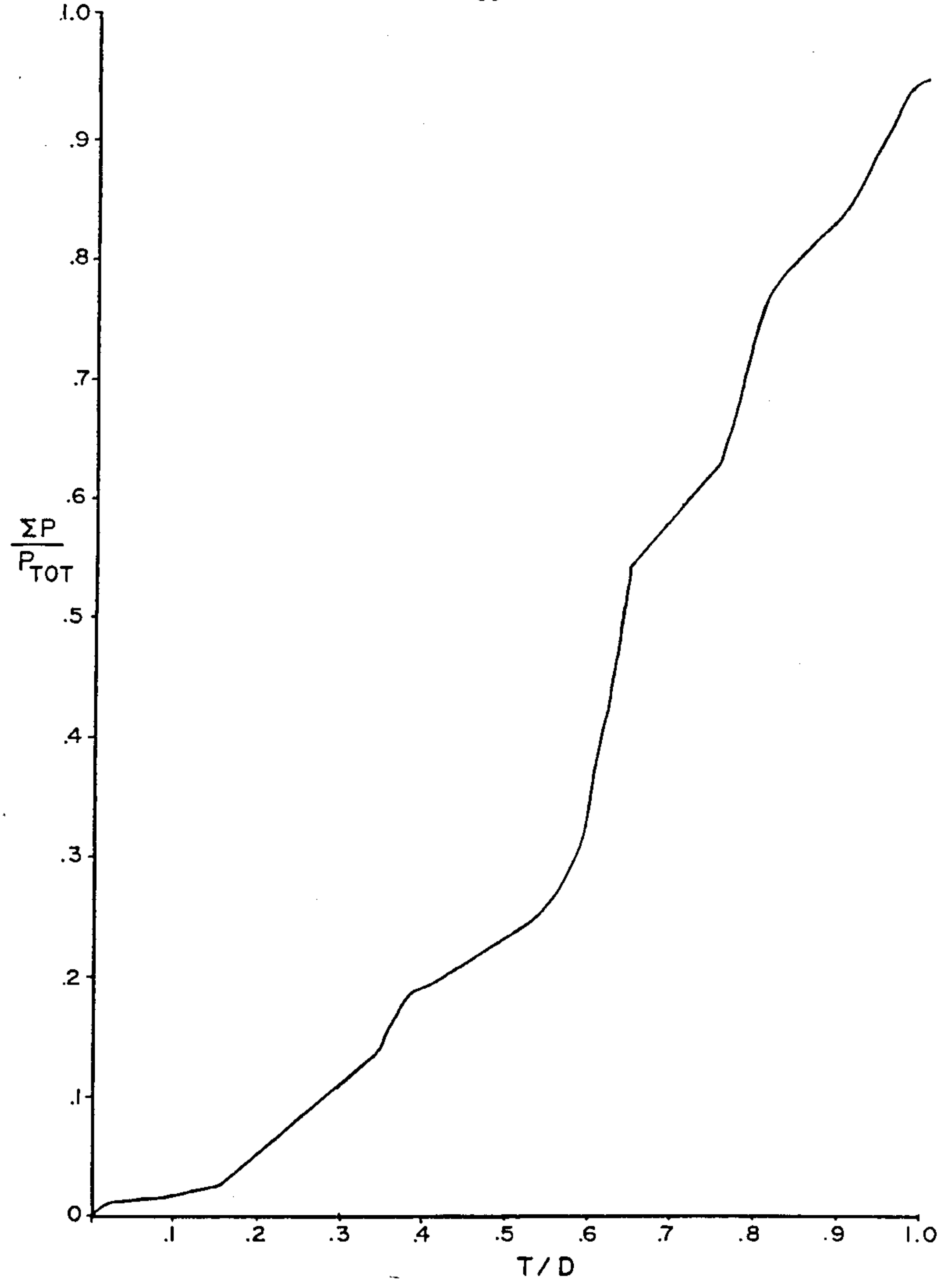


Figure 14. Hyetograph of 6/17/82 St. Petersburg storm used in the SWF rainfall distribution.

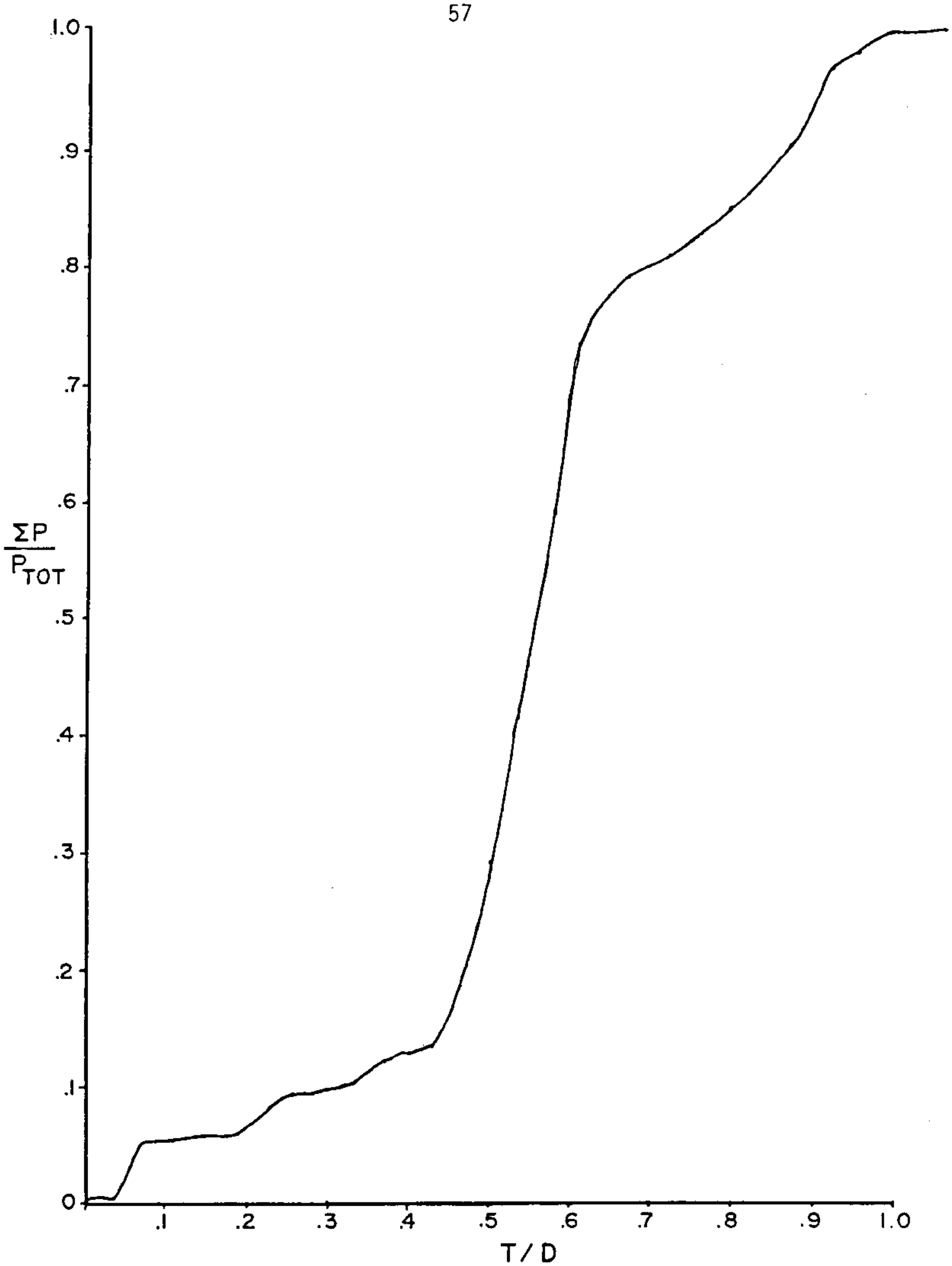


Figure 15. Hyetograph of 9/17/47 Brooksville storm used in the SWF rainfall distribution.

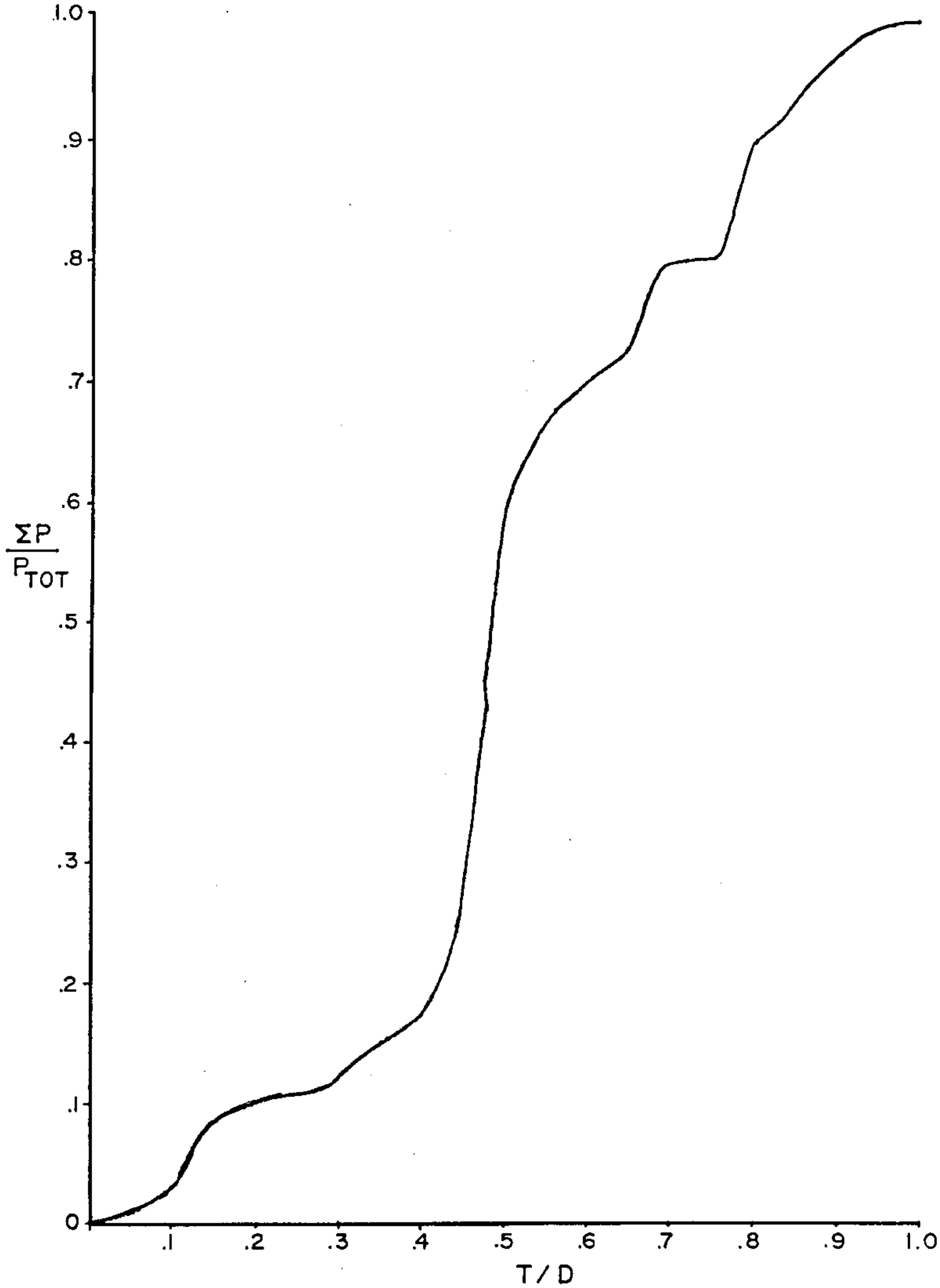


Figure 16. Hyetograph of 11/16/51 Brooksville storm used in the SWF rainfall distribution.

The Derivation of the Rainfall Distribution for Southwest Florida

The storm time-volume data collected for the study was plotted into dimensionless hyetographs. Most of the storms produced an S shaped hyetograph. However, they varied widely in when the most intense period of rainfall occurred. Storms ranged from peaking soon after rainfall began to peaking just before finishing.

The method of derivation for the design rainfall distribution was to produce a hyetograph that was representative of severe storm events occurring in the region. Because these storms varied widely in when their peaks occurred, it was decided to only select mid-peaking storms for use in this study. Mid-peaking storms were selected because their use coincides with the antecedent moisture condition II which is normally used for design (USDA-SCS 1972). As previously discussed, the right amount of antecedent rainfall must be placed before the most intense segment of the storm. Early peaking storms are subject to greater watershed infiltration capacities and surface depression storage which absorb much of the peak rainfall, allowing a lower peak runoff rate. However, if the bulk of the precipitation occurs in the latter portion of the storm, most of the previously mentioned losses will already be satisfied before the time of peak rainfall intensity, resulting in higher peak runoff rates.

There are four storms in the data which fit all of the criteria developed for this study: near to 24-hour duration, at least hourly volume readings, over three inches total volume, and peak occurs near middle of storm (see figures 13 through 16).

Recording Station	Storm Duration (hours)	Total Volume (inches)	Date of Occurrence
Brooksville	23	6.79	9/18/47
Brooksville	24	7.51	11/16/51
Tampa	20	3.90	5/15/76
St. Petersburg	22	5.10	6/17/82

The challenge was to find an equation that reflected the general S shape of these selected storms. A scattergram was made using the dimensionless data points of all the storms. S-shaped curves can be produced from this equation: $y = e^{-\alpha x^{\beta}}$. The data was transformed in several ways in an attempt to produce a linear distribution, thus simplifying the analysis. A double log was taken of the equation, $y = e^{-\alpha x^{\beta}}$ to linearize the resulting plot. This required the data be transformed as follows:

$$y = \ln(-\ln y) \text{ and } x = \ln x$$

where:

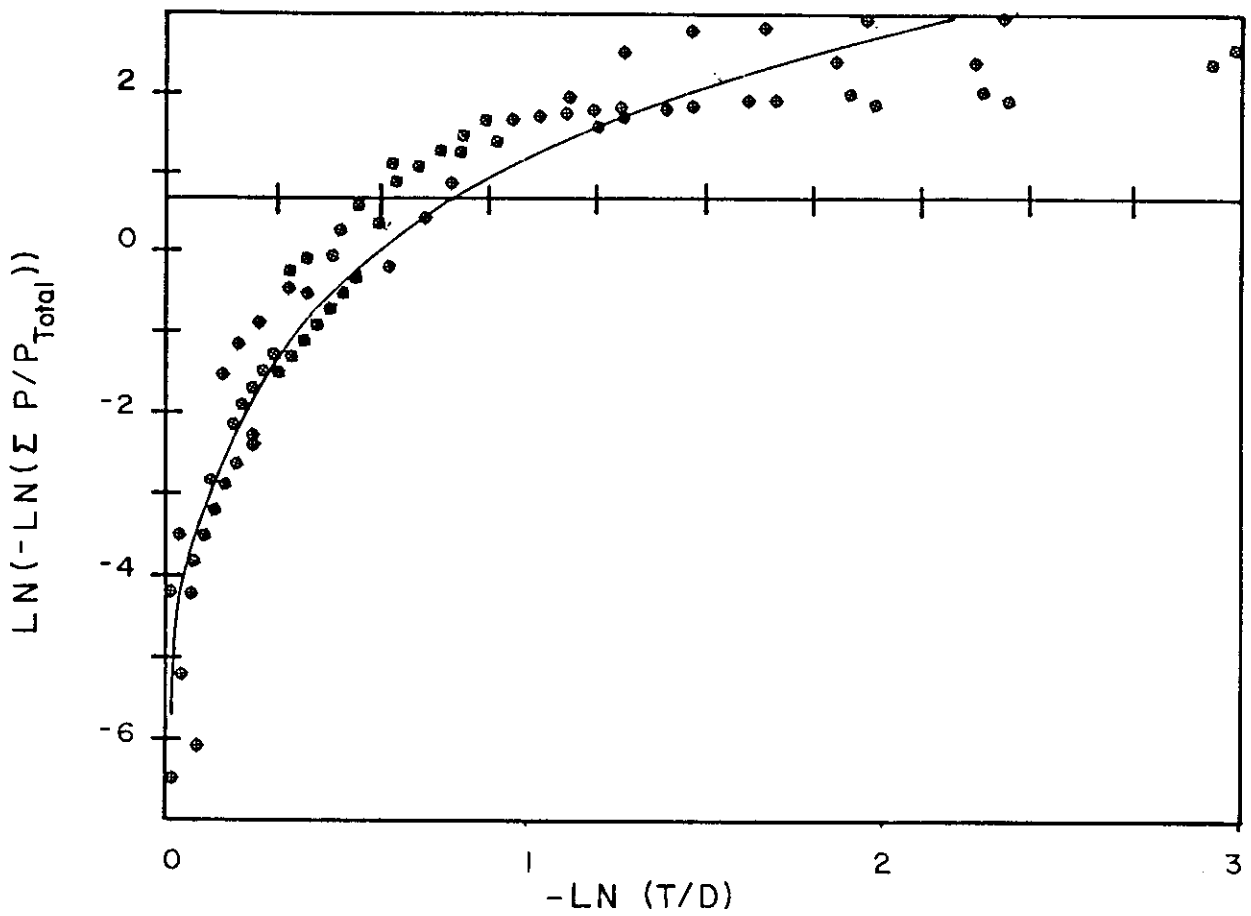
$y = P/P_{total}$	P is precipitation in inches
$x = T/D$	T is elapsed time in hours D is storm duration in hours

The result plotted out resembles a logarithmic equation, however. So, a best fit log equation curve was applied to the transformed data (see Figure 17).

Using an assumed watershed, the log curve was evaluated using the Santa Barbara hydrograph method. The runoff hydrograph generated by the log-curve distribution was compared to these generated by actual storms and other design distributions for a given volume and hypothetical watershed. It was determined from the results that although the log curve showed good overall correlation (R^2 = 0.921), the maximum runoff volume was consistently underestimated (Thompson 1986). Further inspection of the curve gave the answer to this problem. While overall correlation was relatively high, the correlation in the area of greatest influence, time of peak rainfall intensity, was never very close and always a negative error.

Next, the Gumbel Method of curve fitting was tried. The Gumbel Method has been used extensively in statistical analysis of climatological phenomena. It was used by the Weather Bureau in TP 40 an TP 49 and has been used in studies of wind and temperature extremes (South Florida Water Management District TP 81-3). But, this method failed to produce the double curve necessary to match the scattergram S shape. The Weibull distribution also failed to produce the needed shape.

Finally, a polynomial equation was applied to the transformed data. Additional terms were added to the linear equation,



$Y = 0.2614936 + 1.822604 \ln x \quad R^2 = 0.9215023$

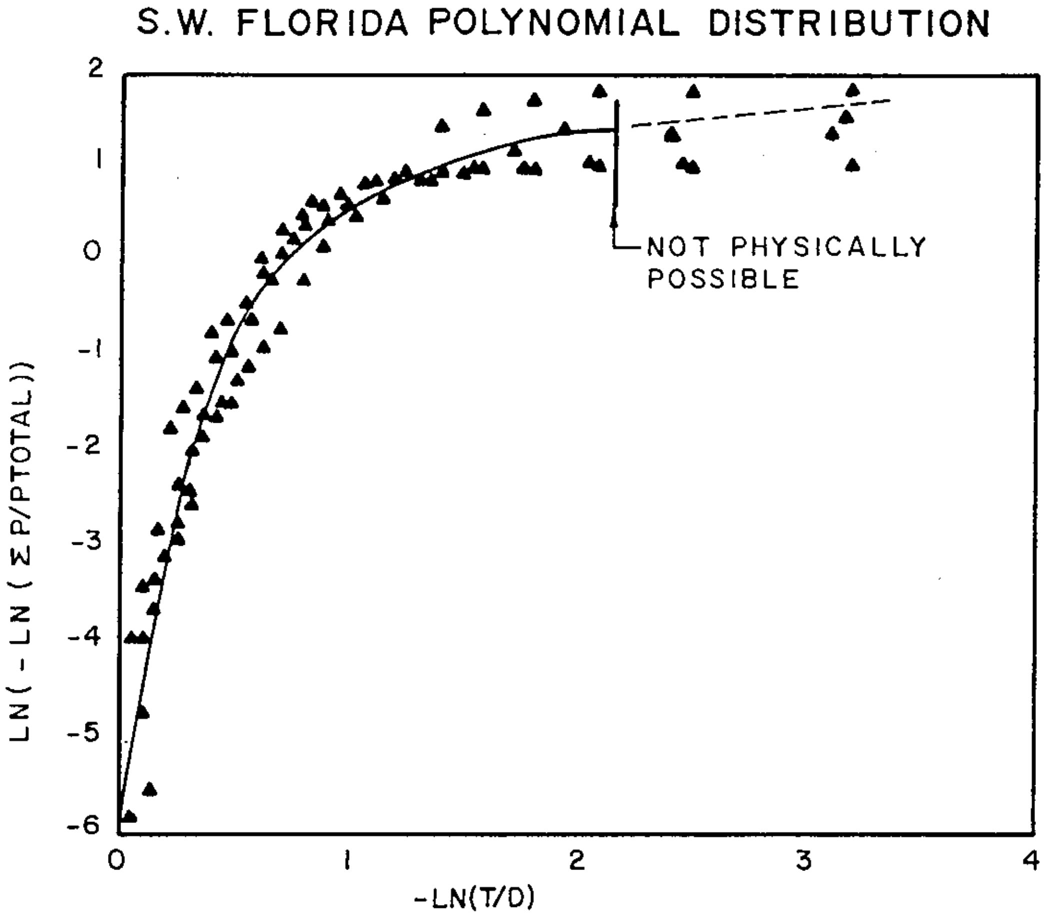
Figure 17. Log equation curve fit.

$y = a + bx$, until an acceptable fit was obtained. The normal equations generated in the solution of the polynomial by the method of least squares were solved on a microcomputer using a program written by Dr. Wanielista (1987). The polynomial, $y = -6.029 + 7.564e\ (x) - 3.603\ e^2\ (x^2) + 8.902\ e^2\ (x^3) - 1.07\ e^3\ (x^4) + 4.886\ e^2\ (x^5)$, provides the best fit. This appears in Figure 18.

The values in the scattergram for the first four hours of the storms vary considerably more than the remainder. In Figure 18, they appear from 2.2 to 3.2 on the x axis in the transformed data plot. Because of this variability, the corresponding rainfall distribution derived from the polynomial equation assumes a constant proportional intensity for the first four hours. The curve generated using this assumption very closely approximates the distributions of the storms used to develop it. The R^2 value for goodness of fit is 0.98.

Once an equation which closely represented the scattergram was obtained, time values stepped in half-hour increments from 0 to 24 hours were inserted into the equation and their corresponding precipitation values determined. These time-precipitation values represent the derived rainfall distribution (see Table 8).

A detailed verification of this distribution was performed as was done for the log-curve distribution. The procedure and results are found in Chapter IX.



THE REGRESSION POLYNOMIAL LINE Y=

$(-6.029E+00) + (7.564E+01) * x + (-3.603E+02) * x^2 + (8.902E+02) * x^3 + (-1.070E+03) * x^4 + (4.886E+02) * x^5$

THE VARIANCE $-2.321E-01$

Figure 18. Polynomial curve fit, Southwest Florida distribution.

TABLE 8

POLYNOMIAL LEAST SQUARES SOUTHWEST FLORIDA
RAINFALL DISTRIBUTION FOR 24 HOURS

HOUR	P/P_{TOTAL}	FRACTION P_{TOTAL}
0.0	0	0
0.5	.006	.006
1.0	.011	.005
1.5	.016	.005
2.0	.021	.005
2.5	.026	.005
3.0	.032	.006
3.5	.037	.005
4.0	.043	.006
4.5	.050	.007
5.0	.057	.007
5.5	.067	.010
6.0	.078	.011
6.5	.093	.015
7.0	.108	.015
7.5	.121	.013
8.0	.132	.011
8.5	.144	.012
9.0	.156	.012
9.5	.168	.012
10.0	.182	.014
10.5	.197	.015
11.0	.216	.019
11.5	.238	.022
12.0	.265	.027
12.5	.296	.031
13.0	.332	.036
13.5	.374	.042
14.0	.421	.047
14.5	.471	.050
15.0	.526	.055
15.5	.583	.057

TABLE 8 -- CONTINUED

HOUR	P/P_{TOTAL}	FRACTION P_{TOTAL}
16.0	.641	.058
16.5	.695	.054
17.0	.747	.052
17.5	.795	.048
18.0	.838	.043
18.5	.875	.037
19.0	.904	.029
19.5	.928	.024
20.0	.948	.020
20.5	.963	.015
21.0	.974	.009
21.5	.982	.008
22.2	.988	.006
22.5	.992	.004
23.0	.995	.003
23.5	.997	.002
24.0	1.000	.003

CHAPTER V

HYDROGRAPH ESTIMATION MODELS

Having reviewed the factors influencing hydrographs and briefly discussing how to determine these factors, this chapter covers the commonly used hydrograph estimation methods. Two of these methods, (1) the Soil Conservation Service Unit Hydrograph and (2) the Santa Barbara Urban Hydrograph, will be used extensively in Chapter IX for determining the peak runoff rate from a hypothetical watershed using the different rainfall distributions already discussed as the dependent variable. In this way, the distributions are compared to one another. By reviewing the procedure used in each estimation method, the benefits of running the comparisons using more than one method become apparent. The methods do not use all of the same input variables in their models, therefore, the relative importance of the rainfall distribution used will vary.

The Rational Method as a Hydrograph Estimator

The Rational Method equation is (Schulz 1973):

$$Q_p = CIA$$

where:

Q_p = peak flow rate in cfs

C = dimensionless runoff coefficient

I = the average rainfall intensity in inches/hour

A = the drainage basin area in acres

whose units are:

cfs = C(in/hr) acre (1 ft/12 in) (43560 ft^2/acre) = (3630 ft^3/hr) or (1.008 ft^3/sec)

This equation can be manipulated easily into a hydrograph estimator, starting with the assumption that the peak flow rate Q_p found with the above equation is the peak of a triangular hydrograph (see Figure 19). Notice that this model has a constant rainfall intensity I and a duration D shown on the rainfall hyetograph. Total rainfall volume can be found by multiplying intensity x area x duration (IAD). The volume of runoff, the lower part of the hyetograph, is equal to CIAD, where C is the runoff coefficient in the Rational Method. This relationship produces units of (in/hr) x acre x hr = in x acre, which is a volume. The upper part of the hyetograph shows the volume of the rainfall infiltrated into the ground or evaporated (1-C) x I x A x D.

From Figure 19, the volume, V_2, of the triangular hydrograph is equal to T_cQ_p, where T_c is the time of concentration of the drainage area. For this model, the time to peak, T_p, is defined

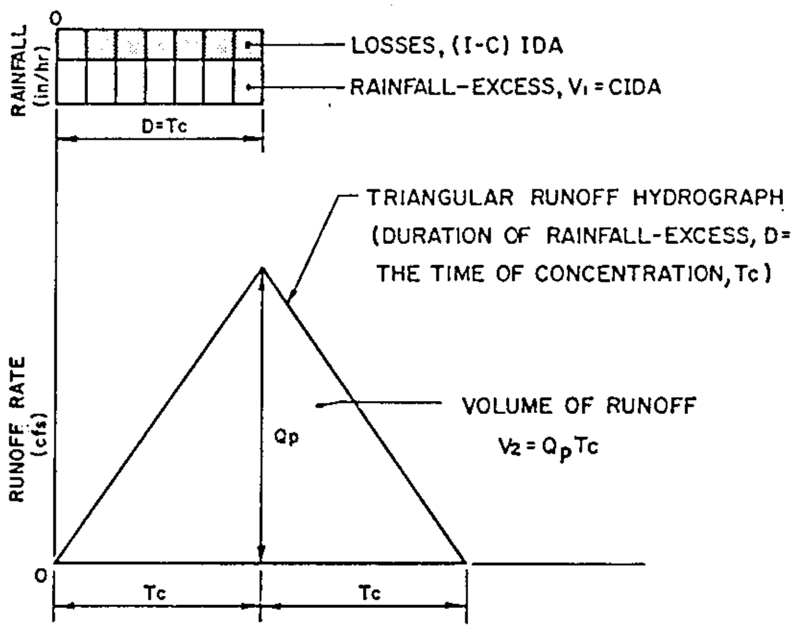


Figure 19. Triangular hydrograph for use with rational method.

as the time from the start of rainfall excess to the peak of the hydrograph and is equal to the time of concentration, T_c.

The volume of runoff, V_1, shown as the lower part of the hyetograph will equal the volume of the triangular hydrograph, V_2. When rainfall duration, D, equals the time of concentration, T_c, of the basin (as Figure 19 indicates), the Rational Method equation can be derived:

Since:

$$V_1 = V_2$$

$$CIAD = Q_p T_c$$

and where

$$D = T_c$$

$$Q_p = CIA$$

So, the peak flow, Q_p, is the peak of an equilateral triangular hydrograph with a base equal to $2T_c$.

The Soil Conservation Service Unit Hydrograph Method

Some of the input variables used in this method, such as runoff curve number (CN) and potential maximum retention (S), require some explanation before the hydrograph estimation method is covered. Also, in order to differentiate between the types of runoff reflected in the method's results, this section begins with definitions followed by the theoretical basis for the Unit Hydrograph Method.

Definitions

This method calculates the peak flow rate of direct runoff. The rainfall reaching the ground surface can result in the following:

1. Surface Runoff: occurs only when the rainfall rate is greater than the infiltration rate. The runoff flows on the watershed surface to the point of reference. This type appears in the hydrograph after the initial demands of interception, infiltration, and surface storage have been satisfied. It varies during the storm and ends during or soon after it.

2. Subsurface Flow: occurs when infiltrated rainfall meets an underground zone of low transmission, travels above the zone to the soil surface downhill, and appears as a seep or spring. This type is often called quick return flow because it appears in the hydrograph during or soon after the storm.

3. Base Flow: occurs when there is a fairly steady flow from natural storage. The flow comes from lakes or swamps, or from an aquifer replenished by infiltrated rainfall. See Appendix II for more on base flow.

The SCS Method combines surface and subsurface flow in its calculations, calling the result direct runoff. Some authors (SCS TR-55) refer to effective rainfall as that portion of the storm volume that results in direct runoff. Using these concepts, the SCS developed their method through the following derivation.

Theory

For a simple rainfall model where initial abstraction is ignored, this equation holds:

$$F/S = Q/P \tag{12}$$

where:

F = actual retention after runoff begins

S = potential maximum retention after runoff begins ($S \geq F$)

Q = actual runoff

P = rainfall ($P \geq Q$)

The retention, S, is a constant for a particular storm because it is the maximum possible. The retention, F, varies because it is the difference between P and Q at any point.

$$F = P - Q \text{ (by definition)} \tag{13}$$

Substituting into equation (13)

$$(P - Q)/S = Q/P \tag{14}$$

Solving for Q:

$$P^2 - PQ = QS$$

$$P^2 = PQ + QS$$

$$P^2/Q = P + S$$

$$Q = P^2/(P + S) \tag{15}$$

Next, consider an initial abstraction (I_a) greater than zero, the amount of rainfall available for runoff is $P - I_a$. Substituting this into equation (12) yields:

$$F/S = Q/(P - I_a) \tag{16}$$

and the total retention for a storm consists of I_a and F. The total maximum retention consists of I_a and S. Continuing with the substitutions:

$$F = (P - I_a) - Q \tag{17}$$

$$\frac{(P - I_a) - Q}{S} = \frac{Q}{(P - I_a)} \tag{18}$$

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{19}$$

This is the rainfall-runoff relation with the initial abstraction taken into account.

This initial abstraction is made up of interception, infiltration, and surface storage, which all occur before runoff begins. The SCS developed a relationship between I_a and S using rainfall and runoff data from experimental small watersheds. The empirical relationship is:

$$I_a = 0.2\ S \tag{20}$$

Substituting into equation (19) gives:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{21}$$

This is the rainfall-runoff relation used in the SCS method of estimating direct runoff from storm rainfall.

To show the rainfall-runoff relationships graphically, S values are transformed into curve numbers (CN) by the following equation:

$$CN = 1000/(S + 10) \tag{22}$$

rearranging:

$$S = (1000/CN) - 10 \tag{23}$$

The SCS has developed, through research on experimental watersheds, a table of curve numbers for different land uses (see Table 9). The curve numbers further vary by what soil classification occurs with the land use. Hydrologic Group A soils have a high infiltration rate even when thoroughly wet. Group B soils have a moderate infiltration rate, Group C soils have a slow infiltration rate, and Group D soils have a very slow infiltration rate. The last section of this chapter discusses runoff curve numbers in more detail.

The SCS Method of Hydrograph Generation

The SCS method uses the triangular unit hydrograph concept (see Figure 20), where:

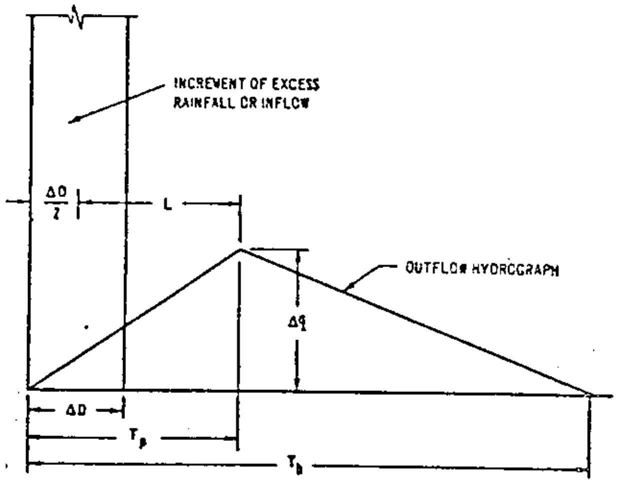
ΔD = increment of storm in hours

ΔQ = runoff in inches during period, ΔD

Δq = peak discharge in cfs for an increment of runoff

A = drainage area in square miles

T_p = time to peak $[(\Delta D/2) + L]$ in hours



$$\Delta q = \frac{484\ A(\Delta Q)}{\frac{\Delta D}{2} + L} \text{ in C.F.S.}$$

Where:

ΔD = INCREMENT OF STORM PERIOD IN HOURS

ΔQ = RUNOFF IN INCHES DURING PERIOD ΔD

Δq = PEAK DISCHARGE IN C.F.S. FOR AN INCREMENT OF RUNOFF

A = DRAINAGE AREA IN SQUARE MILES

T_p = TIME TO PEAK $\left(= \frac{\Delta D}{2} + L\right)$ IN HOURS

T_b = TIME OF BASE $(= 2.67\ T_p)$ IN HOURS

Figure 20. Triangular hydrograph relationships (Kent 1973).

T_b = time of base (= 2.67 T_p) in hours

$T_r = T_b - T_p$

From Figure 20, the total volume under the triangular unit hydrograph is:

$$Q = \frac{q_p T_p}{2} + \frac{q_p T_r}{2} = \frac{q_p}{2}(T_p + T_r) \tag{24}$$

Solving for peak rate, q_p, in inches per hour:

$$q_p = \frac{2Q}{T_p + T_r} \tag{25}$$

Let:

$$K = \frac{2}{1 + \frac{T_r}{T_p}} \tag{26}$$

Now:

$$q_p = \frac{KQ}{T_p} \tag{27}$$

In making the conversion from inches per hour to cubic feet per second and putting the equation in terms ordinarily used, equation (27) becomes the general equation:

$$q_p = \frac{645.33 \times K \times A \times Q}{T_p} \tag{28}$$

where q_p is peak discharge in cubic feet per second (cfs) and 645.33 is the conversion factor to discharge one inch from one square mile in one hour.

The relationship of the triangular unit hydrograph, $T_r = 1.67\ T_p$, gives K = 0.75 using equation (26). Substituting into equation (28) gives:

$$q_p = \frac{484\ AQ}{T_p} \tag{29}$$

Since the volume under the rising side of the triangular unit hydrograph is equal to the volume under the rising side of the curvilinear dimensionless hydrograph, the constant K = 484 or peak rate factor is valid for the dimensionless unit hydrograph in Figure 21.

Incremental Hydrographs

Storm rainfall does not occur uniformly over the duration of the storm event. To use equation (29) for non-uniform rainfalls, it is necessary to divide the storm into increments of duration (ΔD) and compute the increments of runoff (Δq). The peak discharge equation for an increment of runoff is (Kent 1973):

$$\Delta q_p = \frac{484\ A(\Delta Q)}{\frac{\Delta D}{2} + L} \tag{30}$$

where D and L are in hours.

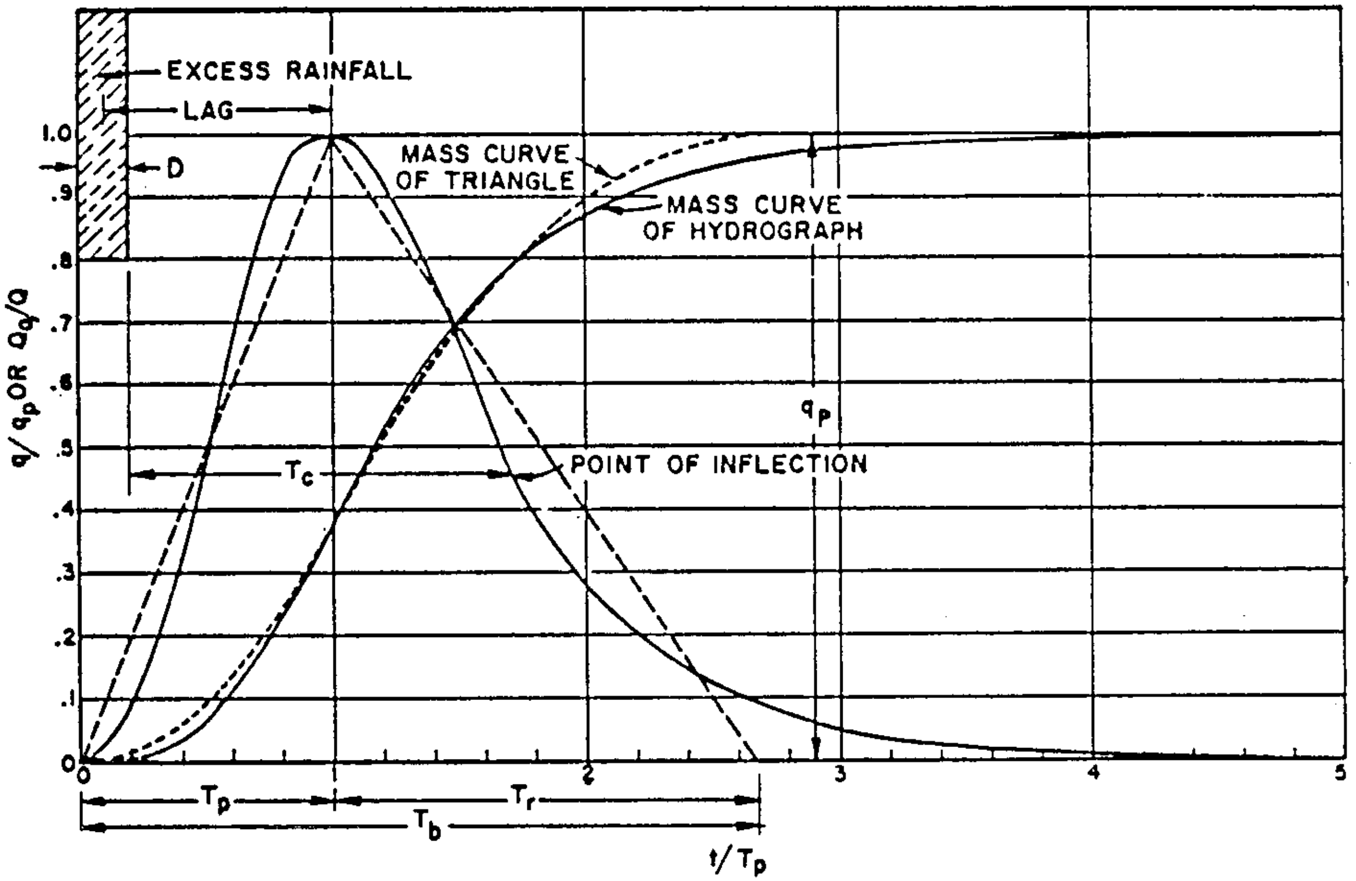


Figure 21. Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph (USDA-SCS 1972).

Equation (30) is applied to each increment of (ΔD) for the storm duration or period of interest. The ordinates of the individual triangular hydrographs for each Δq_r are then added to develop a composite hydrograph. Note that each incremental hydrograph must be displaced one (ΔD) to the right for each succeeding time increment prior to adding to form the composite hydrograph.

The Santa Barbara Urban Hydrograph Method

The Santa Barbara Urban Hydrograph (SBUH) Method computes a hydrograph directly without going through an intermediate process like the SCS unit hydrograph method (Stubchaer 1975). In the SBUH Method, the final outflow hydrograph is obtained by routing the instantaneous hydrograph for each time period through an imaginary linear reservoir. The imaginary reservoir has a routing constant equal to the time of concentration of the watershed. The following procedure describes how the method works.

Runoff depths for each time period are calculated using the following equations:

$R(0) = IP(\Delta t)$ inches: Impervious Area Runoff

$R(1) = (1-I)[P(\Delta t)-f(\Delta t)]$ inches: Pervious Area Runoff

$R(\Delta t) = R(0) + R(1)$ inches: Total Runoff Depth

where:

- I = impervious portion of watershed, fraction
- $P(t)$ = rainfall depth during time increment, Δt, inches
- Δt = incremental time period, hour
- f = infiltration during time increment, Δt, inches
- R = runoff

The impervious portion (I) of the watershed is the directly connected impervious area. The infiltration rate will decrease from a maximum initial rate (inches/hour) as the soil voids fill with water. A final (constant) infiltration rate is used when curve numbers are input into the computer program for this method. Some computer programs allow the infiltration rate to vary by using the Horton Equation in calculating f.

$$\text{Horton's Equation:} \quad f = f_c + (f_o - f_c)e^{-kT}$$

where:

- f = infiltration rate at some time, t, after the start of rainfall, inches per hour
- f_o = initial infiltration rate, inches per hour
- f_c = final infiltration rate, inches per hour
- K = a recession factor, site dependent

The instantaneous hydrograph is then computed by multiplying the total runoff depth R for each time period (t) by the drainage basin area (A) in acres, and dividing by the time increment (t) in hours. The result is the instantaneous inflow (I).

$$I(\Delta t) = R(\Delta t)\ (A/\Delta t)\ ft^3/sec$$

The final outflow hydrograph, $Q(\Delta t)$, is calculated by routing the instantaneous hydrograph, $I(\Delta t)$, through an imaginary reservoir.

$$Q(2) = Q(1) + K[I(1) + I(2) - 2Q(1)]$$

where:

K = $[\Delta t/(2T_c + t)$, empirically derived routing constant

T_c = watershed time of concentration

$I(1)$ = instantaneous flow, $t = t$

$I(2)$ = instantaneous flow, $t = t + \Delta t$

$Q(1)$ = outflow, $t = t$

$Q(2)$ = outflow, $t = t + \Delta t$

This model is particularly sensitive to the time of concentration. The time of concentration is that time required for all parts of the watershed to contribute flow to the point of discharge.

For systems with a high intensity rainfall occurring for a short period of time, as the time of concentration increases, the peak outflow decreases. Systems with longer duration rainfall intensities do not show as pronounced a variation in outflow as time of concentration values are varied. This is a factor to consider when matching a design rainfall distribution to a particular watershed to be modeled with the SBUH method.

Curve Numbers

At this point, some additional background information concerning runoff curve numbers will be useful. Both the Soil Conservation Service Method and the Santa Barbara Method use the concept of Runoff Curve Numbers in their method of hydrograph generation. The selection of the proper CN is critical to the validity of the resulting hydrograph.

The combination of the hydrologic soil group and the land use/treatment class form a hydrologic soil-cover complex. The SCS has done field experiments to determine the CN associated with each complex. This CN indicates the runoff potential of a complex. The higher the CN value, the higher the runoff potential.

The SCS experiments on the various soil-cover complexes involved calculating the average CN for small watersheds (generally under 1 square mile). Rainfall-runoff data was collected for storm durations of one day and under. The storms found to produce runoff equivalent to the region's annual flood were used in the calculation.

An important factor to the runoff potential of a watershed independent of soil-cover is the antecedent moisture condition. The antecedent moisture condition of a soil depends on the watershed's recent weather. The SCS uses three levels of antecedent moisture condition (AMC):

AMC I: for dry, lowest runoff potential. Soils in the watershed are dry enough for plowing or cultivation.

AMC II: the average condition

AMC III: highest runoff potential. Soils in the watershed are practically saturated from antecedent rains

The CN tables published by the SCS (USDA-SCS 1972) most often contain values for the various soil-cover complexes assuming an AMC of II (see Table 9). When this assumption does not apply to the watershed being modeled, a conversion table is available to make the adjustment (see Table 10).

TABLE 9

SCS CURVE NUMBERS FOR HYDROLOGIC SOIL-COVER COMPLEXES

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land[1/]: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover[2/]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious).	81	88	91	93
Residential:[3/]				
Average lot size — Average % Impervious[4/]				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

1/ For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

2/ Good cover is protected from grazing and litter and brush cover soil.

3/ Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

4/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

TABLE 10

CURVE NUMBERS FOR ANTECEDENT MOISTURE CONDITIONS I AND III

CN FOR AMC III	CORRESPONDING CNs	
	AMC I	AMC III
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
20	9	37
15	6	30
10	4	22
5	2	13

SOURCE: SCS TP-149 (1973)

CHAPTER VI

PROCEDURE USED TO CALCULATE PEAK RATE FACTORS

SCS Equation Derivation

To work with the Soil Conservation Service Unit Hydrograph method, a shape factor is required. This factor is also called a peak rate factor (K) because it influences peak runoff rates. The standard SCS unit hydrograph shape was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographical location. This dimensionless curvilinear hydrograph (Figure 21) has its ordinate values expressed in a dimensionless ratio, q/q_r, and its abscissa values as t/T_p. It has a point of inflection 1.67 times the time to peak (T_p) and the time to peak 3/8 of the time of base (T_b). This sets 37.5% of the total volume of runoff under the rising side of the hydrograph (USDA-SCS 1972). Letting the rising side represent one unit of time and one unit of discharge, the curvilinear hydrograph can be represented by an isosceles triangle hydrograph. This allows the base of the triangle to be solved in relation to the time to peak. Solving for the base length of the triangle, if one unit of time, T_p, equals .375 of volume:

$$T_b = 1/.375 = 2.67 \text{ units of time}$$

$$T_r = T_b - T_p = 1.67 \text{ units of time or } 1.67\ T_p$$

These relations will now be used to develop the peak rate equation for use in the unit hydrograph. Still referring to Figure 21, the total volume under the triangular unit hydrograph is:

$$Q = \frac{q_p T_p}{2} + \frac{q_p T_r}{2} = \frac{q_p}{2} (T_p + T_r)$$

where:

Q = runoff in inches

T_p = time to peak in hours

T_r = time to return in hours

q_p = peak runoff rate in inches/hour

$$q_p = \frac{2Q}{T_p + T_r}$$

Let:

$$K = \frac{2}{1 + \frac{T_r}{T_p}}$$

So:

$$q_p = \frac{KQ}{T_p}$$

When converting from inches per hour to cubic feet per second and putting the equation in terms commonly used, with drainage area (A) in square miles and time (T) in hours, the equation becomes:

$$q_p = \frac{645.33\ (K)(A)(Q)}{T_p}$$

where q_p is peak discharge in ft^3/sec, and the conversion factor, 645.33, is the rate required to discharge one inch from one square mile in one hour. The relationship of the triangular unit hydrograph, $T_r = 1.67\ T_p$, gives K = 0.75 and 645.33 x 0.75 = 484. Substituting:

$$q_p = \frac{484\ AQ}{T_p}$$

so the derivation of the peak rate factor using the "standard" hydrograph shape yields K = 484.

Data Analysis/Criteria

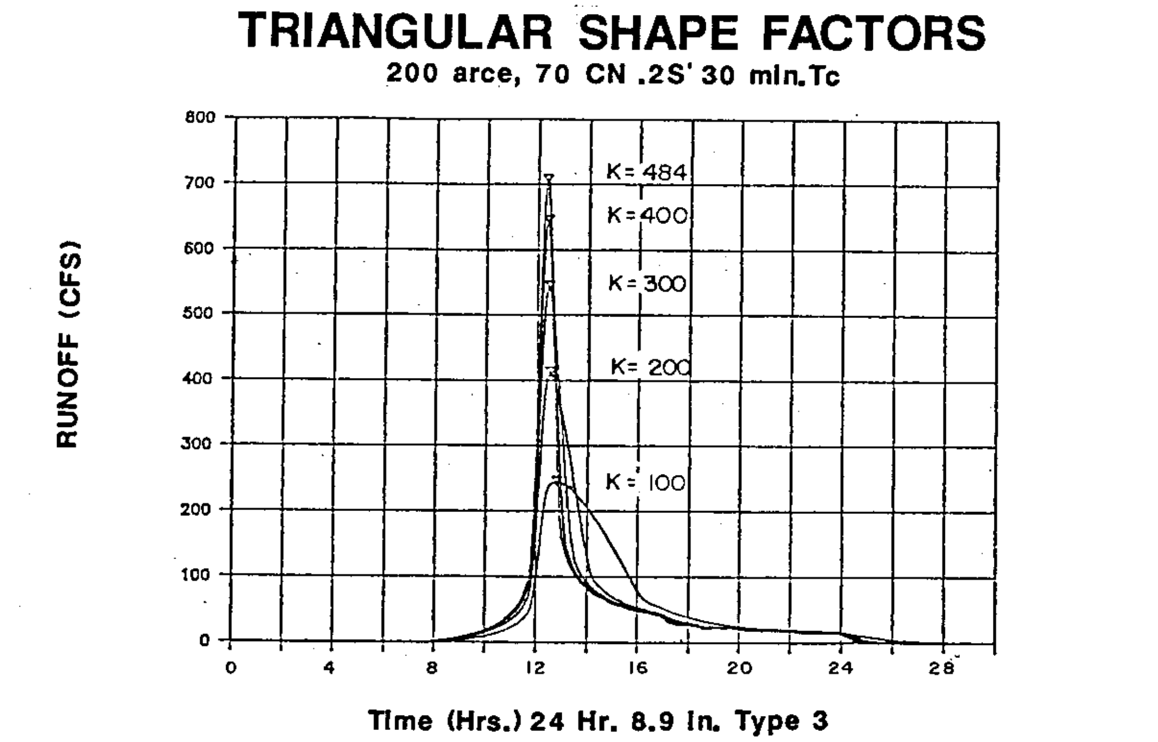
Any change in the dimensionless unit hydrograph which corresponds to a change in the percent of volume under the rising side will cause a change in the shape factor. A study of the hydrographs from watersheds in the west Central Florida area indicates that the standard shape factor does not apply. It is important to select the correct shape factor to accurately approximate the peak runoff from a watershed for hydraulic structure design.

Figure 22 (Vomacka 1986) shows the resulting runoff peaks for a hypothetical watershed when K factors from 100 to 484 are inserted into the unit hydrograph equation. There is a marked increase in peak runoff rate with increasing K factor.

Determining what peak rate factor to use when modeling a watershed requires knowledge of the time to peak and the peak runoff rate. Several factors affect these values, including watershed topography, storage, routing of the runoff, and the characteristics of the storm producing the rainfall excess.

The best way to determine what K factor to use for a watershed is to use field determined time-flow measurements to find T_p and q_p, using the equation, $q_p = KAQ/T_p$. By using field data, all of the variables entering into the calculation of K are included.

This study uses the stream flow records kept by the U.S. Geological Survey for two watersheds in west Central Florida, Hickory Creek and Gallagher Ditch. The USGS set up both recording rain gauges and stream flow gauges in these areas. The Hickory Creek watershed discharge was monitored at 15-minute intervals from February 12, 1982, through September 30, 1984 (USGS 1986). The Gallagher Ditch watershed discharge was monitored at 5-minute intervals from April 1982 through September 1984. The data



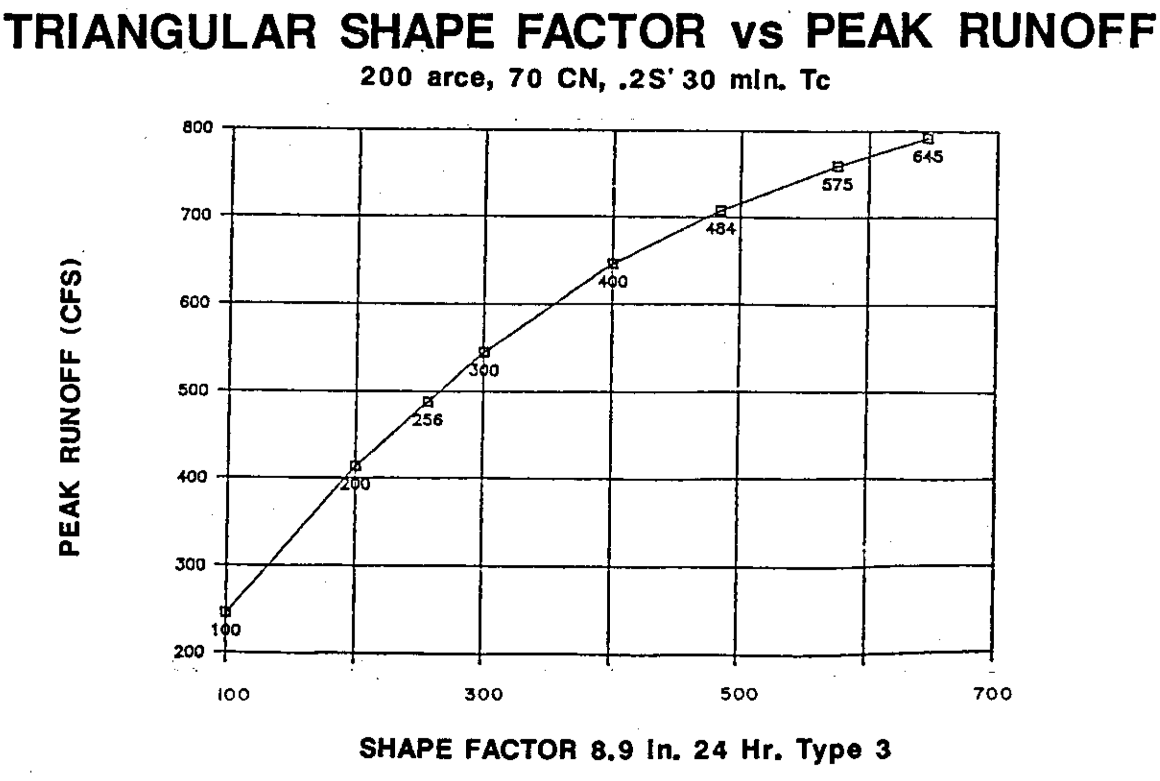


Figure 22. Triangular shape factors (Vomacka 1986).

supplied by the USGS for this study came in the form of four reels of computer tape.

The University of Central Florida computer department read the data for the two watersheds and printed it on hard copy. The data lists the gauge station number, date by year-month-day-hour-minute, and the period's peak stream flow value in cubic feet per second.

The first task in data analysis was to scan the printouts for flow peaks, making a list of the peak values and their date of occurrence. The peak flows were selected because the resulting K factor will be used to model watersheds for design storms. The data points associated with the rising and receding limbs of the peaks were plotted out to form hydrographs. Two criteria were applied to these hydrographs to aid in the selection of the events best suited for analysis. First chosen were hydrographs that had a well defined base flow preceding the rising limb and following the receding limb. This is important because the time to peak and time of recession used to calculate the K factor apply to direct runoff only. The base flow must be removed before these times are measured. Well defined base flow can be separated out with a high degree of confidence that no direct runoff is removed. Three methods are in the literature on how to separate out base flow (see Appendix II). After trying each one, the straight line method was selected because it consistently gave results that compared well with the plots. An example of the straight line method is included in the following section containing the hydrographs and calculations.

The second criteria used to screen the hydrographs was that they have a single peak. Often a second rainfall would occur before the surface runoff from a preceding rainfall had completely passed by the gauge. The SCS method used in this study is not capable of separating the combined flows, so double peaked hydrographs were dropped. Because each watershed also had rainfall data available, these data were used to help determine the causes for irregularities in the hydrographs. By plotting out both gauge readings for isolated events, a clear correlation was seen between storm variations and hydrograph variations. Another complication found in the hydrographs were humps on the rising or receding limbs. These were found from the corresponding hyetographs to be due to surges in the rainfall intensity occurring shortly before or after the major period of rainfall within a storm.

The stream flow data consists of readings every 15 minutes. The hydrographs were plotted using the values that fell on the even hours. For critical sections of the hydrograph, such as the start of the rise or peak flow, every quarter-hour value was plotted to increase shape definition.

Often the peak flow value occurred for longer than a single 15-minute reading. In this case, when the peak was flat and occurred for an hour or longer, it was a matter of judgement of where to draw the line between the rising and receding limbs. The peak was estimated as the center of the flat maximum flow.

Once the hydrographs meeting the criteria were identified, the next step was to calculate the total stormwater runoff (Q, in inches) by adding the area under each curve. The straight line method was used to remove the base flow from the hydrographs before calculating their volume of direct runoff. Adding the area under each curve results in a volume with units of cubic feet. This must be converted to inches over the watershed for use in the peak rate equation, $q_p = KAQ/T_p$.

So, from the plotted hydrograph, q_p, Q, and T_p can be found. Also, the watershed area, A, can be determined from a topographic map. With these values known, K can be calculated, as previously mentioned, $K = (q_p \; T_p)/AQ$.

Factors Affecting Peak Rate Factor

The storm duration is an important factor affecting the time to peak which, in turn, directly influences the calculated peak rate factor, K, for the runoff hydrograph (see Figure 1). For example, when a light rainfall persists for several hours, the corresponding hydrograph will at first show no runoff (unless the soil is saturated from a recent previous storm). As time passes, sufficient rain will fall to saturate the soil so a small runoff begins. As more and more of the watershed starts to allow all of this light rainfall to become direct runoff, the stream flow increases to a peak value. The peak rate factor calculated from this type of storm-hydrograph will be higher than for the same volume of rain that occurs in a more intense storm. The difference is the time to peak inserted into $q_p = KAQ/T_p$.

CHAPTER VII

HICKORY CREEK WATERSHED

Description of the Hickory Creek Watershed

Hickory Creek is located 2.4 miles east of Ona, Florida, in Hardee County (see figures 23 and 24). This watershed is in the Peace River drainage basin. Hickory Creek joins the Peace River as a tributary 5.5 miles downstream from Zolfo Springs. The portion of Hickory Creek considered in this study begins 4.5 miles upstream from the confluence with the Peace River.

Specifically, it is located in the NE quarter of Section 35, Township 34S, Range 24E. The watershed area is 3.75 square miles (2400 acres), as shown in Figure 25. The USGS water stage recording gauge is located on the downstream side of the culvert under state highway 64.

The land use of the watershed is predominantly rural, with scattered farm structures and homes. Approximately 10% of the land area is swamp. The drainage type is dendritic with no large lakes.

The Hickory Creek drainage basin is a low lying swamp area. It has a time of concentration of 24 hours. Long duration storms result in peak runoff flows within 7 to 9 hours. The recession limbs extend for 3 to 5 days, however. Because of this slow

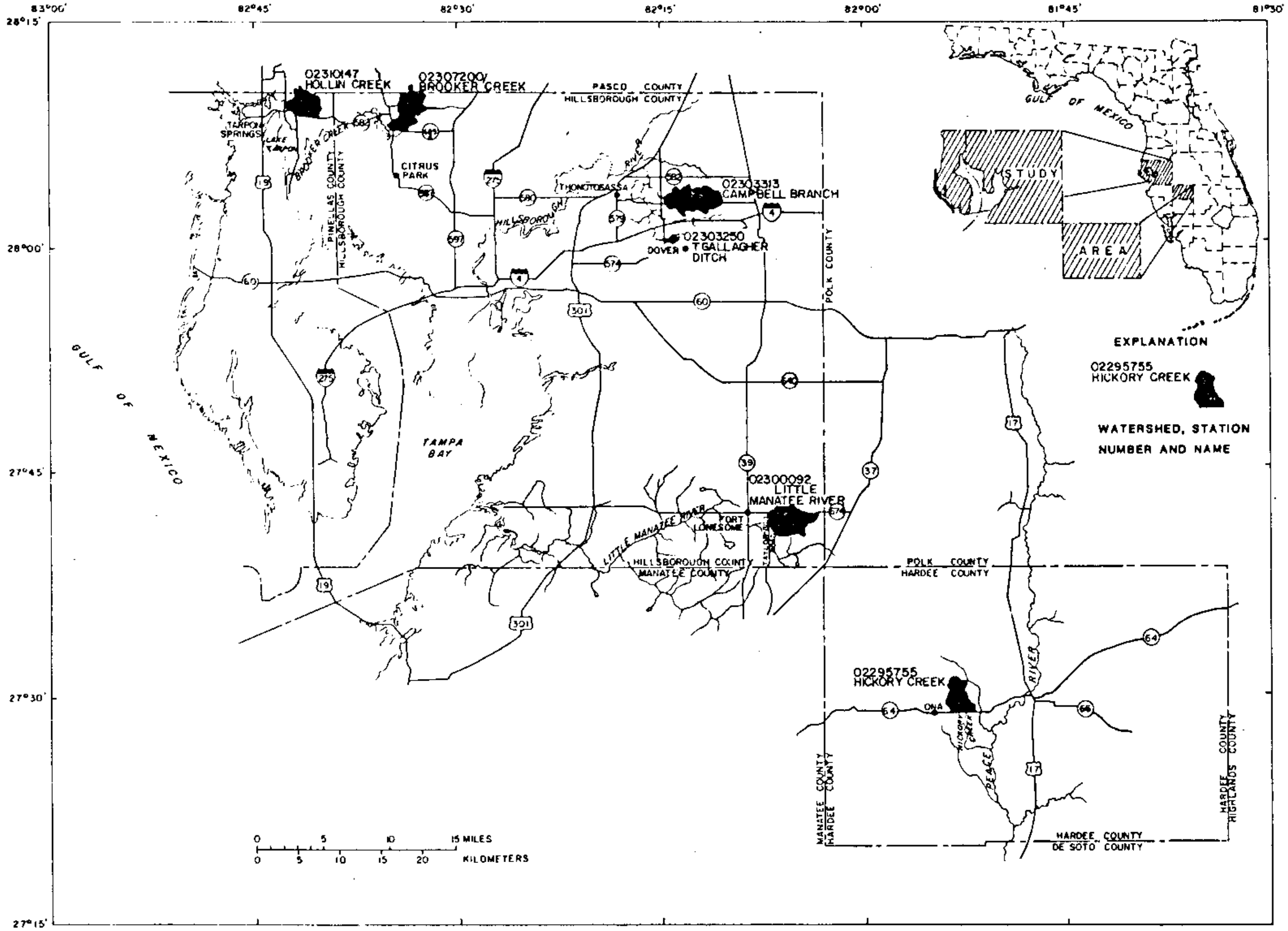


Figure 23. Location of Hickory Creek watershed (USGS 1986).

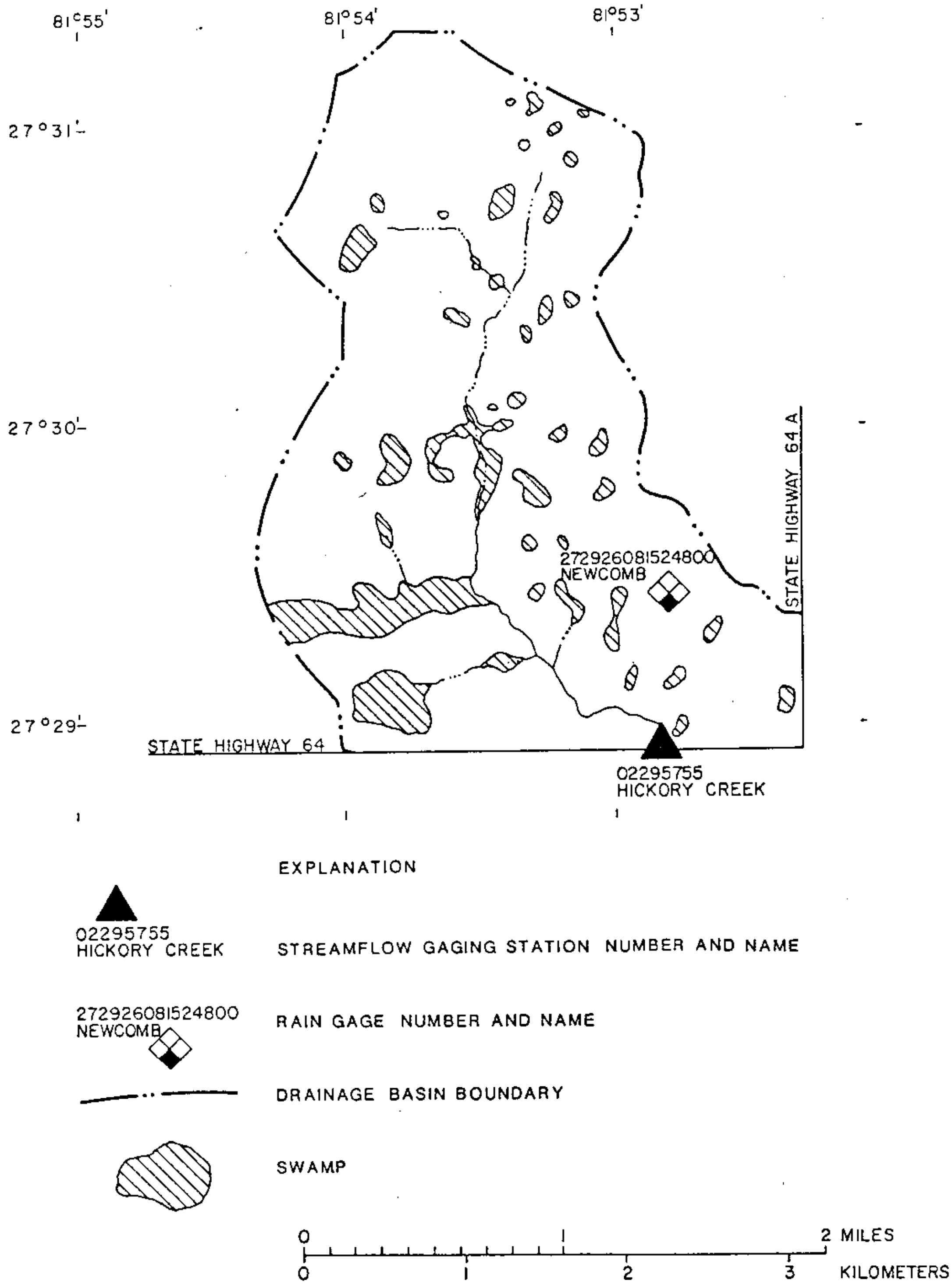


Figure 24. Hickory Creek watershed (USGS 1986).

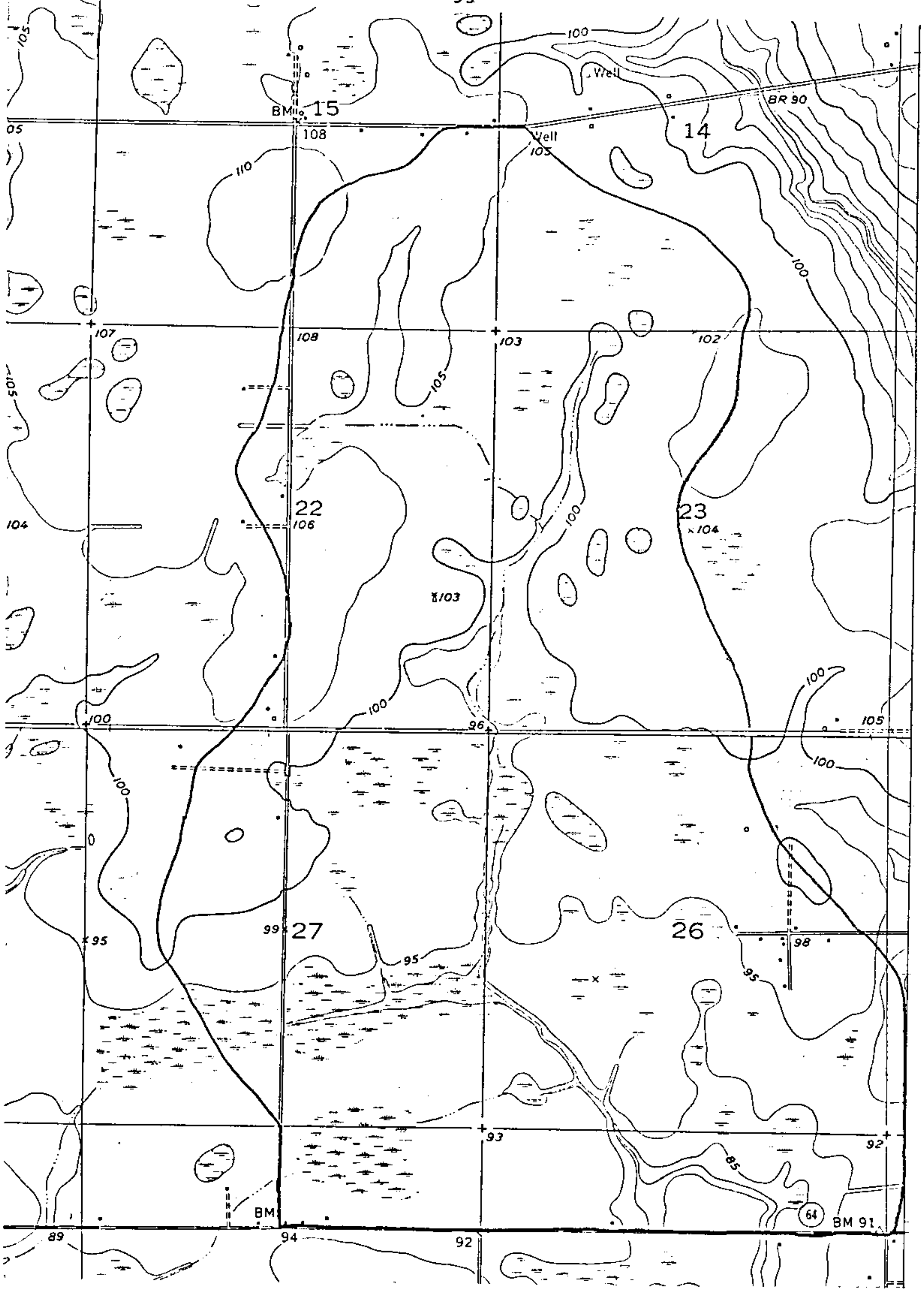


Figure 25. USGS Quadrangle with Hickory Creek watershed highlighted.

return to the no-flow condition (Figure 26), the flows that were analyzed for this study usually had some amount of existing stream flow when the storm began. The straight line method was used to remove this residual flow from the storm's direct runoff measured by the flow gauge. See Appendix II for more on base flow separation.

By following the procedures and criteria covered in Chapter VI, the data set produced seven hydrographs suitable for analysis of peak rate factors. Table 11 lists the date, characteristics and calculated "K" associated with each hydrograph. Figures 27 through 33 present the seven Hickory Creek hydrographs and the accompanying calculations of "K".

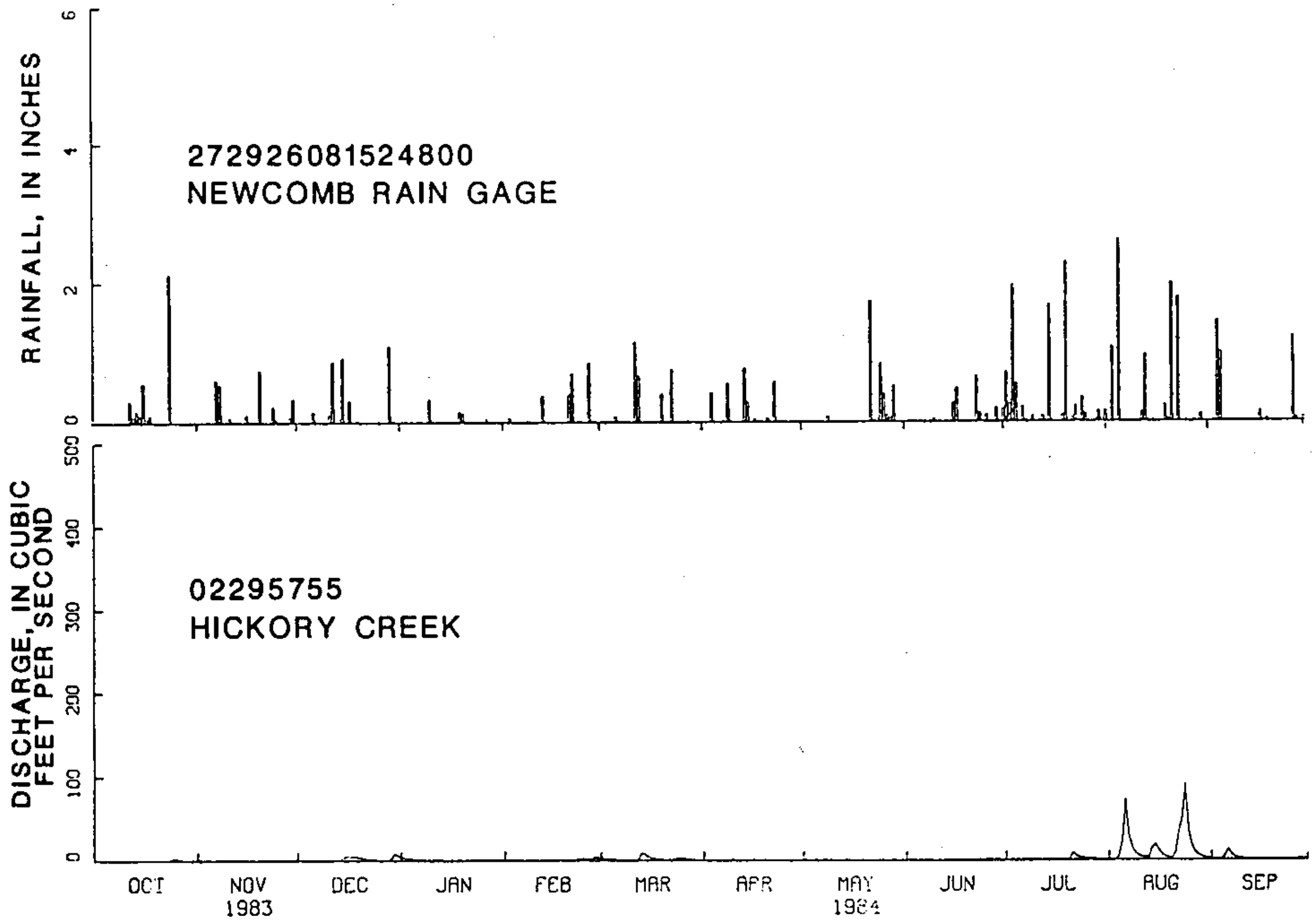


Figure 26. Daily discharge and rainfall for the Hickory Creek watershed (USGS 1986).

TABLE 11

HICKORY CREEK PEAK RATE FACTORS (K)

STORM DATE	RUNOFF VOLUME (Q inches)	PEAK RUNOFF* (q_p cfs)	TIME TO PEAK (T_p hrs)	RETURN TIME (T_R hrs)	PEAK RATE FACTOR (K)
10/05/82	3.54	312	8	99	188
02/13/83	0.87	65	12	83	240
03/08/83	1.03	71	14	126	257
03/24/83	0.89	89	9	68	237
03/28/83	0.80	73	10	72	237
08/05/84	1.24	102	9	99	197
08/24/84	0.87	128	8	105	215

Average Peak Rate Factor: K = 224

* Adjusted for base flow

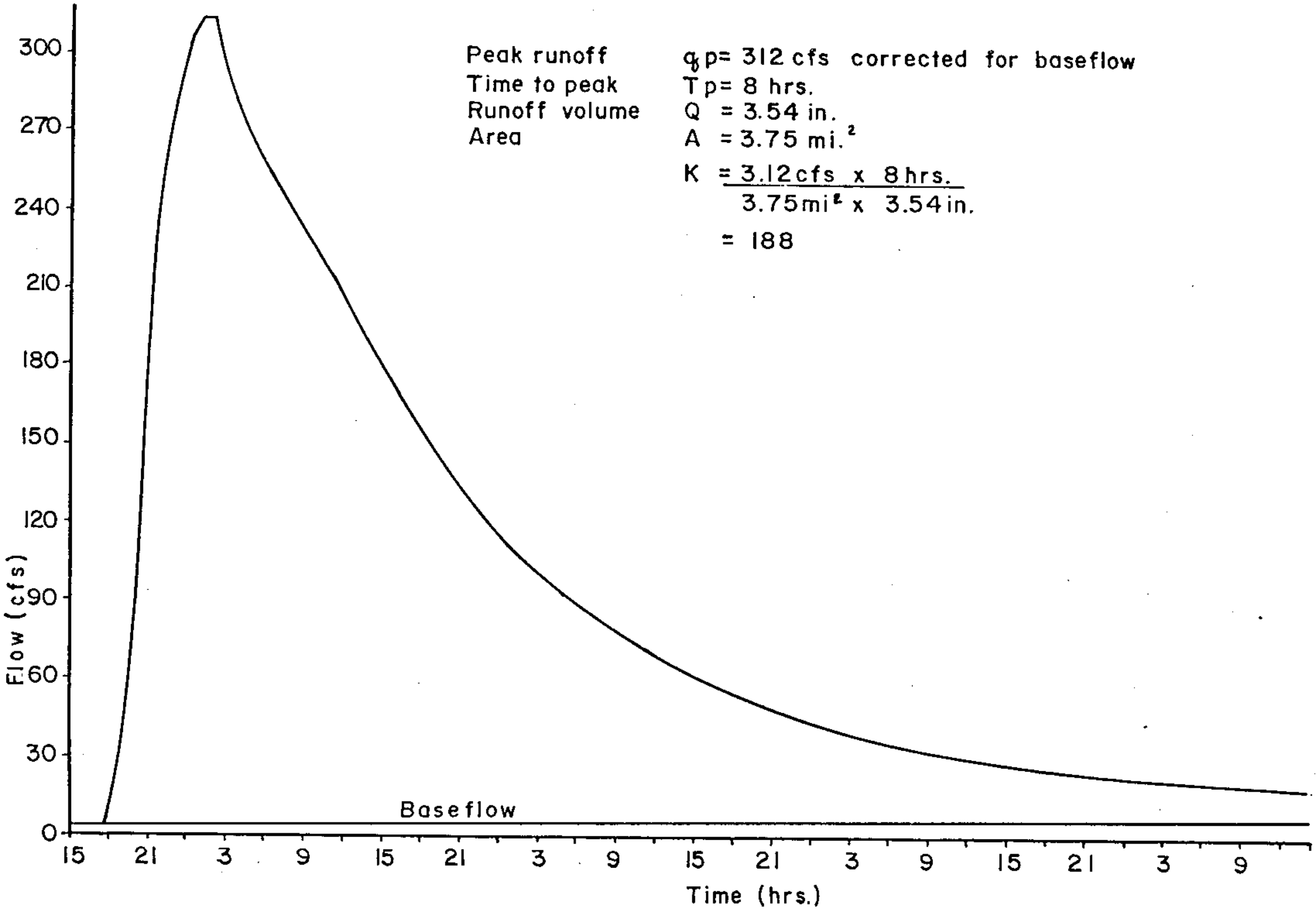


Figure 27. Hickory Creek hydrograph for 10/5/82.

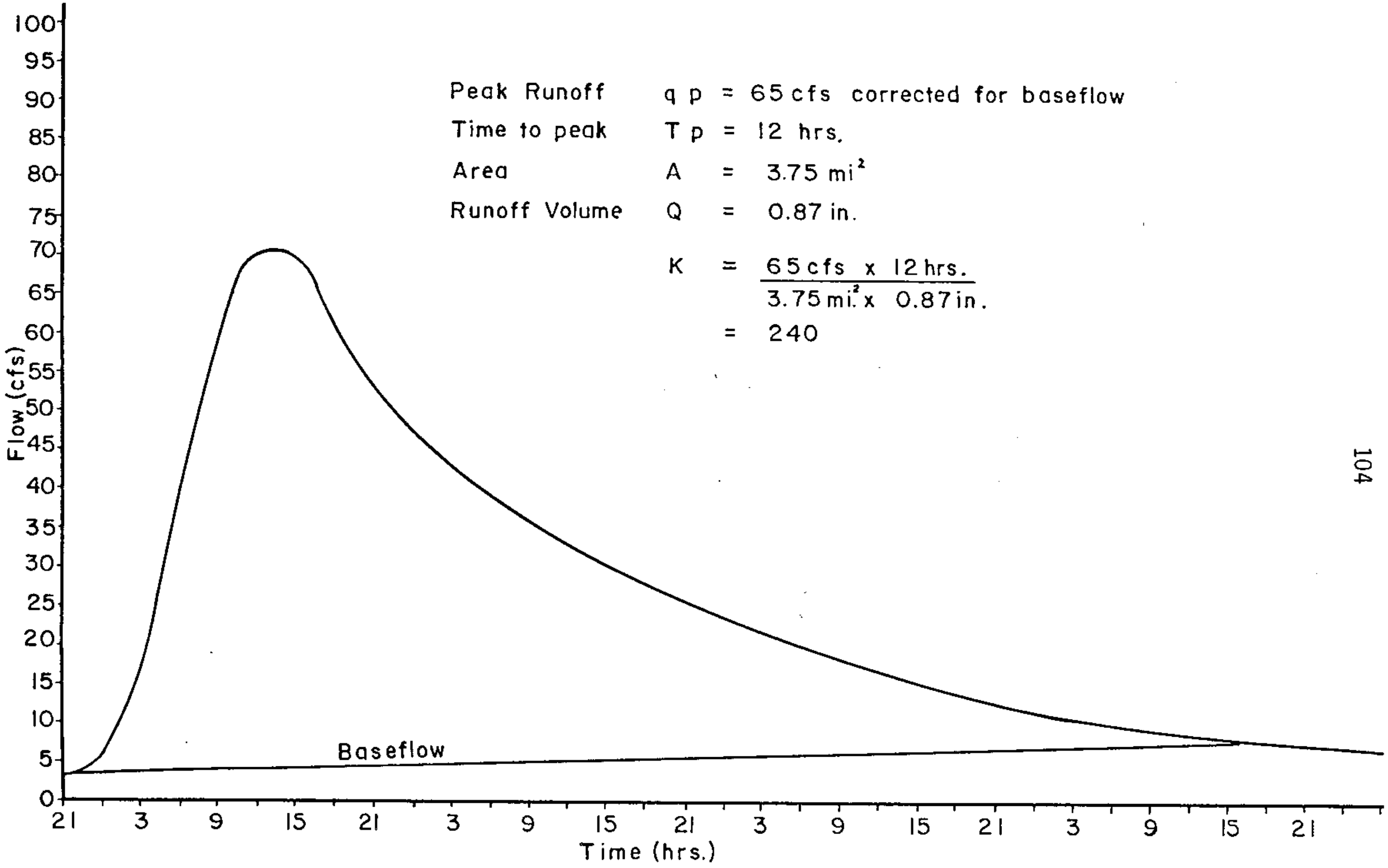


Figure 28. Hickory Creek hydrograph for 2/13/83.

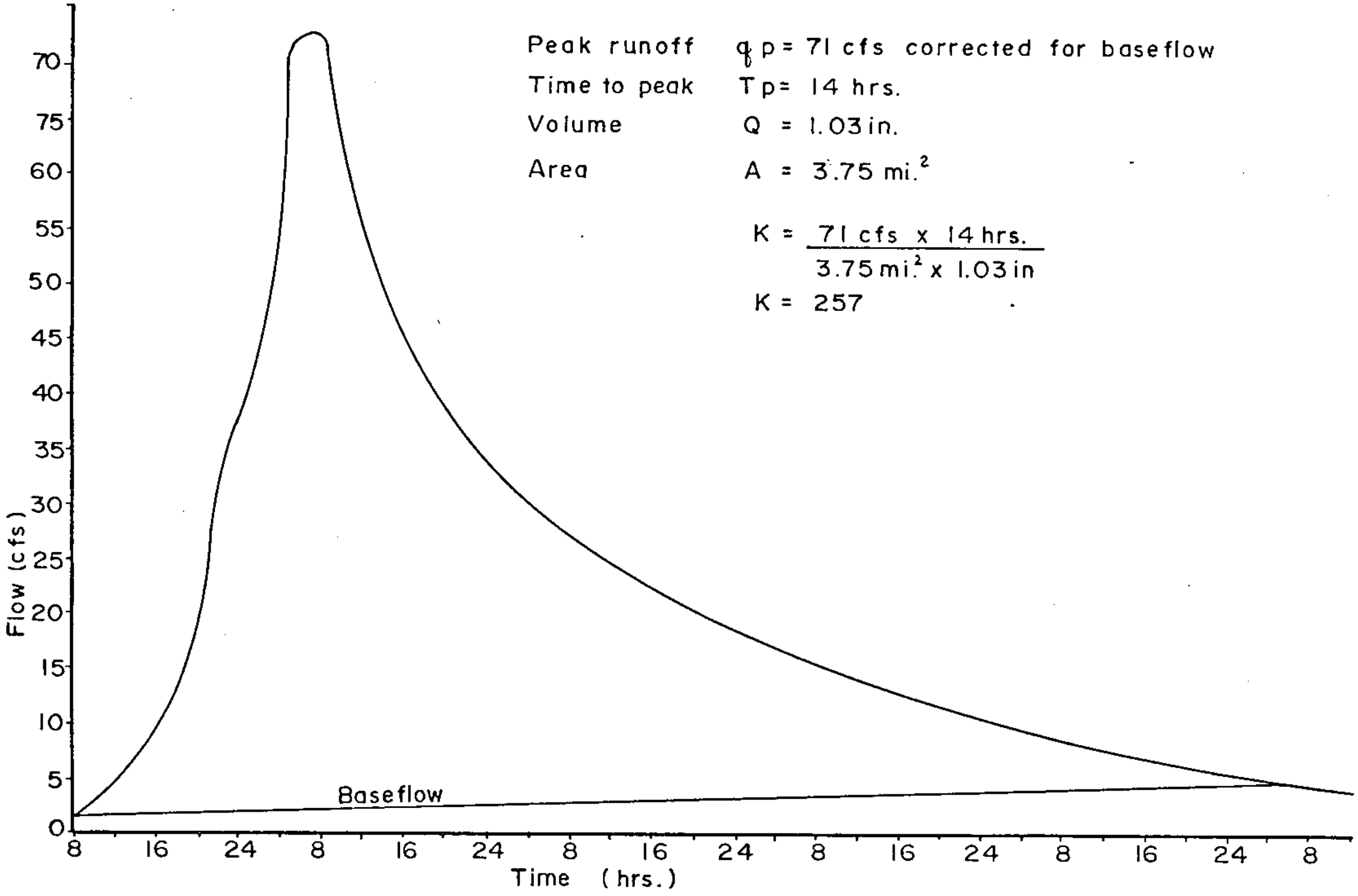


Figure 29. Hickory Creek hydrograph for 3/8/83.

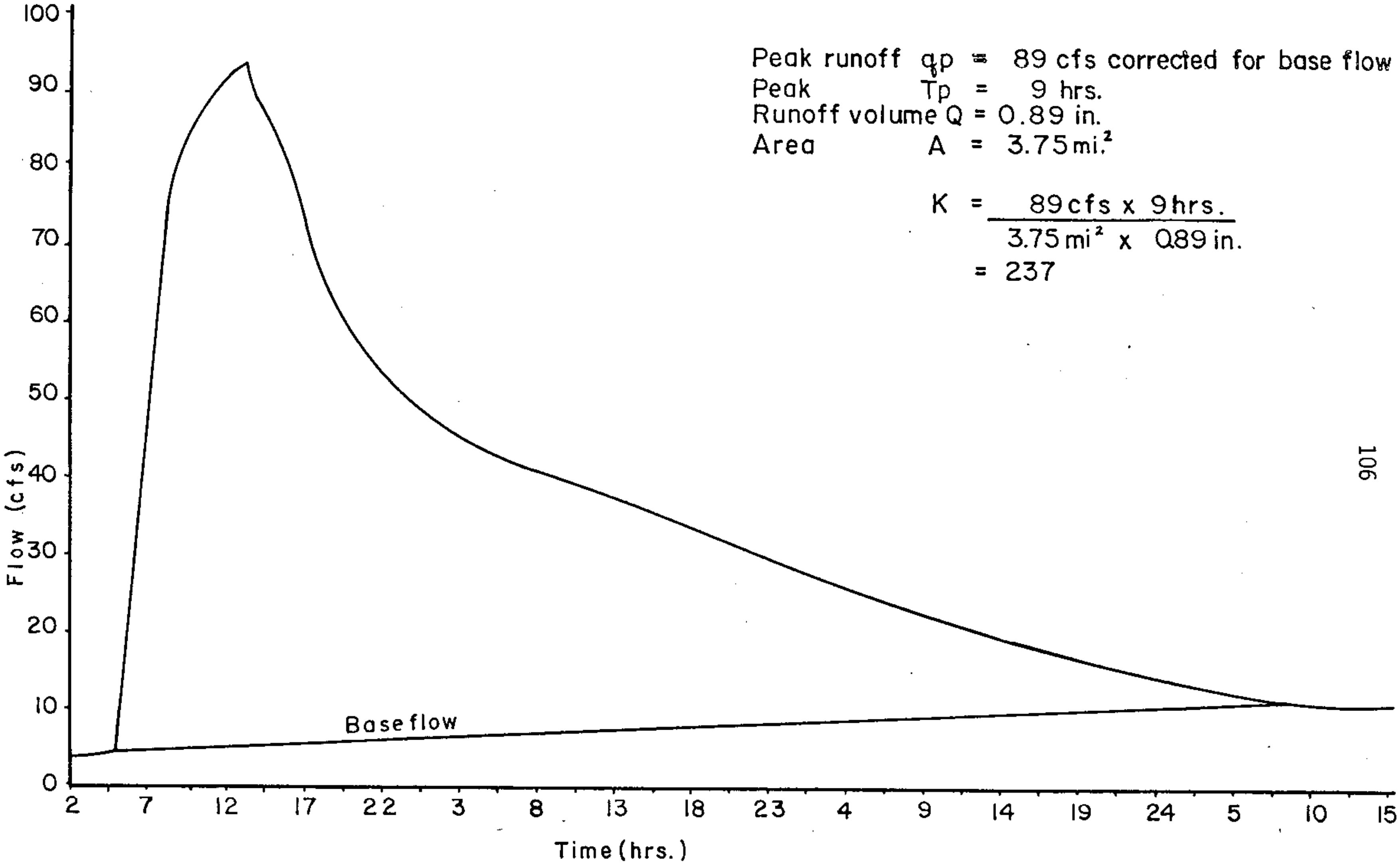


Figure 30. Hickory Creek hydrograph for 3/24/83.

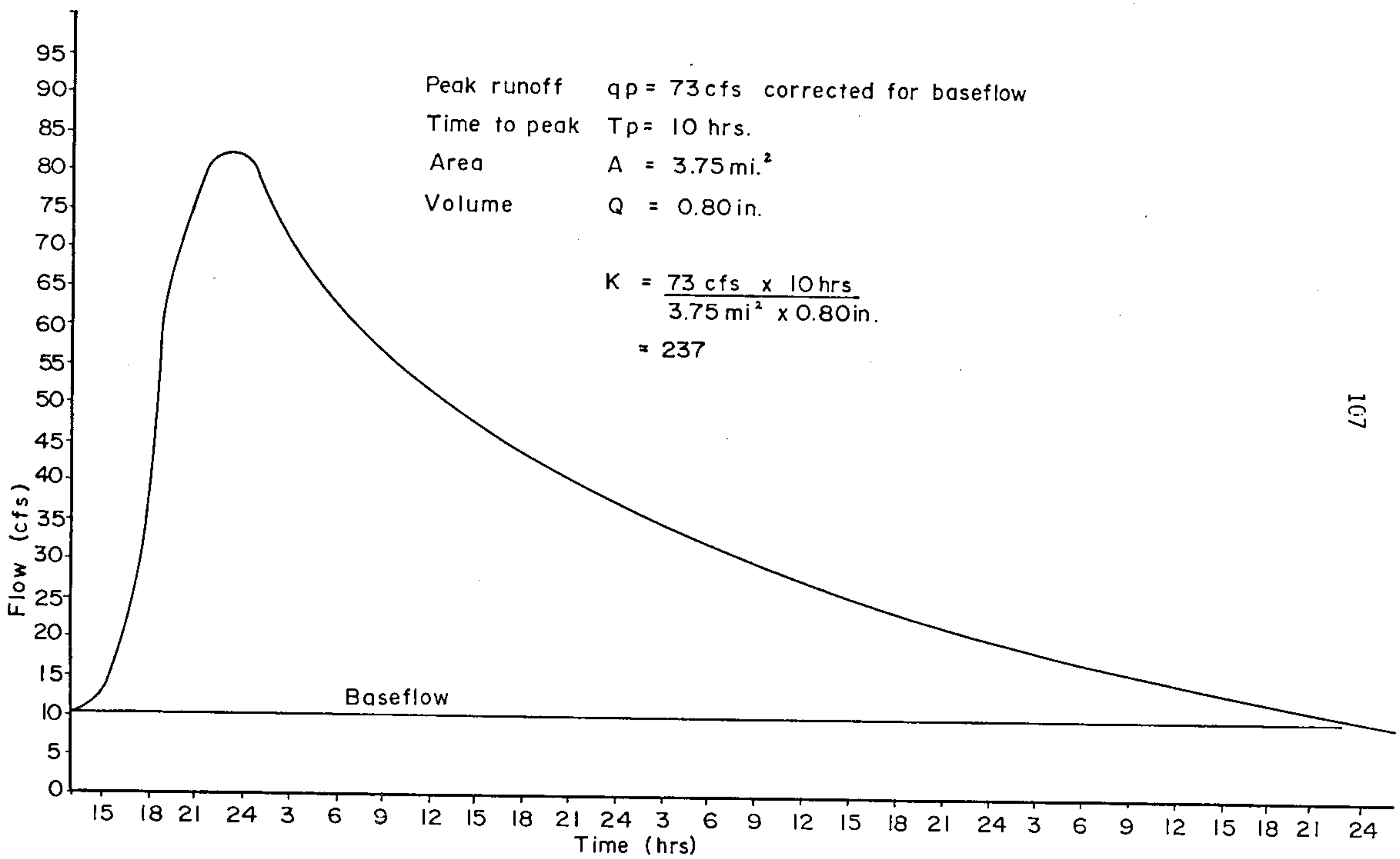


Figure 31. Hickory Creek hydrograph for 3/28/83.

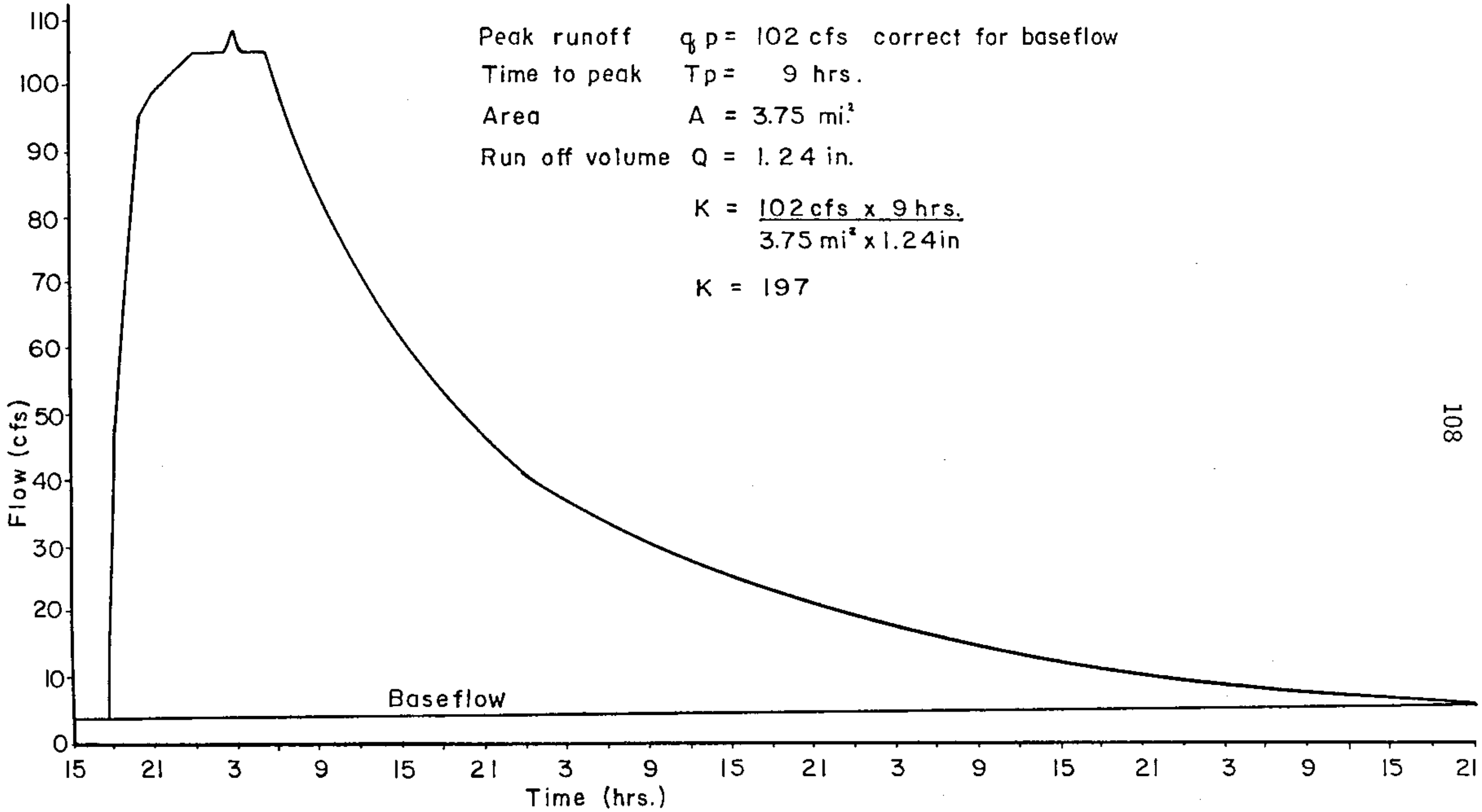


Figure 32. Hickory Creek hydrograph for 8/5/84.

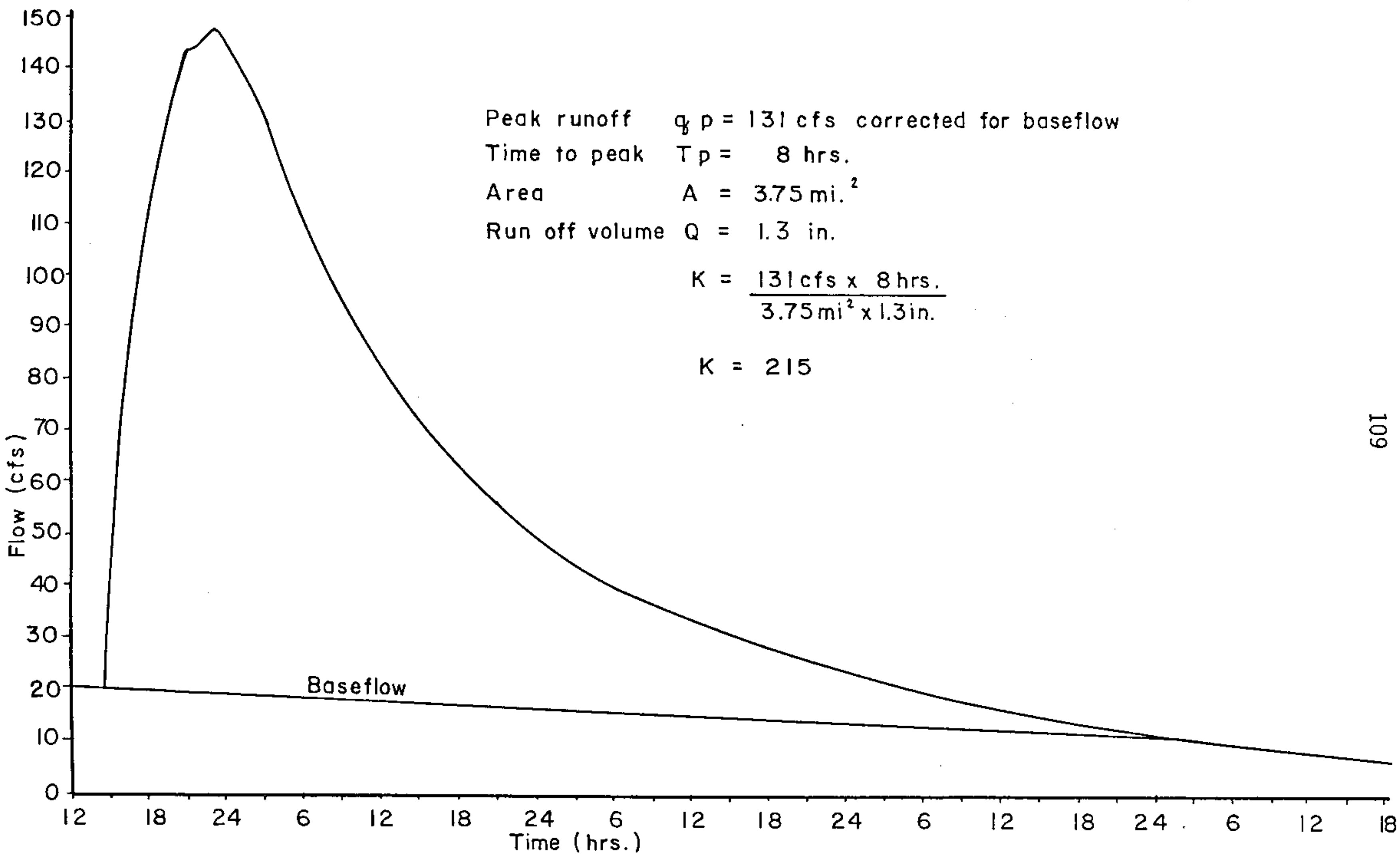


Figure 33. Hickory Creek hydrograph for 8/24/84.

CHAPTER VIII

GALLAGHER DITCH WATERSHED

Description of the Gallagher Ditch Watershed

The Gallagher Ditch watershed is located 2.2 miles northwest of Dover, Florida, in Hillsborough County (see figures 23 and 34). This watershed is in the Hillsborough River drainage basin. The Gallagher Ditch discharges to the Baker Creek Canal which joins the Hillsborough River near Thonotasassa, Florida. Specifically, it is located in the NE quarter of Section 31, Township 28S, Range 21E. The watershed area is 0.47 square miles (300 acres). The recording gauge is located on the downstream side of the culvert crossing McIntosh Road. Datum of this gauge is 50.81 ft NGVD of 1929 (USGS 1986).

The watershed has a gentle slope of 0.75% from east to west and contains two ponds. The area land use is predominantly agricultural, containing farm structures and private homes. Gallagher Road runs north-south through the center of the watershed (see Figure 35).

Storms result in peak runoff flows within 1.25 hours, the recession limbs extend for 8 to 9 hours (Figure 36). This smaller watershed had less of a base flow effect than did Hickory Creek.

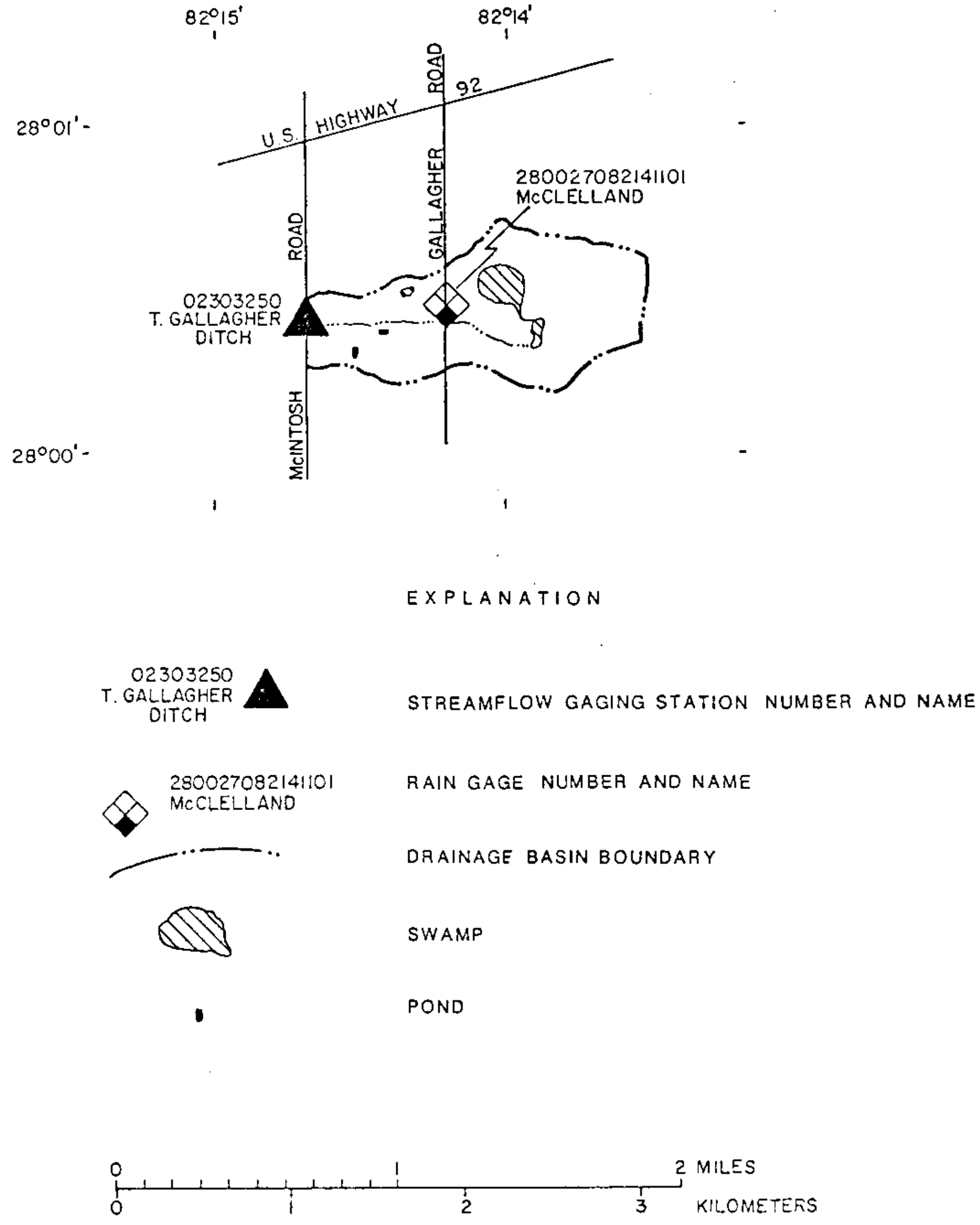


Figure 34. Gallagher Ditch watershed (USGS 1986).

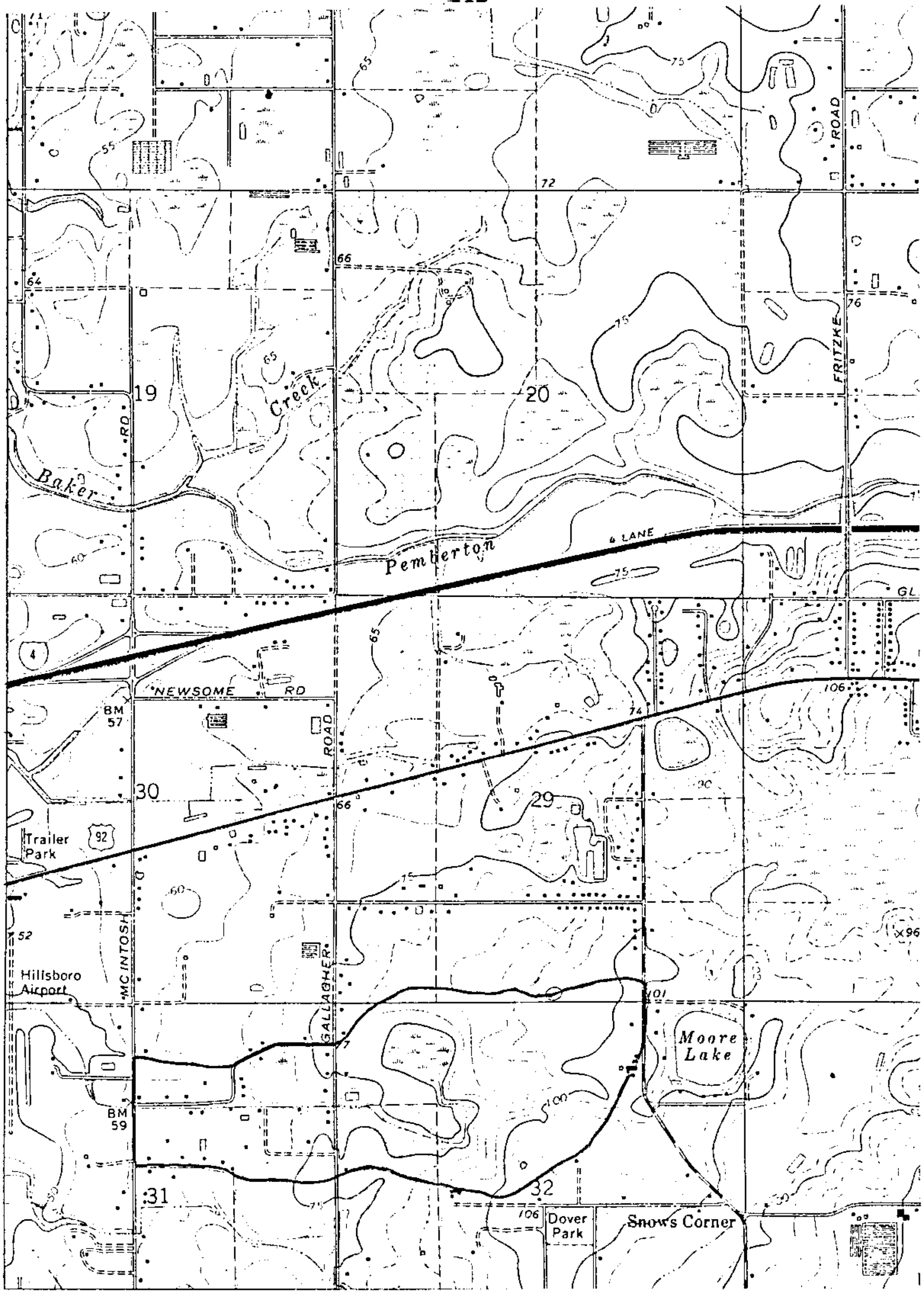


Figure 35. USGS Quadrangle with Gallagher Ditch watershed highlighted.

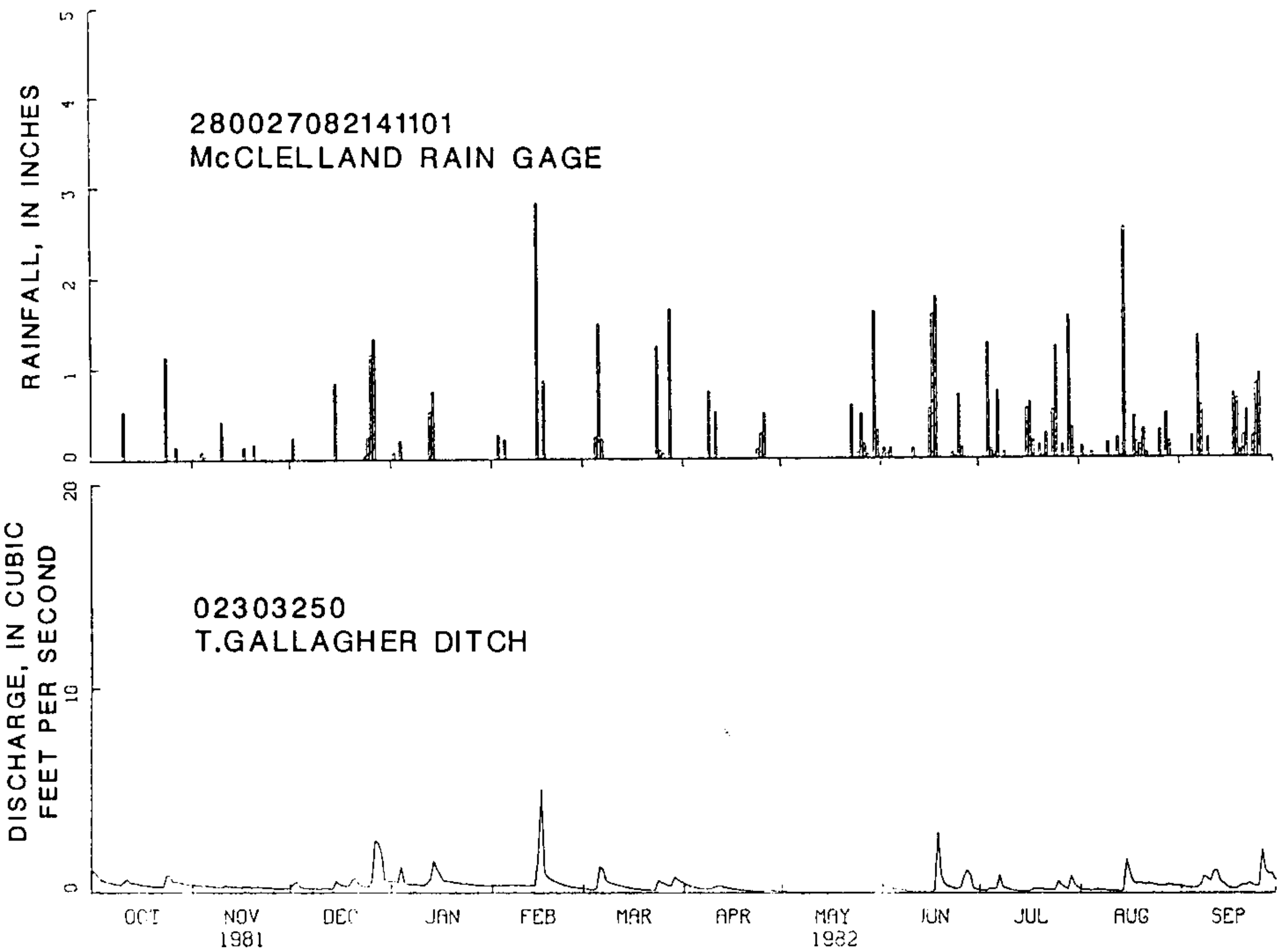


Figure 36. Daily discharge and rainfall for the Gallagher Ditch watershed (USGS 1986).

By following the same procedure for the Gallagher Ditch data as was used for Hickory Creek, four hydrographs were found suitable for analysis of peak rate factors. Table 12 lists the date, characteristics, and calculated "K" associated with each hydrograph. Figures 37-40 present the four Gallagher Ditch hydrographs and the accompanying calculations of "K".

TABLE 12

GALLAGHER DITCH PEAK RATE FACTORS (K)

STORM DATE	RUNOFF VOLUME (Q inches)	PEAK RUNOFF (q_p cfs)	TIME TO PEAK (T_p hrs)	RETURN TIME (T_r hrs)	PEAK RATE FACTOR (K)
06/18/81	0.228	32	1.25	5	373
06/09/83	0.190	27	1.0	5	302
06/26/83	0.157	23	1.25	7	390
05/23/84	0.198	27	1.25	7	361

Average K Factor: 356

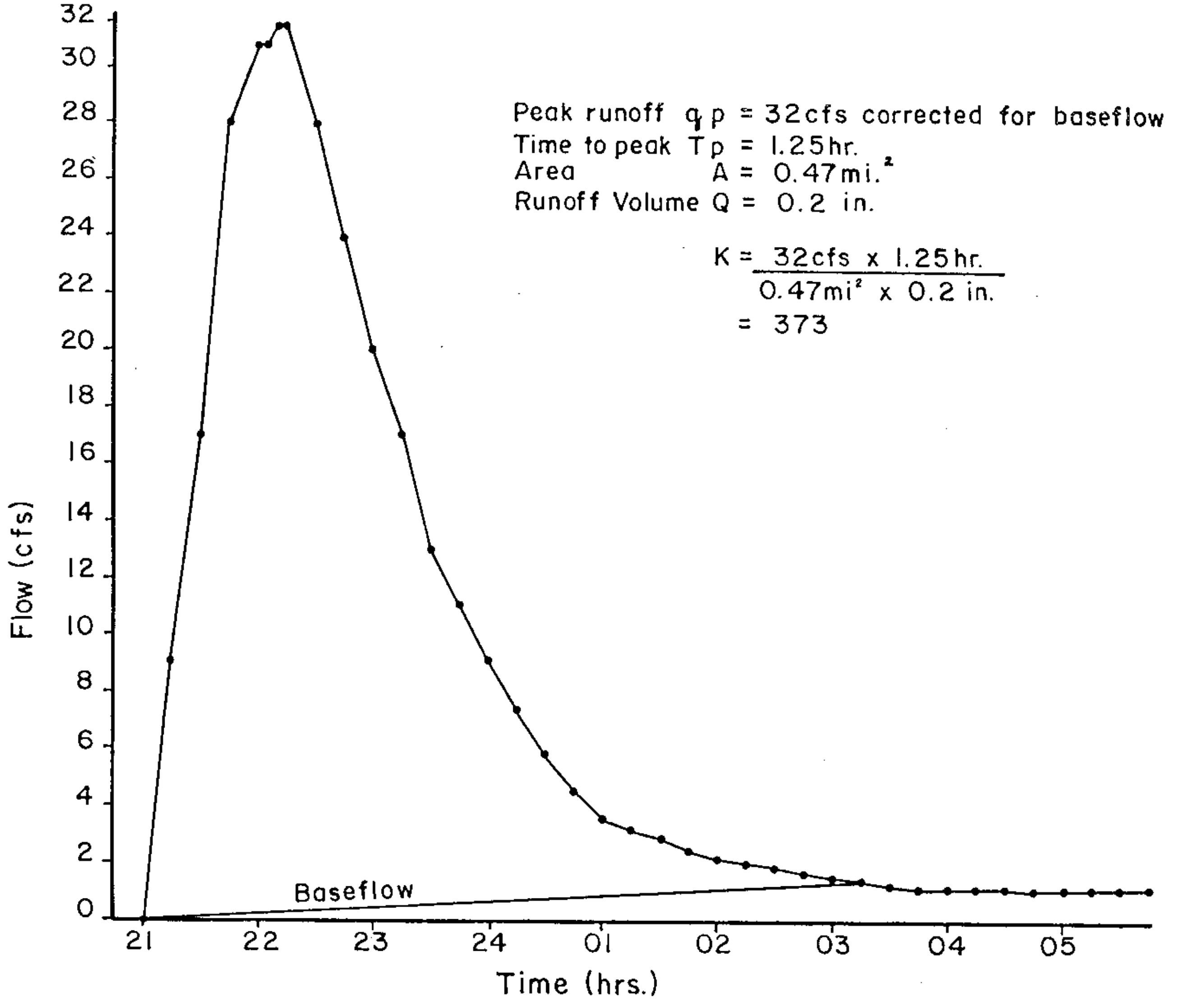


Figure 37. Gallagher Ditch hydrograph for 6/18/81.

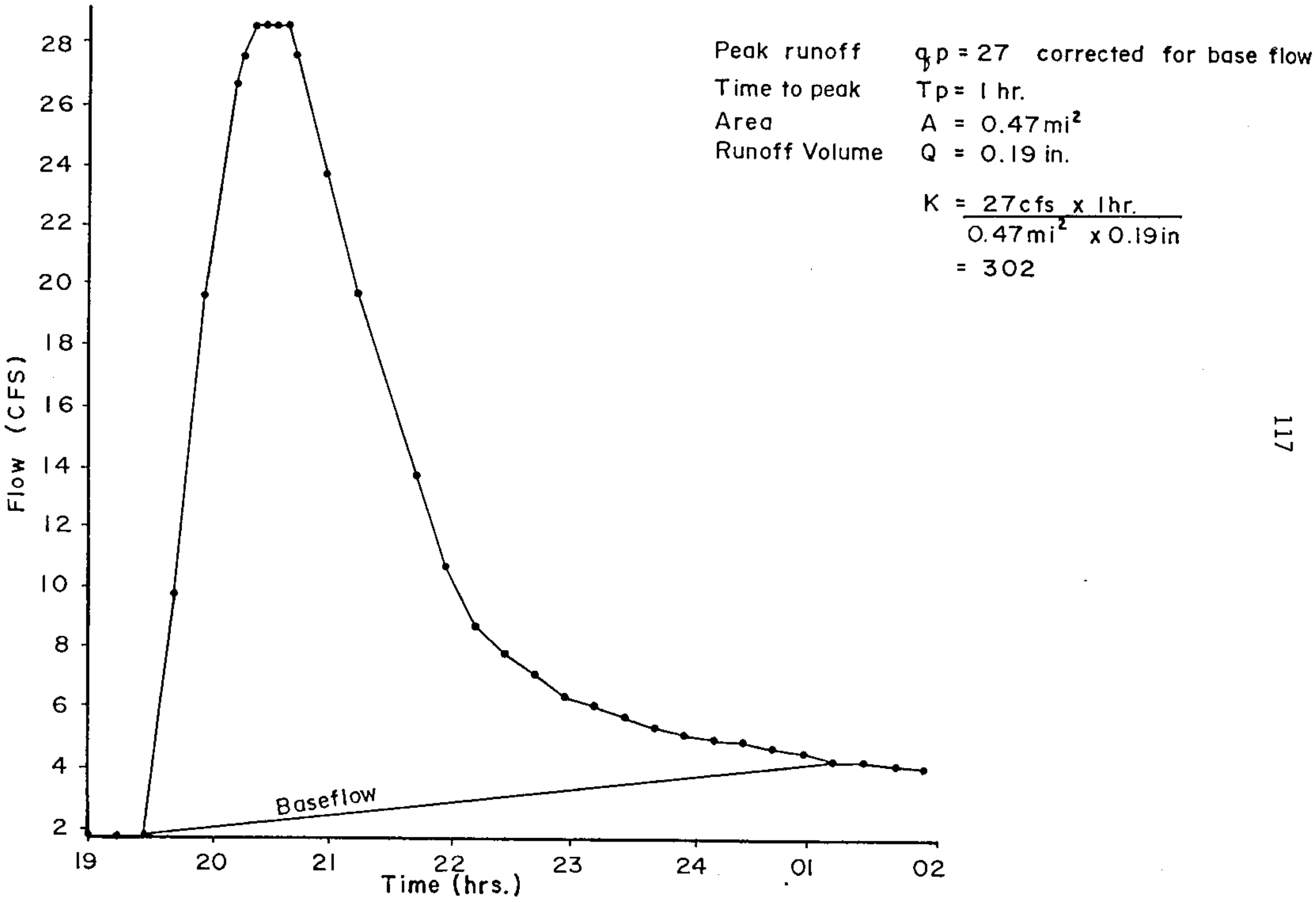


Figure 38. Gallagher Ditch hydrograph for 6/9/83.

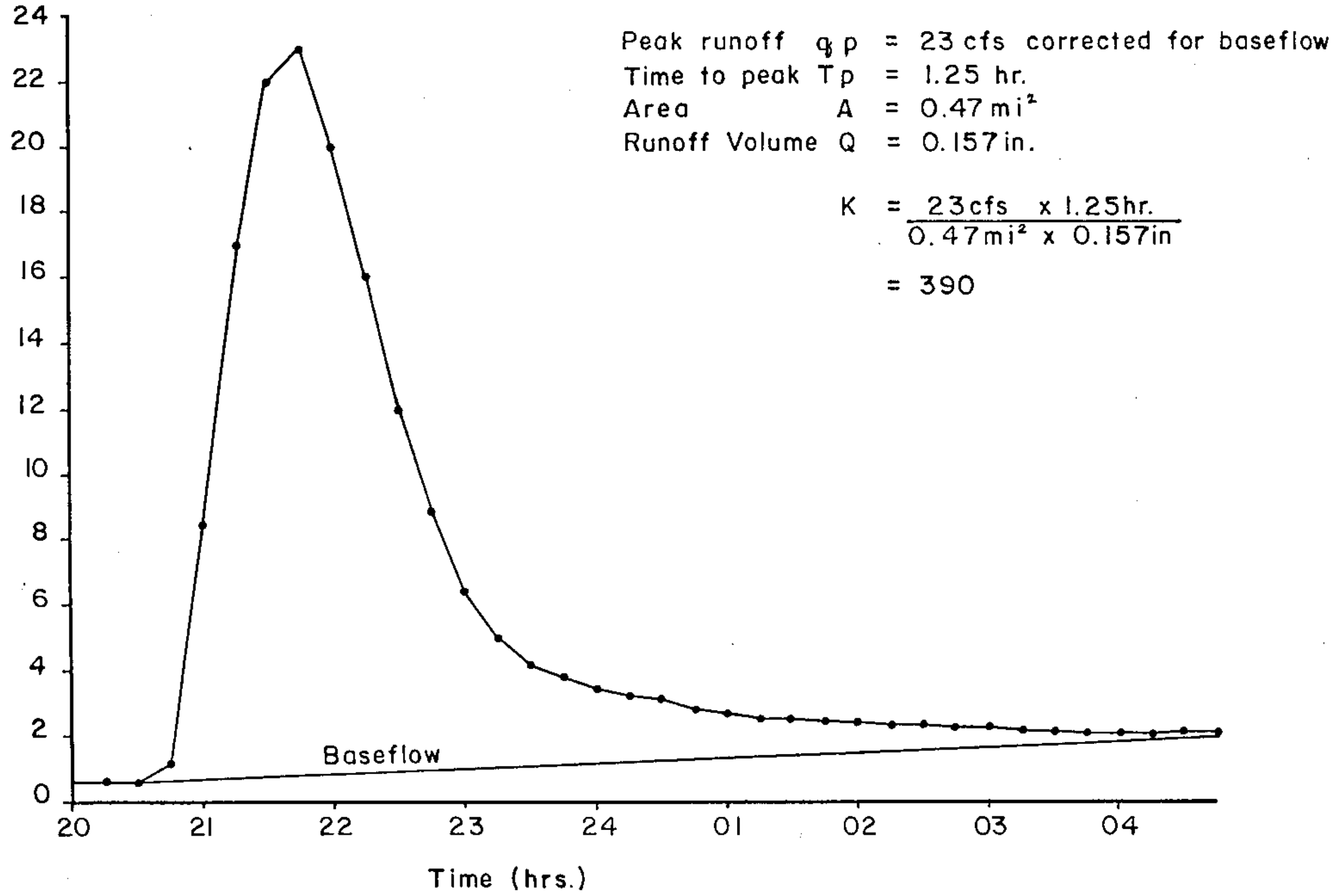


Figure 39. Gallagher Ditch hydrograph for 6/26/83.

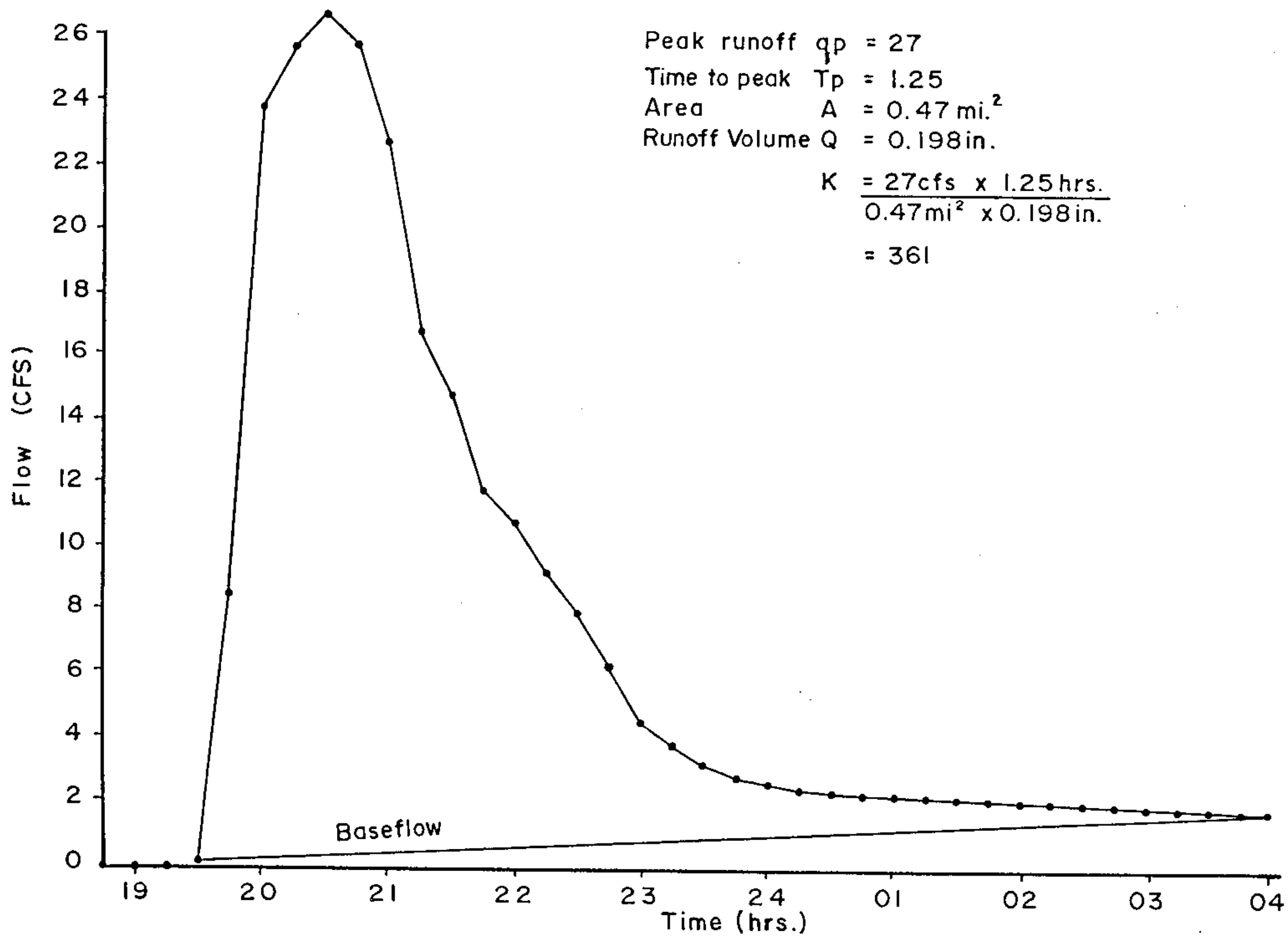


Figure 40. Gallagher Ditch hydrograph for 5/23/84.

CHAPTER IX

COMPARING RAINFALL DISTRIBUTIONS

Once the southwest Florida rainfall distribution was determined, it was compared extensively against the existing distributions commonly in use. The comparisons were run on an IBM microcomputer. Separate results are tabulated for hydrograph peak flow values obtained from the Santa Barbara Method and from the SCS Hydrograph Method.

First, a hypothetical watershed was developed with the following parameters:

Area:	300 acres
% Impervious:	35
% Directly Connected Impervious Area:	79
Time of Concentration:	90 minutes
Peak Rate Factor (K):	220
CN:	80

Each rainfall distribution was applied to the watershed using the 10-year, 24-hour storm volume. The peak flow value found in the resulting hydrograph was noted. Then, to test for the effects of antecedent moisture condition, the curve number was varied to reflect dry conditions (CN = 61) and saturated conditions (CN = 90). The following sets of plots were developed from the results. See plots of the 300-acre, K = 220 Santa Barbara and SCS 7.9 inch hydrographs.

Then, the same procedure was repeated for the 25-year storm (9 inch) and 100-year storm (11 inch). Having compared results of all the distributions for storm return frequency, antecedent moisture conditions, and hydrograph generation method, next the effects of watershed area and, correspondingly, time of concentration were compared.

The watershed size was decreased to 50 acres and time of concentration to 60 minutes. The entire procedure was repeated for these changed watershed conditions except only the Santa Barbara Method was used. These results are plotted out in the figures found in Appendix III. Then, the watershed size was returned to 300 acres and time of concentration back to 90 minutes. To test the effect of varying the K factor, the peak rate factor was changed from 220 to 300 and the procedure was repeated. Only results for the SCS Method are tabulated because the Santa Barbara Method does not use K factors, see Appendix III.

The rainfall distributions used in the comparisons were (see Appendix I, tables 14-20):

1. SCS Type II
2. SCS Type II Florida Modified
3. Army Corps of Engineers Distribution using SPS Index 20 inches
4. Pilgrim Cordery
5. Southwest Florida
6. Tampa actual
7. Brooksville actual

The Tampa rainfall distribution was a storm taken from the Weather Service data. It was not one of the storms used to derive the southwest Florida distribution. For this reason and because of its non-standard shape, it was included to learn how it would affect the resulting hydrograph peaks. Brooksville represents a standard distribution of rainfall.

Two Army Corps of Engineers distributions were developed in Chapter III. To simplify the plotting, only the SPS Index 20 inch distribution was included in the testing. However, a separate bank of tests were run to compare the Index 20 inch to the Index 11 inch distribution. See Appendix III for the plotted comparison results. The plots comparing the two Army Corps of Engineers distributions follow what is expected from reviewing Figure 11. The SPS Index 11 inch distribution has a greater percentage of the storm total volume in the maximum segment than does the SPS Index 20 inch distribution, therefore, it provides higher runoff peaks.

Interpretation of the Plotted Comparisons

Table 13 lists the results of a series of comparisons. These are plotted out and found in Appendix III. The plots of the 300 acre, K = 220 peaks using the SCS Method are well distributed. Notice that the distributions on the top of the chart have greater slopes than do the distributions lowest on the chart. This indicates that even for a simple pre- vs. post-development change in peak flow calculation it is important to select the proper distribution. Using the SCS Type II distribution will imply a greater change in peak runoff rate for a change in curve number and, thus, a larger detention pond than using the SWF distribution which has a less pronounced slope and so less of a difference between the developed and undeveloped conditions.

This is the case for the 50-acre watershed runs, and 300-acre, K = 300 watershed runs, too. The effect is not as pronounced with the 100-acre, K = 220 runs. These plots show all the distributions as parallel. In this case, a pre- vs. post- could be run with identical results using any distribution.

Comparing Santa Barbara Method peak flow rates to SCS Method peak flow rates: The Santa Barbara produces larger peaks for both the 300-acre and 100-acre watershed runs over the peaks resulting from the SCS Method. This can be explained by the K factors used

TABLE 13

COMPARISONS OF RAINFALL DISTRIBUTIONS USING COMPOSITE CURVE NUMBERS 61, 75, 80, AND 90 WITH THE SANTA BARBARA AND SCS HYDROGRAPH METHODS

	VOLUME OF RAINFALL:	7.9 in				9.0 in				11.0 in			
	METHOD OF HYDROGRAPH:	SANTA BARBARA		SCS		SANTA BARBARA		SCS		SANTA BARBARA		SCS	
Rainfall Distribution	SCS Curve Number:	61	80	61	80	61	80	61	80	61	80	61	80
	Tampa - actual	49	58	33	43	57	66.6	40.0	50.5	73	83	52	64
	Brooksville - actual	45	55	27	35	53	64.3	32.4	41.0	68.8	80.9	43	52.5
	SCS Type II	53	66	29	40	63	77	35	46	82	97	47	60*
	SCS Type II-modified	55	64	29	39.5	64.6	74	34.6	46	83.5	92.7	46.3	58.5
	Corps of Engineers	46	56	29	39	54	65	35	45	70	81	47	58
	SWFWMD	45	54	32	40.6	53.4	62	37.7	47.6	68	77.5	49.8	60.0
	Pilgrim Cordery	52	64	29	40	61.4	74	35	47	79	93	47	59
Rainfall Distribution	SCS Curve Number:	75	90	75	90	75	90	75	90	75	90	75	90
	Tampa - actual	55.2	61.4	40.3	48.2	64.3	70.6	47.7	55.6	80.4	86.8	60.8	68.8
	Brooksville - actual	52.7	60.4	32.9	40.2	61.4	69.5	38.7	46.4	77.9	86.2	49.5	58.0
	SCS Type II	63.1	73.4	36.9	45.5	73.7	84.5	43.6	52.5	93.7	104.9	56.5	65.7
	SCS Type III	60.9	69.8	36.7	45.0	70.6	80.0	43.0	51.5	89.2	98.8	55.4	64.2
	Corps of Engineers	53.5	60.5	36.3	43.4	62.2	69.2	42.9	50.1	78.6	85.6	55.0	62.2
	SWFWMD	51.4	57.3	38.2	45.0	60.0	65.9	45.1	52.1	75.2	81.1	57.5	64.4
	Pilgrim Cordery	60.8	70.1	37.1	45.8	71	80.6	44.0	53	89.7	99.8	56.6	65.8

* Don't match

in the SCS Method. Values of 220 and 300 are relatively low. The 50-acre runs were done using K = 484; in this case, the SCS values exceed those calculated using the Santa Barbara Method.

This graphically shows the importance of selecting a K factor that fits the project topography. It also shows that the Santa Barbara Method lacks an important degree of flexibility that is allowed in the SCS Method.

In the majority of the cases, the Santa Barbara and SCS Methods both produce peaks using the SCS Type II, SCS Type II Modified and Pilgrim Cordery which exceed the peaks produced by the actual rainfall distributions and the Army Corps and SWF distributions. The SWF and Army Corps consistently come much closer to matching the peaks generated by the actual storms.

Comparing an Estimated Hydrograph Using the SCS Method to an Actual Hydrograph from the Hickory Creek Watershed

The SWF developed rainfall distribution has been extensively tested against the other distributions using hypothetical watersheds. In this selection, the distribution and the derived peak rate factor are inserted into the SCS Unit Hydrograph Method to compare the result against an actual hydrograph from the Hickory Creek Watershed data.

To do this, the U.S. Geological Survey rainfall record for Hickory Creek was scanned for a long duration storm. The storm selected has a 12.5-hour duration and occurred on March 12, 1984.

Neither the rainfall nor the resulting runoff data have been used previously in this study. The other watershed modeling parameters were either given by the USGS or determined using standard SCS methods.

1. Watershed Size: 2400 acres (given with USGS data)
2. Time of Concentration: 23.3 hours determined from lag where lag time is the interval from the center of mass of the rainfall excess to the hydrograph peak, $T_c = L/.6$
3. Center of Rainfall Excess: 1.83 in. - 0.5 in. (initial abstraction) = 1.33 in from the hyetograph of the actual storm in Figure 41, the 0.5 in. initial abstraction was satisfied after four hours. The remaining 1.33 in. is the rainfall excess. Taking 1.33/2 = 0.66 in. as the center of mass, the lag time begins at 8.5 hours into the storm. The hydrograph peaks at 22.5 hours. Therefore, lag = 14 hours.
4. $T_c = 14/.6 = 23.3$ hours or 1400 minutes

The SCS Curve Number for the pervious portion is CN 70 using Table 9 where land use is forest land with good cover, soil group C for antecedent moisture condition II. Ten percent of the watershed is swampland; this is taken as impervious surface when standing water exists during the wet season.

From Figure 41 it can be seen that the SCS Unit Hydrograph model using the determined K factor and the SWF rainfall distribution closely approximates the actual runoff from the watershed.

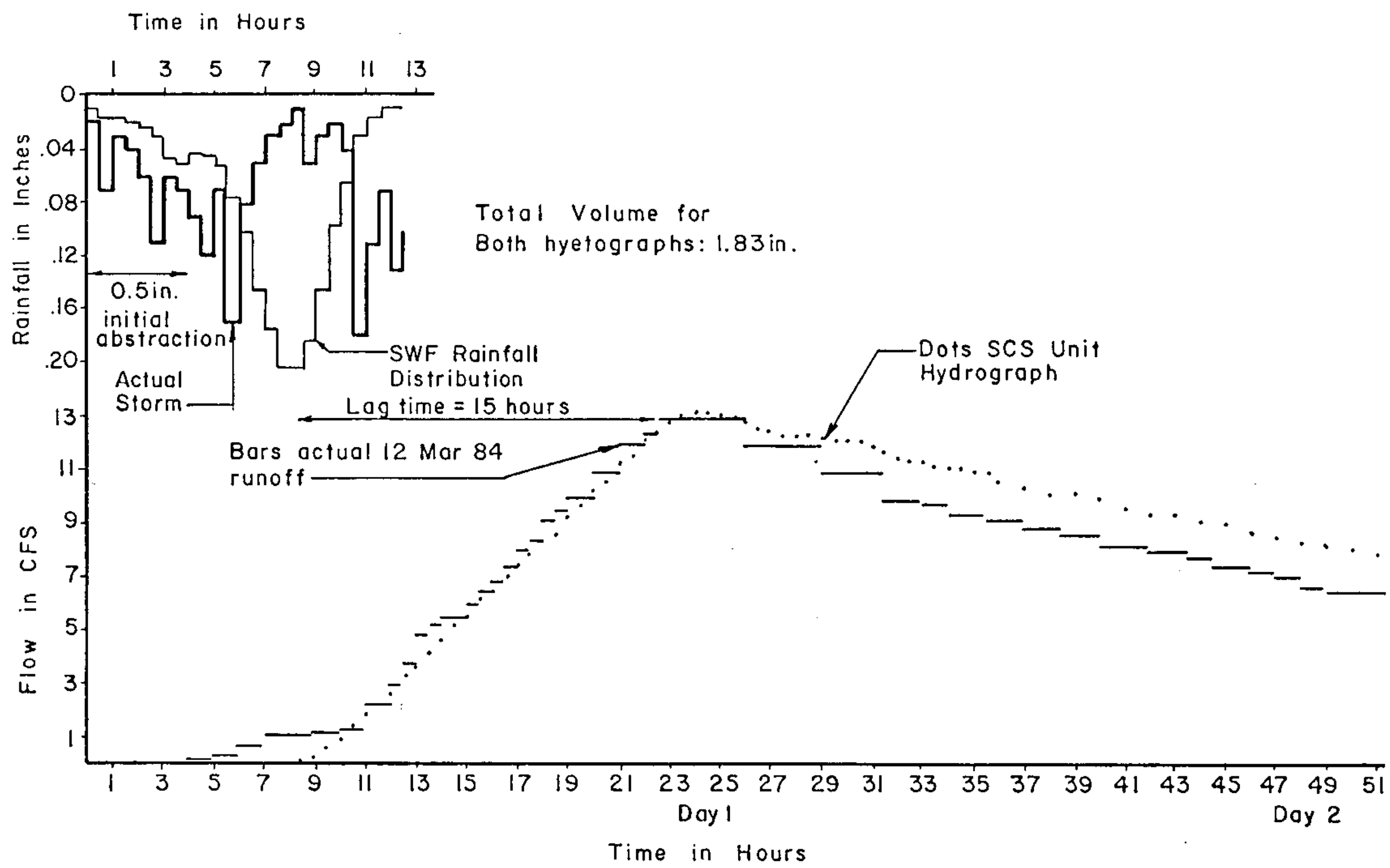


Figure 41. Hickory Creek watershed hyetograph and hydrograph, March 12, 1984 storm, SWF rainfall distribution.

The two rainfalls are plotted as hyetographs on the upper left corner of Figure 41. The March 12, 1984, storm experienced a lull 8 to 9 hours after starting, then picked back up again. The SWF hyetograph peaks during that lull. These storms have distinctly different shapes and yet produce similar results.

For a relatively large watershed which is predominantly pervious, the rainfall distribution used in the model is not a key factor in estimating hydrograph peak or shape. This is due to the long time of concentration associated with a large gently sloping watershed. Look again at the peaks of the two hydrographs. The flat peaking March 12, 1984, hydrograph peaks two hours before the crest of the SWF hydrograph. So a difference is recorded. The hyetographs are dissimilar, but the short time of occurrence of that dissimilarity is overshadowed by the long time of concentration of the watershed.

CHAPTER X

SUMMARY AND CONCLUSIONS

Summary

To obtain the most accurate hydrograph approximations using the SCS Unit Hydrograph and Santa Barbara Urban Hydrograph Methods, it is necessary to use input parameters that closely reflect the particular region under study.

This work developed a dimensionless 24-hour rainfall distribution using rainfall data from the southwest Florida area. The data were screened for storms that yielded three or more inches total volume, had durations close to 24 hours, and that peaked near the middle of the event. The storm data selected were then converted to dimensionless hyetographs. Using the dimensionless forms, a polynomial least squares curve fitting technique was applied to produce a single characteristic rainfall distribution.

The Pilgrim-Cordery Method of determining rainfall distributions was also applied to the data and a second distribution was developed.

Along with a rainfall distribution, accurate hydrograph approximations using the Soil Conservation Service Unit Hydrograph Method require a peak rate factor that closely reflects the watershed under study. The stream flow data from two watersheds in the southwest Florida area were studied to determine their particular peak rate factors.

The effects of the two distributions calculated in the study were compared to those produced by the distributions in common use through a series of hypothetical watersheds. The SCS Unit Hydrograph and Santa Barbara Urban Hydrograph Methods were used to calculate the peak runoff rate for a hypothetical watershed using each of the distributions. The results were plotted and are in Appendix II.

Conclusions

Watershed size is an important consideration in determining the relative importance of rainfall distribution in hydrograph estimation methods. On a 2400-acre watershed, the hour-to-hour intensity variations between two widely differing distributions of equal duration and volume failed to produce significantly different hydrographs. But, the tests between distributions on 50 to 300 acre hypothetical watersheds showed that rainfall distribution variations account for large variations in peak runoff rates. The critical factor behind watershed size is the time of concentration. For large watersheds having times of concentration of several hours, the small hour-to-hour differences between distributions do not produce noticeable hydrograph differences. However, the small watersheds used in this study with times of concentration varying from one to five hours showed marked differences in peak runoff rates with changes in rainfall distribution.

The tests using the small hypothetical watersheds showed that the southwest Florida distribution developed using the polynomial least squares technique better matches the results of area storms than does the distribution developed using the Pilgrim-Cordery Method. This latter method frequently overestimated the peak runoff rates obtained from the area storms.

The peak rate factors developed as part of this study (K = 224, 356) using the Soil Conservation Service methodology show that it is not appropriate to use the default value of 484 for all watersheds in southwest Florida. While the large difference between the two calculated "K" factors possibly reflects the size difference between the Hickory Creek and Gallagher Ditch watersheds, their values may be applicable to other watersheds of similar size and topographic features in southwest Florida. Hydrologists and engineers should always use sound judgement before selecting either a distribution or peak rate factor for use in hydrograph estimation.

Recommendations for Future Work

This work is based on regionally specific data and applied to specific watersheds. Additional study is needed to apply the peak rate factors to other type watersheds. Care should be taken in using the design rainfall distribution outside of the areas contributing to the analysis.

Even though the factors affecting the hydrograph peak and shape are understood for each method of generation, care should be taken when performing design work to insure no unaccounted for factor will further limit the design capacity. As the period of record grows, the data base for this type work strengthens. Perhaps in the next decade this study could be repeated to update and expand the findings. As the population increases in Florida, there may be additional rain gauges installed. Additional gauges would increase the regional accuracy and better define the geographic boundary of application for the design storm.

APPENDICES

APPENDIX I

RAINFALL DATA

This appendix contains the 24-hour rainfall distributions dimensionalized with the incremental volumes corresponding to the 10-year (7.9 inch), 25-year (9.0 inch), and 100-year (11 inch) return frequency storms. These distributions were used for the comparisons discussed in Chapter IX.

TABLE 14

5/8/79 TAMPA STORM DISTRIBUTION WITH
10-, 25-, AND 100-YEAR RETURN FREQUENCY VOLUMES

	(Vol=7.90 in)		(Vol=9.00 in)		(VOL=11.00 in)	
Hr	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
1	.15	.15	.17	.17	.21	.21
2	.02	.17	.03	.20	.03	.24
3	0.0	.17	0.0	.20	0.0	.24
4	.09	.26	.10	.30	.12	.36
5	0.0	.26	0.0	.30	0.0	.36
6	0.0	.26	0.0	.30	0.0	.36
7	0.0	.26	0.0	.30	0.0	.36
8	0.0	.26	0.0	.30	0.0	.36
9	.04	.30	.05	.35	.06	.42
10	.15	.45	.17	.52	.21	.63
11	.62	1.07	.71	1.23	.87	1.50
12	.40	1.47	.45	1.68	.55	2.05
13	.32	1.79	.37	2.05	.45	2.50
14	1.49	3.28	1.70	3.75	2.08	4.58
15	.66	3.94	.76	4.51	.92	5.50
16	.87	4.81	.99	5.50	1.21	6.71
17	1.11	5.92	1.27	6.77	1.55	8.26
18	.23	6.15	.26	7.03	.32	8.58
19	1.05	7.20	1.20	8.23	1.46	10.04
20	.40	7.60	.45	8.68	.55	10.59
21	.26	7.86	.30	8.98	.36	10.95
22	0.0	7.86	0.0	8.98	0.0	10.95
23	.03	7.89	.04	9.02	.04	10.99

TABLE 15

11/16/51 BROOKSVILLE STORM DISTRIBUTION WITH
10-, 25-, AND 100-YEAR RETURN FREQUENCY VOLUMES

	(VOL=7.90 in)		(VOL=9.00 in)		(VOL=11.00 in)	
Hr	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
1	0.10	0.10	0.12	0.12	0.14	0.14
2	0.06	0.16	0.06	0.18	0.08	0.22
3	0.14	0.30	0.16	0.34	0.20	0.42
4	0.53	0.83	0.60	0.94	0.74	1.16
5	0.06	0.89	0.07	1.01	0.09	1.25
6	0.04	0.93	0.05	1.06	0.06	1.31
7	0.22	1.15	0.25	1.31	0.31	1.62
8	0.06	1.21	0.06	1.37	0.08	1.70
9	0.15	1.36	0.17	1.54	0.21	1.91
10	0.40	1.76	0.46	2.00	0.56	2.47
11	1.90	3.66	2.17	4.17	2.65	5.12
12	1.15	4.81	1.31	5.48	1.61	6.73
13	0.94	5.75	1.07	6.55	1.31	8.04
14	0.37	6.12	0.42	6.97	0.52	8.56
15	0.10	6.22	0.12	7.09	0.14	8.70
16	0.01	6.23	0.01	7.10	0.01	8.71
17	0.05	6.28	0.05	7.15	0.07	8.78
18	0.05	6.33	0.05	7.20	0.07	8.85
19	0.89	7.22	1.02	8.22	1.24	10.09
20	0.21	7.43	0.24	8.46	0.30	10.39
21	0.31	7.74	0.35	8.81	0.43	10.82
22	0.06	7.80	0.07	8.88	0.09	10.91
23	0.05	7.85	0.05	8.93	0.07	10.98
24	0.01	7.86	0.01	8.94	0.01	10.99
25	0.01	7.87	0.02	8.95	0.01	11.00
26	0.02	7.89	0.03	8.98	0.03	11.03

TABLE 16

RAINFALL DISTRIBUTION BY THE PILGRIM-CORDERY METHOD FOR 10-, 25-, AND 100-YEAR RETURN FREQUENCIES

		(VOL=7.90 in)		(VOL=9.00 in)		(VOL=11.00 in)	
Hr	P/P_{Total}	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
1	0.001	0.01	0.01	0.01	0.01	0.01	0.01
2	0.006	0.04	0.05	0.04	0.05	0.06	0.07
3	0.012	0.05	0.10	0.06	0.11	0.06	0.13
4	0.022	0.07	0.17	0.09	0.20	0.11	0.24
5	0.032	0.08	0.25	0.09	0.29	0.11	0.35
6	0.042	0.08	0.33	0.09	0.38	0.11	0.46
7	0.054	0.10	0.43	0.11	0.49	0.13	0.59
8	0.094	0.31	0.74	0.36	0.85	0.44	1.03
9	0.143	0.39	1.13	0.44	1.29	0.54	1.57
10	0.205	0.49	1.62	0.56	1.85	0.69	2.26
11	0.320	0.91	2.53	1.03	2.88	1.26	3.52
12	0.462	1.12	3.65	1.28	4.16	1.56	5.08
13	0.702	1.90	5.55	2.16	6.32	2.64	7.72
14	0.808	0.83	6.38	0.95	7.27	1.17	8.89
15	0.844	0.29	6.67	0.33	7.60	0.39	9.28
16	0.872	0.22	6.89	0.25	7.85	0.31	9.59
17	0.900	0.22	7.11	0.25	8.10	0.31	9.90
18	0.928	0.22	7.33	0.25	8.35	0.31	10.21
19	0.954	0.20	7.53	0.24	8.59	0.29	10.58
20	0.980	0.20	7.73	0.23	8.82	0.28	10.78
21	0.998	0.15	7.88	0.16	8.98	0.20	10.98
22	0.999	0.01	7.89	0.01	8.99	0.01	10.99
23	0.9998	0.01	7.90	0.01	9.00	0.01	11.00
24	1.000	0.00	7.90	0.00	9.00	0.00	11.00

TABLE 17

SOIL CONSERVATION SERVICE TYPE II RAINFALL DISTRIBUTION
FOR 10-, 25-, AND 100-YEAR RETURN FREQUENCIES

		(VOL=7.90 in)		(VOL=9.00 in)		(VOL=11.00 in)	
Hr	P/P_{Total}	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
0.0	0.000	----	----	----	----	----	----
0.5	0.005	0.04	0.04	0.05	0.05	0.06	0.06
1.0	0.011	0.05	0.09	0.05	0.10	0.07	0.13
1.5	0.017	0.05	0.14	0.05	0.15	0.07	0.20
2.0	0.022	0.04	0.18	0.05	0.20	0.06	0.26
2.5	0.029	0.06	0.24	0.06	0.26	0.08	0.34
3.0	0.035	0.05	0.29	0.05	0.31	0.07	0.41
3.5	0.042	0.06	0.35	0.06	0.37	0.08	0.49
4.0	0.048	0.05	0.40	0.05	0.42	0.07	0.56
4.5	0.056	0.06	0.46	0.07	0.49	0.09	0.65
5.0	0.064	0.06	0.52	0.07	0.56	0.09	0.74
5.5	0.072	0.06	0.58	0.07	0.63	0.09	0.83
6.0	0.080	0.06	0.64	0.07	0.70	0.09	0.92
6.5	0.090	0.08	0.72	0.09	0.79	0.11	1.03
7.0	0.100	0.08	0.80	0.09	0.88	0.11	1.14
7.5	0.110	0.08	0.88	0.09	0.97	0.11	1.25
8.0	0.120	0.08	0.96	0.09	1.06	0.11	1.36
8.5	0.134	0.11	1.07	0.13	1.19	0.15	1.51
9.0	0.147	0.10	1.17	0.12	1.31	0.14	1.65
9.5	0.163	0.13	1.30	0.14	1.45	0.18	1.83
10.0	0.181	0.14	1.44	0.16	1.61	0.20	2.03
10.5	0.204	0.18	1.62	0.21	1.82	0.25	2.28
11.0	0.235	0.24	1.86	0.28	2.10	0.34	2.62
11.5	0.283	0.38	2.24	0.43	2.53	0.53	3.15
12.0	0.663	3.00	5.24	3.42	5.95	4.18	7.33
12.5	0.735	0.57	5.81	0.65	6.60	0.79	8.12
13.0	0.772	0.29	6.10	0.33	6.93	0.41	8.53
13.5	0.799	0.21	6.31	0.24	7.17	0.30	8.83
14.0	0.820	0.17	6.48	0.19	7.36	0.23	9.06
14.5	0.835	0.12	6.60	0.14	7.50	0.17	9.23
15.0	0.850	0.12	6.72	0.14	7.64	0.17	9.40
15.5	0.865	0.12	6.84	0.14	7.78	0.17	9.57
16.0	0.880	0.12	6.96	0.14	7.92	0.17	9.74
16.5	0.889	0.07	7.03	0.08	8.00	0.10	9.84
17.0	0.898	0.07	7.10	0.08	8.08	0.10	9.94
17.5	0.907	0.07	7.17	0.08	8.16	0.10	10.04
18.0	0.916	0.07	7.24	0.08	8.24	0.10	10.14

TABLE 17 -- CONTINUED

Hr	P/P_{Total}	(VOL=7.90 in) P_{inc}	ΣP	(VOL=9.00 in) P_{inc}	ΣP	(VOL=11.00 in) P_{inc}	ΣP
18.5	0.925	0.07	7.31	0.08	8.32	0.10	10.24
19.0	0.934	0.07	7.38	0.08	8.40	0.10	10.34
19.5	0.943	0.07	7.45	0.08	8.48	0.10	10.44
20.0	0.952	0.07	7.52	0.08	8.56	0.10	10.54
20.5	0.958	0.05	7.57	0.07	8.63	0.07	10.61
21.0	0.964	0.05	7.62	0.07	8.70	0.07	10.68
21.5	0.970	0.05	7.67	0.05	8.75	0.07	10.75
22.0	0.976	0.05	7.72	0.05	8.80	0.07	10.82
22.5	0.982	0.05	7.77	0.05	8.85	0.07	10.89
23.0	0.988	0.05	7.82	0.05	8.90	0.04	10.93
23.5	0.994	0.05	7.87	0.05	8.95	0.04	11.97
24.0	1.000	0.03	7.90	0.05	9.00	0.03	11.00

TABLE 18

ARMY CORPS OF ENGINEERS RAINFALL DISTRIBUTION
FOR 10-, 25-, AND 100-YEAR RETURN FREQUENCIES

	(VOL=7.90 in)		(VOL=9.00 in)		(VOL=11.00 in)	
Hr	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
0.0	--	--	--	--	--	--
0.5	0.06	0.06	0.06	0.06	0.08	0.08
1.0	0.06	0.12	0.06	0.12	0.08	0.16
1.5	0.06	0.18	0.06	0.18	0.08	0.24
2.0	0.06	0.24	0.06	0.24	0.08	0.32
2.5	0.06	0.30	0.06	0.30	0.08	0.40
3.0	0.06	0.36	0.07	0.37	0.09	0.49
3.5	0.06	0.42	0.07	0.44	0.09	0.58
4.0	0.06	0.48	0.07	0.51	0.09	0.67
4.5	0.08	0.56	0.09	0.60	0.11	0.78
5.0	0.08	0.64	0.09	0.69	0.11	0.89
5.5	0.09	0.73	0.11	0.80	0.13	1.02
6.0	0.09	0.82	0.11	0.91	0.13	1.15
6.5	0.09	0.91	0.11	1.02	0.13	1.28
7.0	0.09	1.00	0.11	1.13	0.13	1.41
7.5	0.13	1.13	0.15	1.28	0.19	1.60
8.0	0.13	1.26	0.15	1.43	0.19	1.79
8.5	0.13	1.39	0.15	1.58	0.19	1.98
9.0	0.13	1.52	0.15	1.73	0.19	2.17
9.5	0.14	1.66	0.16	1.89	0.20	2.37
10.0	0.14	1.80	0.16	2.05	0.20	2.57
10.5	0.14	1.94	0.16	2.21	0.20	2.77
11.0	0.14	2.08	0.16	2.37	0.20	2.97
11.5	0.16	2.24	0.18	2.55	0.22	3.19

TABLE 18 -- CONTINUED

	(VOL=7.90 in)		(VOL=9.00 in)		(VOL=11.00 in)	
Hr	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
12.0	0.16	2.40	0.18	2.73	0.22	3.41
12.5	0.19	2.59	0.22	2.95	0.26	3.67
13.0	0.21	2.80	0.23	3.18	0.29	3.96
13.5	0.29	3.09	0.33	3.51	0.41	4.37
14.0	0.30	3.39	0.34	3.85	0.42	4.79
14.5	0.46	3.85	0.52	4.37	0.64	5.43
15.0	0.46	4.31	0.52	4.89	0.64	6.07
15.5	0.73	5.04	0.83	5.72	1.01	7.08
16.0	0.81	5.85	0.92	6.64	1.12	8.20
16.5	0.34	6.19	0.39	7.03	0.47	8.67
17.0	0.33	6.52	0.38	7.41	0.46	9.13
17.5	0.18	6.70	0.21	7.62	0.25	9.38
18.0	0.17	6.87	0.20	7.82	0.24	9.62
18.5	0.11	6.98	0.13	7.95	0.15	9.77
19.0	0.11	7.09	0.13	8.08	0.15	9.92
19.5	0.09	7.18	0.11	8.19	0.13	10.05
20.0	0.09	7.27	0.11	8.30	0.13	10.18
20.5	0.09	7.36	0.10	8.40	0.12	10.30
21.0	0.09	7.45	0.10	8.50	0.12	10.42
21.5	0.08	7.53	0.09	8.59	0.11	10.53
22.0	0.08	7.61	0.09	8.68	0.11	10.64
22.5	0.09	7.70	0.08	8.76	0.10	10.74
23.0	0.07	7.77	0.08	8.84	0.10	10.84
23.5	0.07	7.84	0.08	8.92	0.10	10.94
24.0	0.06	7.90	0.08	9.00	0.06	11.00

TABLE 19

SOUTHWEST FLORIDA RAINFALL DISTRIBUTION
FOR 10-, 25-, AND 100-YEAR RETURN FREQUENCIES

		(VOL=7.90 in)		(VOL=9.00 in)		(VOL=11.00 in)	
Hr	P/P_{Total}	P_{inc}	ΣP	P_{inc}	ΣP	P_{inc}	ΣP
0.0	0.000	----	----	----	----	----	----
0.5	0.006	0.05	0.05	0.05	0.05	0.07	0.07
1.0	0.011	0.04	0.09	0.05	0.10	0.06	0.13
1.5	0.016	0.04	0.13	0.05	0.15	0.06	0.19
2.0	0.021	0.04	0.17	0.05	0.20	0.06	0.25
2.5	0.026	0.04	0.21	0.05	0.25	0.06	0.31
3.0	0.032	0.05	0.26	0.05	0.30	0.07	0.38
3.5	0.037	0.04	0.30	0.05	0.35	0.06	0.44
4.0	0.043	0.05	0.35	0.05	0.40	0.07	0.51
4.5	0.050	0.06	0.41	0.06	0.46	0.08	0.59
5.0	0.057	0.06	0.47	0.05	0.52	0.08	0.67
5.5	0.067	0.08	0.55	0.09	0.61	0.11	0.78
6.0	0.078	0.09	0.64	0.10	0.71	0.12	0.90
6.5	0.093	0.12	0.76	0.14	0.85	0.17	1.07
7.0	0.108	0.12	0.88	0.14	0.99	0.17	1.24
7.5	0.121	0.10	0.98	0.12	1.11	0.14	1.38
8.0	0.132	0.09	1.07	0.10	1.21	0.12	1.50
8.5	0.144	0.09	1.16	0.11	1.32	0.13	1.63
9.0	0.156	0.09	1.25	0.11	1.43	0.13	1.76
9.5	0.168	0.09	1.34	0.11	1.54	0.13	1.89
10.0	0.182	0.11	1.45	0.13	1.67	0.15	2.04
10.5	0.197	0.12	1.57	0.14	1.81	0.17	2.21
11.0	0.216	0.15	1.72	0.17	1.98	0.21	2.42
11.5	0.238	0.17	1.89	0.20	2.18	0.24	2.66
12.0	0.265	0.21	2.10	0.24	2.42	0.30	2.96
12.5	0.296	0.24	2.34	0.28	2.70	0.34	3.30
13.0	0.332	0.28	2.62	0.32	3.02	0.40	3.70
13.5	0.374	0.33	2.95	0.38	3.40	0.46	4.16
14.0	0.421	0.37	3.32	0.42	3.82	0.52	4.68
14.5	0.471	0.40	3.72	0.45	4.27	0.55	5.23
15.0	0.526	0.43	4.15	0.50	4.77	0.61	5.84
15.5	0.583	0.45	4.60	0.51	5.28	0.63	6.47
16.0	0.641	0.46	5.06	0.52	5.80	0.64	7.11
16.5	0.695	0.43	5.49	0.49	6.29	0.59	7.70
17.0	0.747	0.41	5.90	0.47	6.76	0.57	8.27
17.5	0.795	0.38	6.28	0.43	7.19	0.53	8.80
18.0	0.838	0.34	6.62	0.39	7.59	0.47	9.27

TABLE 19 -- CONTINUED

Hr	P/P_{Total}	(VOL=7.90 in) P_{inc}	ΣP	(VOL=9.00 in) P_{inc}	ΣP	(VOL=11.00 in) P_{inc}	ΣP
18.5	0.875	0.29	6.91	0.33	7.91	0.41	9.68
19.0	0.904	0.23	7.14	0.26	8.17	0.32	10.00
19.5	0.928	0.19	7.33	0.22	8.39	0.26	10.26
20.0	0.948	0.16	7.49	0.18	8.57	0.22	10.48
20.5	0.963	0.12	7.61	0.14	8.71	0.17	10.65
21.0	0.974	0.07	7.68	0.08	8.79	0.10	10.75
21.5	0.982	0.06	7.74	0.07	8.86	0.09	10.84
22.0	0.988	0.05	7.79	0.05	8.91	0.07	10.91
22.5	0.992	0.03	7.85	0.02	8.95	0.04	10.95
23.0	0.995	0.03	7.85	0.02	8.97	0.03	10.98
23.5	0.997	0.03	7.88	0.02	8.99	0.01	10.99
24.0	1.000	0.02	7.90	0.01	9.00	0.01	11.00

TABLE 20

SOIL CONSERVATION SERVICE TYPE II-FLORIDA MODIFIED RAINFALL DISTRIBUTION FOR 10-, 25-, AND 100-YEAR RETURN FREQUENCIES

TIME STEP (hours)	P/P_{Total}	P_{inc} RAINFALL (inches)	ΣP RAINFALL	P_{inc} RAINFALL (inches)	ΣP RAINFALL	P_{inc} RAINFALL (inches)	ΣP RAINFALL
1	0.012	0.096	0.096	0.11	0.11	0.134	0.134
2	0.025	0.1	0.196	0.117	0.227	0.143	0.277
3	0.039	0.11	0.306	0.125	0.352	0.153	0.43
4	0.054	0.12	0.426	0.136	0.488	0.166	0.596
5	0.071	0.13	0.556	0.15	0.638	0.182	0.778
6	0.089	0.146	0.702	0.166	0.804	0.2	0.978
7	0.110	0.22	0.922	0.188	0.992	0.23	1.208
8	0.135	0.19	1.112	0.218	1.21	0.27	1.478
9	0.164	0.23	1.342	0.263	1.473	0.32	1.798
10	0.201	0.3	1.642	0.33	1.803	0.414	2.212
11	0.258	0.45	2.092	0.51	2.313	0.626	2.838
12	0.606	2.75	4.842	3.14	5.453	3.836	6.674
13	0.757	1.186	6.028	1.35	6.803	1.65	8.324
14	0.807	0.395	6.423	0.45	7.253	0.55	8.874
15	0.842	0.276	6.699	0.315	7.568	0.385	9.259
16	0.870	0.22	6.919	0.25	7.818	0.306	9.565
17	0.893	0.19	7.109	0.21	8.028	0.256	9.821
18	0.913	0.16	7.269	0.18	8.208	0.111	9.932
19	0.931	0.14	7.409	0.16	8.368	0.197	10.129
20	C.947	0.127	7.536	0.145	8.513	0.178	10.307
21	0.962	0.118	7.654	0.134	8.647	0.164	10.471
22	0.976	0.11	7.764	0.123	8.770001	0.15	10.621
23	0.989	0.1	7.864	0.115	8.885	0.14	10.761
24	1.00	0.089	7.953	0.1	8.985	0.124	10.885

APPENDIX II

BASE FLOW SEPARATION

The unit hydrograph estimation method discussed in Chapter V applies to surface runoff only. In instances where a persistent low flow occurs at the gauge station between storms, it is necessary to separate such base flow values from the total gauge readings.

Three common separation methods are found in the literature:

1. The Straight Line Method - a straight line is drawn from the point of initial rise in the rising limb of the plotted hydrograph to the point on the recession limb where a pronounced change in slope occurred. The slope change is thought to be the point where groundwater seeping into the stream constitutes a greater volume of flow than does direct surface runoff (see Figure 42, line A-B).

2. The second method is illustrated by line A-C-D in Figure 42. This has the initial base flow decrease from A to point C directly below the peak rate of flow. Then, base flow rises to D on the hydrograph. Point D represents N days after the peak and is found using the following equation (Schulz 1974):

   $N = A^{0.2}$ where N is time in days and A is the drainage area in square miles

3. The third method uses a base flow recession curve fitted to the hydrograph in a time decreasing direction (see Figure 42, line A-E-F). Starting at point F, follow the hydrograph recession curve toward the peak to point E which is arbitrarily determined. Next, project the curve back to point A using an arbitrary line.

All of these methods are arbitrary in nature and require the users to exercise judgement in their selection.

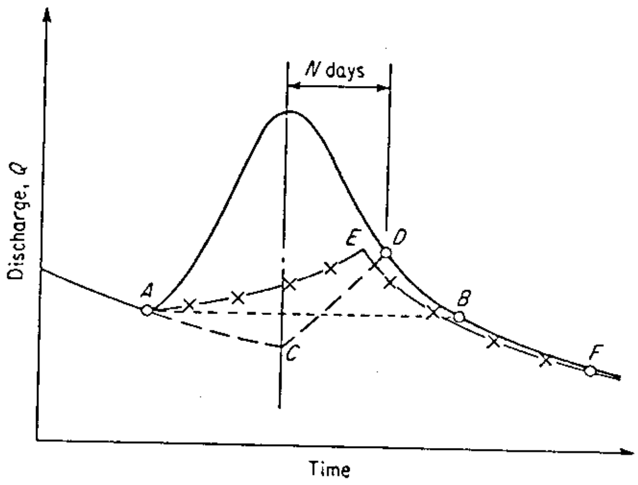


Figure 42. Base flow separation techniques.

APPENDIX III

PLOTTED RESULTS OF THE RAINFALL DISTRIBUTION

The following pages show comparisons using a variety of hypothetical watersheds.

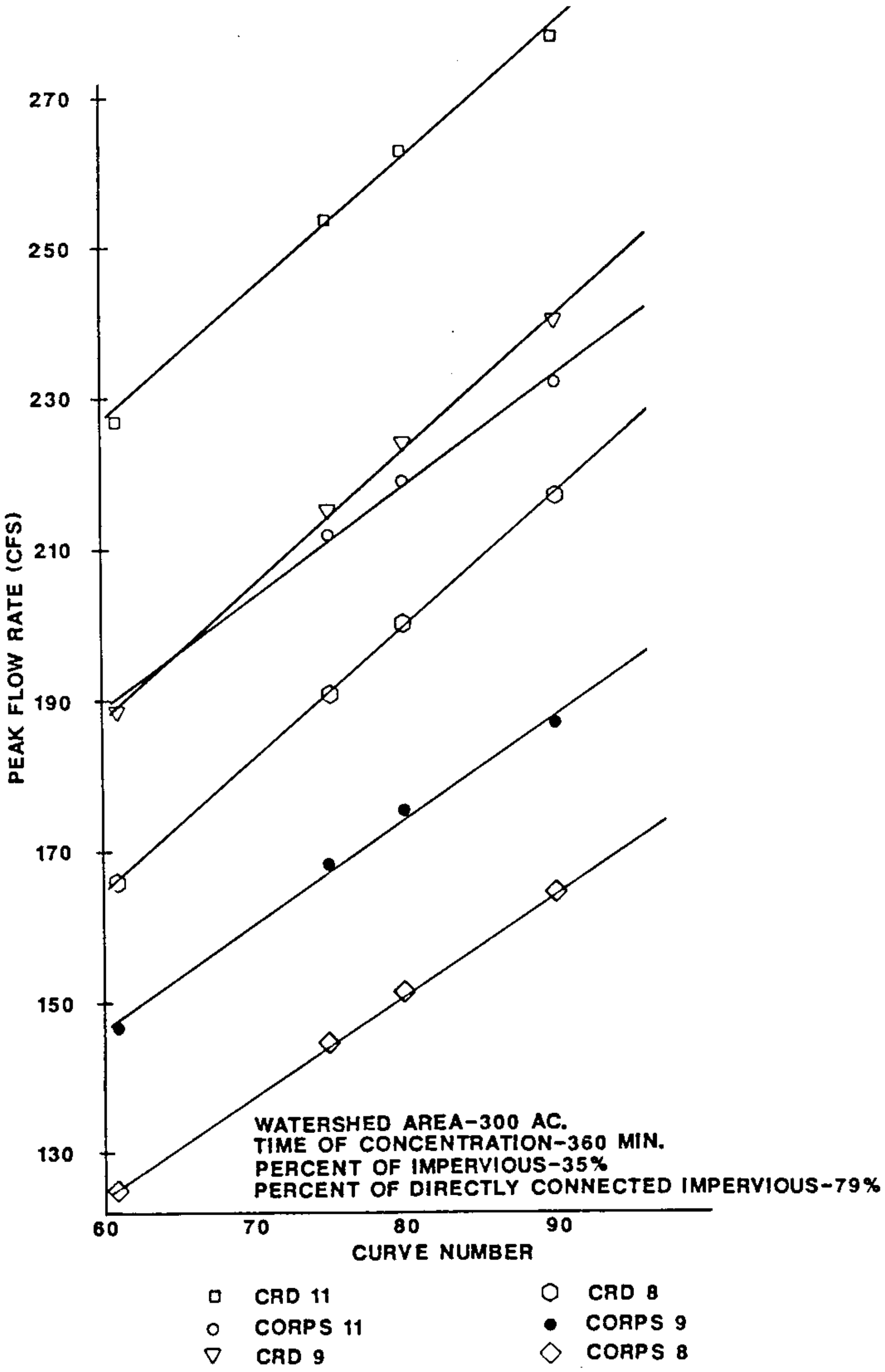


Figure 43. Army Corps of Engineers distribution comparisons using Santa Barbara urban hydrograph method.

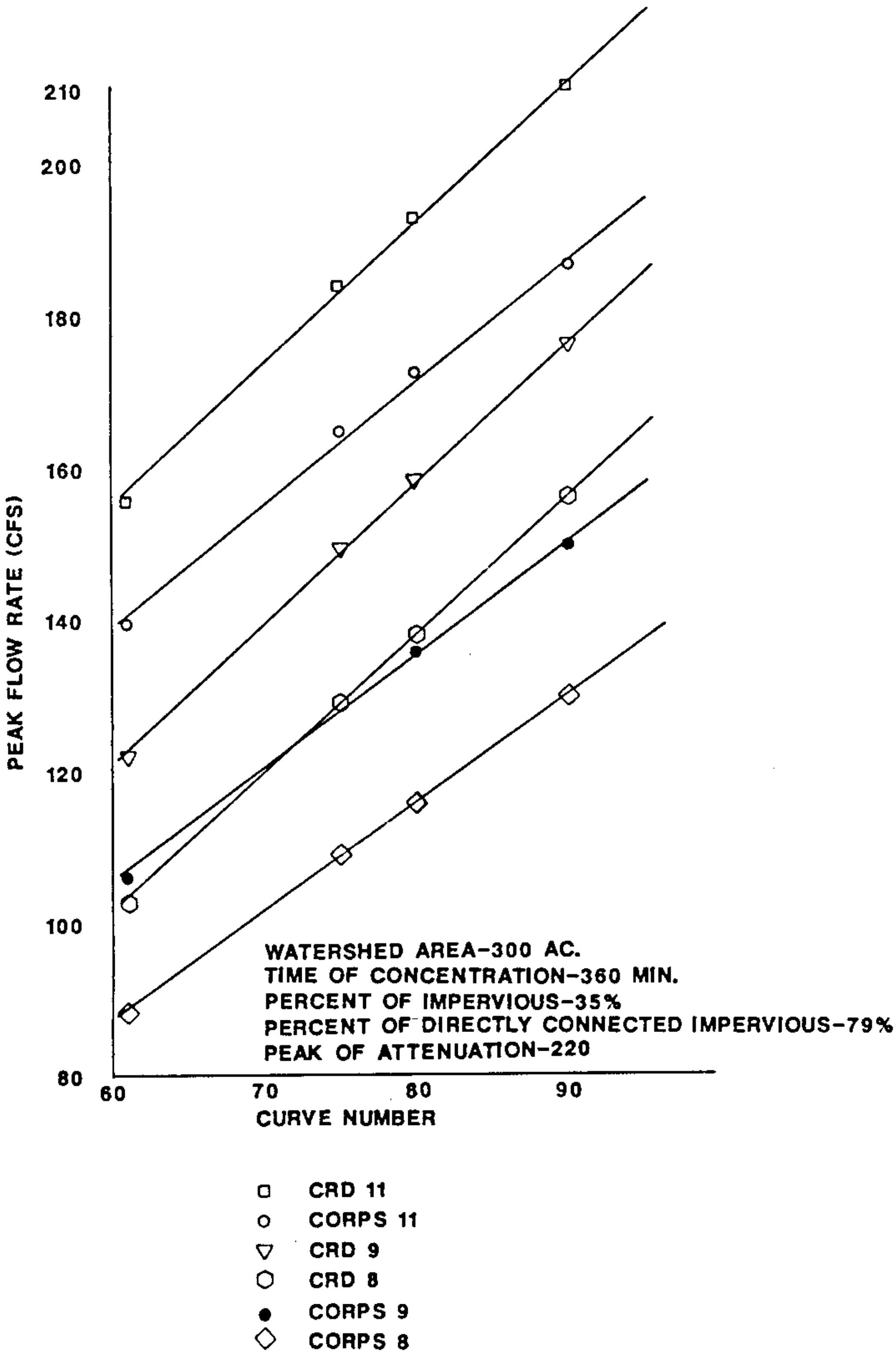


Figure 44. Army Corps of Engineers distribution comparisons using SCS unit hydrograph method.

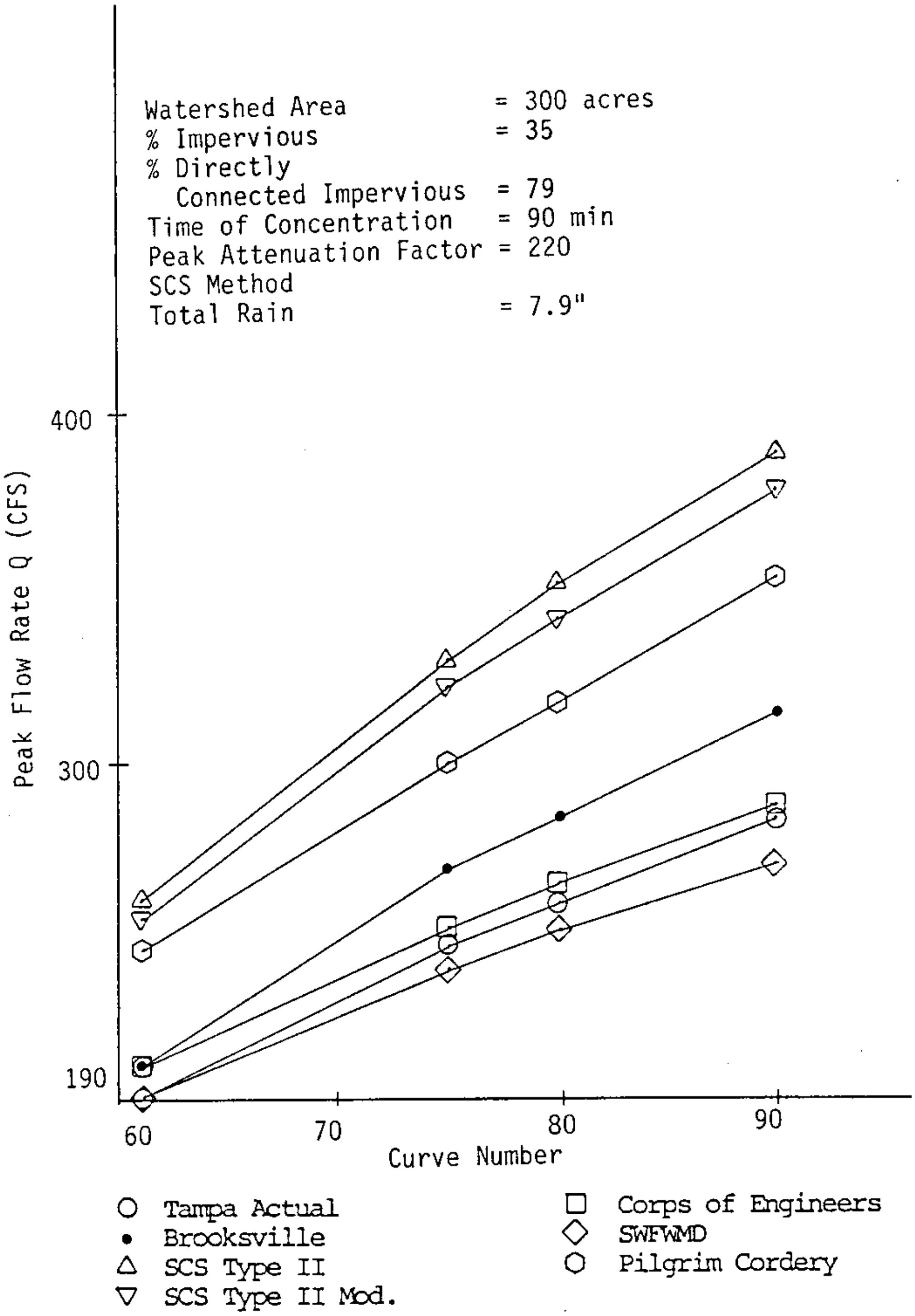


Figure 45. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using SCS Method K = 220, total rainfall = 7.9 inches.

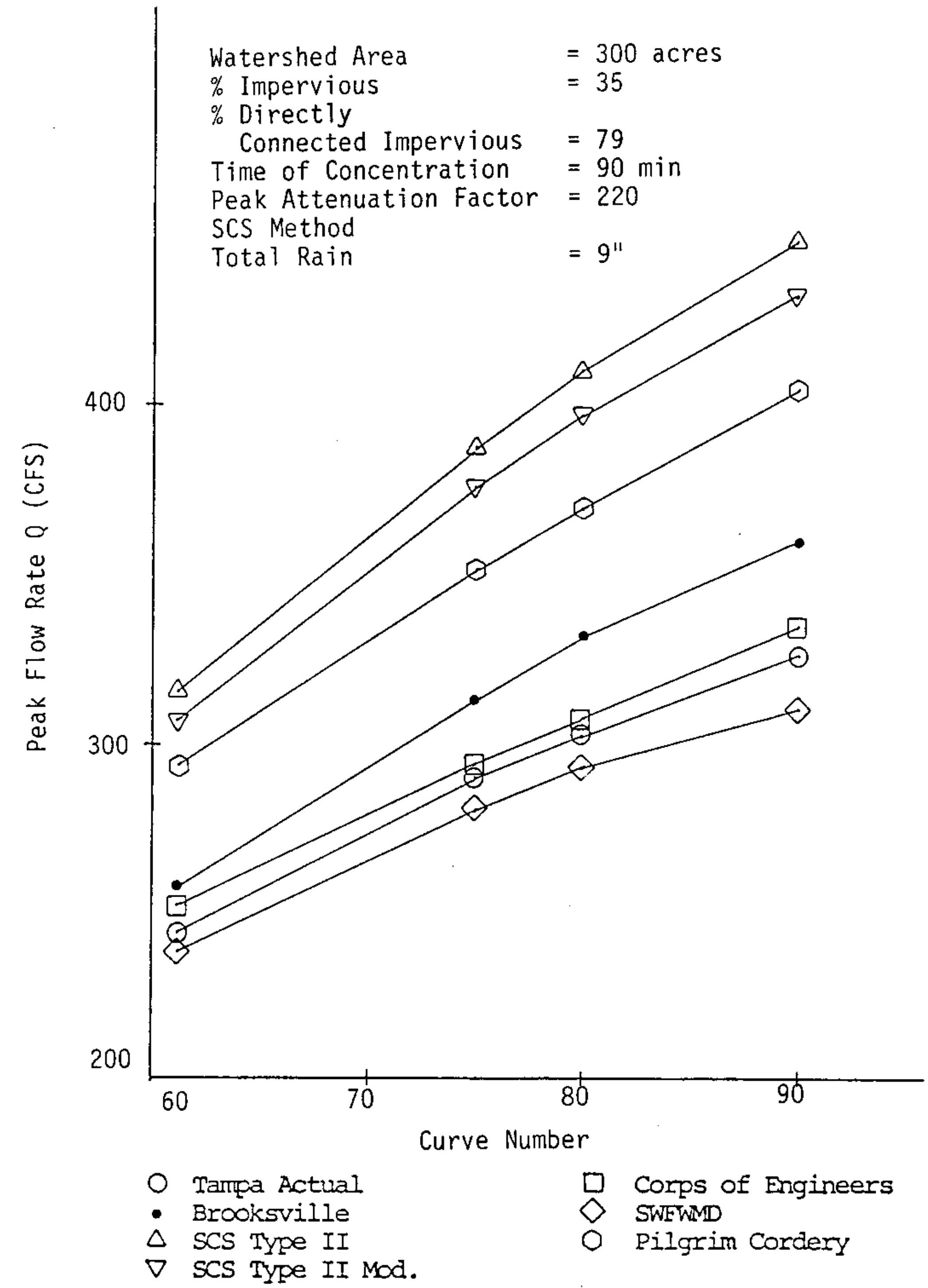


Figure 46. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using SCS Method K = 220, total rainfall = 9 inches.

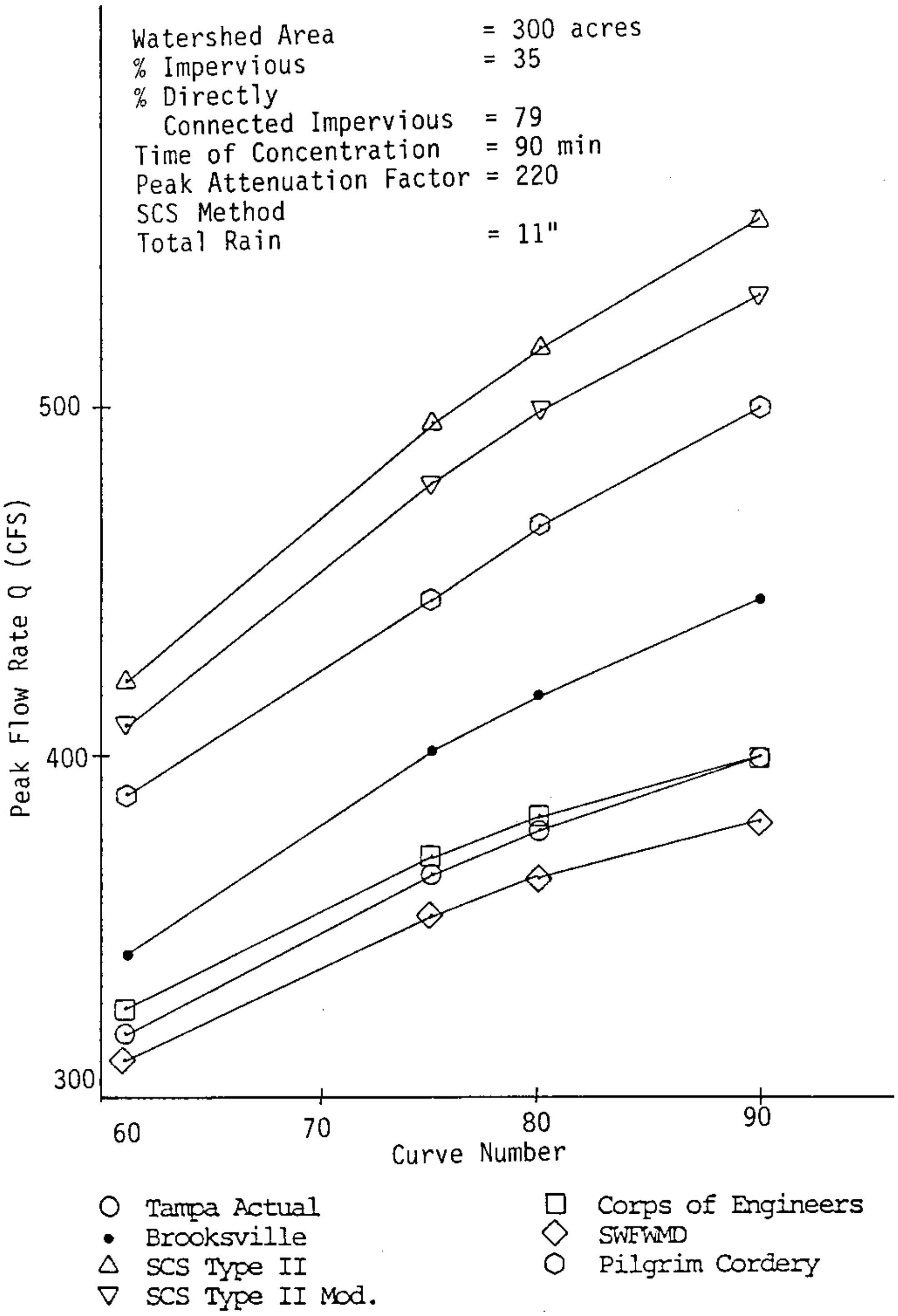


Figure 47. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using SCS Method K = 220, total rainfall = 11 inches.

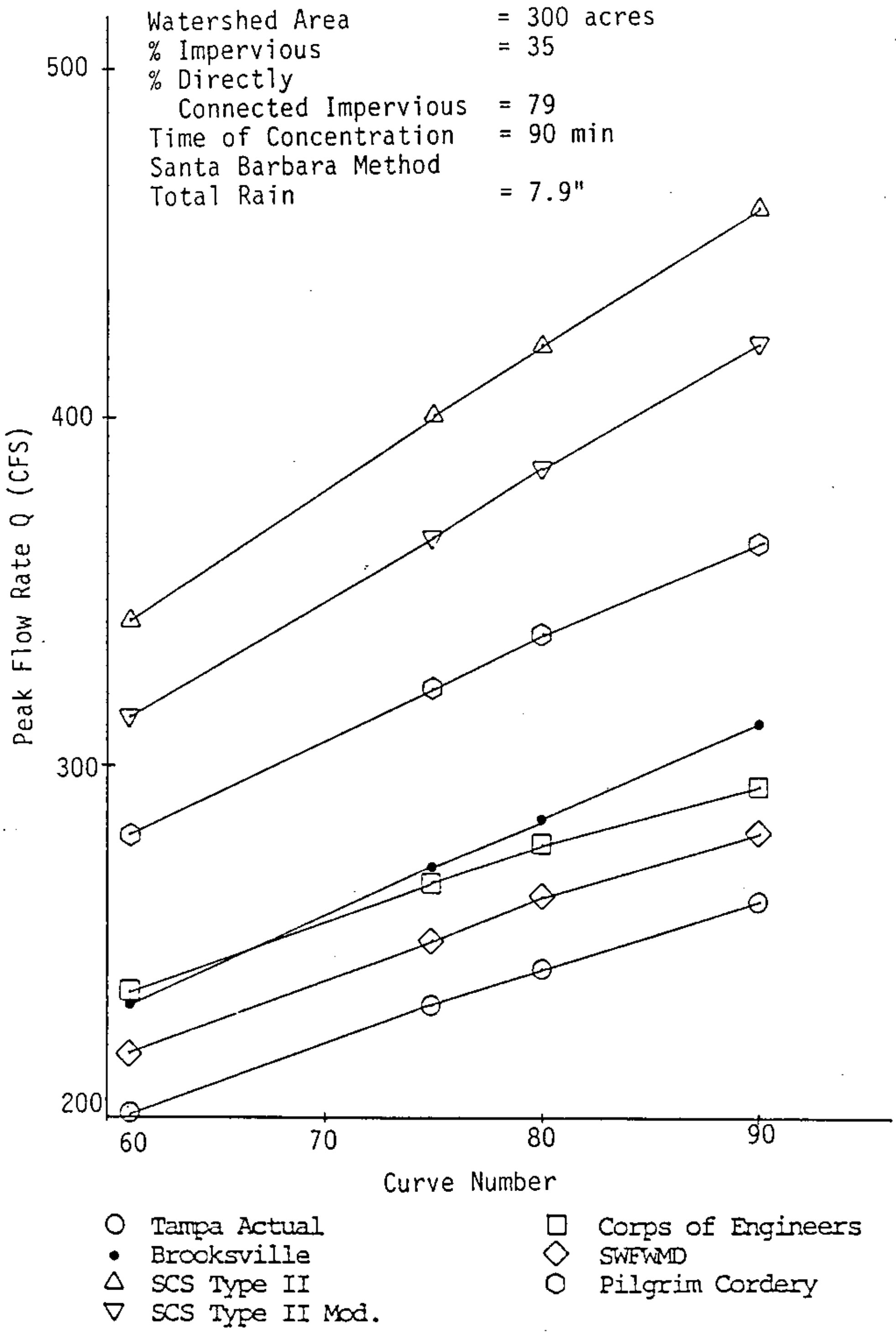


Figure 48. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using Santa Barbara Method, total rainfall = 7.9 inches.

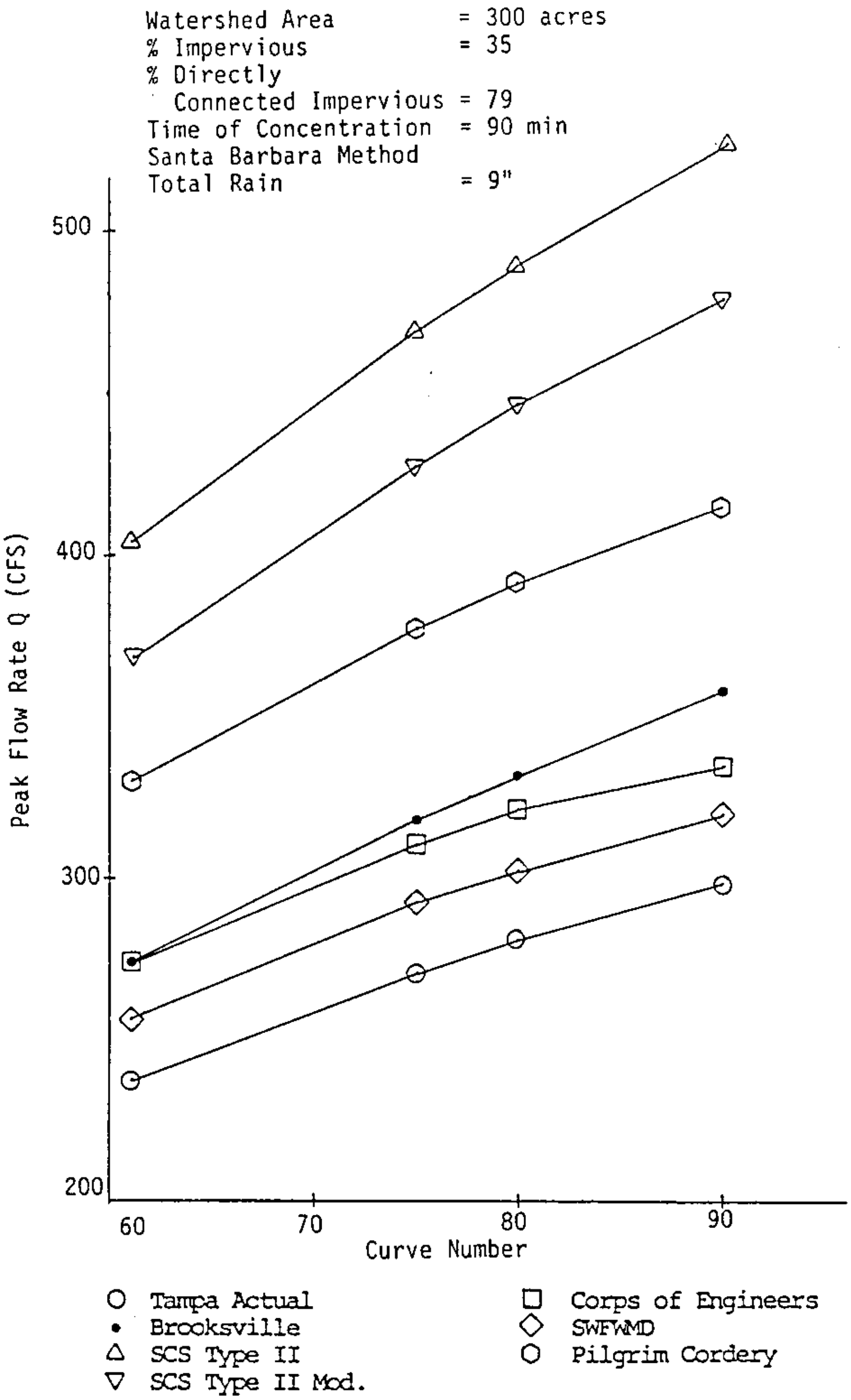


Figure 49. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using Santa Barbara Method, total rainfall = 9 inches.

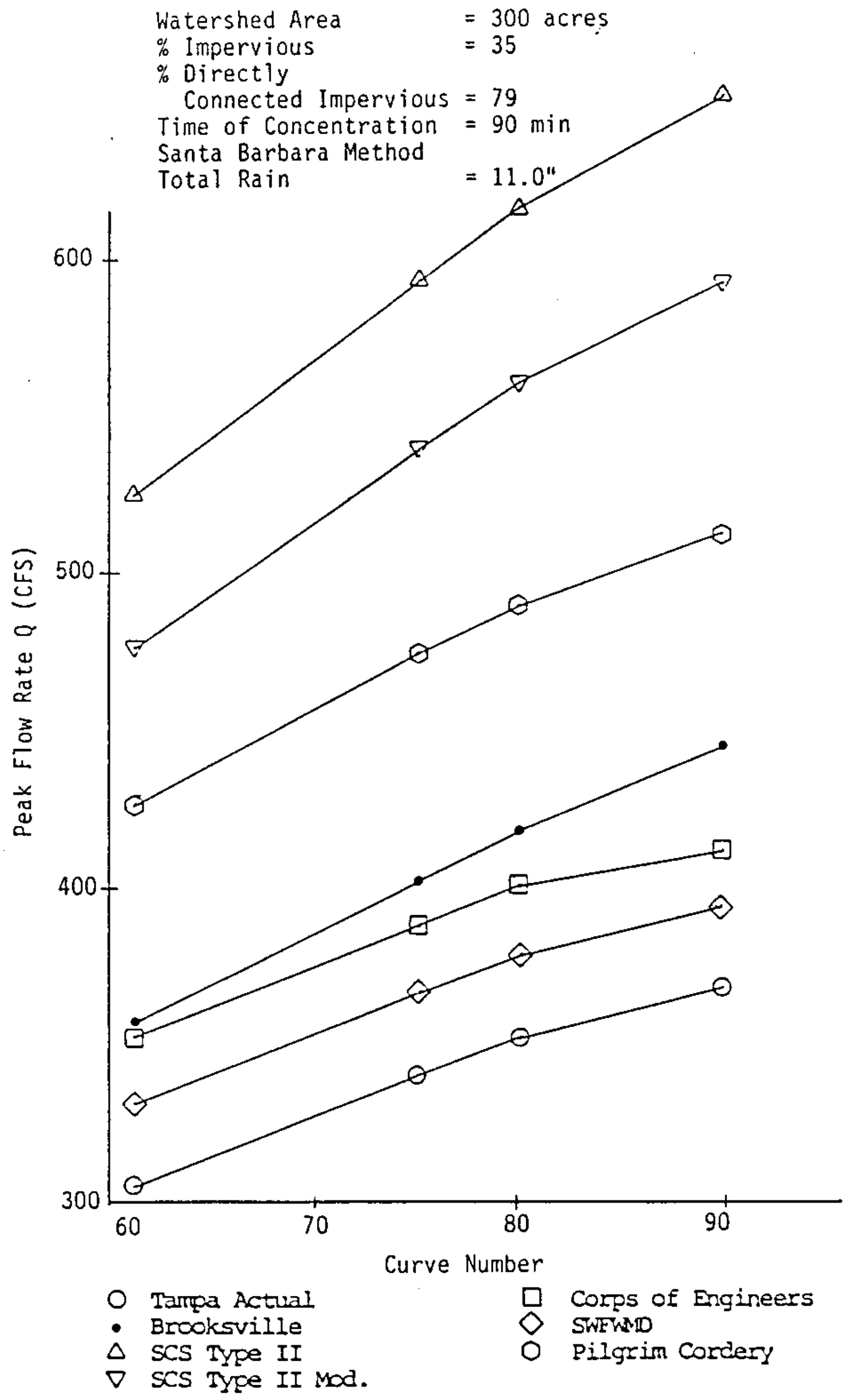


Figure 50. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using Santa Barbara Method, total rainfall = 11 inches.

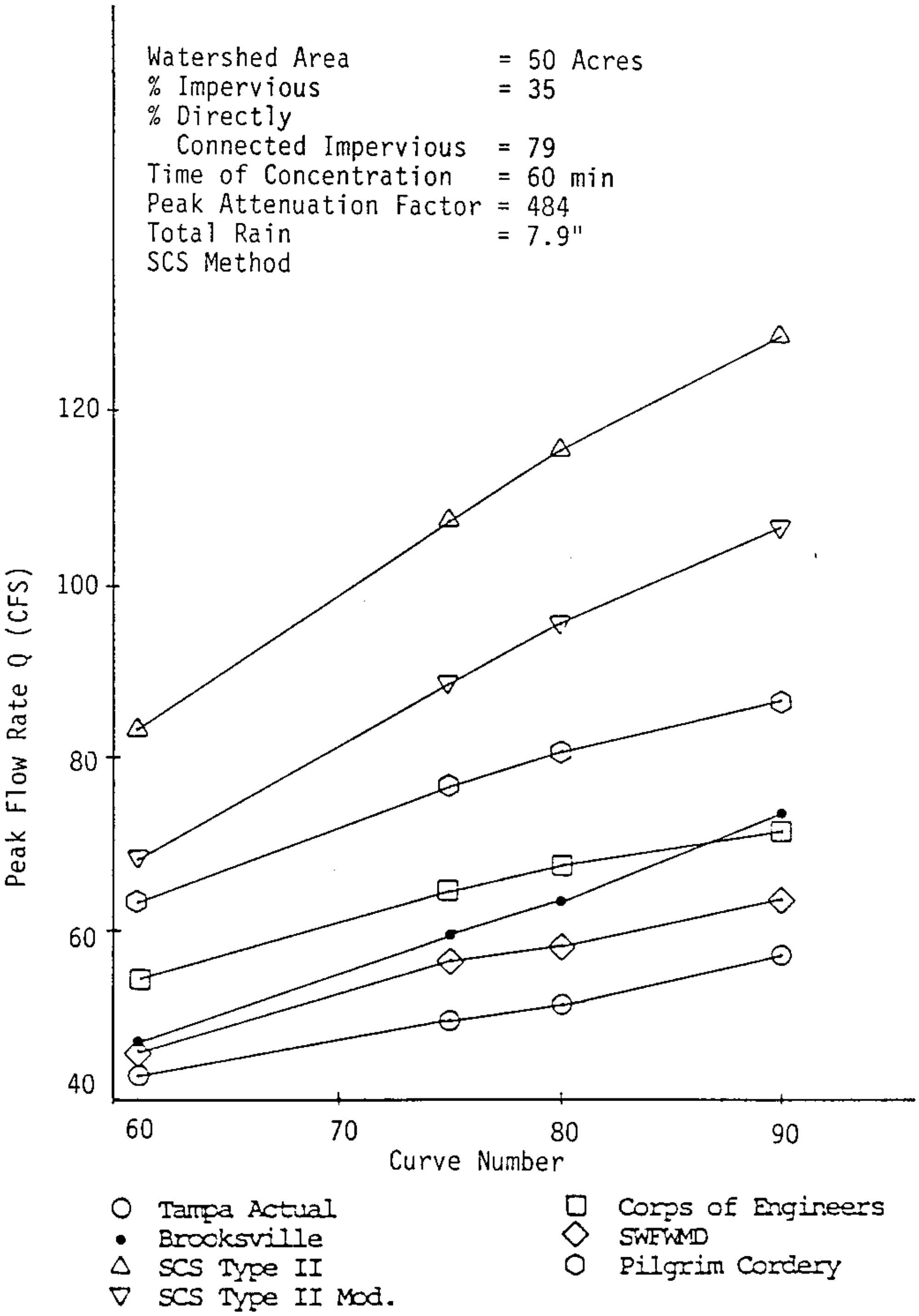


Figure 51. Comparison by rainfall distribution of peak flow rates from a hypothetical 50-acre watershed using SCS Method K = 484, total rainfall = 7.9 inches.

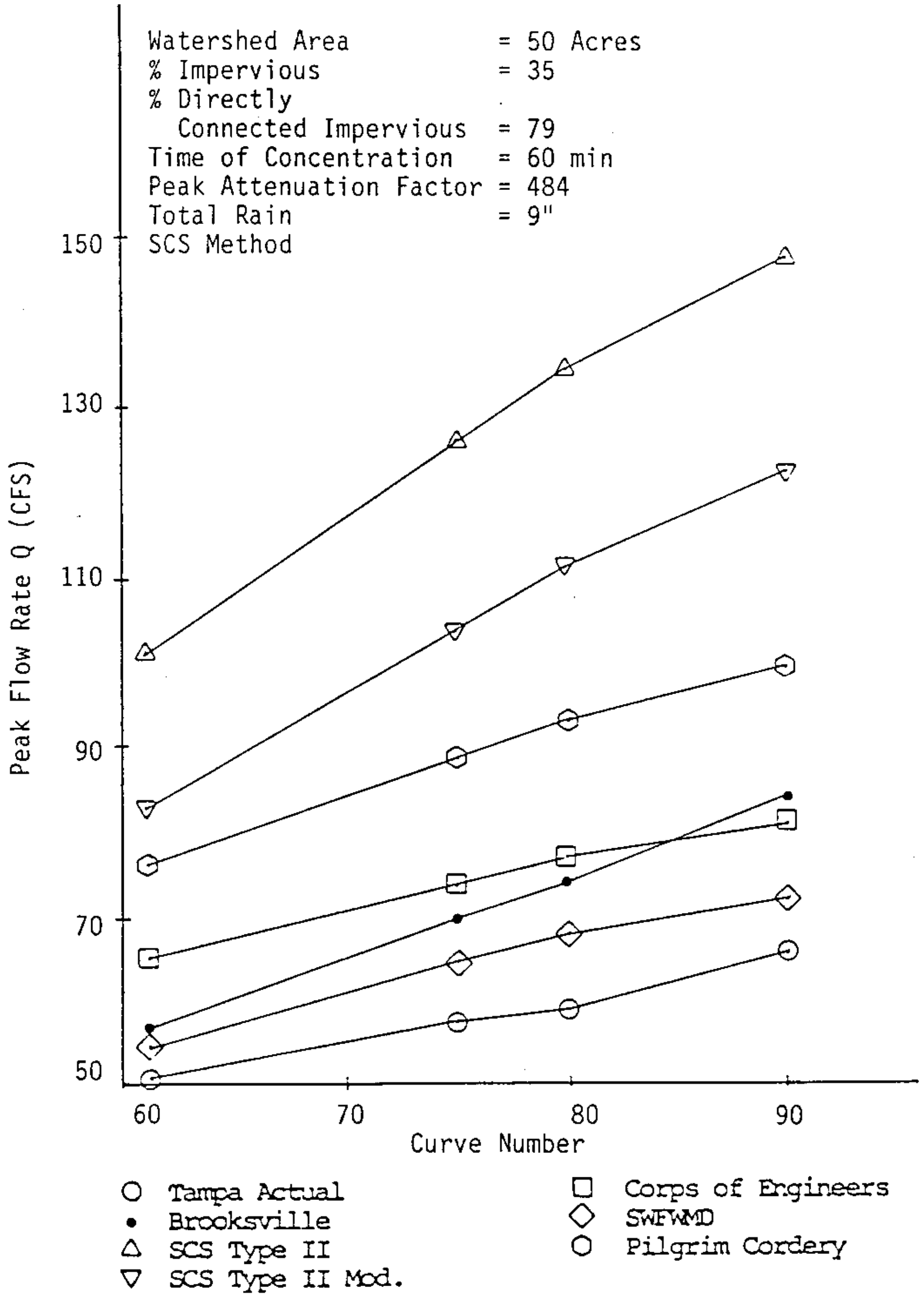


Figure 52. Comparison by rainfall distribution of peak flow rates from a hypothetical 50-acre watershed using SCS Method K = 484, total rainfall = 9 inches.

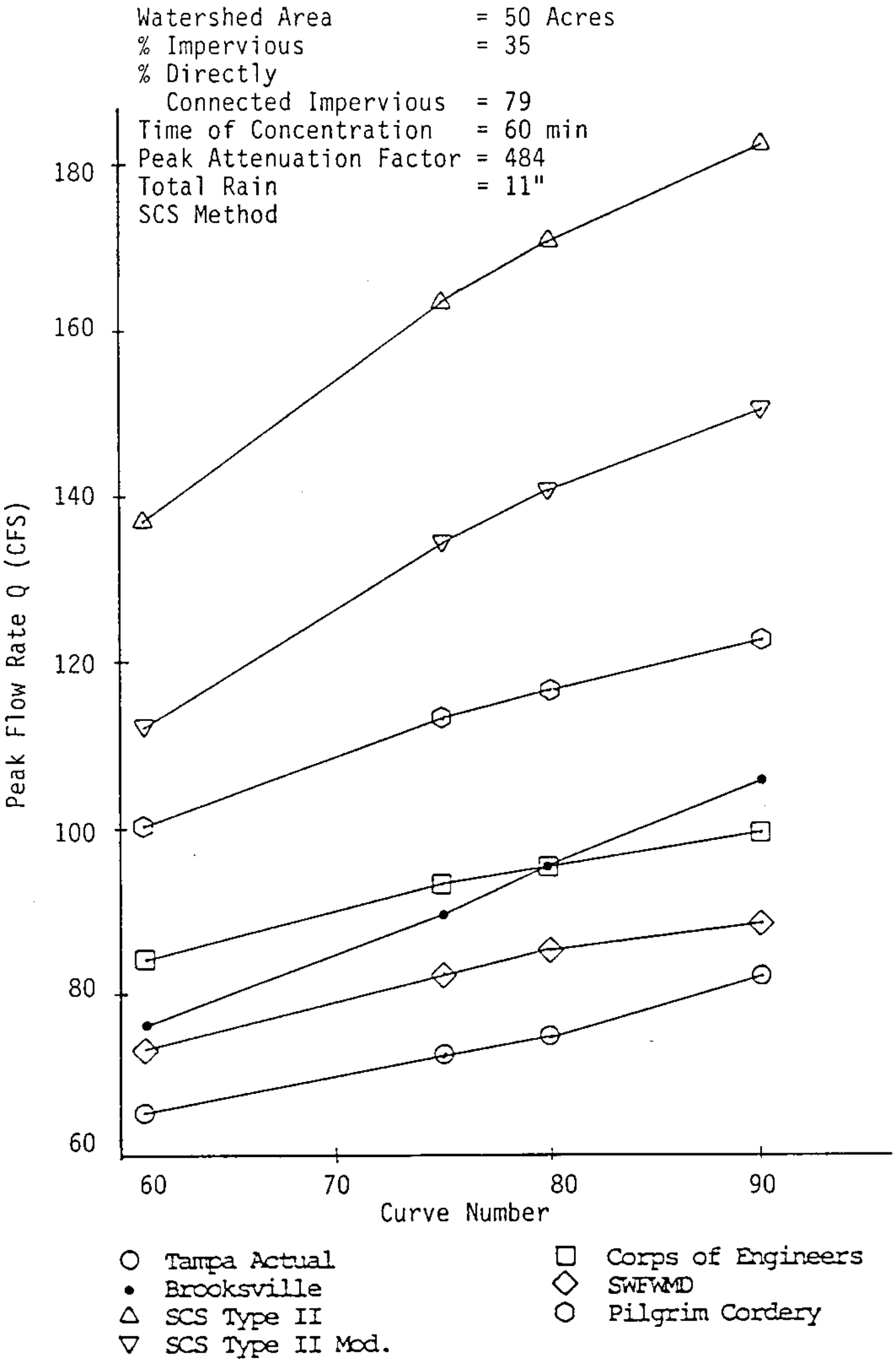


Figure 53. Comparison by rainfall distribution of peak flow rates from a hypothetical 50-acre watershed using SCS Method K = 484, total rainfall = 11 inches.

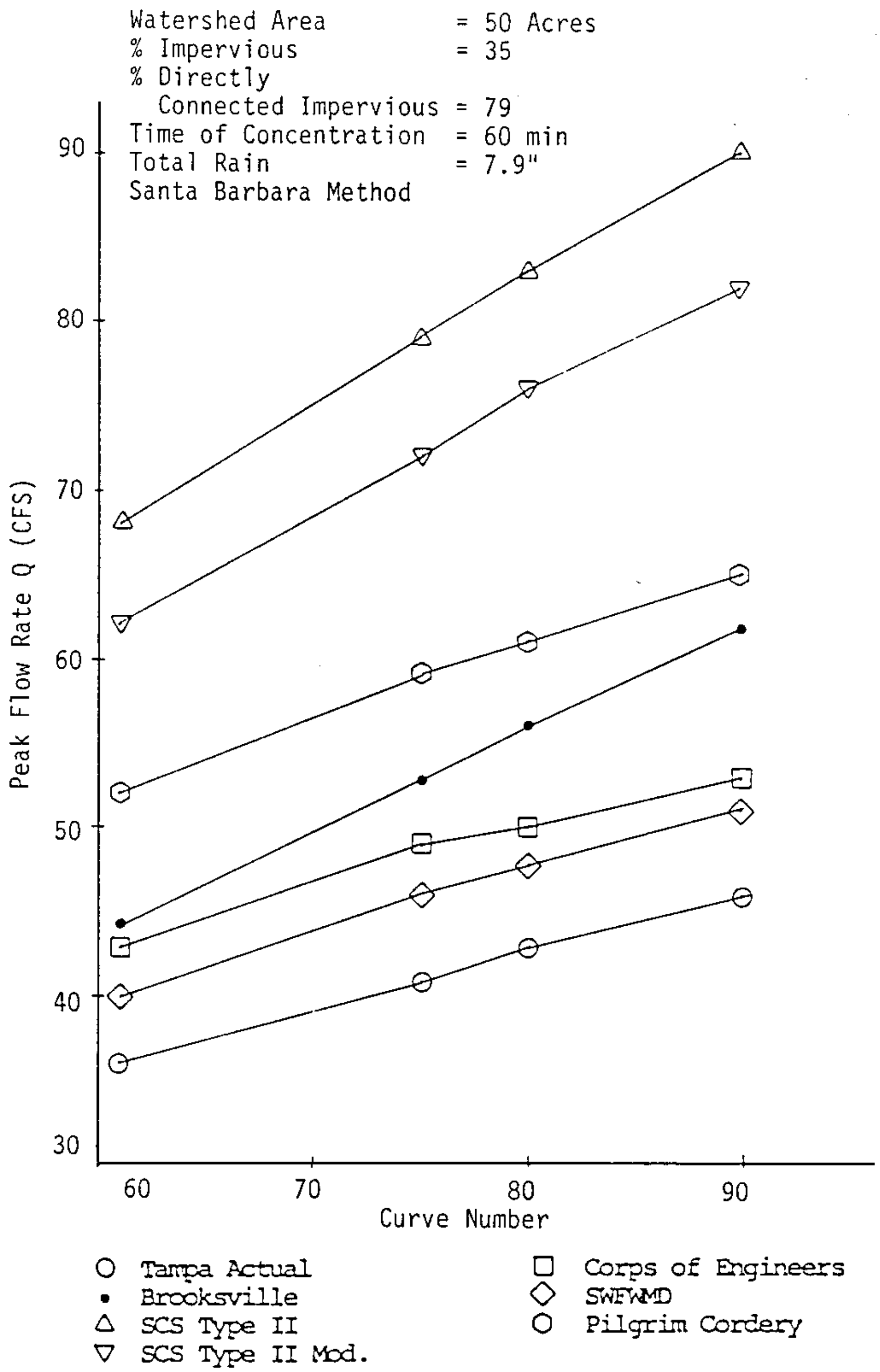


Figure 54. Comparison by rainfall distribution of peak flow rates from a hypothetical 50-acre watershed using Santa Barbara Method, total rainfall = 7.9 inches.

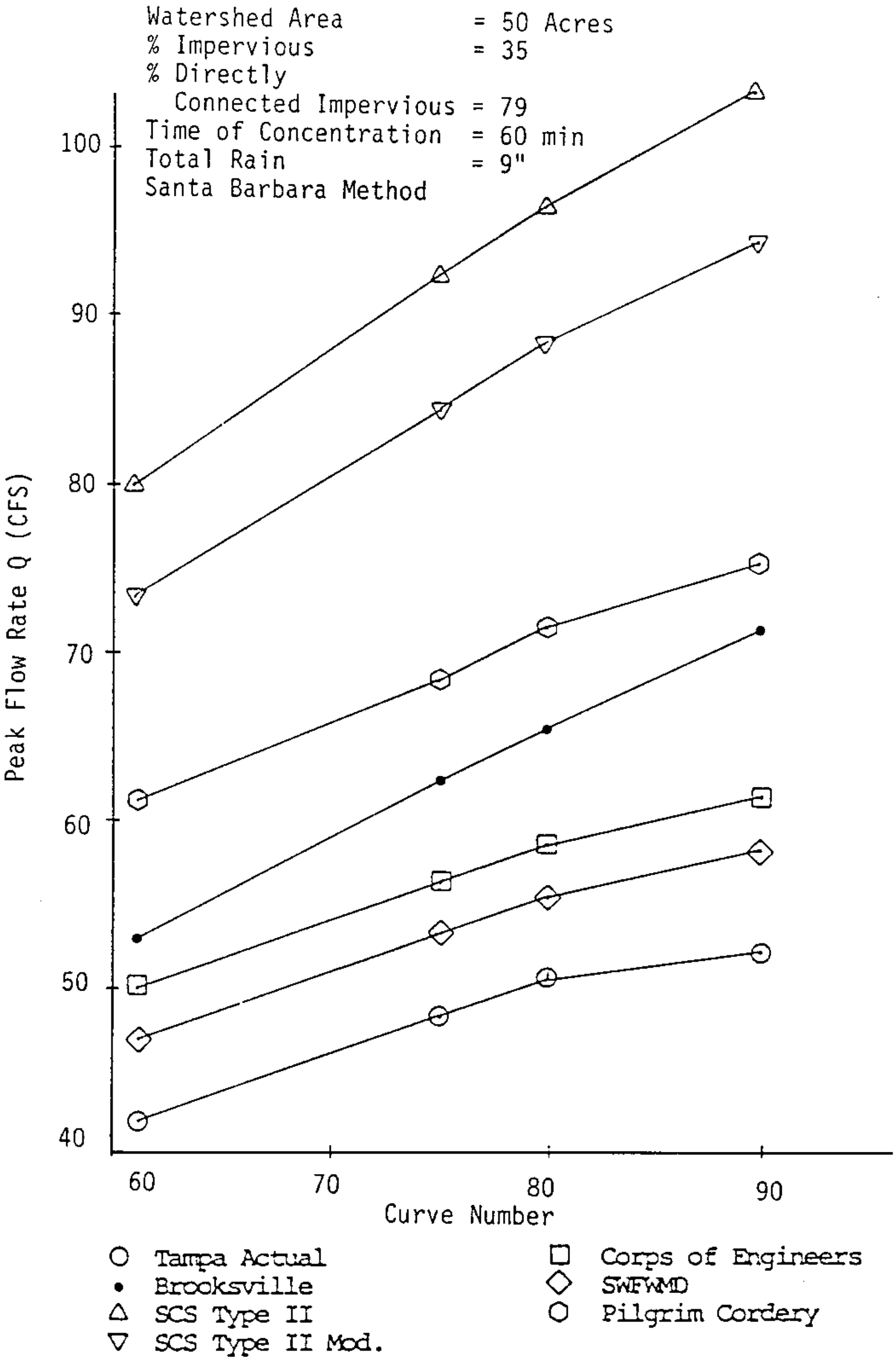


Figure 55. Comparison by rainfall distribution of peak flow rates from a hypothetical 50-acre watershed using Santa Barbara Method, total rainfall = 9 inches.

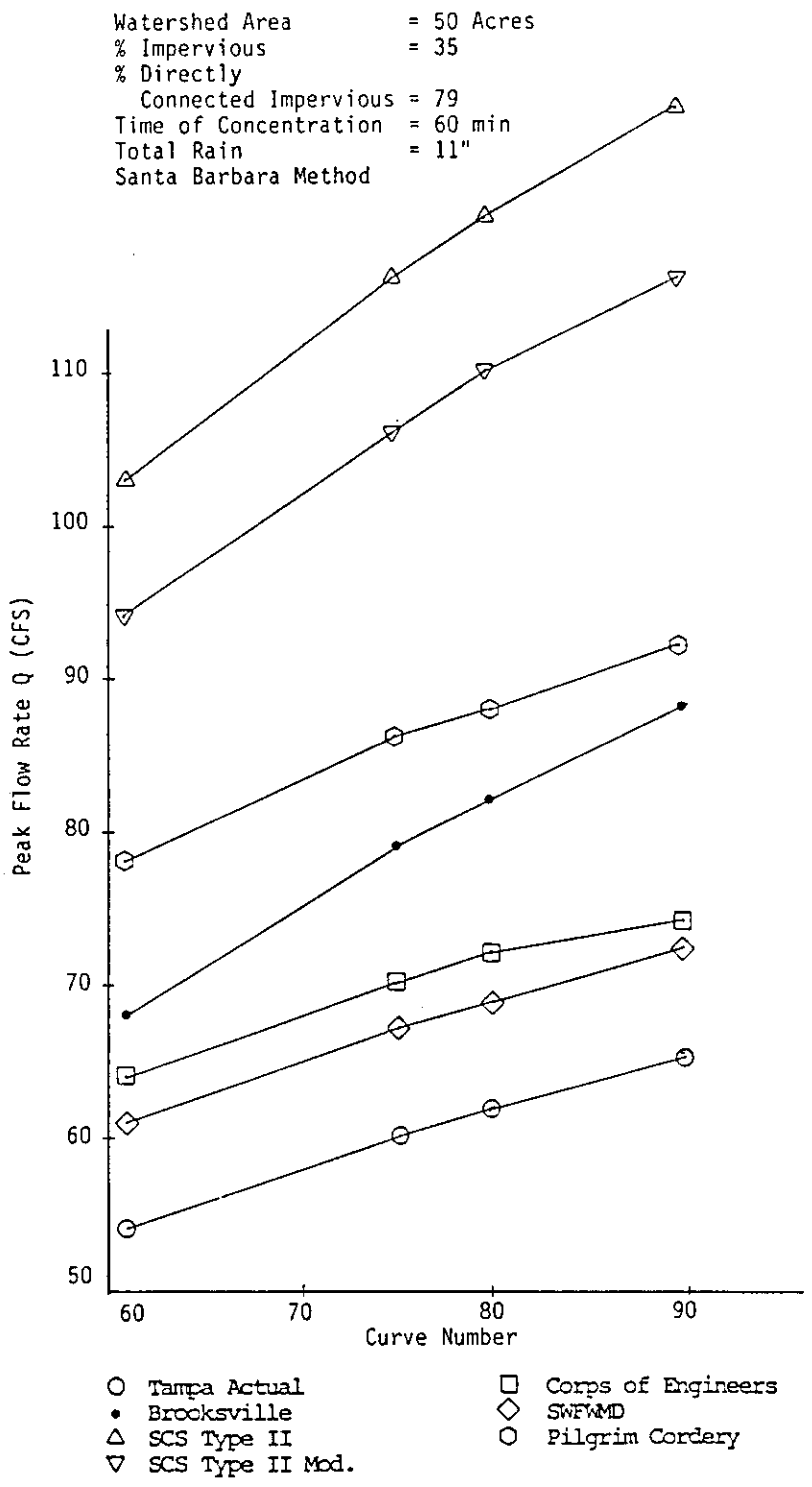


Figure 56. Comparison by rainfall distribution of peak flow rates from a hypothetical 50-acre watershed using Santa Barbara Method, total rainfall = 11 inches.

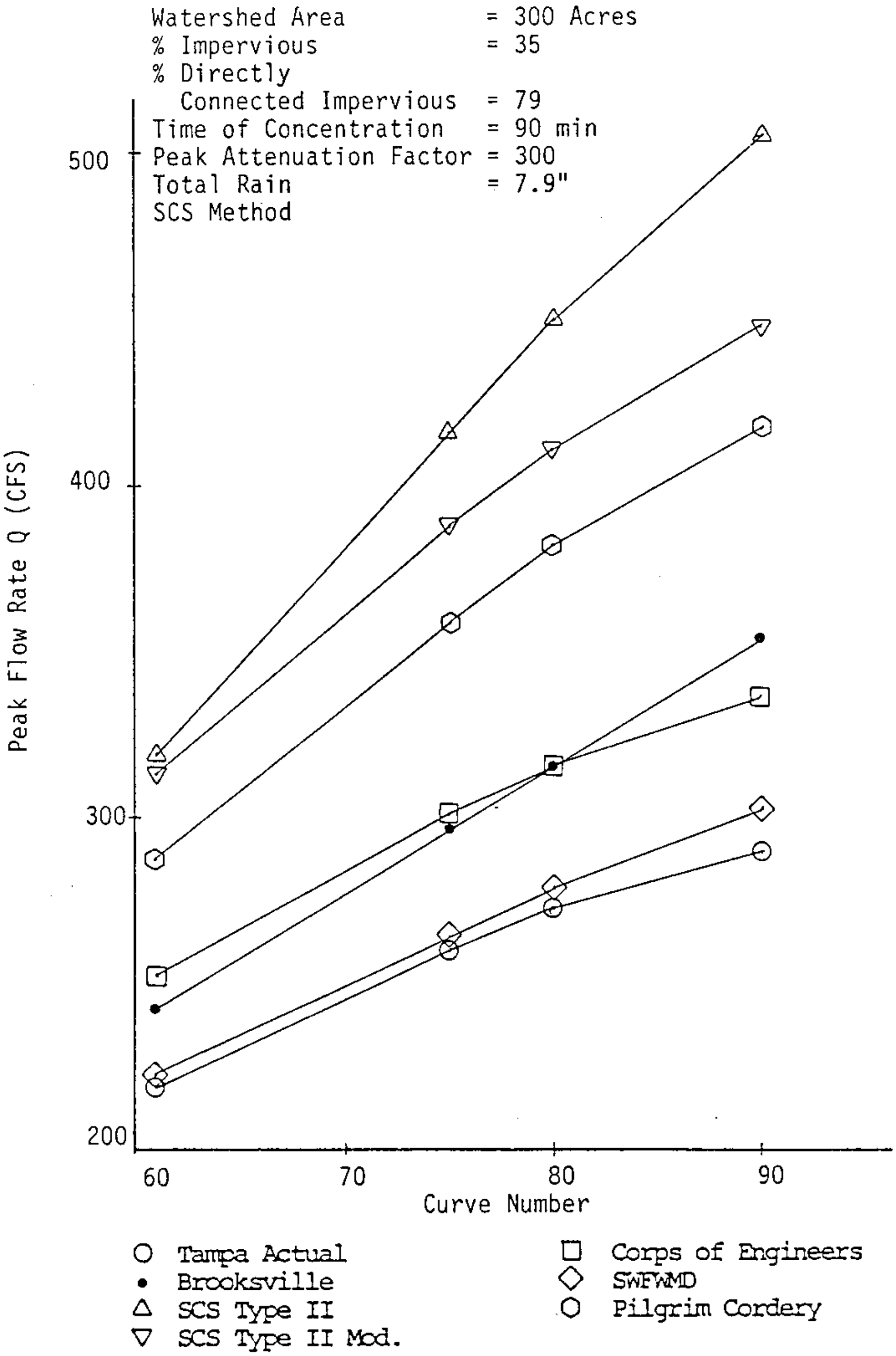


Figure 57. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using SCS Method K = 300, total rainfall = 7.9 inches.

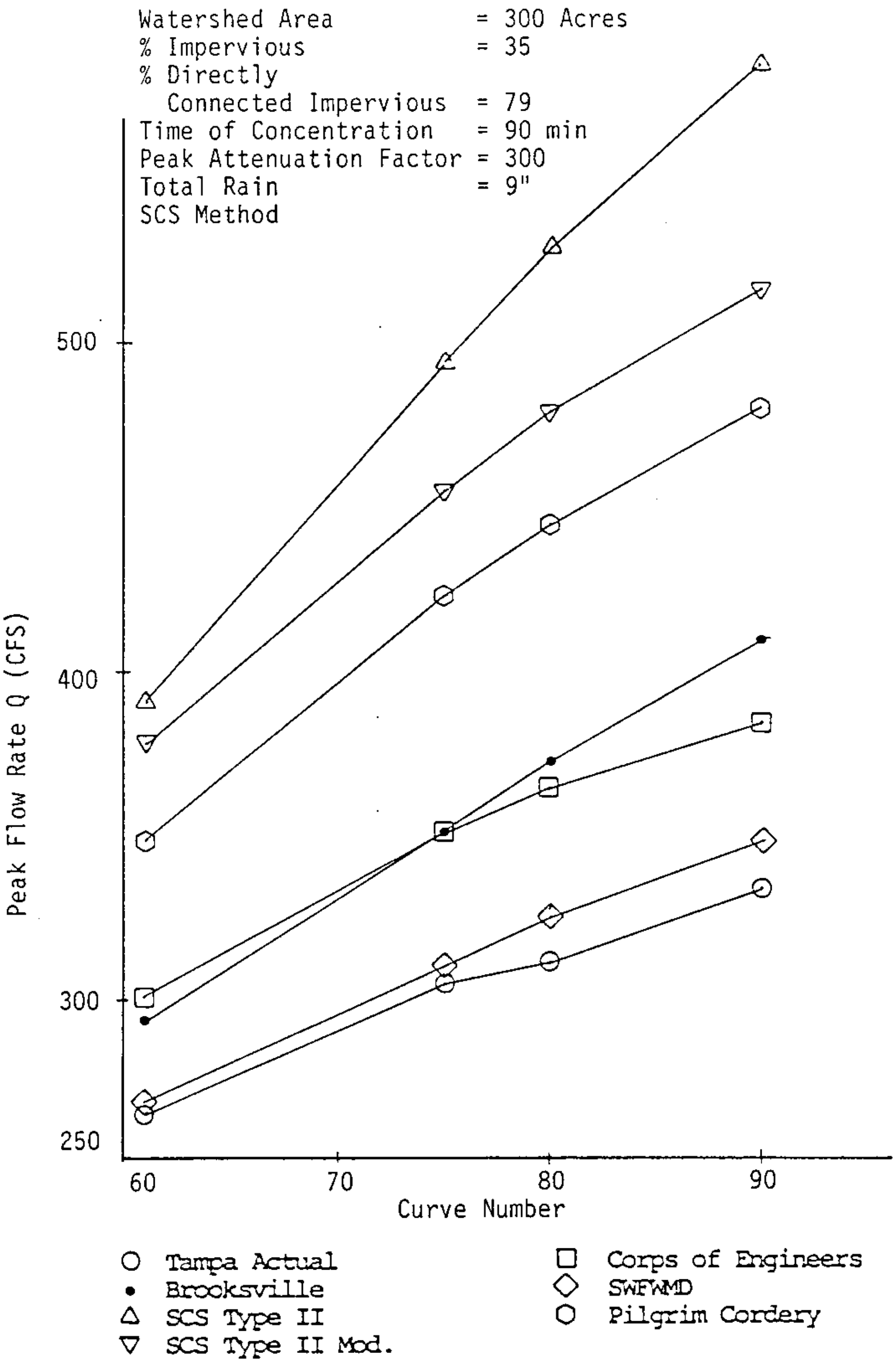


Figure 58. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using SCS Method K = 300, total rainfall = 9 inches.

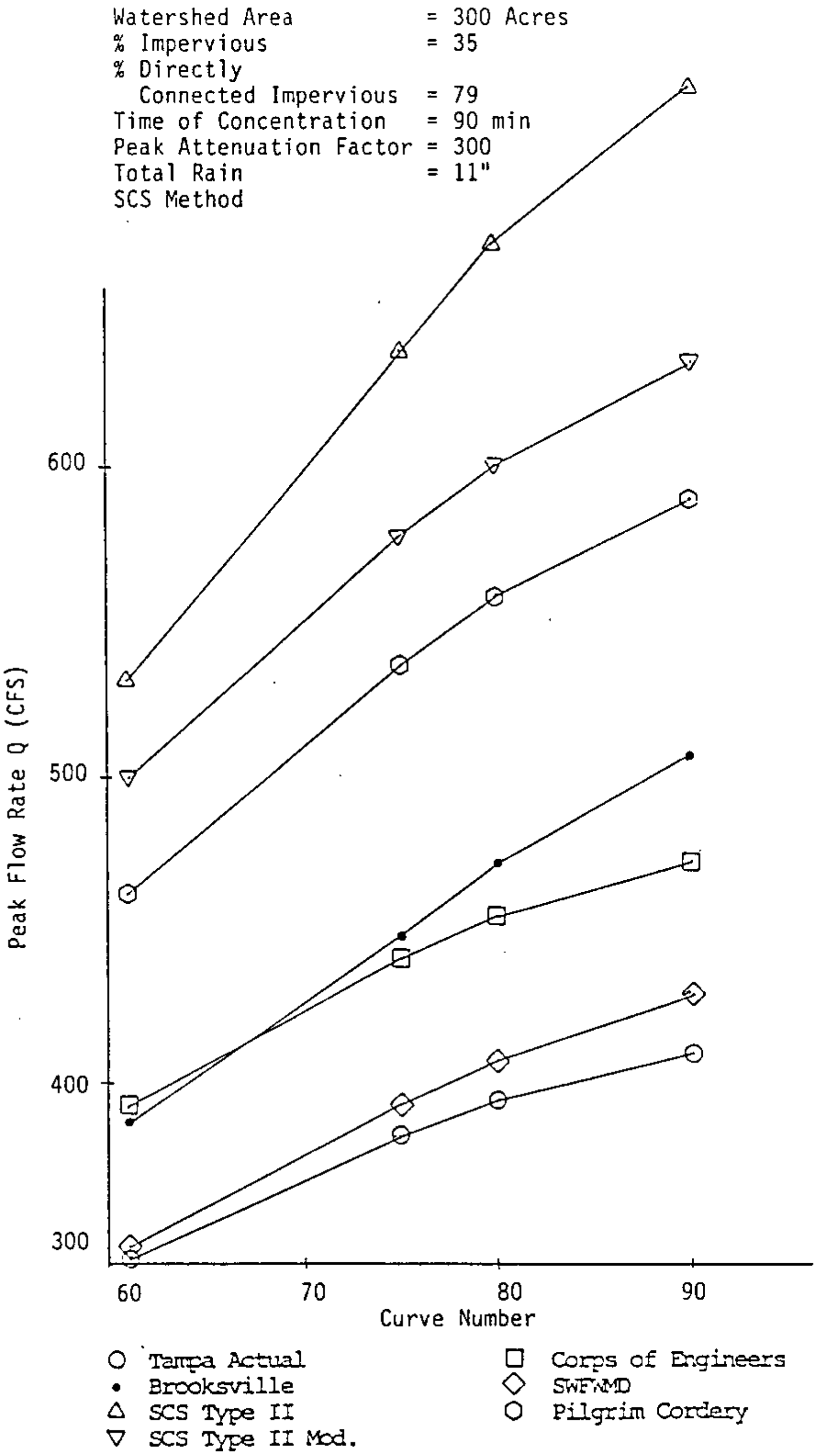


Figure 59. Comparison by rainfall distribution of peak flow rates from a hypothetical 300-acre watershed using SCS Method K = 300, total rainfall = 11 inches.

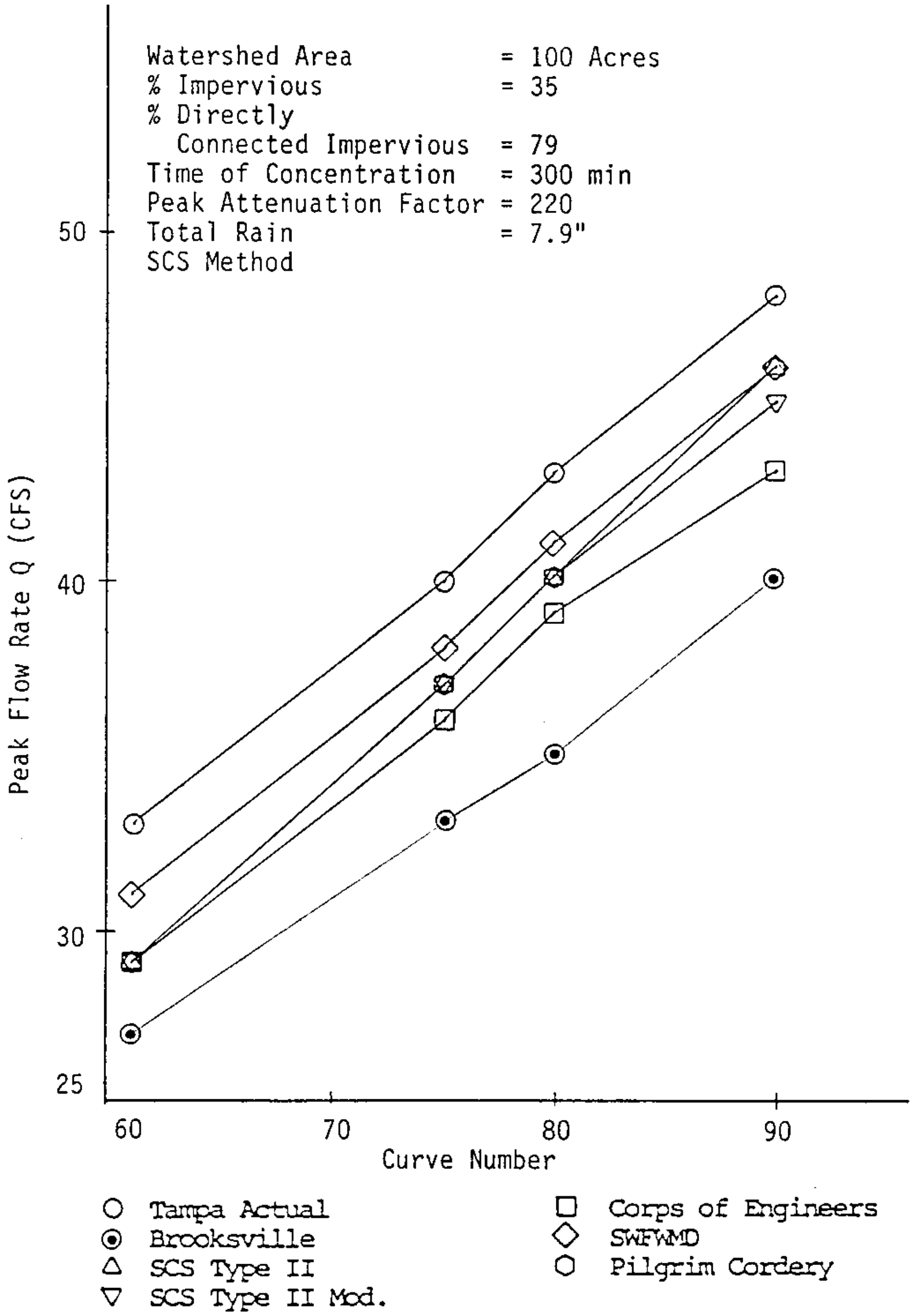


Figure 60. Comparison by rainfall distribution of peak flow rates from a hypothetical 100-acre watershed using SCS Method K = 220, total rainfall = 7.9 inches.

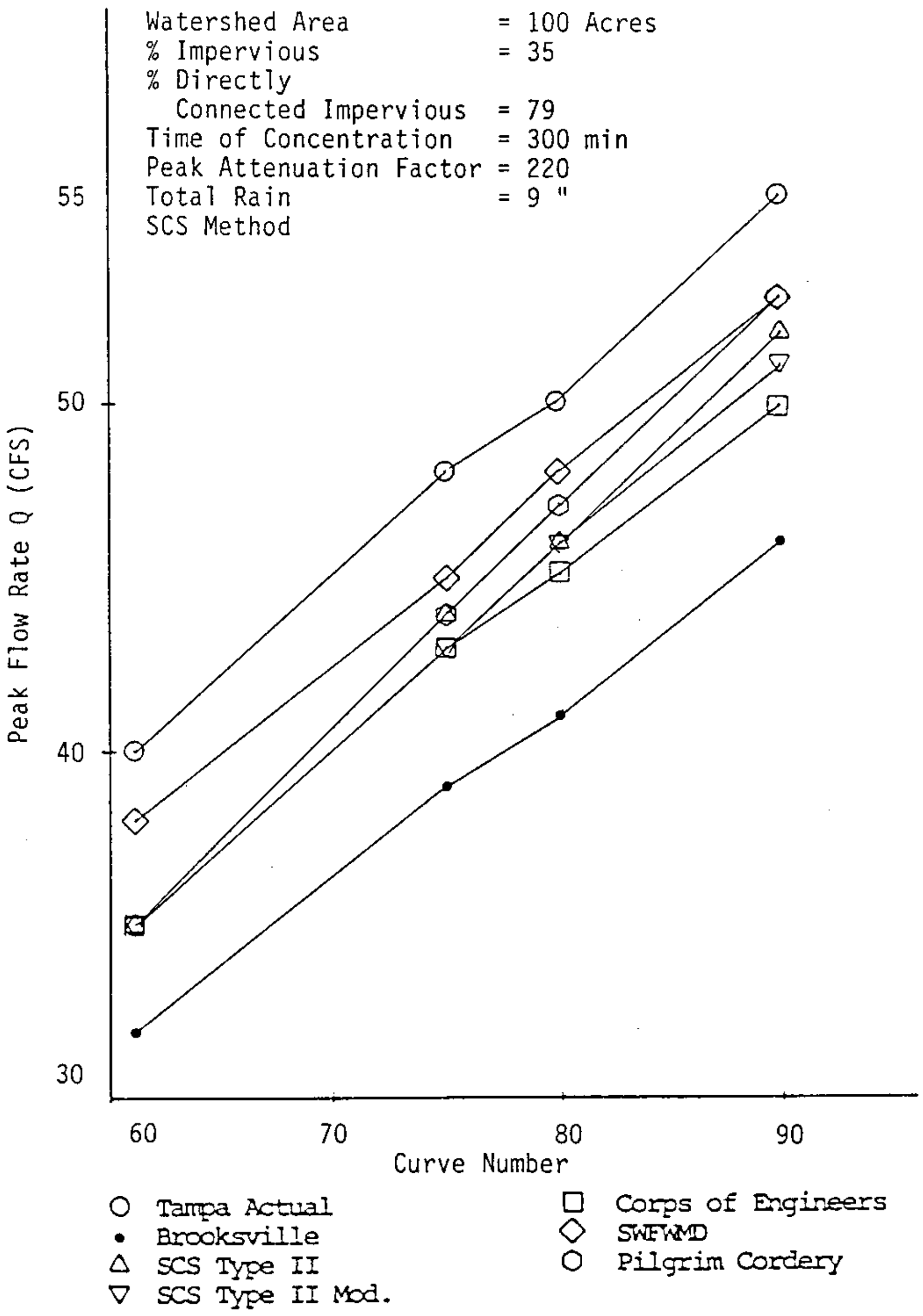


Figure 61. Comparison by rainfall distribution of peak flow rates from a hypothetical 100-acre watershed using SCS Method K = 220, total rainfall = 9 inches.

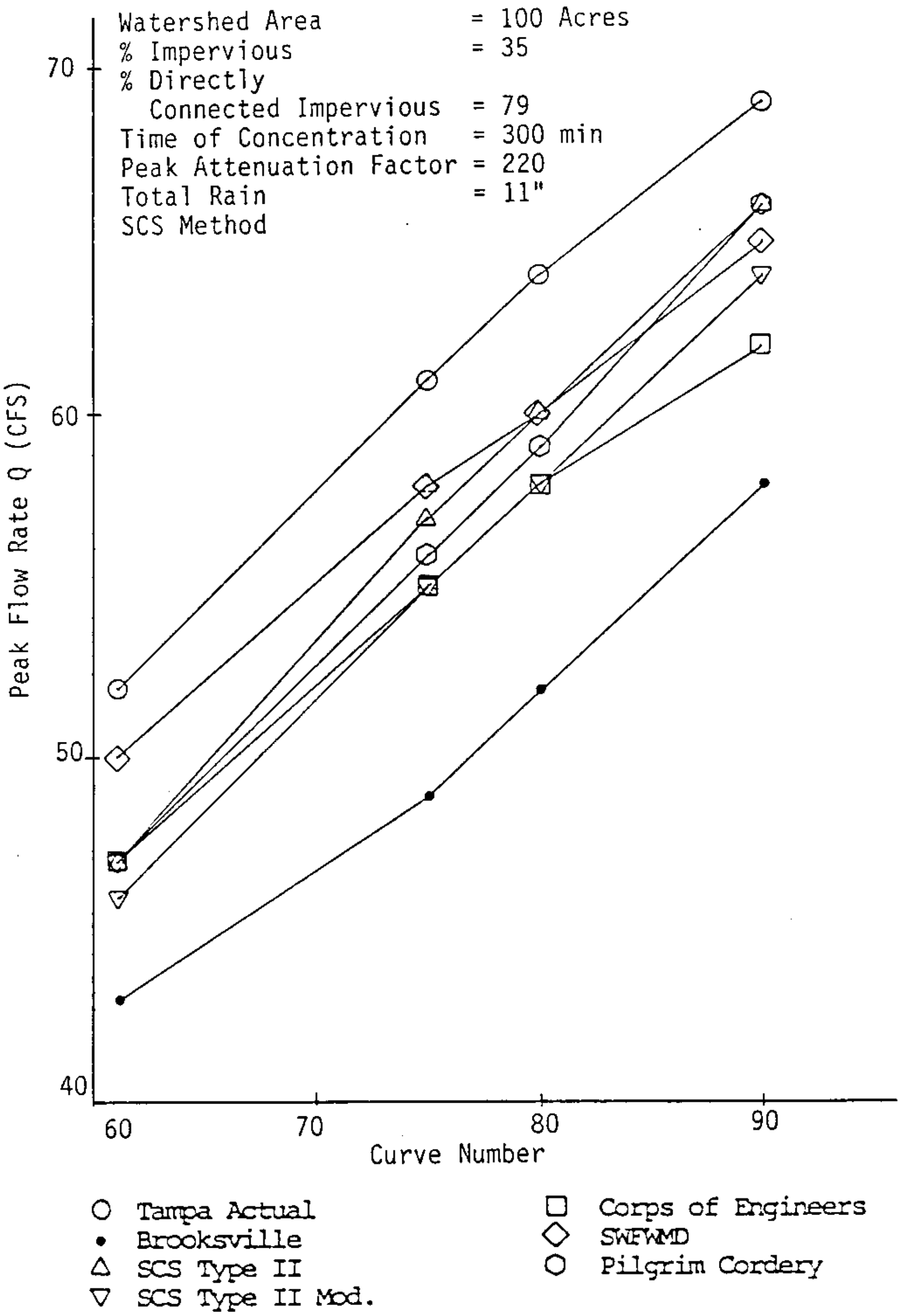


Figure 62. Comparison by rainfall distribution of peak flow rates from a hypothetical 100-acre watershed using SCS Method K = 220, total rainfall = 11 inches.

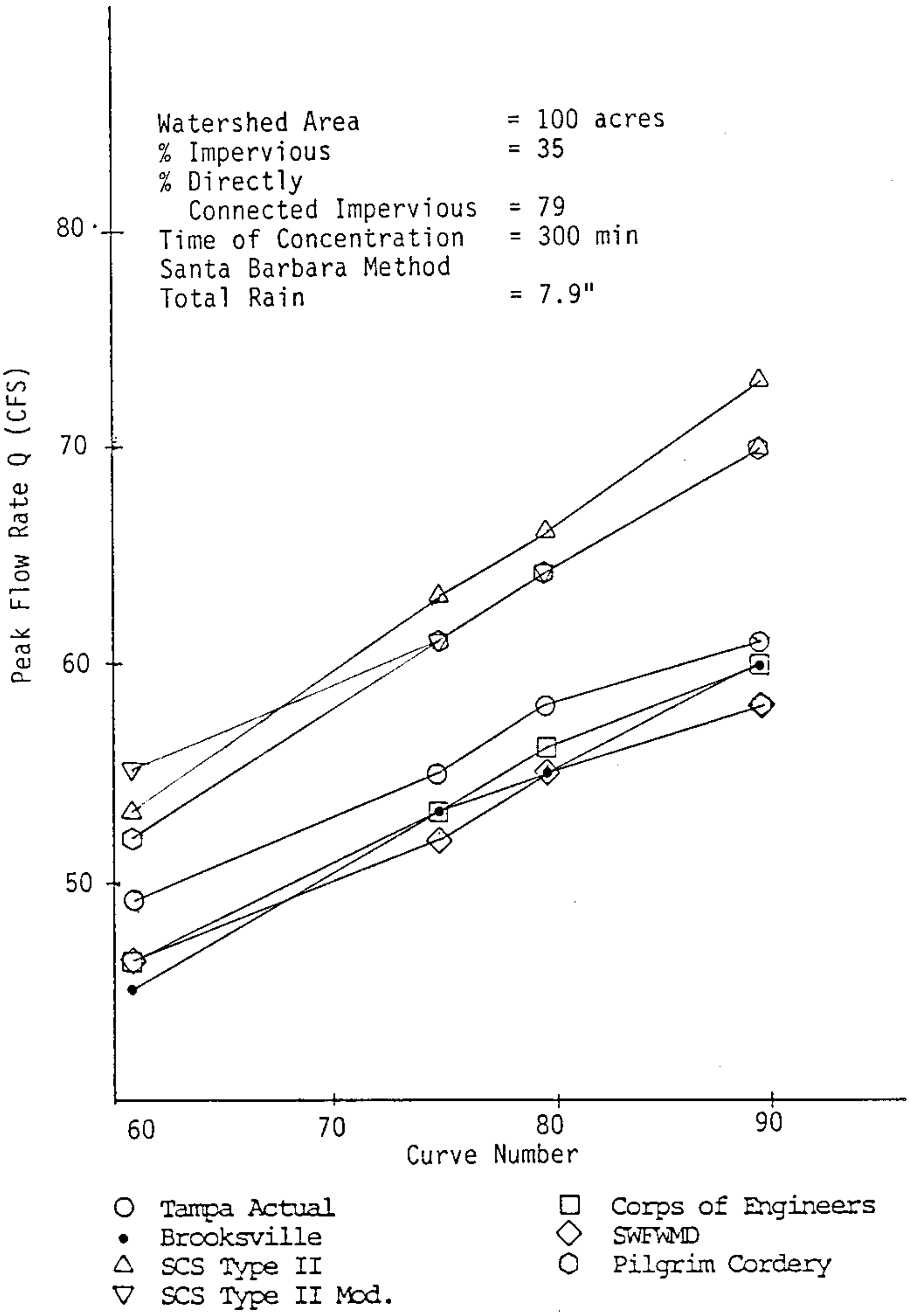


Figure 63. Comparison by rainfall distribution of peak flow rates from a hypothetical 100-acre watershed using Santa Barbara Method, total rainfall = 7.9 inches.

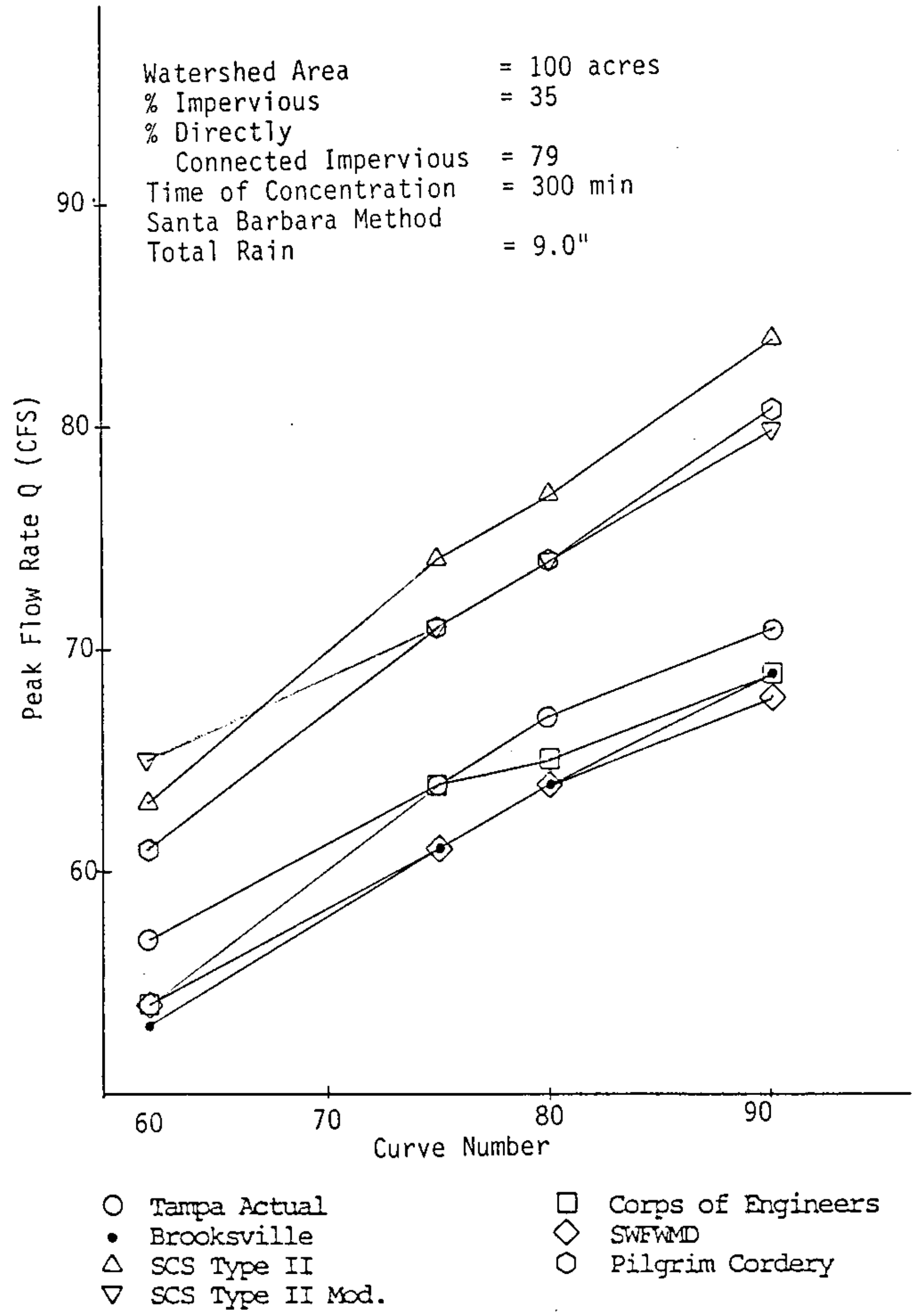


Figure 64. Comparison by rainfall distribution of peak flow rates from a hypothetical 100-acre watershed using Santa Barbara Method, total rainfall = 9 inches.

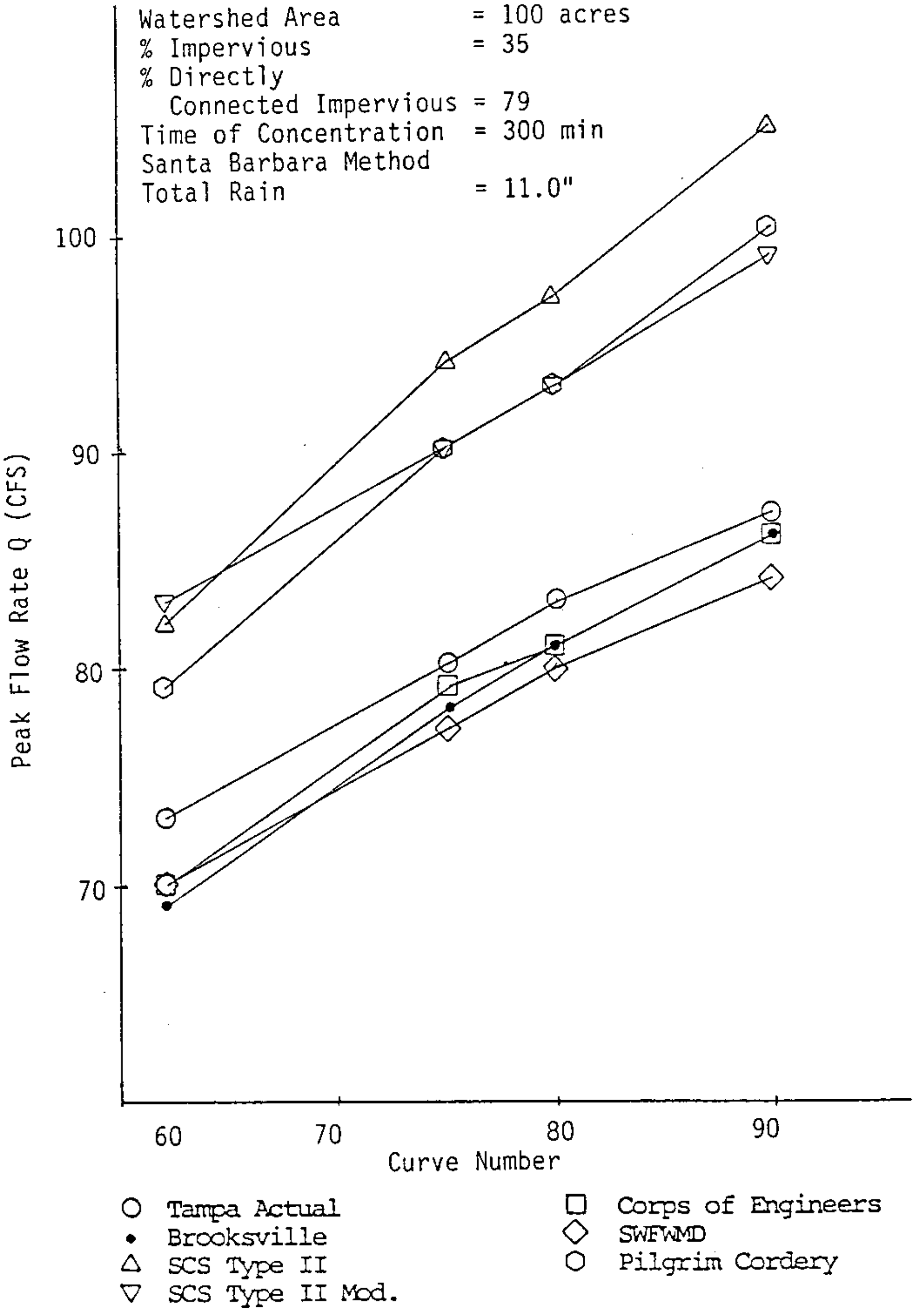


Figure 65. Comparison by rainfall distribution of peak flow rates from a hypothetical 100-acre watershed using Santa Barbara Method, total rainfall = 11 inches.

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